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FACTORS AFFECTING RUTTING OF FLEXIBLE PAVEMENTS IN PAKISTAN

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FACTORS AFFECTING RUTTING OF FLEXIBLE PAVEMENTS IN PAKISTAN

By

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ABSTRACT

FACTORS AFFECTING RUTTING OF FLEXIBLE PAVEMENTS IN PAKISTAN

by

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Flexible pavements are designed to withstand structural and functional failures. Rutting is a structural defect associated with functional implications. Rutting is mainly caused by wheel loads and accelerated by material and environmental factors. Although all pavement layers (base, subbase and roadbed soils) contribute to rutting, the contribution of the Asphalt Concrete (AC) layer alone could be very significant.

The pavement network in Pakistan is experiencing premature rutting problem in the AC layer. The main objective of this study is to identify the AC mix factors and physical properties that contribute to pavement rutting.

Fifty pavement sections were selected from a large pool of pavements. For each selected pavement section rut measurements were made and cores were obtained. The cores were subjected to various laboratory tests to ascertain the asphalt mix properties.

A multivariate regression analysis was performed to determine the relationship of rut depths and AC mix properties. The result showed that air voids in the range of 4 to 7 percent, and low penetration of asphalt, can provide significant improvement in rut resistance of the AC mixes. High loads and the longitudinal grade of pavement contribute to rutting.

In memory of my parents and to wife, children, sister, brothers and all other loving humanity.

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INTRODUCTION

1.1 GENERAL

The structural design of asphalt concrete (AC) pavements and AC overlays has been an evolutionary process based primarily on experience and judgment of the highway engineers, augmented by empirical relationships developed through research and field observations. A proper design of AC pavements requires the consideration of several complex and interrelated factors such as, the AC mix design, environmental factors and engineering properties of various pavement layers. Recently efforts have been made to analyze these factors to enhance existing empirical design procedures and to develop rational design methods/models based on theoretical background such as the theories of elasticity, visco-elasticity and plasticity. Today, design methods for flexible pavements and overlays could be divided into two groups, empirical and mechanistic-empirical. The main design consideration in both groups is to limit the vertical compressive strain induced at the top of the AC layer, base and roadbed soil to control permanent deformation and to minimize the tensile strain induced at the bottom of the AC or the stabilized layers to control fatigue cracking.

Both rutting and fatigue cracking are load associated distresses and are affected by several other factors such as traffic volume, load, gradient of road section(which reduces speed of trucks), material properties, construction quality, layer thicknesses and environments. Therefore, any methodology based on empirical or mechanistic approaches, must model these factors in order to predict the pavement behavior efficiently.

Rutting develops through different mechanisms namely repetitive plastic deformation (no volume change) and densification (volume change). The rate and amount of rutting resulting from repetitive plastic deformation, is a function of the load magnitude, load repetitions, material properties, construction quality, and temperature. Prevention of this type of rutting is handled through pavement structural/material design. Rutting caused by densification can be controlled by better quality control, compaction specifications, and enhanced design.

The principal objective in design of AC pavement structures is to reduce both rutting and fatigue cracking potentials. This objective can be achieved by using a balanced design procedure, a good quality asphalt mix, better specifications and quality control practices.

1.2 PROBLEM STATEMENT

In Pakistan, permanent deformation is the principal distress mode in flexible pavements. Several factors affecting this type of distress have been identified including:

1. High truck loads/volumes and high tire pressure.
2. The asphalt mix properties.

3. Inadequate pavement structural design.
4. Inadequate construction practices (e.g. compaction, segregation).
5. Softening of the asphalt binder due to high temperatures.
6. Strength and type of the materials used in the various pavement layers.
7. Softening of the base, subbase, and roadbed materials due to moisture infiltration in some regions of Pakistan.

Existing rehabilitation methods to correct rutting of asphalt pavements in Pakistan generally involve cold milling followed by AC overlays. The rehabilitation process is found costly and provides no assurance that the overlaid pavement will not rut again. In many instances, newly constructed and/or rehabilitated asphalt pavements have experienced rutting in a short time period after construction. Hence the need to identify the fundamental causes of rutting and to determine the remedial measures have been recognized.

1.3 RESEARCH OBJECTIVES

The objectives of this research study include:

1. Analysis of the asphalt mix properties (i.e. percent aggregate, sand and fine contents, aggregate angularity, binder type and content, and the percent air voids) that affect pavement rutting.
2. Carryout field investigations comprising of measurements of rut depth and road gradients and computation of traffic loads(ESALs).
3. Develop a statistical rut model and identify the variables that influence pavement rutting in Pakistan (i.e. asphalt content, pavement thickness, recovered penetration of asphalt, fines and sand content, aggregate type and content, air voids, AC layer thickness, gradient of road and ESALs etc.

4. Prioritizing the factors in order of their significance on rutting, make conclusions and recommendations to improve the existing asphalt mix design and pavement construction practices.

1.4 SCOPE

To accomplish the stated objectives, 54 flexible pavement sections, were selected for this study from SAHIWAL (km-1102 distance measured from Karachi) to PESHAWER (km-1700) areas by physically surveying the routes. The criteria used in selection was that the selected sections should represent the spectrum of pavement cross sections, paving material, traffic volume and loads, found in most of the areas of Pakistan and exposed to similar weather conditions(hot weather). Detailed locations can be viewed from figures 1.1 and 1.2. Full factorial experiment matrix (Table 1.1) had been formed. Volume of traffic(yearly ESALs) and ranges of AC thickness have been considered as variables for this task. All cells of the matrix were studied depending upon availability of pavement sections within desired parameters of dependent variable (Rut). Each test section of varying length(1 to 5 km), has test sites of 30 meter long with varying levels of distress severity. Distress levels were measured at five test locations(6 meter apart) within one test site, as per AASHTO instructions(2) and then the average was taken (Figures 1.3 and 1.4). The study encompasses a synthesis of available information on rutting, measurements of rut depths and road gradients, extraction of representative core samples and laboratory testing of these samples to include, measurement of AC thickness, calculation of bulk specific gravity/air voids, recovered asphalt penetration, percent asphalt content, aggregate, sand and fines. The test data has been analyzed, statistical rut model developed and sensitivity analysis has been under taken to offer conclusions and recommendations

Figure 1.1. Map of the Islamic Republic of Pakistan

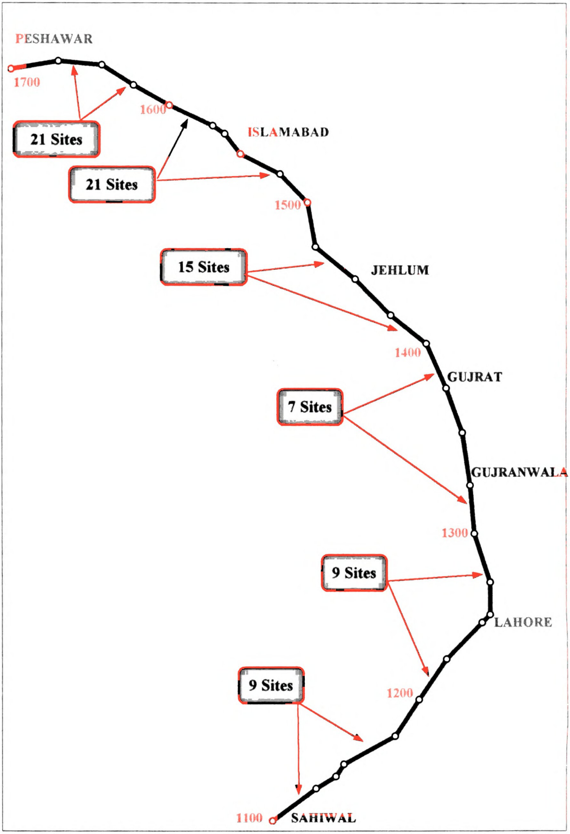

Figure 1.2: Detailed location map of areas included in the study.

Table 1.1: Experimental Design Matrix.

		YEARLY ESALS (millions)		
		0 - 2	2 - 4	>4
ASPHALT CONCRETE THICKNESS (cm)	5 - 10	1	4	7
	10 - 18	2	5	8
	> 18	3	6	9

*** Including asphalt wearing course and asphalt base course**

Test Site (30 (30 M)	Variable Distance	Test Site (30 M)	Variable Distance	Test Site (30 M)

Figure 1.3: A schematic representation of a pavement test section with three test sites.

X X X X X

6M	6M	6M	6M	6M

X = Test location

Figure 1.4 : A schematic representation of the five test locations along 30 meter long test site.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The load carrying capacity of flexible pavements is brought about by the load-distributing characteristics of their layered systems (1). In general, flexible pavements consist of a series of layers with the highest quality material placed at or near the surface. Hence, their strengths are the result of building up thick layers and thereby distributing the load over the roadbed soil (1).

Two types of load related distress (rut and fatigue) are most commonly found in flexible pavements. Rut is a surface depression in the wheel paths. Pavements uplift may occur along the sides of the rut; however, in many instances, ruts are noticeable only after rainfall, when the wheel paths are filled with water. Rutting stems from a permanent deformation in any of the pavement layers or subgrade, usually caused by consolidation or lateral movement of materials due to traffic loads. Rutting may be caused by plastic movement in the AC mix due to high atmospheric temperatures or inadequate compaction during construction. Significant rutting can lead to major structural failure of the pavement and hydroplaning potential. Wear of the surface in the wheel paths from studded tires can also cause a type of "rutting" (2).

It should be noted that the contribution of the AC layer to the total pavement rutting due to densification is negligible since this layer is typically compacted to near its theoretical maximum density during construction. Permanent deformation in the AC layer is mainly the result of lateral distortion due to repeated shear deformation (3).

Existing flexible pavement design methods can be divided into two categories; empirical and mechanistic-empirical. Most of the empirical design methods are based on statistical equations derived from field observations of pavement rutting and surface roughness. Mechanistic-empirical design methods, on the other hand, are mainly based on two criteria:

1. Minimizing the rut potential of each pavement layer by limiting the magnitude of the compressive stress induced at the top of that layer by a moving wheel load.
2. Maximizing the fatigue life of the AC layer by minimizing the induced tensile stress at the bottom of the layer due to a moving wheel load.

Regardless of the pavement design method (empirical or mechanistic-empirical) employed in the design of flexible pavements, the design process involves two major steps as follows:

1. The design of the asphalt mix which involves the proportioning of the different ingredients (coarse and fine aggregates, mineral fillers, and asphalt cement) in the mix and the compaction effort.
2. The thickness design of the AC course and the other pavement layers (base and subbase), which involves the evaluation of the behavior of these layers under the anticipated traffic load and environmental conditions. Since the outcomes of the asphalt mix design process (step 1) affects the engineering properties of the mix, the thickness design of the pavement, and the pavement performance, the two steps (mix design and thickness design) and construction practices must be considered in a comprehensive way so that the desired pavement performance is assured.

Rut potentials of pavement can be minimized by taking balanced engineering steps during the material design (asphalt mix design), pavement design process and construction. These include (4):

1. Engineered asphalt mix design that can withstand the expected traffic loading without plastic yielding, resists the induced tensile stress without cracking, and resists low temperature cracking.

2. Balanced pavement design process that provides balanced and adequate layer stiffness and thickness to reduce the induced compressive stresses at top of the base, subbase layers, and roadbed soil, which cause plastic deformation (rutting) of these layers and to minimizes the tensile stress and strain induced at the bottom of the AC thereby increasing its fatigue life (see figure 2.1) (5).
3. Good construction practices that deliver adequate and uniform compaction of the various layers.

Existing information related to rutting is reviewed in succeeding paragraphs.

2.2 RUTTING

Rutting (permanent deformation) is a load related distress (caused by high axle loads and volumes) and is accelerated by material, environmental factors and by inadequate construction practices. Rut is the accumulation of plastic (permanent) deformation in the roadbed soil, in base and subbase layers due to compaction and/or consolidation under traffic loading. Rutting of asphalt layer may be caused by combination of volume change and shear deformation resulting from the application of traffic loads. In general, the rut potential can be minimized by using good quality materials, coupled with adequate pavement design and construction practices.

Rutting occurs as a combination of densification (volume change) and shear deformation. It has been observed by a number of investigators that the shear portion is more significant than the volumetric one. The major portion of permanent deformation occurs during initial loading (6).

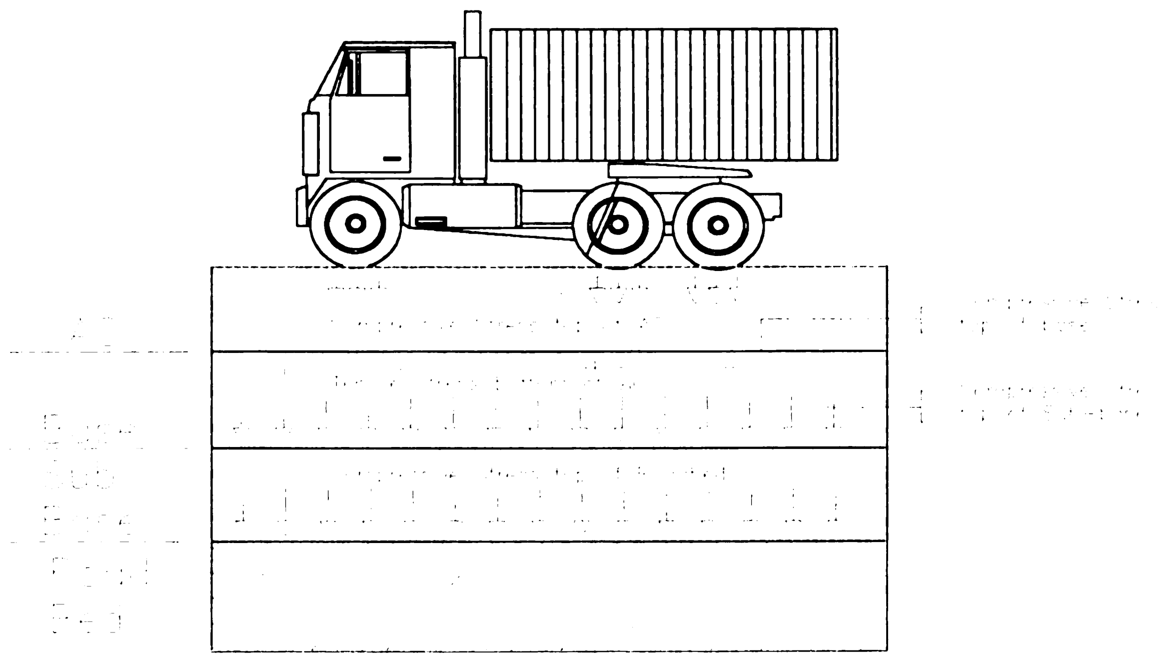

Figure 2.1: Illustration of critical stress/strain locations in a typical flexible pavement structure (4).

The initially soft material is being load-compacted and stiffened until the yield point is reached where additional plastic shearing accompanied by dilation (increase in volume due to the rearrangement of particles) occurs. The irreversible deformation during the entire process is strongly time dependent, because of the viscous nature of the bituminous binder. The factors which influence the rutting behavior include (6) :

- aggregate gradation, size, angularity and surface roughness;
- mixture composition;
- compaction procedure;
- temperature;
- load magnitude, distribution and frequency;
- thickness of the asphalt concrete layer;
- Air voids in the AC mix; and
- longitudinal grade of the pavement.

For all granular materials (and for that matter, for most other materials), the distinction between volumetric and deviatoric deformation is essential, once the linear threshold has been exceeded. For some stress levels, both volumetric and deviatoric deformation have their reversible and irreversible(permanent) portions. Each of these may or may not depend on the duration of the load and is modeled accordingly as either time-dependent or instantaneous . In addition, volumetric deformation can be such that volumetric stress performs positive or negative work. The latter is termed as dilation. The volumetric irreversible deformation due to volumetric stress occurs not only for high stress but also initially which is referred to as densification - an irreversible initial volume change. The major portion of the irreversible deformation is due to shear stresses in the uncompacted material, which suggests that better compaction of the asphalt layer should significantly improve rutting behavior. Also low asphalt content is shown to be beneficial (6).

For any flexible pavement section , its rut is typically determined by computing the mean rut depth (mean difference in elevation between the rutted wheel paths and the surrounding) of the pavement surface along the length of the project. Based on the mean rut depth, three severity levels can be defined as follows (2):

1. Low -Mean rut depth of .25 to .5 inch (6 to 12 mm).
2. Medium -Mean rut depth of .5 to 1.0 inch (12 to 25 mm).
3. High- Mean rut depth of more than 1.0 inch (25 mm).

To determine the mean rut depth, a straight edge of 4 feet long, should be laid across the rut and the maximum depth is measured. The mean depth should be computed from measurements taken at every 20 feet interval along the length of the rut (2).

2.2.1 Mechanism

As mentioned in foregoing paragraphs, pavement rutting is the sum of the rut in the AC, base and subbase layers and in the roadbed soil. Figure 2.2 shows the results of a study of the transverse profile of loops 4 and 6 of the AASHO Road Test (7). It can be seen that rutting had taken place in all pavement layers and in the roadbed soil. The contribution of each layer to the total pavement rut varies from one pavement to another. For example, the average rut in each layer as a percent of the total pavement rut of section 51 of the AASHO

Road Test, is shown in Table 2.1. In general, a narrow rut channel indicates that the primary rut is in the AC layer. The wider the rut channel, the deeper is the seat of rutting.

Table 2.2 provides a list of the relative percentages of the permanent deformation of section 51 of the AASHO Road Test that are attributable to distortion alone. It can be seen that, in general, permanent deformation is largely due to lateral distortion.

Based on laboratory data, Morris (3) concluded that densification and lateral distortion in compacted asphalt concrete specimens are largely a function of the test conditions. He added that, in the field, asphalt pavements are subjected to densification in the compression zone and to lateral distortion in the tension zone. Tests on asphalt concrete used at the Brampton Test Road (9), confirmed the above observations. It was noted that the rut potential in the tension zone in asphalt concrete pavements is higher than that in the compression zone because of distortion.

Rutting occurs by virtue of consolidation, compaction, or lateral distortion of the pavement due to repeated traffic loads. Several factors affect the magnitude of rut and its time rate of accumulation. These include (5):

1. Construction factors including inadequate compaction (either low compaction effort or compaction at lower temperatures than those specified).
2. Asphalt mix factors that include soft (low viscosity or high penetration) asphalt cement, high air voids, rounded aggregate, and excess sand in the mix.
3. Environmental factors that include high temperatures which soften the AC layers, and high moisture content or saturation of the lower layers (base and subbase) due to inadequate drainage.

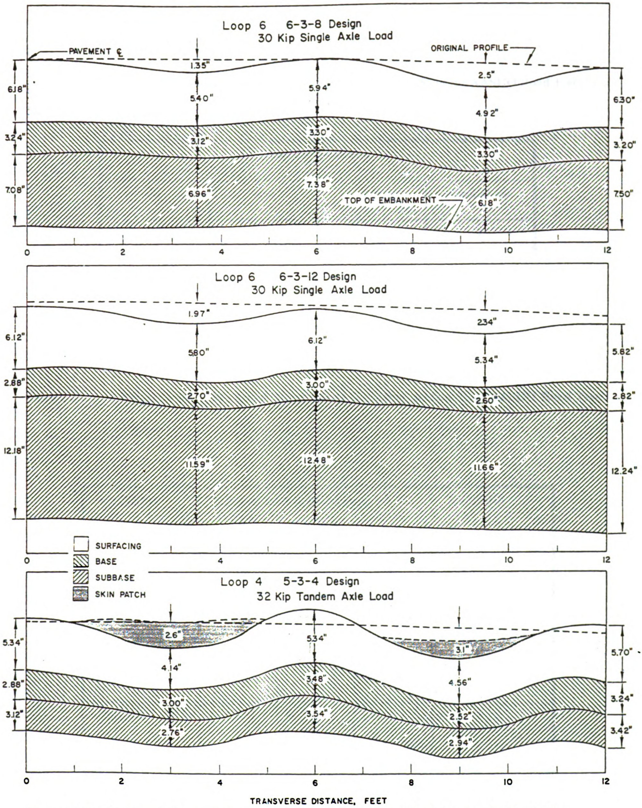

Figure 2.2 : Study of the transverse profile of loops 4 and 6 of the AASHO ROAD TEST (7).

Table 2.1 : Average rut (percent of total rut) in each layer of section 51 of the AASHO Road Test (7).

Percent of the total rut.			
AC	Base	Subbase	Roadbed soil
32	14	45	9

Table 2.2 : Percent rutting in various layers due to distortion, section 51 of AASHO Road Test, 1960, (7).

MATERIAL	% Rutting due to distortion during a given season			AVERAGE
	SPRING	SUMMER	FALL	
A/C SURFACE	82	82	76	80
BASE	0	70	NO MEASURE.	Figs.inconsistent
SUBBASE	97	96	NO MEASURE.	95.5

4. Tire factors such as studded tires and high tire pressures.

Available information related to these and other factors affecting pavement rutting is presented in the next subsections.

Analysis of data from field test sites in Alabama indicate that permanent deformation causing rutting is generally confined to the top 3 to 4 inches (surface and binder courses). There was little evidence that lower base/subbase courses or subgrade were significant contributors to rutting (10).

Permanent deformation is affected by the degree of compaction of the asphalt concrete mix. Two types of permanent deformation occur - consolidation and lateral distortion. Consolidation deformation, which is a relatively minor problem, comes about when traffic reduces the air void content of the mix from the value at the time of construction to a minimum air void content which generally occurs after several years of traffic. Permanent deformation which is due to lateral distortion is primarily a function of the properties of the mix. A tender mix, compacted to a low air void content will undergoes less permanent deformation than if it is compacted to higher air void contents. Lateral distortion can also follow traffic consolidation if the mix densifies down to a critical air void content (normally less than 3 percent) and in fact becomes over lubricated with asphalt cement. Environmental factors that allow the compaction equipment to obtain density with less compactive effort thus affect both types of permanent deformation (11).

Kang W. Lee and Mohammad A. Al-Dhalaan (12), concluded from road test carried out in Saudi Arabia that the rut damage was limited to the wearing course; that is, the subgrade and the base course are intact and the major cause of failure was the inadequacy of the specified mix for the loading and environmental conditions, for example, high temperature.

2.2.2 Factors Affecting Pavement Rutting

2.2.2.1 Tire Inflation and Tire Pavement Contact Pressures

In U S A as well as in Pakistan, asphalt surfaced pavements are experiencing premature rutting due to increased traffic volume, loads and/or increased truck tire pressure. Surveys in the States of Illinois and Texas indicate that the tire pressures have increased substantially over the last few decades. An average tire pressure of 96 psi with a maximum of 130 psi were recorded in the Illinois survey. The Texas survey showed an average tire pressure of 110 psi with a maximum pressure of 155 psi (13). Similar study revealed that the tire pressure in PAKISTAN ranges from 95 to 145 psi (15).

Typically, the rut potential of asphalt pavements has been evaluated on the basis of the magnitude of the compressive stresses induced at the top of the base layer and roadbed soil due to an 18-kip (80 KN) single axle load and a constant tire pressure (typically 85 psi). Experimental studies conducted by GoodYear tires and Rubber Company indicate that (16):

1. For a constant tire load, increasing the tire inflation pressure causes a shift in the point of maximum contact pressure to the center region of the contact area between the tire and the pavement surface.
2. Regardless of the tire load and tire pressure, the tire-pavement contact pressure is not uniform within the tire-pavement contact area. The distribution of the contact pressure is a function of the tire type and design. For example, contact pressures as high as twice the tire inflation pressure were measured for three tire types as shown in Table 2.3.

Smith and Bonquist (17) studied the influence of tire type and tire inflation pressure on pavement performance. They conducted full-scale pavement tests using the Federal

Highway Administration (FHWA) Accelerated Loading Facility (ALF) located at Turner Fairbank Highway Research station in Mclean, Virginia. The study examined the effects of tire pressure, tire load, pavement cross-section, and environmental conditions on pavement response (stresses and strains) and on pavement performance (rut potential and fatigue life). They made the following conclusions:

1. The effect of wheel load on pavement responses is greater than the effect of tire pressure. The measured pavement responses (stresses and strains) doubled for an increase in load from 9,400 pounds to 19,000 pounds, while increasing the tire inflation pressure from 76 psi to 140 psi resulted in a less than 10 percent increase in the measured response. This conclusion supports the results of mechanistic analysis of flexible pavement structures. For example, Baladi used MICHPAVE (linear/nonlinear finite element computer program) to analyze the stresses and strains induced in the pavement layers due to various wheel loads and tire inflation pressures. He reported that the effects of increasing tire pressure on the induced stresses and strains in the pavement are much smaller than those due to increasing wheel load.
2. The effects of tire pressure and/or wheel load on pavement rutting are much higher for thin pavement sections (less than 2-inch AC surface) than for thick sections (more than 4-inch thick AC surface).

Further, higher temperatures cause higher rut potential. Hence, the combinations of high tire pressure, high wheel load, high temperatures, and thin pavement sections are extremely damaging to flexible pavements. Although, the average truck tire inflation pressure has increased by about 20 psi over the last thirty-year period, the above findings suggest that this increase has an insignificant effect on pavement rutting (4).

Table 2.3 : Effect of tire type and tire inflation pressure on the maximum tire - pavement contact pressure, (16).

Tire type and load	Tire inflation pressure (TIP) (psi)	Max. contact pressure (MCP) (psi)	Ratio (MCP/TIP)
11 - 24.5	65	117	1.8
4250 lbs	75	114	1.5
	95	118	1.2
11r24.5	75	122	1.6
4250 lbs	95	152	1.6
	115	182	1.6
385/65R22.5	100	200	2
	120	226	1.9
	140	252	1.8

On the other hand, the reported increase in wheel load(e.g., the average wheel-load in Michigan and in Pakistan, has increased by 4000 and 8000 pounds respectively over the last twenty-year period) has significant effect on the rut potential (4 , 8).

Chen (14) employed the three dimensional Texas Grain Analysis (TEXGAP-3D) program (a 3-D linear finite element computer program) to study the effects of various wheel loads, nonuniform tire pressures, and nonuniform tire-pavement contact pressures on the induced stresses and strains and the resulting damage in asphalt concrete pavements. Results of the analysis were also compared with those of the ELSYM5 computer program which uses a circular contact area with uniform contact pressure distribution. Chen utilized a rutting model developed from the AASHO Road Test data to analyze pavement rutting. He made the following observations:

For a constant axle load of 4500 pounds, a 47 percent increase in the tire inflation pressure produces less than 2 percent increase in the compressive strain developed at the top of the subgrade as shown in Figure 2.3.

2. For a constant inflation pressure, a 20 percent increase in the axle load produces approximately a 20 percent increase in the subgrade compressive strain at the top of the subgrade as shown in Figure 2.4.

3. For the same tire inflation pressure, increase in axle load from 4500 pounds to 5400 pounds results in a 50 percent reduction in pavement life based on subgrade compressive strain as shown in Figure 2.5.

Based on these observations Chen et al. concluded that the axle load, not the inflation pressure has a major influence on subgrade rutting.

2.2.2.2 Consolidation and Field Compaction

Proper specifications and quality assurance regarding asphalt mixture composition and compaction decrease the rutting potential due to densification.

Barksdale(18) cited the work of Raithby, Epps and Acott, and reported that asphalt mixes placed in the field at a compaction level equal to the 50 blows Marshall compaction experienced gradual compaction and densification due to traffic loading (18).

Huges and Maupin(13) monitored the rut depth of asphalt pavements made by using different asphalt mixes (see Figure 2.6). They measured the rut depth right after construction, six months, and one year after construction. They reported that mix 1 with the highest initial voids in the total mix (VTM) has rutted more quickly than Mix 4 which had the lowest initial VTM. This supports the widely held belief that improving density during construction reduces rut depth.

Miller (20), based on results obtained from 24 test sites in Arkansas, stated that asphalt mixtures placed and compacted at air voids between 2.5 to 5 percent provide an acceptable level of ruts. He added that pavements with deep rutting have air void contents of less than 1.0 percent (high asphalt content). It should be noted that to prevent bleeding, the air voids in the asphalt mix should be higher than 2 percent (18).

Brown (22) stated that air void contents of more than 3 percent are needed to decrease the probability of premature rutting throughout the life of the pavement, whereas, air voids of less than 3 percent greatly increase the probability of premature rutting.

Huber and Heiman (8) reached a similar conclusion and suggested that asphalt mixes should be compacted above a threshold value of 4 percent air voids.

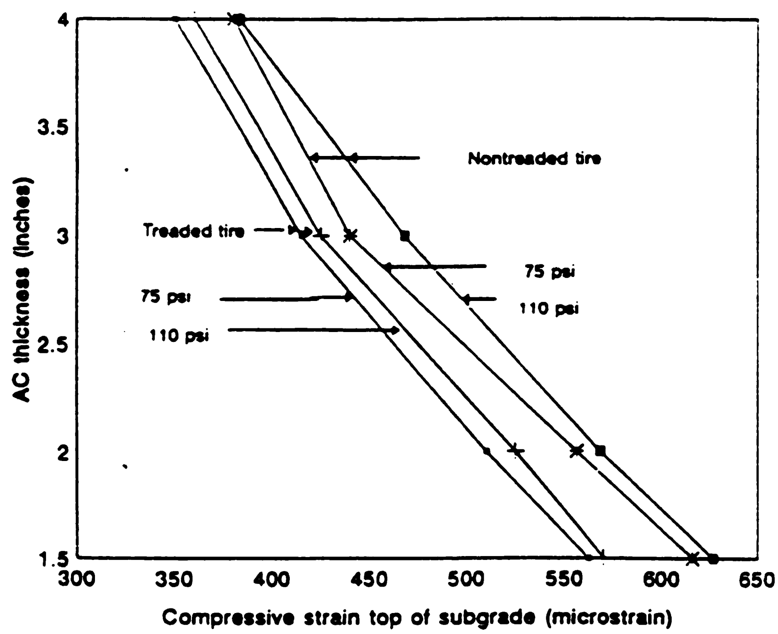

Figure 2.3 : Variation of compressive strain at the top of the subgrade for different thickness and tire pressure, (14).

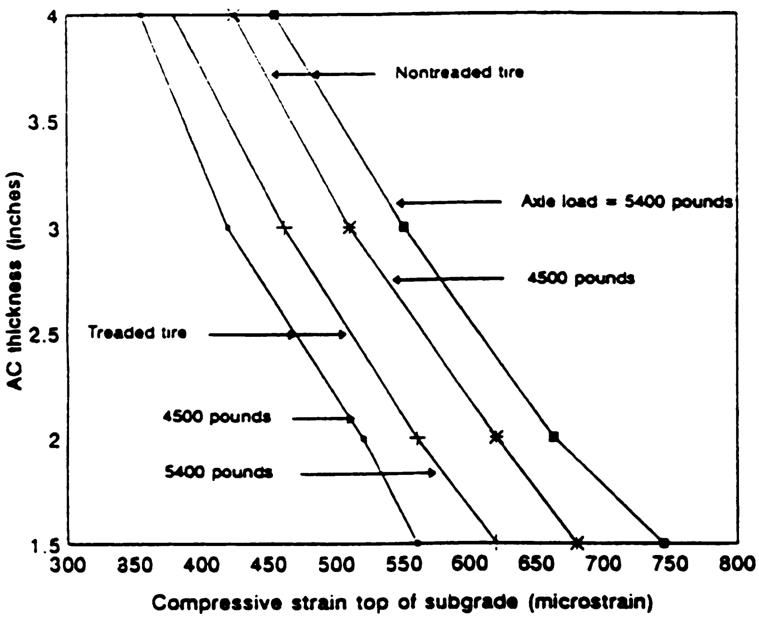

Figure 2.4 : **Variation of compressive strain at the top of the subgrade for different thickness and two level of loads (14).**

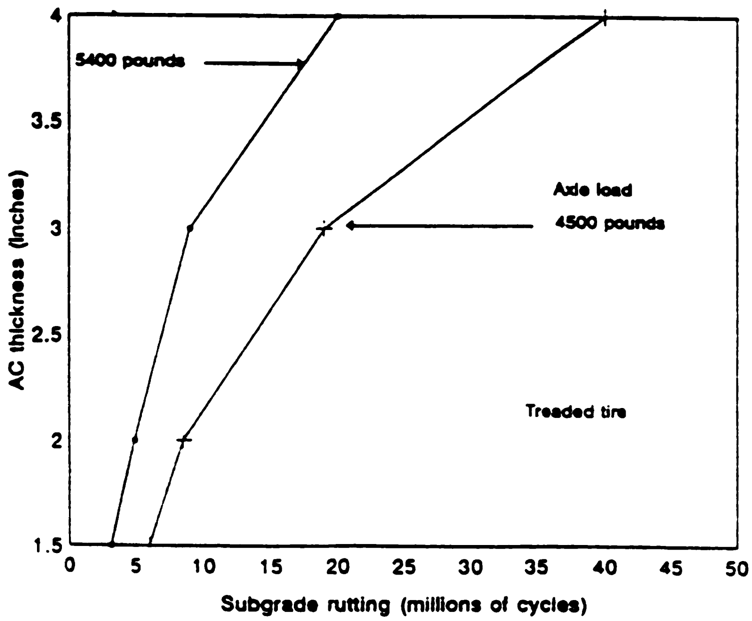

Figure 2.5 : **Pavement subgrade rutting damage life for different thickness and loads (14).**

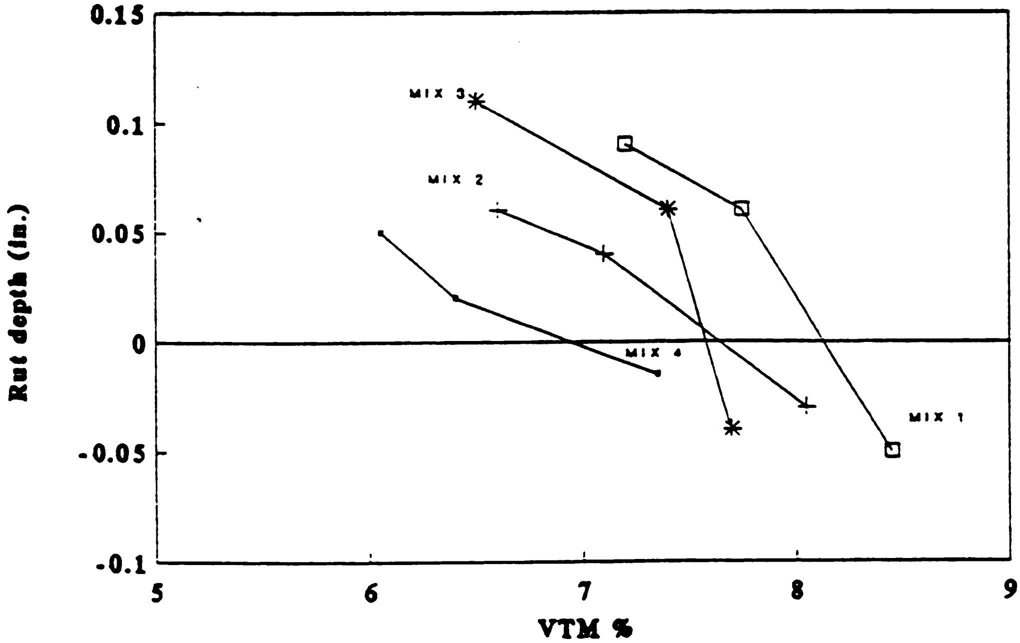

Figure 2.6 : Rutting as a function of the voids in the total mix (VTM) (9).

Lateral distortion can also follow traffic consolidation if the mix densifies down to a critical air void content (normally less than 3 percent) and in effect becomes overlublicated with asphalt cement (11).

A study carried out in Albama revealed that in properly designed mixes densification stabilizes at about 4% air voids and rut depth development ceases or decreases to very low rates as illustrated in figure 2.7. At about 4% air voids, the rut resistance in properly designed mixes is optimum. At this stage it is critical that the aggregate skeletal structure have the ability to resist further densification, and this is best accomplished with well graded aggregate with angular rough textured particles. Asphalt content is also critical as the mix reaches about 4% air voids. For pavements that experience severe rutting, densification continues and second phase conditions develop. When the air voids reach about 2%, the mix becomes very unstable and plastic flow develops, as illustrated in figure 2.8 (10).

2.2.2.3 Aggregate

The performance of asphalt mixes depends on providing adequate aggregate interlock for resisting and distributing the wheel load rather than on the shear strength of the asphalt cement. Therefore, the size, shape and angularity, and quality of the aggregates play an important role in the performance of the AC mix. Hence, every precaution should be taken to insure that a face to face aggregate contact and aggregate interlocking are provided for the high quality AC mixes. The maximum aggregate size, aggregate shape, and angularity, and the percent aggregate content in the mix influence the rut potential of asphalt pavements and Marshall stability and flow of the asphalt mix. Higher aggregate

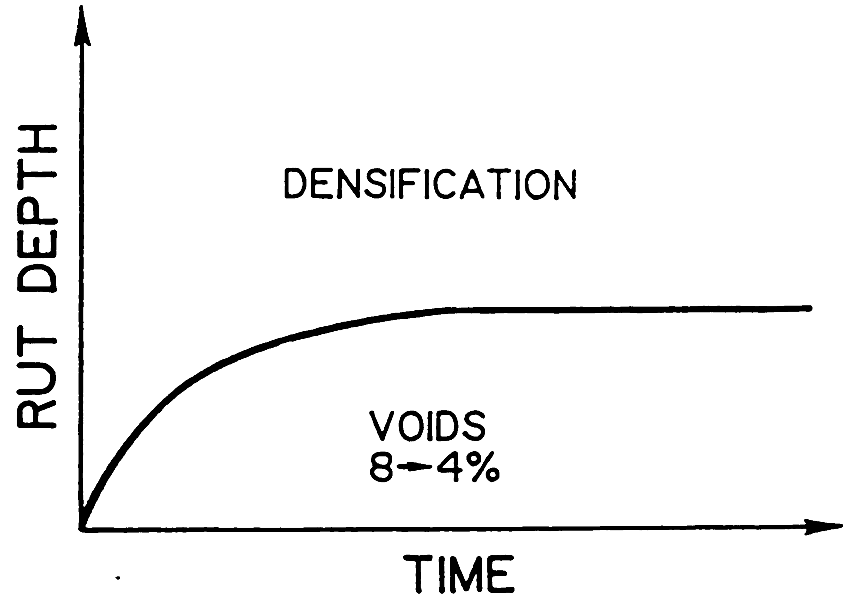

Figure 2.7 : Model for rut development in asphalt concrete. Pavement performing as designed (10).

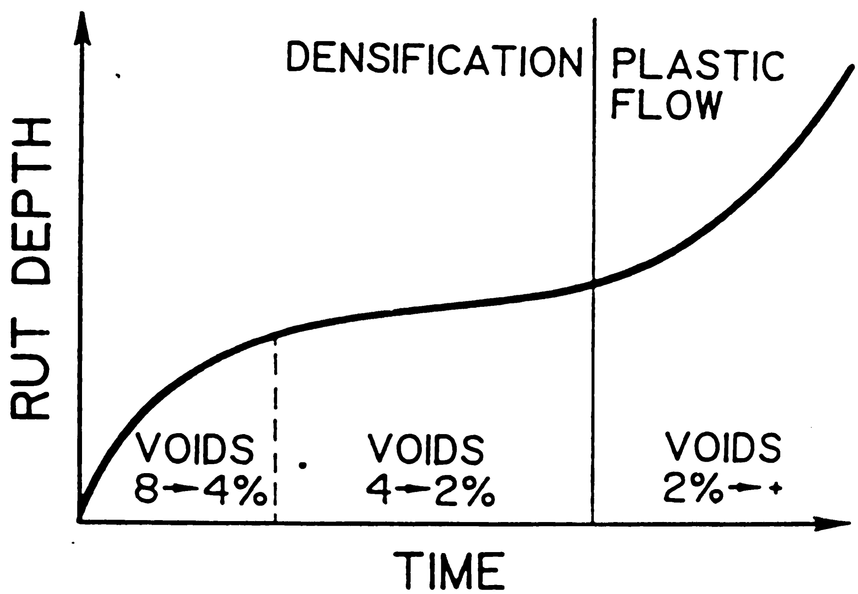

Figure 2.8 : Model for rut development in asphalt concrete. Pavement rutting excessively (10).

angularity produces higher Marshall stability, lower flow, and lower pavement rutting (20, 23, 25, 26). Figure 2.9 depicts the rut depth as a function of Marshall stability. Larger top size aggregate in the asphalt mix, on the other hand, produces higher stability, higher resilient modulus, higher compressive strength and lower rut potential (28, 29).

Herrin and Goetz (30) studied the effects of the percent aggregate in the mix and aggregate shape on the stability of the mix. They observed that as the percent aggregate in the mix decreases, its influence on the stability of the mix decreases. This may be explained by the fact that as the percent aggregate decreases and the percent sand increases, face to face aggregate contact is lost producing a ball bearing effect. That is higher percent sand contents cause the aggregate particles to float in the sand matrix thereby, reducing friction and stability.

Indeed the European Stone Mastic Asphalt (SMA) mix consists of 70 to 80 percent crushed aggregate. The SMA is well known for its low rutting potential. In addition, for a constant asphalt content, large size aggregates cause an increase in the density of the compacted asphalt mixtures (31). The increase in density is more pronounced when the aggregate top size is increased from 3/4 to 2-inch as it can be seen from Table 2.4. Further, increasing the aggregate top size in a mix causes an increase in the percent air void and in the percent void in the mineral aggregates (VMA) as shown in Figure 2.10 and 2.11 (31). Similarly, based on literature review conducted by Button at el. (32), he concluded that the rutting problem can be addressed by using large top size aggregates (1 to 1½-inch), increasing the specified percent void in mineral aggregate (14 to 15 percent minimum), replacing most or all the natural sand in the mix with manufactured angular particles, increasing the minimum allowable air void in the laboratory compacted mix to 4 percent and limiting the ratio by weight of mineral filler to bitumen to about 1.2.

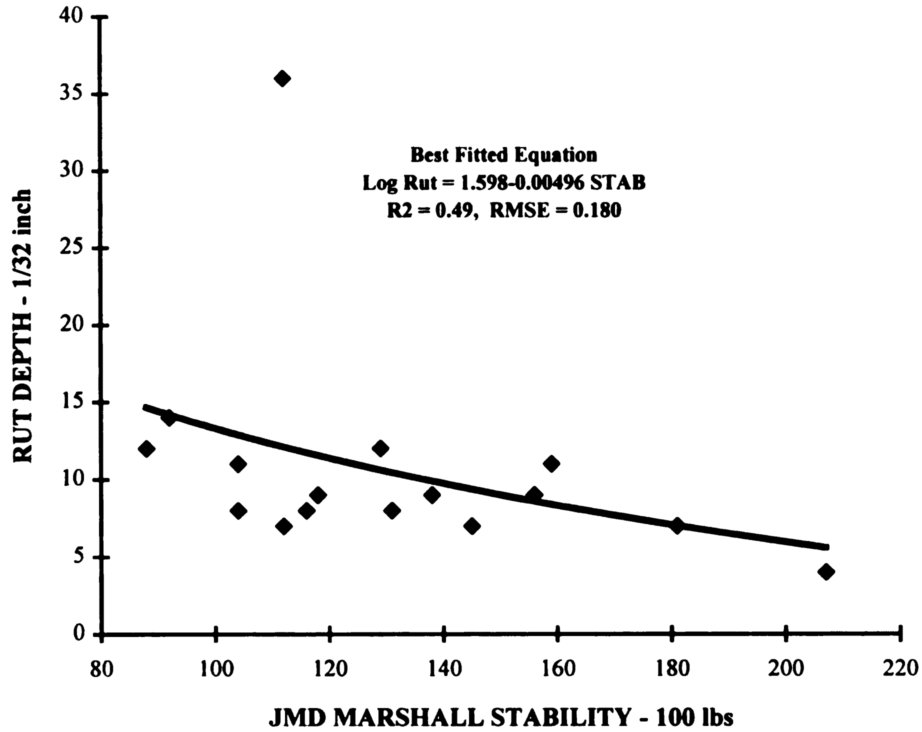

Figure 2.9: Relationship between rut depth and stability (20).

Epps and Monismith (21) reported that crushed (angular) aggregates produce AC mixtures with low rut potential. Typical requirement for crushed coarse aggregate are:

1. A minimum of 75 percent of particles with two crushed faces.
2. A minimum of 90 percent of particles with one crushed face.

The analysis also indicates that rutting varies geographically and that this variation can be explained by quality of locally available aggregate. Those areas with crushed stone and angular natural sands are less susceptible to rutting (10). Changes in aggregate properties will likely be most fruitful in improving rut resistance.

Button and Perdomo concluded that the round shape and smooth texture of natural (uncrushed) aggregate particles was one of the chief mixture deficiencies that contribute to rutting. Aggregate comprises over 90% of asphalt concrete mixtures and provides the basic load carrying skeletal structure. Well graded angular aggregate, proper selection of asphalt properties and asphalt contents can produce mixes that are not only rut resistant, but also durable and resistant to fatigue and thermal cracking (10). Coarse aggregate retained on the 4.75-mm (No.4) sieve should continue to be at least 85 percent of particles with two or more fractured faces for wearing and binder courses (32).

The Michigan Department of Transportation (MDOT) has developed a special provision for designing the asphalt paving mixtures for heavy duty pavements to minimize rutting. This special provision is applicable to the wearing, binder and base course mixtures used on the main line, ramps and cross-overs of all interstate highways , and other highways carrying more than 20,000 ADT or more than 1,000 daily Equivalent 18 Kip Single Axle Load (ESAL) applications.

Table 2.4 :Summary of mix densities (lbs/c.ft), (31).

Asphalt Content	Aggregate Top Size				
	¾"	1"	1½"	2"	2½"
3%	--	--	--	148.77	150.00
4%	143.71	145.50	148.19	151.30	151.38
5%	144.14	148.02	149.68	150.25	149.65
6%	142.96	145.88	148.98	--	--

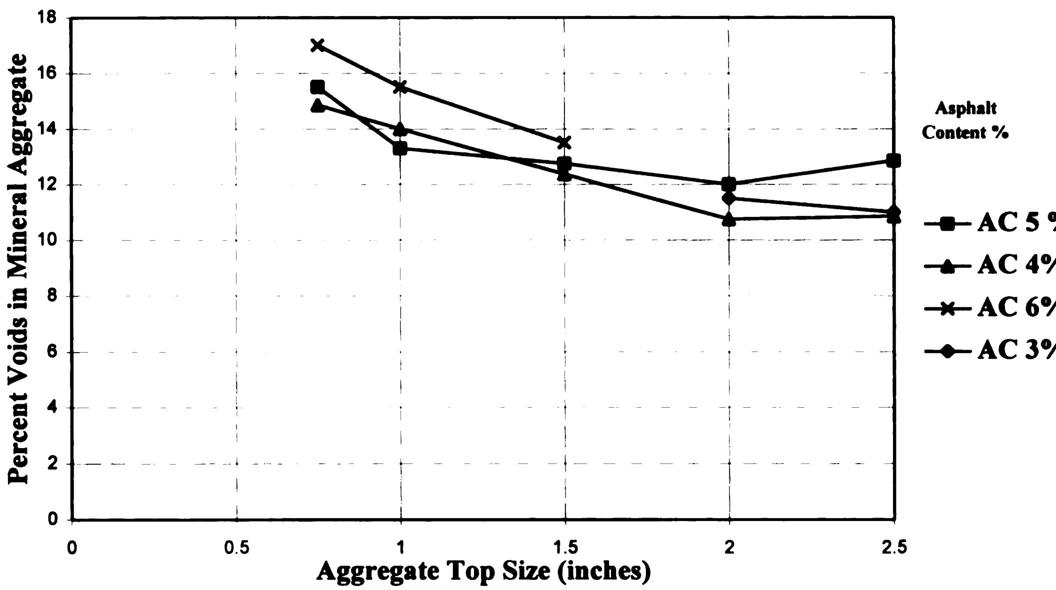

Figure 2.10: Effect of aggregate top size upon the percent voids in mineral aggregate, (31).

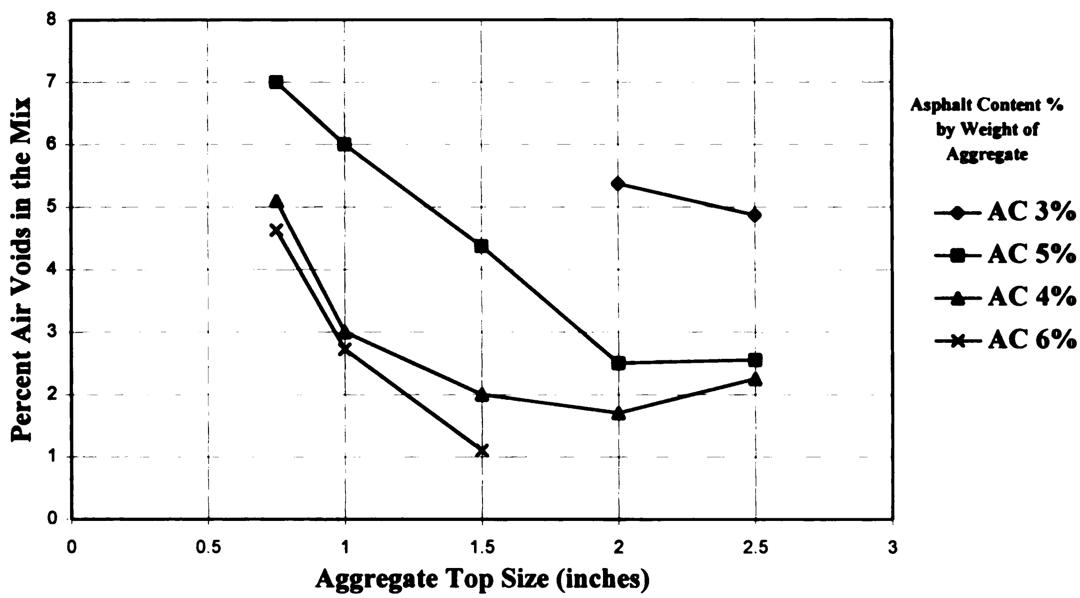

Figure 2.11: Effect of aggregate top size upon the percent air voids in the mix, (31).

The salient features contained in the special provision are:

- Use of AC-20 asphalt cement only;
- Use of larger size aggregate for binder and base course mixtures;
- More angular coarse aggregate. This is being increased to at least 85 percent two-face crushed particles;
- At least 75 percent manufactured sand in the fine aggregate;
- At least 4 percent minus No.200 fraction in the mixture to stiffen the asphalt.

The binder type does not appear to be as important as the gradation in minimizing early rutting. This conclusion is certainly related to the gradation used and possibly the type of aggregate (13).

2.2.2.4 Sand and Mineral filler

The angularity and shape of the sand particles, the percent sand content in the AC mix, and the type of mineral filler influence the rut and fatigue life performance of asphalt pavements. The Federal Highway Administration (FHWA) recommends that natural sand (rounded sand particles) be limited to 15 to 20 percent of the total weight of aggregates for high volume roads and to 20 to 25 percent for medium and low volume roads (33).

Similarly, Button et al. (34) suggested that for high traffic volume roads limiting the natural (uncrushed) sand particle content of the asphalt mixes to about 10 to 15 percent reduces rut potentials.

Young (35) cited Barksdale and Hicks, and reported a significant increase in pavement rutting due to an increase in the percent sand contents. It appears that mixes with less than 20 percent natural sand in the fine aggregate contained fewer fair to poor pavements than mixes with more than 20 percent natural sand in the fine aggregate (32).

The significant effect of a reduction in fines content of only 3.0 percent from the prescribed gradation is surprising. On the other hand, it is logical when one considers that this essentially cuts in half the volume of fines in the mixture, reducing the amount of cementing material in the mix, and changing the asphalt film thickness at the points of aggregate contact. This indicates that variations of fines content, even within typical gradation specifications, can have serious effects on mix performance (11).

2.2.2.5 Asphalt Type and Content

The type of the asphalt cement and its content in an asphalt mix impact the rut of the asphalt mixes. Typically, the type of asphalt cement used in an asphalt mix depends on the environmental conditions. Higher temperature regions require harder (higher viscosity or lower penetration) asphalts. On the contrary, softer asphalt's are used in the construction of asphalt pavements in cold regions. The asphalt cement hardness or viscosity affects three types of distress.

1. **Fatigue life** which is affected by the AC viscosity at in service temperatures.
2. **Rutting-** Softer asphalt tends to cause higher rutting potential. Hence rut is affected by the AC viscosity at high temperatures.
3. **Cold temperature cracking**: Stiff asphalt cements tend to harden at lower temperatures. Thus, cold temperature cracking is affected by the AC viscosity at low temperatures.

The effects of the asphalt binder on the various distress modes vary. For example, a study conducted by the National Research council, Strategic Highway Research Program (SHRP), found that (4):

1. The binder contribution to the rut potential of the asphalt course is about 40 percent as shown in Figure 2.12. The other 60 percent is attributed to the other ingredients in the mix (aggregate, sand and mineral filler) and their proportions and to construction practices.
2. The 40 percent is related to the pavement layer thickness and properties. Finally, the asphalt binder contribution to the low temperature cracking potential is about 80 percent.

Traditionally, asphalt binders are classified and specified according to their viscosities at a given temperature. Two AC binders of the same grade (e.g., AC-20) may have the same viscosity at the specified temperature. However, their viscosities at higher or lower temperatures may differ substantially and so their behaviors. This fact has been recognized in the new SHRP binder specifications which consider three types of distresses: permanent deformation, thermal cracking and fatigue cracking . These distresses are related to the rheological properties of the binder at high, low, and intermediate temperatures, respectively (36).

Huber and Heiman (8) stated that penetration and viscosity of the AC binder do not possess a significant effect on rutting rate because an asphalt grade typically produces a

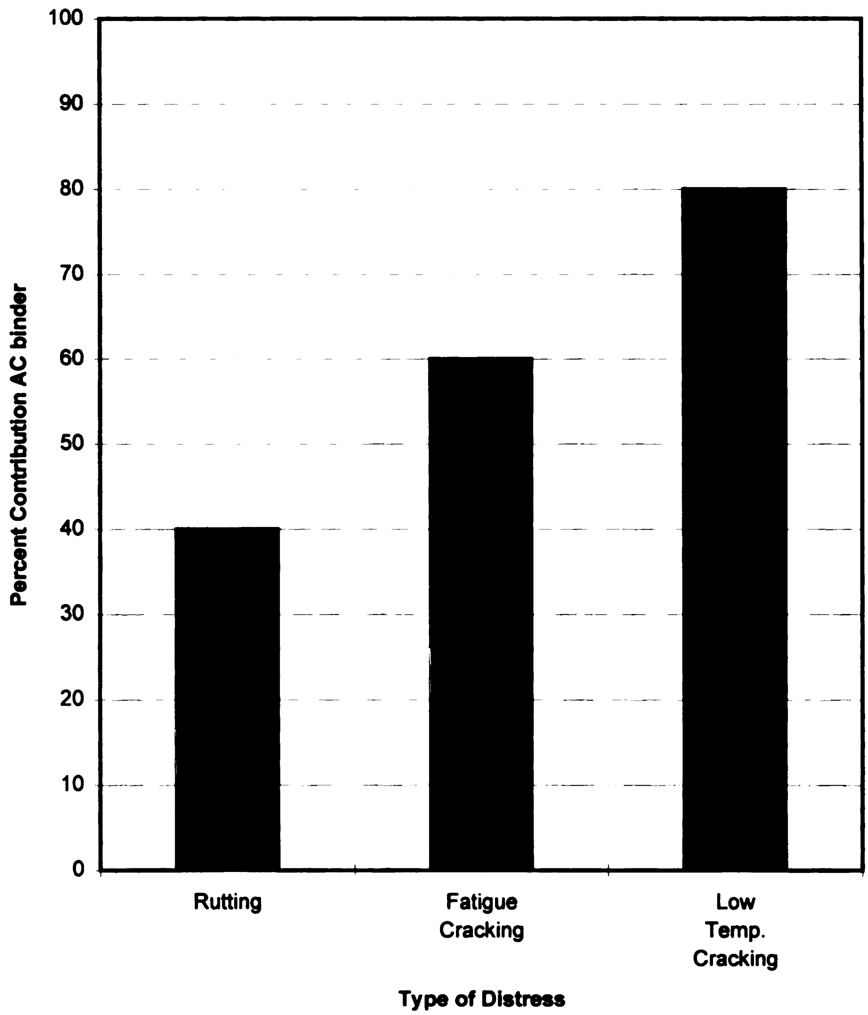

Figure 2.12: Binder contribution to difference distress (4).

similar penetration and viscosity after mixing. They attributed the rutting performance to air voids, voids filled with asphalt and asphalt content.

Huges and Maupin (13) on the other hand, stated that binder type is not as important as aggregate gradation in minimizing early pavement rutting.

Kruts (37) evaluated the impact of moisture on rutting potential of polymer modified mixtures. He reported that moisture conditioning of laboratory specimens yielded fairly consistent permanent deformation results for all mixtures regardless of the grade of the asphalt cement binder. An excessive amount of asphalt or a very soft grade asphalt can often produce a mix with high Marshall flow values. A maximum Marshall flow of 16 is often specified for mix design and construction control. Brown and Cross (38) reported that Marshall flow is a good indicator of rutting potential. Higher amount of rutting is associated with mixes having Marshall flow values above 10.

Mixes with large aggregates have less asphalt content requirement to coat the aggregate. Case studies of rutted pavements in Oklahoma conducted by Hensley and Leahy (39) showed that missing intermediate fines in stone-filled mixtures make them less sensitive to asphalt content. This is due to the fact that intermediate fines increase the surface area to be coated with asphalt cement and increase their ball bearings effects. Thus, their absence not only reduces the required asphalt content but also reduces the sensitivity of the mix to increased asphalt contents.

Decker and Goodrich (40) ranked five asphalt cement factors relative to their effects on asphalt pavement distresses. They used a ranking scale from 0 to 5. A ranking of 0 indicates no effect whereas 5 indicates significant effect. Since, asphalt cement properties are mainly governed by the source of the crude, they found that the refining process has a very minor effect on rutting and fatigue cracking. Decker and Goodrich studied and ranked the physical properties (such as stiffness, temperature susceptibility) and rheology of the AC binders (see Table 2.5). They concluded that increase in the viscosity of the AC binder causes an increase in the stiffness of the AC mix which in turn improves the rut resistance.

For asphalt pavements with high rutting in the top courses of the asphalt concrete, it was observed that these courses often contained fine aggregate gradations and high asphalt contents. One of the biggest causes of rutting is excessive asphalt content in asphalt mixtures. Steps should be taken to ensure that the proper asphalt content is selected and provided during mix production. The asphalt content should be selected to provide void contents of 4 to 5 percent in laboratory compacted mixtures for high traffic volume roads. Asphalt content should not arbitrarily be increased to facilitate compaction, to minimize segregation, or for any other reason except to provide satisfactory voids in the laboratory compacted asphalt mixture (38).

The study in Alabama revealed that the asphalt content is also critical as the mix reaches about 4% voids structures. Excessive asphalt will decrease inter granular contacts, weakening the aggregate skeletal structure and leading to further densification. Excess asphalt can weaken otherwise very stable aggregate structures (10).

According to Barksdale, rutting was found to be directly related to asphalt content, it was not extremely sensitive to material type, gradation and level of compaction used in the Marshal Mix Design Method. For the relative low asphalt content of 4.5 to 4.8 percent presently used in base course mixes in Geogria,. An increase 1/4 to 1/2 percent asphalt should result in rut depths of about 0.20 to 0.25-in (18).

Table 2.5: Effect of asphalt cement factors on pavement distress (40)

ASPHALT FACTORS	PAVEMENT PERFORMANCE			
	THERMAL CRACKING	FATIGUE CRACKING	RUTTING	MOISTURE DAMAGE
REFINING TECHNIQUE	1	1	1	0
PHYSICAL PROPERTIES	4	3	2	1.5
CHEMICAL PROPERTIES	2	2	2	2
AGING	1	3	1	1.5
MODIFICATION	2.5	2.5	2.5	2.5

Based on the observations of Dhahran - Abqaiq Road in Suadi Arabia, it was concluded that one of the reasons for rutting at the Abqaiq section was overasphalting (4.61% asphalt content in the field) which also indicates that the Marshal stability test might not be appropriate for the measurement of properties of asphalt mixtures, especially for the crushed limestone. Some of the potential reasons for the rutting were over asphalting, fine-graded mix, and poor quality control (12).

2.2.2.6 Environmental Factors

Exposure to environments causes the bituminous material to harden over time. With passage of time the bituminous binder becomes so brittle that it can no longer sustain the strains, affiliated with daily temperature changes and traffic loads. The rate of hardening is a function of oxidation resistance of binder, the air voids content, temperature, and thickness of the asphalt film. Therefore, the rate of hardening varies with the binder type, climate and material design. It should be noted that most of the asphalt hardening takes place during mixing, agitating, transporting, and construction.

Asphalt durability can be defined as its resistance to change in its original properties during mixing, construction, and service life. Durability of the asphalt is not a factor that effects rutting directly. In fact the embrittlement of the asphalt due to aging can cause increase in both the cohesion and the stiffness of the binder, which result in a greater resistance to permanent deformation under loading. Asphalt hardening on the other hand, adversely affects its fatigue life and low temperature cracking. Further, aging of the binder is not sufficient to deter rutting in an under designed pavement structure. Cases have been reported where rutting has occurred suddenly in older pavements which were exposed to higher tire pressure and heavier axle loading than expected (38).

Huges and Maupin (10) reported that early pavement rutting is a function of the temperature of the pavements when it is opened to traffic. They suggested pavement temperatures of less than 150 °F lead to a stable asphalt mix under traffic.

2.2.2.7 Ranking of Factors

A comprehensive study of the effects of asphalt mixture variables on the performance of flexible pavements and on the engineering properties of the mixtures was sponsored by the National Cooperative Highway Research Program (NCHRP). The title of the project is Asphalt Aggregate Mixture Analysis System (AAMAS). The asphalt mix design factors included in the study are: compaction, resilient and creep modulus, indirect tensile strength and strain at failure and the compressive strength. Each factor was ranked on a scale from 0 to 5. A ranking of 0 indicates that the factor has no effect on the pavement performance. A ranking of 5 indicates significant effects (11).

Two static creep and repeated load tests were used in the AAMAS study, the uniaxial compression and the indirect tensile. The values of the resilient modulus obtained from the test were used to evaluate the rut potential of the compacted asphalt mixes.

Table 2.6 provides a list of the ranking of the various test outputs. It can be seen that:

1. The creep modulus of the AC mixture has significant effect on the rut and thermal cracking potentials, a moderate effect on fatigue cracking, and a minor effect on moisture damage potential.
2. The compressive strength has a moderate effect on rut potential, a minor effect on moisture damage, and no effects on the fatigue and thermal cracking potentials of the mixes.

Other mix factors such as aggregate particles orientation, which is affected by the compaction method (the gyratory compaction technique was recommended by the AAMAS study), the slope and intercept of the creep-time curve (also known as the alpha and gnu parameters), the tensile stress to tensile strength ratio, and the compressive stress to compressive strength ratio were also included in the AAMAS study. Their respective ratings are listed in Table 2.6.

Polymer modifications resulted in improved rutting resistance in wheel tracking tests, carried out by Collins, Bouldin and Berker. They reported that, with increasing polymer content, the systems will be more resistant to deformation and exhibit enhanced elastic recoil. For this specific mix valkerting, et al found that already at polymer concentrations as low as 2 % by wt, the rutting resistance was increased by a factor of approximately 2. The blend containing 6 % by wt leads even to a 7.5 fold increase (41).

2.2.2.8 Rut Prediction Models

The proper pavement design and pavement management systems must be capable of predicting the performance, and the remaining service life of the pavement structures. Rut is one type of distress that must be predicted during the pavement design and the pavement evaluation processes. In several countries, an overlay is applied when the rut depth of the pavement is of the order of 20 to 30 mm. In the U.S.A, the maximum acceptable rut varies from one State Highway Agency (SHA) to another and it depends on the pavement class (interstate versus farm to market roads). Current rut prediction models can be divided, in general, into two groups: mechanistic and mechanistic/empirical. The mechanistic models are based either on the theory of elasticity, theory of plasticity, or the viscoelastic theory.

Barksdale (42, 43), Heukelon and Klomp (44), Romain (45) and others have suggested that layered elastic theory can be used to calculate the induced stresses and strains in the pavement due to traffic. The plastic strain can be assumed to be proportional to the elastic strain and the number of load repetitions.

Table 2.6 : Summary of Ranking of Mix Properties Related to Pavement Performance (11).

FACTORS	DISTRESS MANIFESTATIONS			
Fundamental Engineering Properties	**Permanent Deformation**	**Thermal Cracking**	**Fatigue Cracking**	**Moisture Damage**
Resilient Modulus	3	3	5	1
Creep Modulus	5	5	3	1
Tensile Strain at Failure	1	4	5	1
Indirect Tensile Strength	3	0	0	1
Other Properties & Factors				
Particle Orientation	3	3	3	0
Alpha and Gnu	5	1	2	0
Tensile Strength Ratio	2	0	3	5
Resilient Modulus Ratio	3	0	2	4
Compressive Strength Ratio	2	0	0	2

In this type of analysis, the relationship between the plastic strain and the applied stress for each pavement component can be obtained from laboratory repeated load test along with either linear or nonlinear elastic theory. Nonlinear theory should provide more accurate results, but its use has been limited because of its complex nature. Rut prediction cannot be made directly from plastic stress-strain relationship although, it provides a considerable insight regarding material behavior.

Barksdale and Leonards (46) suggested that rutting can be estimated by assuming that the pavement can be represented as a viscoelastic layered system. The plastic strain can then be predicted by using the material characteristics obtained in the laboratory from the creep tests. Elliot and Moavenzadeh (47) suggested the use of the linear rather than the nonlinear viscoelastic theory due to its mathematical complexities for predicting structural response.

Morris (3) cited Kenis and a study conducted at Washington State University Test Track and reported that the rut prediction obtained by using linear viscoelastic theory were substantially lower than the actual measured rut depth. For the test track, after 247,000 load repetitions, the linear viscoelastic theory predicted rut depth in the order of 5 to 10 percent of the actual measured ones.

The complexities involved with the theoretical prediction models tempted the researchers to develop mechanistic/empirical models to predict rutting. Some of these models are presented below.

Leahy and Witczak (48) assessed the influence of repeated triaxial test conditions and mix parameters on the permanent deformation characteristics of asphalt concrete and presented a statistical model of the form:

$$\log_{10}\varepsilon_p = 15.83+7.132[\log_{10}(T)]+1.105[\log_{10}(S)]0.118[\log_{10}(V)]+2.155[\log_{10}(EAC)] + 1.117[\log_{10}(VOL)]+[0.986(TTEMP^{-0.102}\ VMA^{-0.158})\log_{10}(LN)] \qquad (2.1)$$

$$R^2 = 0.82$$

Where;

ε_p	=	permanent deformation;
T	=	test temperature (°F);
S	=	deviator stress (lb/in^2);
V	=	viscosity at 70° F (10^6 poise);
EAC	=	effective asphalt content (percent by volume);
VOL	=	percent air void;
TTEMP	=	test temperature (°F);
VMA	=	percent void in mineral aggregates; and
LN	=	load repetition.

A sensitivity analysis of the above equation showed that the temperature is the most significant variable. The model is less sensitive to loading conditions, material type and mix parameters(48).

Based on laboratory test data Morris et al. (9), presented the following regression equation, to predict pavement rutting:

$$\varepsilon_p = f\,(s_1, s_2, T, N) \pm E \qquad (2.2)$$

Where

ε_p	=	amount of permanent strain;
s_1	=	vertical stress;
s_2	=	horizontal stress;
T	=	temperature;
N	=	number of load applications; and
E	=	The estimate of error associated with any attempt to predict ε_p as a function of the other four factors.

The laboratory developed model was used to predict the observed rutting at the Barmpton Road Test. Although the model does not account for the effect of the AC thickness, the binder type, and the percent air voids, the overall predicted and observed values were comparable as shown in Figure 2.13.

Baladi (49) correlated AC mix and other pavement layers properties to predict the rut depth in the AC layer. He presented an equation of the form:

$$
\begin{aligned}
LOG(RD) = {} & -1.6 + 0.067(AV) - 1.4[\log(TAC)] + 0.07(AAT) \\
& - 0.000434(KV) + 0.15[\log(ESAL)] - 0.4[\log(MR_{RB})] \\
& - 0.50[\log(MR_B)] + 0.1[\log(SD)] + 0.01[\log(CS)] \\
& - 0.7[\log(TB_{EQ})] + 0.09[\log\{50-(TAC + TB_{EQ})\}] \qquad (2.3)
\end{aligned}
$$

where

LOG	=	natural log;
RD	=	rut depth (inch);
AV	=	the percent air void in the mix;
TAC	=	thickness of AC layer (inch);
AAT	=	average annual temperature (°F);
KV	=	kinematic viscosity at 275 °F (AASHTO T-201) (centistrokes);
ESAL	=	the number of 18-Kip ESALs at which the rut depth is being calculated;
MR_{RB}	=	resilient modulus of roadbed soil (psi);
MR_B	=	resilient modulus of the base material (psi);
SD	=	pavement surface deflection (inch);
CS	=	compressive strain at the bottom of the AC layer; and

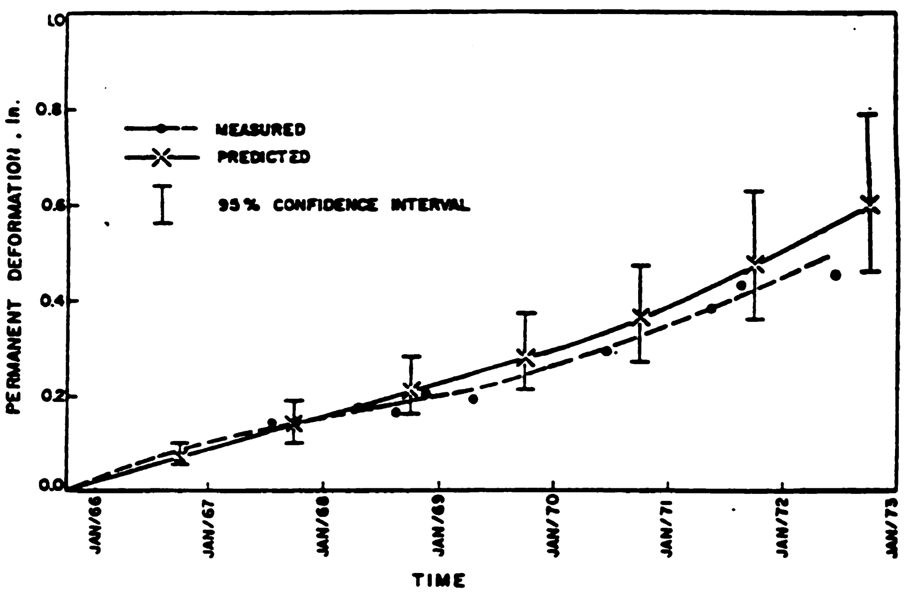

Figure 2.13 : Permanent Deformation as a Function of Time (Brampton Road Test Section 3) (9)

TB_{EQ} = equivalent thickness of base material (inch), which is the actual thickness of the base layer plus the equivalent thickness of the subbase layer; equivalent thickness of the subbase layer is equal to the actual thickness of the subbase layer reduced by the ratio of the modulus of the subbase to that of the base material.

Baladi, concluded that air voids is the most important factor affecting rut and higher air voids lead to higher rut potential.

Based on a study of 19 pavement sites conducted at Michigan State University, Mukhtar (4) developed an empirical rut model to predict pavement rutting. The model is of following form :

$$RUT = ESAL\,[-.3656 + .00068\,\{(FINE)^3 + SAND\} + .00012\,\{EXP(AV)\} + .0289\,(AV^2) - .0037\,(AV^3) + .00314\,(AC^3) - .6614\,(LOG(ANG))] \qquad (2.4)$$

where

Rut = rut depth (inch);

ESAL = 18-kip cumulative ESAL during the service life;

AC = asphalt content (percent by weight);

SAND = sand in AC mix (percent by weight);

FINE = fine in AC mix (percent by weight); and

ANG = angularity of coarse aggregate (plus #4) in AC mix.

Basing on the sensitivity analysis of the model Mukhtar concluded :

1. There is no appreciable increase in rut depth as the percent fine content increases from 0 to about 5 percent. Further increase in the percent fine content causes substantial increase in pavement rutting.
2. Increasing the percent air voids from 3 to 6 percent causes an insignificant increase in the rut depth. Increasing the percent air voids above the 6 percent level causes a substantial increase in rut.
3. Increasing asphalt content from 4 to 8 percent causes an increase in the rut depth.
4. Increasing aggregate angularity from 1 (rounded aggregate such as river gravel) to 5 (aggregate crushed on all sides) causes a substantial decrease in the rut depth.

Miller and Ford (20) carried out regression analysis to determine the most significant relationships between pavement ruts and mixture of physical measurements. The pavement mixture characteristics found to have an appreciable coefficient of correlation with rutting or the logarithm of rutting include, respectively, air voids, 0.674; voids filled, 0.658; and resilient modulus, 0.602. The effect of the amount and type of asphalt and the aggregate character may be reflected in the above factors. Stepwise linear regression analysis was conducted to determine the best-fitted equation for each dependent variable and its relationship to other mix characteristics. One best-fitted equation that illustrates the relationship between rutting and the mixture properties is the following:

$$RUT = -73.8 + 0.937\,(VF) + 0.582\,(D40) + 2.33\,(BAV) - 0.0236\,(STAB) \quad (2.5)$$

where

RUT = rut depth, 1/32 in.;

VF = voids filled (percent);

D40 = hump in grading curve at #40 sieve (percent). The value is determined on a plot of the gradation on 0.45 power paper. A line is extended from origin to #4 sieve data point. The difference in % between the straight line and the gradation at #40 is hump;

BAV = air voids between wheel path (percent);

STAB = Marshal stability (psi).

The coefficient of determination (R^2) value is 0.495 for this equation with a high confidence level. This indicates that only about 50 percent of the rutting is explained by this equation. Additional factors to be considered in design that affect rutting include traffic speed and character, environmental conditions, longitudinal grade and support from the underlying pavement structure and subgrade. These factors were not part of this analysis. Their evaluation would increase the understanding of the rutting problem. The results of this study indicated that the mixture air voids of 2.5 to 5 percent will provide asphalt mixtures that have an acceptable level of rutting. Deep ruts are associated with pavements having air voids of less than 1.0 percent and with the traffic that is slow moving and channeled (20).

Thompson and Nauman (50) developed a model to predict permanent strain as a function of load repetitions. The model is expressed as:

$$\text{Log}\ {}^{E}{}_{P} = a + b\ \text{Log}\ N \tag{2.6}$$

or: $\quad {}^{E}{}_{P} = AN^{b}$

where: ${}^{E}{}_{P}$ = permanent strain;

a & b = experimentally determined factors; and

A = antilog of “a”.

Ohio State University (OSU) researchers have proposed a permanent strain accumulation prediction model for use in a pavement design system developed for the ohio DOT. The OSU permanent strain accumulation model is:

$$^{E}{}_{p}/N = An^{m} \tag{2.7}$$

where

$^{E}{}_{P}$ = plastic strain at N number of load repetitions;

N = number of repeated load applications;

A = experimental constant dependent on material and state of stress conditions; and

m = experimental constant depending on material type;

Note: If the “b” term from the log strain - log N model is known, m is equal to b-1 (50).

James Mark Matthews and B.B. Pandey, (51) developed a regression equation basing on the performance of flexible pavements:

$$RD = -0.265 + 6.79\ (SVS) + 3.08\ (ESAL) \tag{2.8}$$

where

RD = average rut depth (mm),

SVS = subgrade vertical strain (x 10^{-3}) obtained from the computer output (RAOPAVE), and

ESAL = (x 10^{7}).

Equation 2.8 has a squared multiple regression coefficient of 0.951, a calculated F value of 116.0, and a critical F value 6.93 at 0.01 significance level. Because the calculated F value is greater than the critical F value, the null hypothesis is rejected, leaving the linear terms in the model. It would be useful, and seems a natural extension, to relate rut depth to gravel surfacing depth and cumulative ESALs. This relationship is shown by the following equation:

$$RD = 3.878 \text{ (ESAL)} - 0.0459 \text{ (TWBM)} \qquad (2.9)$$

where

TWBM equals the thickness of WBM, in millimeters.

Equation 2.9 has a squared multiple regression coefficient of 0.727, a calculated F value of 32.0, and a critical F value of 6.93 at 0.01 significance level.

For granular pavements, the loss of serviceability is caused mostly by excessive permanent deformation along the wheel-path. Hence, in this study rutting was adopted as the index for judging pavement performance, which is in agreement with British practice. An average rut depth of 15 mm is considered adequate for an overlay done at the optimal time. Putting this value of rut depth in Equation 2.8, the following rutting model was developed for the traffic range covered in this study:

$$SVS = 4.93 * 10^{5} \text{ (ESAL}^{-1.22}\text{)} \qquad (2.10)$$

where SVS equals the subgrade vertical strain in 0.001 mm/mm.

Equation 2.10 was compared with that of Shell

$$SVS = 2.8 * 10^{-2} \text{ (ESAL}^{-0.25}\text{)} \qquad (2.11)$$

The difference in the coefficients of these equations is caused by the difference in the material characteristics. Although the difference of the corresponding coefficients in the two equation is considerable, the curves are close to one another when plotted. For an ESAL value of 30 million (a typical value - the ESAL range in this study is 8 to 45 million), both equations yield a subgrade vertical strain of 0.00038 mm/mm. An average rut depth of 20 mm is considered adequate for the complete failure of a pavement structure. For this rut depth, both equations gave the same subgrade vertical strain (0.000348 mm/mm), for an ESAL value of 42 million (51).

CHAPTER 3

RESEARCH PLAN

3.1 GENERAL

Over the last two decades, the number of heavy multi-axle trailers in Pakistan has risen many fold. This resulted in premature manifestation of rutting and its rapid development of high severity levels.

Pakistan today, has a prematurely and severely deteriorated road network of AC pavements. These present loss of precious infra-structure worth billions of rupees. If this problem (rutting) is continued to be neglected, then new AC pavement projects will also crumble prematurely. The rehabilitation costs will be a formidable obstacle to the socio-economic development of the country.

The resistance of the AC mixes to rutting is a function of the AC mix properties (percent coarse and fine aggregate, angularity of the aggregate, and the asphalt binder content and viscosity, air voids), the pavement design process, construction practices and environmental factors. Recall that (see Chapter 2) one of the SHRP studies concluded that forty percent of rutting, sixty percent of fatigue cracking and eighty percent of thermal cracking potential may be attributable to the asphalt binder alone whereas, the remaining percentages of the respective distresses can be attributed to the other properties of the AC mix and to the overall design of the pavements.

Analysis of data from field test sites in Alabama indicates that permanent deformation causing rutting is generally confined to the top 3 to 4 inches (surface and binder courses). There was little evidence that the lower base/subbase layers or the subgrade were significant contributors to rutting (10). This suggests that rut resistance of AC mixes can be enhanced by changing the properties of the AC mixes to withstand the present day traffic conditions and to perform satisfactorily over the design life. Research studies need to be conducted to establish the AC mix factors that enhance the resistance against rutting of the mix.

3.2 RESEARCH PLAN

To accomplish the objectives of this study (see chapter 1), a research plan was framed. The plan comprises of seven steps as follows(figure 3.1):

1. Select pavement sections having various asphalt grades and percent asphalt contents, aggregate types and angularity, gradation, various types of fines and percent fine contents, various AC thicknesses, different levels of traffic volume and load, and different service lives.
2. Obtain traffic data (traffic counts and axle loads) for each selected section.
3. Secure construction history (date of completion/date of opening traffic on the road sections) from the National Highway Authority (NHA) of Pakistan and other consultant/construction agencies.
4. Measure the rut depths at different site/sections, the longitudinal grade and obtain core samples.
5. Conduct laboratory tests on the core samples to determine the physical(AC layer thickness) and asphalt mix properties. These include aggregate gradation, asphalt content, bulk specific gravity, penetration of recovered asphalt, and the angularity of the aggregate.

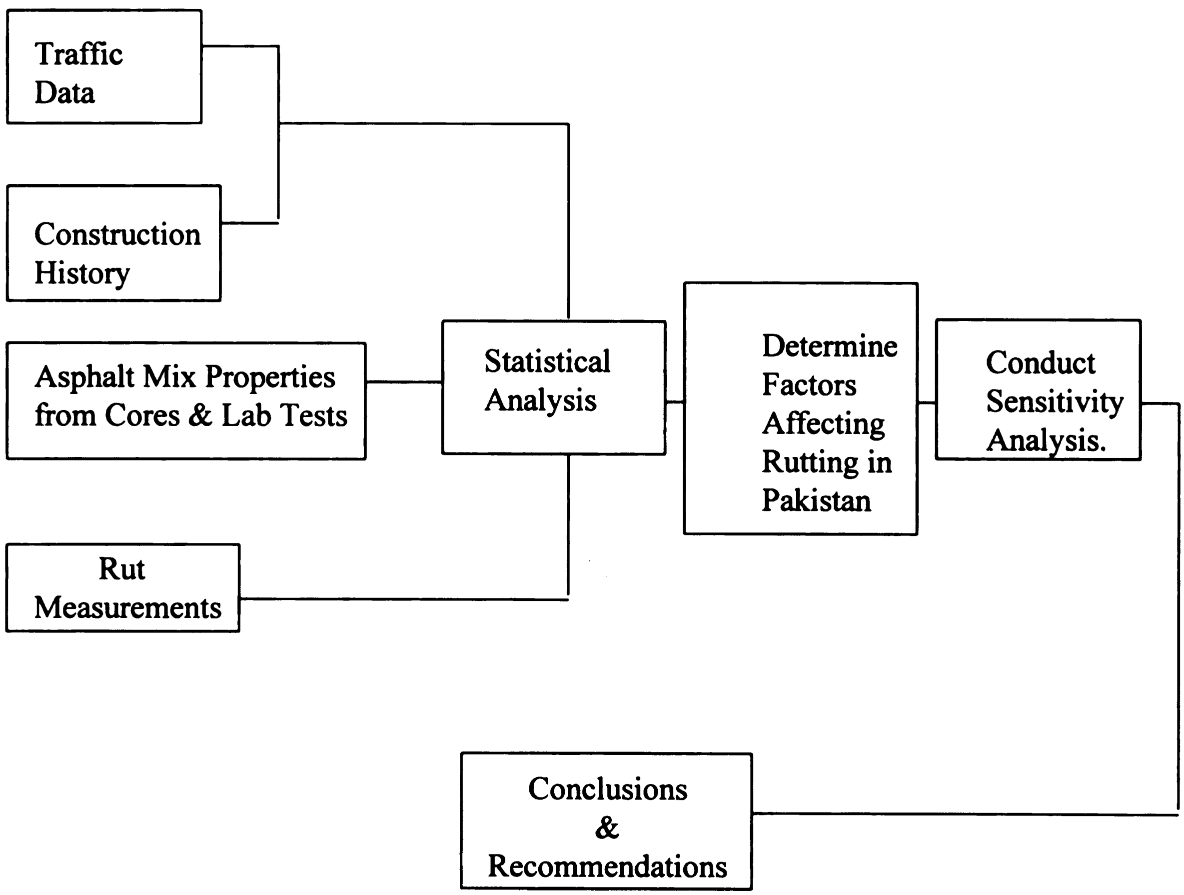

Figure 3.1: Overall Research Plan

6. Perform statistical analysis relating the rut depths to the AC mix properties, traffic and road attributes.

7. Conduct sensitivity analysis whereby the effects of the various parameters of the statistical models on the rut life are assessed.

3.2.1 Pavement Selection and Investigations

The selection of pavement test section/site/locations was accomplished in consultation with personnel from the National Highway Authority (NHA) of Pakistan. The subject study has been designed to analyze pavement sections in areas from Peshawar to Sahiwal. In this regard, a full factorial matrix with the following two independent variables has been formed. (see Figure 1.3)

1. **Asphalt Course Thickness** - Thin (5 to 10 cm), moderate (10 to 18 cm), thick (more than 18 cm) asphalt surface.

2 **Traffic Volume and loads** - Light (0 to 2), moderate (2 to 4), heavy (>4) million 18 kips equivalent single axle loads (ESALs).

The matrix consists of 9 cells, each cell represents pavement sections, to be selected. Each section will have variable length of 1 to 5 kilometers depending on the number of sites, the various degrees of severity levels of rutting (See Figure 1.3). Each test site is 30 meter long and it contains 5 equally spaced test locations (See Figure 1.4).

For each test site/location, the rut depth was measured and one core sample was obtained. Because of economical reasons and time constraints, the number of cores was limited (one per each site).

The rut depths were measured by using a 3 & 2 meter straight-edge leveling rods with a graduated steel ruler with an accuracy of 1 to 2 mm. The ruts were measured in the outer wheel path over each marked test location (five test locations 6 meter long at each test site). The average of the five measurements was then calculated for each test site.

The cores were brought to the laboratory, where the sample thicknesses were recorded and tests were performed according to the American society for Testing & Material (ASTMs) Standard test procedures. The laboratory tests were designed to determine, the bulk specific gravity, gradation, the asphalt content, recovered penetration of the asphalt and the angularity of the aggregate (for more details, see chapter 4).

CHAPTER 4

FIELD AND LABORATORY INVESTIGATION

4.1 FIELD INVESTIGATIONS

4.1.1 Pavement Selection

The pavement section selection was accomplished by surveying the routes physically. The main criterion used in this selection is that the selected sections should represent the spectrum of pavement cross-sections, paving materials, and traffic volume and load found in most areas in Pakistan. In this regard, the following variables were identified and prioritized prior to the selection of the pavement sections.

1. **Asphalt Course Thicknesses** - Thin (5 to 10 cm), moderately thick (10 to 18 cm), and thick (more than 18 cm) asphalt surface.
2. **Traffic Volume and Load** - Heavy, moderate, and light traffic loads and volumes in terms of the 18-kip equivalent single axle load (ESAL).
3. **Pavement Type** - flexible pavement without overlays.
4. **Cross-Sections** - Three layers (AC, base and subbase).
5. **AC Mixes** - Stability based and standard type mixes.
6. **Roadbed Type** - Cohesive soils.
7. **Pavement Surface Age** - Newly constructed(less than 2-years old) and older pavement sections.
8. **Distress Type** - Rutting.

Based on the above criteria and the various variables, 54 flexible pavement sections of variable lengths (one to five kilometers) were initially selected. For each pavement section, the National Highway Authority (NHA) referencing system (kilometer stone, distances measured from Karachi) were used to obtain the location reference point. The construction and rehabilitation history, was obtained from NHA and different consultant/construction agencies in Pakistan. The rut depths and the longitudinal grade were measured. Certain sections had to be excluded from the study due to lack of data regarding date of completion/opening traffic of these sections. Traffic volume and load, were acquired from the National Transport Research Center (NTRC) Islamabad, a traffic study conducted by Mr. Zafar Iqbal Farroqui, and other Provincial Highway agencies. Rut depths, cross-sectional data (layer thicknesses and types), were obtained by actual field/laboratory measurements. The data was then tabulated in a spreadsheet. Table 4.1 provides lists of the flexible pavement sections chosen in this study. Other data such as pavement type, route number, direction (north, south, east, or west bound), section number, and the beginning and ending kilometer post of each pavement section are also listed in the Table.

Each pavement section was examined/surveyed and, within each section, several 100-feet long pavement sites showing rutting problem were identified and they were labeled as test sites. For some pavement sections, the test sites were adjacent to each other while for some others, they were quite apart. During the Winter of 1994, a four member research team of the Military College of Engineering (MCE) visited each test site. The purpose of the visit was to:

1. Verify the location reference point, pavement type, and general conditions.

Table 4.1 : Selected Flexible Sections

Section & SITE	Route NH = 11 PH = 22 LH = 33	Direction N = 01 E = 02 S = 03 W = 04	Kilometer Post From Karachi		Distance From Rawal-pindi	Distance From Main Road	Sample Designation Number
			From	To			
3A	11	4	1696	1697	156	-	11043A156
4B	11	4	1685	1699	145	-	11044B146
4C	11	4	1685	1699	159	-	11044C159
5A	11	4	1669	1670	130	-	11045A130
5B	11	4	1669	1670	130	-	11045C130
6A	11	4	1661	1662	121	-	11046A121
6B	11	4	1661	1662	122	-	11046B122
7B	11	4	1656	1657	116	-	11047B116
7C	11	4	1656	1657	116	-	11047C116
8B	11	4	1641	1642	102	-	11048B102
9A	11	4	1637	1638	98	-	11049A98
9B	11	4	1637	1638	98	-	11049B98
10A	11	4	1637	1638	97	-	110410A97
10B	11	4	1637	1638	97	-	110410B97
12A	11	1	1577	1585	37	-	110112A37
12C	11	1	1577	1585	44	-	110112C44
13A	11	1	1555	1556	15	-	110113A15
13B	11	1	1555	1556	15	-	110113B15
14A	11	1	1561	1564	22	-	110114A22
14B	11	1	1561	1564	22	-	110114B22
14C	11	1	1561	1564	24	-	110114C24
16A	33	2	1550	1550	10	1	330216A101
17A	33	2	1550	1550	10	8	330217A108
17B	33	2	1550	1550	10	9	330217B109
18A	33	2	1540	1540	0	7	330218A07
18B	33	2	1540	1540	0	8	330218B08
19C	33	2	1540	1540	0	14	330219C014
20B	11	3	1505	1506	35	-	110320B35
21A	11	3	1508	1509	32	-	110321A32
22A	11	3	1497	1498	43	-	110322A43

Table 4.1 : Selected Flexible Sections (Continued)

Section & SITE	Route NH = 11 PH = 22 LH = 33	Direction N = 01 E = 02 S = 03 W = 04	Kilometer Post From Karachi		Distance From Rawal-pindi	Distance From Main Road	Sample Designation Number
			From	To			
23B	11	3	1497	1498	42	-	110323B42
24A	11	3	1486	1487	53	-	110324A53
24B	11	3	1486	1487	54	-	110324B54
25A	11	3	1478	1487	53	-	110325A53
26B	11	3	1452	1452	88	-	110326B88
27A	11	3	1452	1458	82	-	110327A82
27B	11	3	1452	1458	88	-	110327B88
28A	11	3	1441	1442	98	-	110328A98
29A	11	3	1442	1442	98	-	110329A98
30A	11	3	1416	1417	124	-	110330A124
31B	11	3	1416	1417	123	-	110331B123
31C	11	3	1416	1417	124	-	110331C124
32B	11	3	1408	1409	132	-	110332B132
33A	11	3	1408	1409	131	-	110333A131
34A	11	3	1387	1395	145	-	110334A145
34B	11	3	1387	1395	153	-	110334B153
36A	11	3	1365	1366	175	-	110336A175
37A	11	3	1381	1382	159	-	110337A159
38A	11	3	1364	1365	176	-	110338A176
38B	11	3	1364	1365	175	-	110338B175
42A	11	3	1291	1291	249	-	110342A249
43A	11	3	1275	1276	265	-	110343A265
44A	11	3	1226	1227	314	-	110344A314
45A	11	3	1227	1228	312	-	110345A312
45B	11	3	1227	1228	312	-	110345B312
45C	11	3	1227	1228	313	-	110345C313
46A	11	3	1175	1179	361	-	110346A361
46B	11	3	1175	1179	364	-	110346B364
47B	11	3	1175	1176	365	-	110347B365
47C	11	3	1175	1176	364	-	110347C364
48A	11	3	1125	1126	414	-	110348A414
48B	11	3	1125	1126	415	-	110348B415

Table 4.1 : Selected flexible sections (Continued)

Section & SITE	Route NH = 11 PH = 22 LH = 33	Direction N = 01 E = 02 S = 03 W = 04	Kilometer Post From Karachi		Distance From Rawal-pindi	Distance From Main Road	Sample Designation Number
			From	To			
49A	11	3	1122	1123	418	-	110349A418
51A	22	3	1250	1250	290	5	220351A2905
52A	22	3	1268	1268	272	15	220352A27215
52B	22	3	1268	1268	272	25	220352B27225
53A	22	3	1334	1334	206	1	220353A2061
54A	22	4	1661	1661	121	3	220454A1213
55A	11	1	1585	1585	45	3	110155A453
58A	11	4	1554	1559	14	-	110458A14
58C	11	4	1554	1559	19	-	110458C19
59A	11	4	1586	1587	46	-	110459A46
59B	11	4	1586	1587	47	-	110459B47
61A	33	2	1540	1540	0	14	330261A014

2. Mark the test sites.

3. Inspect, measure, and record the extent and severity of rutting.

4. Identify those test sites to be cored.

4.1.2 Measurements of Rut Depths

The rut depths were measured by using a three/two meter straight-edge leveling rod, a graduated steel ruler with an accuracy of 1 to 2 mm. The ruts were measured in the outer wheel path over each marked core location and at five other test locations along the 30 meter long test site and the average rut depth was calculated. For each test section of 1 to 5 kilometer length one or more test sites were selected, depending upon the availability of severity levels of rutting. For each severity level, one test site was identified (For more details refer to Table 4.3).

4.1.3 Core Locations

A total of 75 locations were designated for pavement coring. Cores were taken for three severity levels within one test section depending upon availability. Core locations were given specific designation numbers. The coring method and the designation numbers are presented in the next section.

4.1.4 Marking Coding and Coring

Each test site was designated by a number and an alphabet system . The number designates the pavement section and the alphabet designates the test site. For example, a test site designation of 29-A indicates the first test site of pavement section number 29.

All test sections were cored. The cores were extracted from the outer wheel path, at the start of the test site. Each core location was designated using a nine digit number and a letter. Starting at the left-most digit, the first two digits indicate the type of route (National-11, Provincial-22 or local-33), the next two digits designate direction from base station, Islamabad,(North-01, East-02, West-03 and South-04), the fifth and sixth digits indicate the section number, the alphabet designates the site number, the eighth digit indicates the distance of the core location from Islamabad. The ninth digit indicates the distance of core location from a main route(if located on the branch route). For example, a core location designation of 11043A156 indicates (left to right) that the test is conducted on section of National Highway, west of Islamabad, test section number 3 and site A, located 156 kilometers from Islamabad.

The cores were obtained by using a powered auger equipped with a 6/4-inch coring bit. For thin AC layers 6 inch diameter coring bit was used and for thicker layers 4 inch coring bit was used. These produced enough material for testing according to ASTM. Most of the cores obtained from the wheel paths were utilized for measurements of bulk specific gravity, layer thickness and extraction testing to determine material properties. The laboratory test procedures are presented in section 4.3 of this chapter.

4.1.5 Traffic Volume and Loads

In view of the calculation of yearly and cumulative ESALs over the designated sections, the traffic volumes (the number of trucks of different configuration, passing over a test section in 24 hours) were obtained from the records of National Transport Research Center in Islamabad (Traffic Volume Data 1994-95), Provincial Governments and the traffic counts of Capital Development Authority in Islamabad traffic (52, 53, 54).

The annual growth rates of trucks calculated by NTRC from 1983-93 were used (55). These rates are shown below. In this study, a traffic growth rate of 8 to 9 percent per year was used.

Years	Annual Growth Rate
1983-87	6%
1987-90	7.7%
1990-93	9.8%

To calculate the ESALS, different axle load studies carried out in Pakistan, were consulted. These include the axle Load survey 1993 (Taxilla to Muredike) by Zafar Iqbal Farooqui (56), Axle Load Study on National Highways 1995 carried out by NTRC (57), and Axle Load Studies in Punjab (58). Out of these studies, Mr. Farooqui and NTRC studies were found relatively more accurate and they were used in this study.

Truck factors of different categories of truck in Pakistan over the designated pavement section were used. In case where the Truck Factor of a particular section was not available, the nearby Truck Factor was used. AASHTO LEFs (Pt=2.5 and SN=5) and growth factor has been used in calculation of truck factor and cumulative ESALS.

Some of the truck factors at a particular section are appended in Table 4.2 for reference (56, 57, 58).

4.1.5.1 Definition - Truck Factor

Number of ESALS applied by each vehicle is known as ESALS factor. Since the majority of the load on a pavement is due to truck traffic. The vehicle specific ESAL factor commonly called a truck factor which is in unit of ESALS/Truck, is most important factor in computing the design ESALs (Commulative ESALs) (56, 57, 58).

4.1.5.2 Service Life

The ages of test sections and test sites were calculated based on the date of opening the pavement to traffic (obtained from records of NHA and other consultant/construction agencies) and the date of taking rut measurements. The data is shown in Table 4.3.

4.2 LABORATORY INVESTIGATIONS

This section provides a summary of the laboratory test procedures used to determine the physical and engineering characteristics of the materials. Various laboratory tests were conducted to assess the properties of the core materials obtained from the various test sites. Since these tests are standards of the American Society for Testing and Material (ASTM), only the ASTM test designation numbers and the purpose

Table 4.2: Truck Factors at Taxilla (56, 57)

Axle Configuration	Truck Factor
1.2	6.03
1.22	11.851
1.2+22	4.386
1.2+2.2	6.996
1.2+222	14.73
1.22+222	15.821
1.1+1	2.264
1.1+2	3.35

Table 4.3: Field Data - All Test Sites

Section & Site	Sample Designation Number	Sample Thick (cm)	Yearly ESALS (millions)	Service Life (years)	Cum ESALS (millions)	Rut Depth (cm)
3A	11043A156	8.9	0.850	8.000	4.250	0.953
4B	11044B146	8.9	1.250	8.000	6.250	0.953
4C	11044C159	9.8	1.250	8.000	6.250	2.096
5A	11045A130	11.4	1.000	8.000	5.000	0.635
5B	11045C130	10.2	1.000	8.000	5.000	1.270
6A	11046A121	8.9	1.250	8.000	6.250	1.143
6B	11046B122	10.2	1.250	8.000	6.250	1.905
7B	11047B116	5.1	2.023	7.435	9.613	1.270
7C	11047C116	5.1	2.023	7.435	9.613	2.54
8B	11048B102	5.1	2.800	7.435	13.340	2.54
9A	11049A98	10.8	2.030	7.435	9.680	0.953
9B	11049B98	16.8	2.030	7.435	9.680	2.223
10A	110410A97	12.7	2.800	7.435	13.340	1.270
10B	110410B97	11.4	2.800	7.435	13.340	2.223
12A	110112A37	11.4	3.070	1.403	5.310	0.635
12C	110112C44	7.6	3.070	1.403	5.310	2.159
13A	110113A15	15.2	10.810	0.890	9.620	2.54
13B	110113B15	15.2	10.810	0.890	9.620	5.08
14A	110114A22	16.5	6.060	1.370	8.300	0.953
14B	110114B22	10.8	6.060	1.370	8.300	4.445
14C	110114C24	6.4	6.060	1.370	8.300	1.905
16A	330216A101	10.2	3.110	5.930	13.320	3.81
17A	330217A108	14	2.580	5.000	9.630	5.398
17B	330217B109	7.9	2.580	5.000	9.630	1.270
18A	330218A07	18.4	2.652	7.410	12.600	3.81
18B	330218B08	19.1	2.652	7.410	12.600	1.27
19C	330219C014	14	2.970	5.000	11.130	1.143
20B	110320B35	17.1	4.510	0.268	1.210	0.953
21A	110321A32	17.8	1.480	0.268	0.400	0.635
22A	110322A43	19.4	2.290	0.362	0.830	0.635
23B	110323B42	20	1.840	0.362	0.660	0.635
24A	110324A53	18.7	2.170	0.420	0.910	0.953
24B	110324B54	18.7	2.170	0.420	0.910	0.445
25A	110325A53	21.6	1.480	0.420	0.620	0.635

Table 4.3: Field Data - All Test Sites (Continued)

Section & Site	Sample Designation Number	Sample Thick (inches)	Yearly ESALS (millions)	Service Life (years)	Cum ESALS (millions)	Rut Depth (inches)
26B	110326B88	21.6	2.020	0.670	1.350	1.270
27A	110327A82	24.1	1.440	0.670	0.960	0.635
27B	110327B88	24.1	1.440	0.670	0.960	1.397
28A	110328A98	20.3	2.070	1.420	2.940	0.635
29A	110329A98	22.9	2.160	1.420	3.070	1.270
30A	110330A124	21.6	1.845	1.830	3.185	0.635
31B	110331B123	20.3	1.710	1.830	2.970	0.635
31C	110331C124	21	1.710	1.830	2.970	2.54
32B	110332B132	10.8	1.845	1.430	3.185	0.953
33A	110333A131	24.1	1.790	0.625	1.120	0.635
34A	110334A145	12.7	4.110	1.420	5.840	0.635
34B	110334B153	15.2	4.110	1.420	5.840	2.54
36A	110336A175	15.9	3.870	1.416	5.336	1.270
37A	110337A159	21.3	1.770	0.350	0.620	0.318
38A	110338A176	14.6	3.380	0.830	2.800	1.207
38B	110338B175	16.5	3.380	0.830	2.800	2.413
42A	110342A249	10.2	4.780	1.250	5.970	1.270
43A	110343A265	7.6	4.360	0.500	2.180	0.953
44A	110344A314	21	1.575	10.830	9.795	0.635
45A	110345A312	20.95	3.160	10.830	19.710	0.635
45B	110345B312	20.6	3.160	10.830	19.710	1.080
45C	110345C313	21	3.160	10.830	19.710	3.810
46B	110346B364	21	1.420	11.670	10.185	1.905
47B	110347B365	20.3	2.600	11.920	18.610	1.270
47C	110347C364	20.3	2.600	11.920	18.610	4.128
48A	110348A414	21.6	2.560	10.420	14.920	1.270
48B	110348B415	21.6	2.560	10.420	14.920	3.810
49A	110349A418	21.6	3.420	10.500	19.500	5.715
51A	220351A2905	5.1	0.590	1.670	1.030	0.635
52A	220352A27215	12.1	1.270	3.500	3.420	1.270
52B	220352B27225	9.2	1.270	3.500	3.420	0.508
53A	220353A2061	10.8	0.510	3.080	1.250	0.635
54A	220454A1213	5.1	0.900	7.420	4.300	0.635
55A	110155A453	11.4	0.670	1.250	0.830	0.889
58A	110458A14	5.1	2.880	1.580	4.550	1.016
58C	110458C19	14.9	2.880	1.580	4.550	1.842
59A	110459A46	11.4	2.820	1.580	4.450	3.810
59B	110459B47	8.9	2.820	1.580	4.450	0.635
61A	330261A014	5.1	2.970	6.000	11.130	4.128

of the tests are mentioned. For details of the test procedure, the reader is referred to the "Annual Book of ASTM Standards" Volume 4.03.

As noted earlier, all pavement cores were obtained by a four person research team. The cores were then transported to the MCE laboratories and they were:

1. Inspected to document the possible existence of distress such as cracking and stripping, and to determine the thickness of the AC course.
2. Subjected to specific gravity tests to determine the bulk specific gravity (ASTM D-2726) before handling.
3. Washed, dried and oven heated upto the desired temperature to avoid breaking of coarse aggregate during the process of preparation for extraction test. The material was then subjected to extraction tests. The purpose of the extraction tests are to determine:
   a) The percent asphalt, fine, sand, and coarse aggregate contents.
   b) The penetration of the recovered asphalt cement.
   c) The top size and gradation of the aggregates.
   d) The coarse aggregate angularity.

   A brief description of each test procedure and data reduction is presented in the next sections.

4.2.1 Bulk Specific gravity Tests (ASTM-D2726-90)

4.2.1.1 Definition

The ASTM D-2726 standard test procedure defines bulk specific gravity as the ratio of the mass of a given volume of material at 25° C to the mass of an equal volume of water at the same temperature.

4.2.1.2 Summary of Test Method

The specimen is immersed in a water bath at 25^0 C (77^0 F). The mass under water is recorded, and the specimen is taken out of the water, blotted quickly with a camp towel, and weighed in air. The difference between the two masses is used to measure the mass of an equal volume of water at 25^0 C. Correction factors are provided for converting the mass of water to that of the reference 25^0 C if from a practical point of view the weighing was done at different temperatures.

This test method provides guidance for determination of the oven dry or thoroughly dry mass of the specimen. The bulk specific gravity is calculated from these masses. Then the density is obtained by multiplying the specific gravity of the specimen by the density of the water.

4.2.1.3 Significance and Use

This test method is useful in calculating the percent air voids, as given in ASTM D 3203, and the unit weight of compacted dense bituminous mixtures. These values in turn may be used in determining the relative degree of compaction.

Since specific gravity has no units, it must be converted to density in order to do calculations that require units. This conversion is made by multiplying the specific gravity at a given temperature by the density of water at the same temperature.

4.2.1.5 Sampling

The samples were obtained in accordance with ASTM D979. The thickness of specimens be at least one and one half times the maximum size of the aggregate. Pavement specimens were taken from pavements with a diamond drill. Care was taken to avoid distortion, bending, or cracking of specimens during and after removal from pavements . Specimens were stored in a safe, cool place. Specimens were free of foreign materials such as seal coat, tack coat, foundation material, soil, paper, or foil.

4.2.1.6 Procedure

4.2.1.6.1 Specimens That Contain Moisture:-

Mass of Specimen in Water - Immerse the specimen in a water bath at 25^0 C (77^0 F) for 3 to 5 min then weigh in water. Designing this mass as C. If the temperature of the specimen differs from the temperature of the water bath by more than 2^0C (3.6^0F), the specimen shall be immersed in the water bath for 10 to 15 min.

Measure the temperature of the water and if different from $25^0 \pm 1^0$C ($77^0 \pm 1.8^0$ F) a correction to the bulk specific gravity to 25^0 C must be made in accordance with procedure.

Mass of Saturated Surface Dry - After removal from the water, dry the surface of the specimen by blotting quickly with a damp towel and then weigh in air. Designate this mass as B.

Mass of Oven-Dry Specimen - Oven dry the specimen to constant mass at 110^0 C $\pm 5^0$ C (230^0 F) (15 to 24 h is usually sufficient). Allow the specimen to cool and weigh in air. Designate this mass as A.

4.2.1.6.2 Thoroughly Dry Specimens:

Mass of Dry Specimen in Air - Weigh the specimen after it has been standing in air at room temperature for at least 1 h. Designate this mass A.

Mass of Specimen in Water - Use the same procedure as described above.

Mass of Saturated Surface Dry - Use the same procedure a described above.

4.2.1.7 Calculations

Calculate the bulk specific gravity of the specimen as follows:

$$\text{Bulk sp gr} = A/(B - C) \tag{4.1}$$

where:

A = mass of the dry specimen in air (gms);

(B - C) = mass of the volume of water for the volume of the specimen at 25^0C;

B = mass of the saturated surface-dry specimen in air (gms); and

C = mass of the specimen in water (gms).

After the test, the percent air voids in the AC mix were calculated by using the following equation. It should be noted that because of lack of data from NHA regarding theoretical maximum specific gravity (G_{mm}) of materials, a value of 2.56 was assumed.

$$P_a = 100 \left[\frac{G_{mm}-G_{mb}}{G_{mm}} \right] \tag{4.2}$$

Where

P_a = the air voids in the compacted asphalt mixture, percent of total volume;

G_{mm} = the maximum theoretical specific gravity of the paving mixture;

G_{mb} = the bulk specific gravity of the compacted mixture.

$$G_{mm} = \frac{P_{mm}}{P_s/G_{se} + P_b/G_b} \tag{4.3}$$

Where

P_{mm} = percent by weight of total loose mixture =100;

P_s = aggregate content, percent by total weight of mix;

P_b = asphalt content, percent;

G_{se} = effective specific gravity of aggregate;

G_b = specific gravity of asphalt.

4.2.2 Extraction Tests

After the determination of the maximum theoretical specific gravity, the samples were subjected to extraction test in accordance with the ASTM D 2172 standard test procedure. The tests were conducted to determine the AC mix composition. This test separates the asphalt cement in the AC mix from the aggregate. Thus, the AC mix composition such as, the percent asphalt, fine, sand and aggregate contents can be determined. It should be noted that, the test results may be affected by the age of the asphalt mix. Older mixes tend to yield slightly lower bitumen content due to aggregate absorption. It is difficult to remove all the asphalt when some aggregate types are used and some chlorides may remain within the mineral matter affecting the measured asphalt content. Nevertheless, Methylene Chloride was used as solvent and bitumen content was established by difference from the mass of the extracted aggregate, moisture content, and mineral matter in the extract. The bitumen content (see Table 4.4) was expressed as percent by the weight of the moisture-free mixtures as follows:

$$AC = \left[\frac{(W_1 - W_2) - (W_3 - W_4)}{(W_1 - W_2)} \right] \times 100 \qquad (4.4)$$

AC	=	asphalt content (percent by total weight of mix);
W_1	=	mass of test portion;
W_2	=	mass of water in test portion;
W_3	=	mass of the extracted mineral aggregate; and
W_4	=	mass of the mineral matter in the extract.

After extraction of the bitumen from the AC mix, the recovered bitumen, the aggregate and the fine material were further tested to determine the recovered asphalt penetration and the gradation and angularity of the aggregate.

4.2.3 Recovered Asphalt Penetration

The ASTM D5-86 standard test procedure was used to measure the penetration (consistency) of the recovered bituminous material. Higher values of penetration indicate softer asphalt or lower viscosity. Penetration is defined as the consistency of a bituminous material expressed as the distance in tenths of a millimeter that a standard needle vertically penetrates a sample of the material under known conditions of loading, time, and temperature. The recovered asphalt penetrations are listed in Table 4.4.

4.2.4 Sieve Analysis

The particle size distribution of the aggregate and sand obtained from the extraction tests was determined by using sieve analysis in accordance with the ASTM C 136-84a standard test procedure. The tests were conducted by passing a weighed sample of dry aggregate through a series of sieves of progressively smaller openings. Table 4.6 lists the percent by weight of aggregate passing through each sieve.

4.2.5 Aggregate Angularity

Since, there is no standard test procedure for determining the coarse aggregate (retained on sieve number 4) angularity, the number of crushed faces of an individual aggregate particle was used as a measure of the angularity. Numbers ranging from 1 (rounded and subrounded) to 5 (crushed on all four faces) were assigned to the aggregates depending on the number of crushed faces. The test procedure was devised by Hamid Mukhtar during his Phd research (6) and it consists of the following steps:

1. A 300 gram aggregate sample was obtained irrespective of the amount of the coarse aggregates in the AC mix.
2. The aggregates were then divided into different piles depending on the number of crushed faces. Each pile was inspected and an angularity number was assigned according to the number of crushed faces of the aggregates. For example, the pile which consists of aggregate particles crushed on only one face (one side) was given an angularity of 2. Likewise, the pile that consists of aggregate particles crushed on all four faces was assigned an angularity number of 5.
3. The weight of each pile as a percent by weight of the 300 grams sample was determined.
4. The angularity of the aggregate sample was then determined as the sum of the percent by weight of each aggregate pile times the assigned angularity of each pile and divided by the total weight of 300 grams.

Table 4.4 : **Results of extraction, penetration and Bulk specific gravity tests for AC course of the cored pavement sections.**

Sample Designation Number	Sample Thick (cm)	Sample Sp Gty	Rec Pen	Percentage			
				AC	Sand	Fine	Agg
11043A156	8.9	2.445	34.333	4.340	39.747	3.434	52.479
11044B146	8.9	2.449	34.333	4.120	42.168	5.149	48.563
11044C159	9.8	2.418	83.333	4.230	43.882	3.409	48.479
11045A130	11.4	2.438	24.667	3.820	49.937	3.549	42.694
11045C130	10.2	2.234	24.667	4.370	49.039	3.720	42.871
11046A121	8.9	2.447	34.667	4.000	40.272	2.736	52.992
11047B116	10.2	2.468	25.333	4.810	46.691	2.770	45.729
11047C116	5.1	2.470	95.000	6.170	42.711	2.834	48.285
11048B102	5.1	2.485	95.000	7.380	45.563	5.241	41.817
11049A98	10.8	2.425	33.000	5.390	45.820	3.945	44.845
11049B98	16.8	2.454	55.000	5.550	42.975	5.535	45.940
110410A97	12.7	2.481	33.000	5.580	36.484	3.796	54.140
110410B97	11.4	2.438	55.000	5.570	44.911	3.891	45.629
110112A37	11.4	2.356	34.000	3.700	46.503	1.839	47.957
110112C44	7.6	2.395	65.333	3.400	29.917	2.763	63.920
110113A15	15.2	2.530	82.667	4.890	38.492	2.528	54.090
110113B15	15.2	2.535	82.667	4.930	38.409	2.402	54.259
110114A22	16.5	2.367	46.000	4.850	36.623	0.866	57.661
110114B22	10.8	2.554	96.000	6.860	41.554	2.366	49.215
110114C24	6.4	2.427	96.000	6.470	43.856	2.862	46.812
330216A101	10.2	2.472	98.000	4.600	40.478	2.477	52.444
330217A108	14	2.535	91.333	5.820	31.029	3.246	59.906
330217B109	7.9	2.429	68.000	5.510	37.900	3.275	53.315
330218A07	18.4	2.349	62.667	5.330	43.310	2.353	49.007
330218B08	19.1	2.333	42.667	4.600	40.941	3.403	51.056
330219C014	14	2.384	33.000	3.160	40.227	2.140	54.473
110320B35	17.1	2.367	72.333	3.570	29.373	3.983	63.075
110321A32	17.8	2.401	72.333	3.840	29.608	3.856	62.696
110322A43	19.4	2.390	44.667	4.340	35.251	2.879	57.530
110323B42	20	2.400	44.667	4.360	38.026	2.917	54.697
110324A53	18.7	2.315	36.000	3.920	44.082	2.056	49.942
110324B54	18.7	2.367	36.000	3.480	34.960	1.216	60.344
110325A53	21.6	2.340	36.000	4.150	40.899	3.834	51.117
110326B88	21.6	2.439	43.000	5.070	42.557	3.873	48.500
110327A82	24.1	2.350	34.667	3.400	43.547	4.038	49.015
110327B88	24.1	2.382	34.667	4.470	37.992	4.070	53.468
110328A98	20.3	2.410	35.000	4.820	45.705	4.331	45.144

Table 4.4 : **Results of extraction, penetration and Bulk specific gravity tests for AC course of the cored pavement sections (Continued)**

Sample Designation Number	Sample Thick (cm)	Sample Sp Gty	Rec Pen	Percentage			
				AC	Sand	Fine	Agg
110329A98	22.9	2.411	35.000	4.040	44.525	4.472	46.963
110330A124	21.6	2.419	42.333	4.510	41.280	4.077	50.132
110331B123	20.3	2.472	42.333	4.860	43.156	3.473	48.512
110331C124	21	2.454	46.333	5.220	45.314	3.422	46.044
110332B132	10.8	2.429	40.333	5.020	43.748	4.445	46.787
110333A131	24.1	2.356	32.000	4.270	44.495	3.475	47.760
110334A145	12.7	2.411	46.000	4.040	41.570	4.165	50.225
110334B153	15.2	2.476	92.333	4.640	32.441	2.212	60.706
110336A175	15.9	2.450	46.000	4.620	28.709	3.930	62.741
110337A159	21.3	2.323	42.000	4.260	43.906	4.155	47.679
110338A176	14.6	2.259	65.000	4.450	38.583	2.303	54.664
110338B175	16.5	2.464	94.000	4.710	36.382	2.497	56.412
110342A249	10.2	2.347	67.000	3.710	33.923	2.754	59.613
110343A265	7.6	2.313	66.333	3.030	32.068	2.987	61.915
110344A314	21	2.311	44.000	4.160	39.007	4.524	52.309
110345A312	20.95	2.348	74.000	4.910	40.575	4.098	50.417
110345B312	20.6	2.384	92.333	5.830	40.107	4.106	49.955
110345C313	20.95	2.516	86.667	5.740	40.786	2.969	50.505
110346A361	20.95	2.323	43.000	4.820	43.088	3.940	48.152
110346B364	20.96	2.360	95.000	4.750	38.738	3.648	52.864
110347B365	20.3	2.404	85.000	4.440	36.638	3.478	55.444
110347C364	20.3	2.525	95.000	5.250	34.394	3.373	56.983
110348A414	21.6	2.431	52.667	5.180	30.997	4.219	59.604
110348B415	21.6	2.522	52.667	5.260	35.471	2.975	56.295
110349A418	21.6	2.536	83.667	5.310	33.204	2.746	58.739
220351A2905	5.1	2.510	35.667	5.220	38.433	3.668	52.679
220352A27215	12.1	2.390	34.667	4.220	28.983	3.850	62.947
220352B27225	9.2	2.351	34.667	3.330	35.314	4.128	57.229
220353A2061	10.8	2.431	68.333	5.190	42.181	4.722	47.907

Table 4.4 : **Results of extraction, penetration and Bulk specific gravity tests for AC course of the cored pavement sections (Continued)**

Sample Designation Number	Sample Thick (inches)	Sample Sp Gty	Rec Pen	Percentage			
				AC	Sand	Fine	Agg
220454A1213	5.1	2.438	36.667	4.530	51.458	4.172	39.840
110155A453	11.4	2.424	64.333	4.730	42.776	3.858	48.635
110458A14	5.1	2.265	82.667	2.700	32.946	3.824	60.530
110458C19	14.9	2.432	82.667	3.810	29.280	3.251	63.659
110459A46	11.4	2.533	66.333	4.950	40.444	2.813	51.793
110459B47	8.9	2.408	66.333	4.870	38.119	2.968	54.043
330261A014	5.1	2.526	64.333	5.650	39.155	4.255	50.940

The above procedure implies that the angularity of the coarse aggregate is independent of the amount of the coarse aggregate in the AC mix. That is two samples with 20 and 50 percent coarse aggregate contents may yield the same angularity. The drawback of this procedure is that, asphalt mixes with low coarse aggregate contents may have the same aggregate angularity as those mixes made with high coarse aggregate contents. At low coarse aggregate contents, the aggregate particles float in the sand matrix which causes separation of the particles. Hence, the aggregate interlocking and friction are decreased. This scenario implies that care should be taken when analyzing the effects of the coarse aggregate angularity on the AC mix performance. The aggregate angularity must be considered in view of the percent aggregate contents in the mix. That is, higher coarse aggregate content in the mix leads to more mobilization of the aggregate angularity. Nevertheless, the test procedure and the method of calculating the angularity were adopted to minimize the dependency of the angularity on the percent aggregate. This procedure of calculating aggregate angularity follows different approach than those used by other researchers. In their test method, the aggregate angularity is defined by three categories (rounded, subrounded, and angular). The sample angularity is calculated as the weighted average angularity of the three categories based on their weights. The method does not differentiate between aggregates crushed on 1, 2, 3, or 4 faces.

The angularity of sand size particles (passing standard sieve number 4 and retained on sieve number 200) can be determined by procedure adopted by the Michigan transport department as standard test method (MTM) 118-90. The test procedure however, requires a minimum sample size of 1500 grams which was not available in this study. Hence, the sand angularity was not determined and it is not considered in any further analysis.

Finally, in all test sites the coarse aggregates were crushed on either 3 or 4 faces. Because of this consistency (lack of variability in the aggregate angularity), the data was not included in the study.

Table 4.5 : Results of Sieve Analysis for AC Course

Sieve Size (mm)											
Sec.	**25**	**19**	**12.5**	**9.5**	**#4**	**#8**	**#50**	**#100**	**#200**	**-200**	**Gravel**
Percent Retained											
1A	0.00	0.00	12.35	12.87	26.09	13.23	24.74	3.78	1.73	5.18	64.54
3A	1.82	3.12	16.62	9.70	23.60	11.92	22.33	4.76	2.54	3.55	66.78
4B	0.00	0.00	17.90	11.14	21.61	8.87	24.96	7.17	2.98	5.35	59.52
4C	1.65	4.83	12.79	7.91	23.44	12.18	25.56	5.53	2.55	3.52	62.80
5A	2.12	1.34	9.27	7.7	23.96	15.94	29	4.54	2.44	3.65	60.33
5B		6.78	10.02	6.85	21.18	15.08	28.09	5.2	2.91	3.84	59.91
6A	2.01	6.37	14.82	9.82	22.18	11.15	22.91	5.13	2.76	2.81	66.35
6B	2.38	4.4	15.64	8.79	17.41	12.22	25.79	4.7	4.1	4.51	60.84
7B		2.75	21.04	9.32	14.93	12.63	25.91	8.04	2.47	2.87	60.67
7C		3.77	18.37	10.8	18.52	11.46	25.51	6.38	2.17	2.98	62.92
8B		0.46	12.96	10.73	20.4	9.11	30.51	7.41	2.7	5.67	53.66
9A		6.54	11.86	5.53	23.47	12.8	27.81	5.54	2.28	4.13	60.20
9B	2.26	2.49	5.4	8.77	29.72	14.23	21.55	6.8	2.92	5.82	62.87
10A	0.79	2.58	19.46	12.37	22.14	8.31	22.99	5.07	2.27	3.98	65.65
10B		3.72	10.55	7.72	26.33	12.25	28.03	4.93	2.35	4.05	60.57
11A	3.46	5.6	25.32	10.69	16.58	7.41	10.38	5.9	3.13	3.51	69.06
11C	2.46	9.76	27.67	7.99	14.81	8.31	19.08	3.99	2.38	3.52	71.00
12A	1.31	9.57	19.44	6.19	13.29	9.57	30.82	4.03	3.87	1.88	59.37
12C		6.41	27.76	11.01	20.99	8.3	18.74	2.49	1.44	2.82	74.47
13A	1.1	7.5	20.02	9.72	18.98	8.73	24.88	4.54	1.9	2.6	66.05
13B		8.16	20.67	9.38	19.31	10.98	24.16	3.35	1.49	2.47	68.50
14A		4.72	20.32	12.12	23.44	10.23	20.75	6.56	0.95	0.89	70.83
14B	3.1	3.29	21.07	9.49	15.89	9.04	29.3	3.385	2.89	2.41	61.88
14C		5.32	19.36	8.75	16.62	9.6	29.91	3.99	3.46	3.06	59.65
16A		1.75	19.92	11.04	22.73	11.73	23	4.5	2.76	2.53	67.17
17A	6.86	4.55	17.34	13.58	21.66	9.13	15.15	5.93	2.39	3.36	73.12
17B		1.36	19.2	11.85	24.47	11.25	17.58	6.3	4.56	3.41	68.13
18A			15.99	10.41	25.87	12.71	24.24	6.47	1.85	2.41	64.98
18B		0.53	15.53	11.06	26.91	13.32	19.98	7.5	1.67	3.53	67.35
19C			18.59	13.22	24.44	10.96	20.36	7.6	2.62	2.17	67.21
20B	5.78	11.76	16.28	8.04	23.55	9.47	13.67	4.57	2.75	4.08	74.88
21A	9.49	4.19	17.82	7.18	26.52	9.9	13.27	5.02	2.6	3.96	75.10

Table 4.5 : Results of Sieve Analysis for AC Course (continued)

Sieve Size (mm)											
Sec.	25	19	12.5	9.5	#4	#8	#50	#100	#200	-200	Gravel
Percent Retained											
22A	4.36	6.25	14.1	9.21	26.22	11.24	17.82	4.87	2.92	2.97	71.38
23B	1.22	10.01	13.53	8.74	23.69	10.78	18.82	6.65	3.51	3.02	67.97
24A	1.64	5	19.33	6.29	19.72	16.67	18.48	5.49	5.24	2.09	68.65
24B	5.39	13.25	16.44	5.39	22.05	8.25	18.36	5.86	3.75	1.23	70.77
25A	3.33	3.77	13.07	8.42	24.74	13	24.06	3.61	2	3.96	66.33
26B	2.19	7.63	16.05	4.41	20.81	14.02	22.11	5.62	3.08	4.05	65.11
27A	3.41	4.12	11.75	3.81	27.65	11.8	23.4	6.08	3.8	4.15	62.54
27B		6.8	16.64	5.29	27.24	11.08	21.32	4.78	2.59	4.26	67.05
28A		1.6	14.16	7.44	24.23	11.57	27.68	6.01	2.76	4.51	59.00
29A		0.61	15.38	7.56	25.39	12.16	25.37	5.97	2.9	4.61	61.10
30A		5.67	14.06	8.66	24.11	11.13	22.81	6.1	3.19	4.22	63.63
31B	1.03	6.51	11.07	6.6	25.78	9.92	25.01	7.08	3.35	3.64	60.91
31C	1.23	3.56	11.31	5.41	27.07	12.53	25.3	6.9	3.08	3.56	61.11
32A	3.62	7.09	12.87	14.8	14.92	19.06	18.94	3.48	1.88	3.3	72.36
32B		5.83	13.5	14.24	15.82	17.25	21.28	5.04	2.82	4.68	66.64
33A		3.27	16.05	5.47	25.1	13.34	24.16	5.62	3.36	3.59	63.23
34A		4.92	20.25	10.37	16.8	15.35	20.65	4.83	2.49	4.31	67.69
34B	5.9	12.85	17.12	10.37	17.42	10.13	14.85	5.75	3.29	2.3	73.79
36A	10.84	9.32	12.42	11.77	21.43	8.58	14.3	4.7	2.52	4.11	74.36
37A	5.78	2.45	13.83	5.02	27.72	16.26	22.83	3.98	2.79	4.31	71.06
38A	4.1	5.68	17.88	9.15	20.4	11.05	17.24	8.37	3.72	2.36	68.26
38B	2.91	5.82	23.43	9.31	17.73	11.36	16.49	7.18	3.15	2.6	70.56
42A		2.98	22.36	11.92	24.65	12.67	16.18	4.22	2.16	2.82	74.58
43A	1.51	5.27	21.61	12.21	23.25	12.21	16.9	2.44	1.52	3.05	76.06
44A	1.18	3.35	18.43	9.98	21.64	10.85	20.52	5.8	3.53	4.69	65.43
45A		5.26	18.93	9.04	19.79	9.3	22.72	7.3	3.35	4.3	62.32
45B		6.81	18.03	7.61	19.72	9.22	21.85	8.03	3.49	4.33	61.39
45C		4.15	20.15	8.88	20.4	9.57	22.55	7.19	3.96	3.12	63.15
46A		4.5	17.23	8.24	20.62	8.98	22.75	10.07	3.47	4.12	59.57
46B		5.53	19.15	9.7	21.12	8.71	21.77	6.88	3.31	3.8	64.21
47B	2.33	6.33	18.2	9.3	21.86	11.26	17.7	6.01	3.37	3.62	69.28
47C	2.38	8.83	17.63	9.63	21.67	9.22	18.9	5.46	2.72	3.55	69.36
48A	2.45	5.79	26.8	10.3	17.52	7.57	18.77	3.32	3.03	4.42	70.43
48B	7.62	8.05	20.04	8.35	15.36	8.2	20.7	4.57	3.97	3.1	67.62
49A		4.96	29.67	9.45	18.39	6.96	21.15	4.29	2.3	2.87	69.43
50A	1.15	0.64	18.22	11.94	26.83	8.17	20.79	4.31	2.79	5.12	66.95
51A		2.81	12.52	12.83	27.42	10.38	23.47	3.92	2.18	3.84	65.96

Table 4.5 : Results of Sieve Analysis for AC Course (continued)

Sieve Size (mm)											
Sec.	**25**	**19**	**12.5**	**9.5**	**#4**	**#8**	**#50**	**#100**	**#200**	**-200**	**Gravel**
Percent Retained											
52A		5.27	30.16	13.74	16.55	7.23	17.58	3.52	1.93	3.98	72.95
52B		4.44	18.42	10.44	25.9	12.12	18.96	3.56	1.89	4.25	71.32
53A		1.06	21.32	10.29	17.86	9.07	25.42	7.47	2.53	4.95	59.60
54A		1.49	11.22	9.7	19.32	11.43	28.21	11.72	2.54	4.35	53.16
55A		1.08	14.45	11.82	23.7	11.08	26.23	5.37	2.22	4.02	62.13
58A			14.29	15.55	32.37	15.63	13.87	2.81	1.55	3.9	77.84
58C	14.35	3.25	19.42	10.58	18.58	8.6	16.33	3.5	2.01	3.35	74.78
59A			19.81	12.65	22.03	15.57	16.84	18.19	1.95	2.93	70.06
59B	1.72	1.12	20.02	12.98	20.97	10.6	17.73	8.88	2.86	3.08	67.41
60A		3.58	23.18	10.2	20.47	10.93	22.02	3.35	2	4.22	68.36
61A		4	20.38	10.89	18.72	6.84	22.95	9.2	2.51	4.47	60.83

CHAPTER 5

ANALYSIS AND DISCUSSION

5.1 General

The response of a pavement structure to wheel loads can be divided into elastic, viscoelastic, and plastic. Although the three responses are dependent on the material properties, the elastic one is time independent while the viscoelastic and plastic responses are time dependent. The plastic response manifests itself in the form of rutting and fatigue cracking.

Like most other solids, asphalt concrete pavements exhibit a greater variety of mechanical behavior than liquid or gases. It is extremely difficult to arrive at a single set of equations that realistically describes or models the interplay of the elastic, viscoelastic, and plastic responses under a combined set of stresses and boundary conditions. Even if such equations could be developed, they would be far too complex for the practical analysis of stresses and strains induced in the pavement structure. To simplify the problem, in the early work on plastic solids, the yield condition and plastic flow rule were treated as independent ingredients of theory of plasticity. For example, an experimental based yield condition was advanced by Tresca (59). Later, Saint Venant (60) and Levy (61) adopted Tresca's condition and developed a plastic flow rule whose form was inspired by the theory of elasticity. In his theory of plasticity, Von (62) retained the plastic flow rule and modified (for the purpose of mathematical convenience) the Tresca yield condition.

Recently, the plastic flow rule and yield condition for asphalt surfaced pavements were further simplified and rut models were developed. The models are based on a single variable, the compressive strain. The vertical compressive strain is calculated by using elastic layer theory. Examples of these models are those developed by the AASHTO/ARE (63) and Majidzadeh and Ilve (64). The main limitations of the models are that:

1. They are independent of the plastic properties of the paving materials.
2. They cannot be used to assess the effects of material properties on the rut potential of asphalt pavements.
3. Their accuracy is poor at best.

Numerous statistical rut depth models for AC surfaced pavements have also been developed. The models express the rut depth in terms of the asphalt-aggregate mix properties and compositions. These models are presented in Chapter 2 of this thesis. It should be noted that the rut depth of asphalt pavements are functions of not only the AC layer properties but also the properties of the base and subbase layers, the characteristics of the roadbed soils, the traffic load and volume, relief of the area and the environmental factors. In Pakistan, most of the asphalt pavement network is experiencing premature rutting and it is mainly contributed by AC layer.

The objectives of this study (as explained in chapter 1) are to determine the asphalt mix variables that affect the structural performance (rutting) of flexible pavements and to recommend changes in the existing asphalt mix design procedure and standard specifications in Pakistan. To accomplish these objectives, the outputs of this study must include relationships between pavement performance and asphalt mix variables such that the sensitivity of the pavement performance to the various asphalt mix variables can be assessed. Therefore, the data analysis procedure to be employed, must account for the variability of the asphalt mix and must be capable of:

1. Providing a good description of the sensitivity of the response of the dependent variable (rut) to the input parameters (independent variables).
2. Developing realistic models whereby the relationships between the dependent and independent variables are reasonable and have good engineering interpretations.
3. Predicting pavement responses in terms of rutting potentials for the range of the given variables.

Two types of analysis can be employed; mechanistic and statistical. The former is typically based on existing theory (e.g., elastic, plastic, viscoelastic) and it requires substantial inputs regarding material properties. Because of the limited resources, such inputs cannot be obtained. Further, the resulting mechanistic models are not practical and they cannot be used by Highway Agencies. Consequently, a statistical type analysis was selected in this study. The inputs to the statistical models include some of the engineering and physical properties of the AC materials as well as traffic loading, volume, grade, and pavement service life (age).

The main advantage of the statistical analysis is its simplicity and low cost. The main disadvantage is that the resulting statistical models are applicable only within the range of data from which the models have been developed. This disadvantage can be minimized if the range of material properties used in developing the statistical model represents the spectrum of the properties used in the field. Therefore, the selection of the experiment is very crucial to the success of the analysis. The experiment should include pavement sections that:

1. Have a representative range of the properties and variability of the asphalt mixes used in the construction of various pavement sections;
2. Experience light, medium, and heavy traffic;
3. Are located in the various environmental regions;
4. consist of various cross-sections.

The details of the experiments in this study are presented in Chapter 4. The statistical package (computer program) used in the study is "Statistical Package for Social Sciences (SPSS)". The reasons for this selection include (4):

1. The simplicity and user friendliness of the program.
2. The various options that are embedded in the program which allow the users to examine the various statistical parameters of the output models.
3. Its graphic capability whereby each input and output variable can be plotted and examined for engineering interpretation.
4. The capability of the program to access the data through a common spreadsheet and/or data base.
5. The flexibility of the program which allows the users to input the desired form of equations relating the dependent and independent variables.

Most of the statistical procedures are generally based on the least squares criteria (minimization of the sum of the square of the differences between the predicted and the observed data). These procedures are simple and easy to use provided that the proper relationship between each independent variable and the dependent one is specified. In this study, emphasis were laid so that the relationship would:

1. Have the proper engineering interpretations.
2. Be representative of the observed trend between the dependent and independent variables.

Several aspects relative to these issues and some specific comments regarding the statistical procedure used in this study are presented in the next section.

5.2 STATISTICAL ANALYSIS TECHNIQUES AND ISSUES

The most important issues regarding statistical analysis of engineering data include (65, 67):

1. The number of variables to be included in the analysis, their significance level, and their collinearity if any.
2. The form of the equation(s) relating each independent variable and/or clusters of independent variables to the dependent one.
3. The engineering interpretations of the final correlation equation.

If the only purpose of the statistical analysis is to express the behavior of the response (dependent) variable in terms of the independent variables and to minimize the residual sum of squares, then the best suited model is the one that includes all available independent variables regardless of whether or not, the resulting relationship is realistic. A model having fewer variables, on the other hand, has the appeal of simplicity, and economic advantage in terms of obtaining the necessary information. The elimination of some variables from the model, however, is obtained at the expense of biases and loss of predictability of the model. Thus, the exclusion of any variable with a significant correlation coefficient causes a bias penalty (65).

Regardless of the number of variables to be included in the final model, the results of any statistical analysis that is based solely on the least square technique reflects only the correlational structure of the data being analyzed. This structure may or may not be representative of the true engineering relationships between the dependent and independent variables. Another problematic aspect of variable selection is that the relative importance of a variable as manifested in the sample may not necessarily reflect its relative importance in the population. Important variables in a sample may appear unimportant in the population and vice versa. That is, the best set of variables in a sample might not be the best set in the population. The choice of variables in the final model depends on prior knowledge and experience and on the variable clusters used to model perfect behavior. Further, statistical correlation between independent variables (e,g., material properties), and the dependent variable (e,g., pavement performance in terms of rut resistance) may lead to several possible outcomes including:

1. Certain material properties (e.g., air void) appear to have specific effects on pavement performance which can be related to certain observed patterns.
2. Certain material properties appear to have no effect on pavement performance. That is, regardless of the range of the property and its variation, the pavement performance is more or less constant for the entire range of that property.

3. Variation in the values of the pavement performance appear to have similar pattern that can be related to variations in the material properties.

Nevertheless, in this study, the statistical analysis were conducted by using the SPSS computer program. The statistical models (regression equations) were developed by using four steps. A brief discussion of each of these steps is provided here and a detailed discussion is contained in the model development (67).

Step 1 Determination of a Simple Correlation Matrix

In this step, a simple correlation coefficient between the dependent and each of the independent variables was computed and tabulated in a matrix format. Each coefficient was then examined to determine the trend between the dependent and each of the independent variables and the significance of the latter on the former. The trends and the significant levels were also compared to field observations and experience. The simple correlation matrix also provides information regarding the degree of collinearity between any two independent variables (a situation when two or more independent variables are correlated among themselves). A higher degree of collinearity between any two independent variables, causes more difficulty in separating the effect of each independent variable on the response variable(66). Because of its importance, the degrees of collinearity between the independent variables were also examined in steps 2 and 3.

Step 2 Computation of Eigenvalues and Condition Indices

The eigen values of a symmetric (square) matrix "A(n*n)" are a set of "n" non-negative scalars "λ_i^2" such that their product with n non-zero vectors "z_i, i =1...n" of the matrix is the same as the product of the matrix with the z_i vectors. That is "$Az_i = \lambda_i^2$ z". The eigenvalues are typically used to examine the collinearity between the independent variables. In addition, the scaled uncentered cross-products matrix and the decomposition of the regression variance corresponding to these eigen values were computed. There is an indication of near dependency of variables, when there is a high proportion of the variance of two or more coefficients that are associated with the same eigen value (67). Hence, the condition index which is defined below can be calculated.

$$\text{Cond. Index} = [\ \text{eigenvalue}_{max} / \text{eigenvalue}_i\ ]^{0.5} \qquad (5.1)$$

The presence of collinearity results in small eigen values and consequently, larger condition index values. Furthermore, the number of large condition index values corresponds to the number of cases of collinearity between the independent variables.

Step 3 Computation of Variable Tolerance and Variance Inflation Factor (VIF)

Variable tolerance and variance inflation factors (VIF) are other measures of collinearity between the independent variables. The variable tolerance and the VIF are defined as (64, 67):

$$\text{Tolerance} = (1\text{-}R_i^2) \tag{5.2}$$

$$\text{VIF} = [\ 1/\ (1\text{-}R_i^2)] \tag{5.3}$$

where

R_i = the multiple correlation coefficient of the ith independent variable when it is predicted from other independent variables.

If the tolerance of an independent variable is small, or its VIF is high, then the variable is collinear with one or more independent variables.

Step 4 Variable Selection Methods

In this step, the independent variables to be included in the statistical model are selected. Several variable selection methods are available to the user of the SPSS program. In this study, the stepwise regression method is used. This method identifies a good (not necessarily the best) set of variables to be included in the development of the statistical model. The statistical model is constructed by considering one variable at a time that having the greatest or the least impact on the residual sum of squares of the model. Three stepwise techniques can be identified (62, 67); forward selection, backward elimination and stepwise selection. These three techniques are addressed below.

Forward Selection - In the forward selection technique, the first independent variable to be considered in the development of the statistical model is that which accounts for the largest amount of variation in the dependent variable. At each successive step, the independent variable (from the independent variable pool) that causes the largest decrease in the residual sum of squares of the existing model is added. In the absence of any termination rule the process continues until all the variables are included in the model.

Backward Elimination - In the backward elimination technique, first the statistical model is developed by including all the independent variables in the pool. At each consequent step, a variable whose deletion causes the least increase in the residual sum of squares is dropped. If no termination rule is specified the deletion process continues until only one independent variable is left in the model.

Stepwise Selection - The stepwise selection technique is basically a forward selection process that at every step reexamines the significance of the previously added variables. If the partial sum of squares of any previously added variable does not meet a minimum specified criterion, the variable is dropped from the model and the selection process changes to backward elimination. The variable elimination process continues until all of the variables remaining in the model meet a minimum specified criterion after which the forward stepwise selection process is resumed. It should be noted that, at every step, the significance of all variables relative to the model is reexamined. Hence, any variable that has been deleted from the model in earlier steps may be added back to the model when it meets the minimum specified criterion (67).

Neither the forward selection nor the backward elimination techniques consider the impacts of adding or deleting a variable on the remaining variables in the model. A variable added to the model in the forward selection technique may become insignificant after other variables have been added. Similarly, the significance of a deleted variable may increase as other variables are deleted from or added to the model.

The stepwise selection technique was used in this study because the process allows a better chance of selecting the best independent variables relative to the other two techniques. Nevertheless, the forward selection and the backward elimination techniques were occasionally used to compare the resulting statistical models. For each statistical model, the null hypothesis of no relationship between the dependent and independent variables were tested. Finally, the residual analysis and the comparison of the predicted versus observed values were performed.

5.3 ANALYSIS AND DISCUSSION OF RUT DATA

Various Rut Models for asphalt concrete pavement have been established by numerous investigator based on field performance and laboratory data. The laboratory based rut model generally predict failure much sooner than is observed in the field.

Rut or plastic deformation of a pavement structure is a function of the construction practices, the properties and thickness of the AC, base, and subbase layers, the properties of the roadbed soil, environmental factors and traffic load and volume (1). Unfortunately, the properties of the base, subbase, and roadbed materials are not available to this study. Hence, the following assumptions were made at the onset of this analysis.

1. Rutting in flexible can be analyzed as a function of the AC layer and its material properties.

2. The material properties, layer thickness, construction specifications/practices and the construction quality remain the same within one site.

The two assumptions are reasonable because of the following reasons:

1. The pavement sections included in this study and most of the flexible pavement network in Pakistan are experiencing premature rutting. The rut is mainly contributed by the AC layer (the rut channel is narrow and deep). Hence, the properties of the base, subbase layers and the roadbed soil have insignificant effects on pavement rutting.

2. In this study, the test sites within one pavement test section number, were selected from the same job number (a number, used by Highway agencies to identify a construction/rehabilitation project). It is the practice of Highway agencies, to divide longer pavement control sections into several smaller job sections for construction and/or rehabilitation purposes. The pavement thickness design, the material properties, and the specifications are then kept the same within the same job section (number) depending on the existing pavement. Thus, any pavement section within the same job number will " theoretically " have similar layer thicknesses and material properties.

Due to the first assumption, one should expect some bias in the developed rut model. That is, given the lack of information regarding the properties of the base, subbase and roadbed soils, their effects cannot be incorporated. Further, the assumption of constant material properties may not hold. Even though, in the presence of the same specifications and design, variations due to construction practices are always present in the form of the AC mix manufacturing, placement, and compaction. Nevertheless, the thickness of the AC layer in the pavement was obtained from cores that were extracted at one or more locations, and the AC mix properties for each core were determined as presented in the next section. Thus, the bias in the data is minimized and whatever exists, is mainly because of variation in the thickness and properties due to construction.

5.3.1 Test Results

A total of 75 six/four inch diameter full depth cores were obtained from cored sections/site. Each core was carefully examined and defects such as stripping, cracking and smoothness of the core sides were noted and the thickness of the AC layer in each core was measured and recorded. Prior to extractions, the bulk specific gravity of each core was ascertained using ATSM D 2726 standard test procedures. The test result and field data are listed in Table 5.1.

5.3.2 Determination of Material Properties for a Core Location

Based on the second assumption and due to economic and time constrains, and test requirements (the materials from more than one core must be combined to obtain the required amount of material), it was decided to perform one extraction test for each pavement section. Once again, prior to testing, two or more cores obtained from one pavement site were combined to yield enough material for a single extraction test. Table 5.1 summarizes the test results and field measurements. These include the sample designation number, thickness, longitudinal grade, specific gravity, air voids, the penetration of the recovered asphalt cement, the percent by weight of the AC, fine, sand,

and coarse aggregate in the asphalt mix, the service life of the pavement section (the period in years between construction and coring), the cumulative 18 kip ESALs experienced by pavement section during the service life and the average rut depth of the core location. Table 5.2 summarizes the minimum, maximum, average, and standard deviation of each data entry of Table 5.1.

5.3.3 Statistical Analysis

The objectives of this analysis are to :

1. Determine the influential factors affecting pavement rutting.
2. Recommend changes to the existing asphalt mix design and pavement construction practices to decrease the rut potential.

Statistical analysis was conducted to relate the measured pavement rut depth to the AC mix variables, the longitudinal grade and the traffic volume. During the analysis, several collinearity problems were encountered. For example, by normal practice, higher angular aggregate contents, better construction practices, thick AC layer, and strict quality control are commonly used for most pavements designed for higher traffic volumes. By the nature of this practice, collinearity between several independent variables (e.g., AC thickness, and traffic volume) exists as shown by the simple correlation matrix in Table 5.3. For example, the term ESAL (the cumulative 18-kip equivalent single axle load applications) has positive collinearities with the AC thickness (.1), the percent asphalt content (.48), bulk specific gravity (.35) and the penetration (.45).

Table 5.1 : Field / Lab data for test samples

Section Site	Sample Designation Number	AC Thick (cm)	Bulk Sp Gty	Air Voids %	Rec Pen	Percent by Weight				Service Life (years)	Yearly ESALS (million)	Cum ESALS (million)	Grade %	Rut Depth (cm)
						AC	Sand	Fine	Agg					
3A	11043A156	8.9	2.445	4.48	34	4.34	39.75	3.43	52.48	8.00	0.85	4.25	0	0.953
4B	11044B146	8.9	2.449	4.32	34	4.12	42.17	5.15	48.56	8.00	1.25	6.25	0	0.953
4C	11044C159	9.8	2.418	5.54	83	4.23	43.88	3.41	48.48	8.00	1.25	6.25	0	2.096
5A	11045A130	11.4	2.438	4.77	25	3.82	49.94	3.55	42.69	8.00	1.00	5.00	0	0.635
5B	11045C130	10.2	2.234	12.73	25	4.37	49.04	3.72	42.87	8.00	1.00	5.00	0	1.270
6A	11046A121	8.9	2.447	4.41	35	4.00	40.27	2.74	52.99	8.00	1.25	6.25	0	1.143
6B	11046B122	10.2	2.433	4.96	35	4.88	44.53	4.35	46.25	8.00	1.25	6.25	0	1.905
7B	11047B116	5.1	2.468	3.60	25	4.81	46.69	2.77	45.73	7.44	2.02	9.61	0	1.270
7C	11047C116	5.1	2.470	3.50	95	6.17	42.71	2.83	48.28	7.44	2.02	9.61	0	2.540
8B	11048B102	5.1	2.485	2.93	95	7.38	45.56	5.24	41.82	7.44	2.80	13.34	0	2.540
9A	11049A98	10.8	2.425	5.26	33	5.39	45.82	3.95	44.85	7.44	2.03	9.68	0	0.953
9B	11049B98	16.8	2.454	4.16	55	5.55	42.97	5.53	45.94	7.44	2.03	9.68	0	2.223
10A	110410A97	12.7	2.481	3.08	33	5.58	36.48	3.80	54.14	7.44	2.80	13.34	0	1.270
10B	110410B97	11.4	2.438	4.75	55	5.57	44.91	3.89	45.63	7.44	2.80	13.34	0	2.223
12A	110112A37	11.4	2.356	7.96	34	3.70	46.50	1.84	47.96	1.40	3.07	5.31	0	0.635
12C	110112C44	7.6	2.395	6.43	65	3.40	29.92	2.76	63.92	1.40	3.07	5.31	0.6	2.159
13A	110113A15	15.2	2.530	1.17	83	4.89	38.49	2.53	54.09	0.89	10.81	9.62	2.35	2.540
13B	110113B15	15.2	2.535	0.98	83	4.93	38.41	2.40	54.26	0.89	10.81	9.62	3	5.080
14A	110114A22	16.5	2.367	7.54	46	4.85	36.62	0.87	57.66	1.37	6.06	8.30	1.5	0.953
14B	110114B22	10.8	2.534	1.02	96	6.86	41.55	2.37	49.22	1.37	6.06	8.30	2.5	4.445
14C	110114C24	6.4	2.427	5.20	96	6.47	43.86	2.86	46.81	1.37	6.06	8.30	1.8	1.905

Table 5.1 : Field / Lab data for test samples(continued)

Section Site	Sample Designation Number	AC Thick (cm)	Bulk Sp Gty	Air Voids %	Rec Pen	Percent by Weight				Service Life (years)	Yearly ESALS (million)	Cum ESALS (million)	Grade %	Rut Depth (cm)
						AC	Sand	Fine	Agg					
16A	330216A101	10.2	2.452	4.22	98	4.60	40.48	2.48	52.44	5.93	3.11	13.32	2.5	3.810
17A	330217A108	14.0	2.545	0.59	91	5.82	31.03	3.25	59.91	5.00	2.58	9.63	3	5.398
17B	330217B109	7.9	2.429	5.13	68	5.51	37.90	3.28	53.32	5.00	2.58	9.63	1	1.270
18A	330218A07	18.4	2.349	8.26	63	5.33	43.31	2.35	49.01	7.41	2.65	12.60	0	3.810
18B	330218B08	19.1	2.333	8.87	43	4.60	40.94	3.40	51.06	7.41	2.65	12.60	0	1.270
19C	330219C014	14.0	2.384	6.87	33	3.16	40.23	2.14	54.47	5.00	2.97	11.13	0	1.143
20B	110320B35	17.1	2.367	7.54	72	3.57	29.37	3.98	63.07	0.27	4.51	1.21	0	0.953
21A	110321A32	17.8	2.401	6.21	72	3.84	29.61	3.86	62.70	0.27	1.48	0.40	0	0.635
22A	110322A43	19.4	2.390	6.64	45	4.34	35.25	2.88	57.53	0.36	2.29	0.83	0	0.635
23B	110323B42	20.0	2.400	6.23	45	4.36	38.03	2.92	54.70	0.36	1.84	0.66	0	0.635
24A	110324A53	18.7	2.315	9.56	36	3.92	44.08	2.06	49.94	0.42	2.17	0.91	0	0.953
24B	110324B54	18.7	2.367	7.55	36	3.48	34.96	1.22	60.34	0.42	2.17	0.91	0	0.445
25A	110325A53	21.6	2.340	8.61	36	4.15	40.90	3.83	51.12	0.42	1.48	0.62	0	0.635
26B	110326B88	21.6	2.439	4.73	43	5.07	42.56	3.87	48.50	0.67	2.02	1.35	0	1.270
27A	110327A82	24.1	2.350	8.19	35	3.40	43.55	4.04	49.01	0.67	1.44	0.96	1.5	0.635
27B	110327B88	24.1	2.382	6.96	35	4.47	37.99	4.07	53.47	0.67	1.44	0.96	2.75	1.397
28A	110328A98	20.3	2.410	5.87	35	4.82	45.71	4.33	45.14	1.42	2.07	2.94	2.2	0.635
29A	110329A98	22.9	2.411	5.82	35	4.04	44.53	4.47	46.96	1.42	2.16	3.07	1.3	1.270
30A	110330A124	21.6	2.419	5.51	42	4.51	41.28	4.08	50.13	1.83	1.85	3.19	2.15	0.635
31B	110331B123	20.3	2.472	3.45	42	4.86	43.16	3.47	48.51	1.83	1.71	2.97	1	0.635
31C	110331C124	21.0	2.454	4.14	46	5.22	45.31	3.42	46.04	1.83	1.71	2.97	2.15	2.540

Table 5.1 : Field / Lab data for test samples(continued)

Section Site	Sample Designation Number	AC Thick (cm)	Bulk Sp Gty	Air Voids %	Rec Pen	Percent by Weight				Service Life (years)	Yearly ESALS (million)	Cum ESALS (million)	Grade %	Rut Depth (cm)
						AC	Sand	Fine	Agg					
32B	110332B132	10.8	2.429	5.12	40	5.02	43.75	4.45	46.79	1.43	1.85	3.19	0	0.953
33A	110333A131	24.1	2.356	7.98	32	4.27	44.50	3.47	47.76	0.63	1.79	1.12	0	0.635
34A	110334A145	12.7	2.411	5.81	46	4.04	41.57	4.16	50.23	1.42	4.11	5.84	0	0.635
34B	110334B153	15.2	2.476	3.30	92	4.64	32.44	2.21	60.71	1.42	4.11	5.84	1	2.540
36A	110336A175	15.9	2.450	4.30	46	4.62	28.71	3.93	62.74	1.42	3.87	5.34	0	1.270
37A	110337A159	21.3	2.323	9.27	42	4.26	43.91	4.16	47.68	0.35	1.77	0.62	0	0.318
38A	110338A176	14.6	2.259	11.76	65	4.45	38.58	2.30	54.66	0.83	3.38	2.80	1.2	1.207
38B	110338B175	16.5	2.464	3.75	94	4.71	36.38	2.50	56.41	0.83	3.38	2.80	1.75	2.413
42A	110342A249	10.2	2.347	8.31	67	3.71	33.92	2.75	59.61	1.25	4.78	5.97	0	1.270
43A	110343A265	7.6	2.313	9.66	66	3.03	32.07	2.99	61.92	0.50	4.36	2.18	0	0.953
44A	110344A314	21.0	2.311	9.72	44	4.16	39.01	4.52	52.31	10.83	1.58	9.80	0	0.635
45A	110345A311	21.1	2.348	8.30	74	4.91	40.58	4.10	50.42	10.83	3.16	19.71	0	0.635
45B	110345B312	20.6	2.384	6.87	92	5.83	40.11	4.11	49.96	10.83	3.16	19.71	0	1.080
45C	110345C313	21.0	2.466	3.67	87	5.74	40.79	2.97	50.50	10.83	3.16	19.71	0	3.810
46A	110346A363	21.1	2.323	9.30	43	4.82	38.72	3.94	52.54	11.67	1.42	10.19	0	0.317
46B	110346B364	21.0	2.360	7.82	95	4.75	38.74	3.65	52.86	11.67	1.42	10.19	0	1.905
47B	110347B365	20.3	2.404	6.10	85	4.44	36.64	3.48	55.44	11.92	2.60	18.61	0	1.270
47C	110347C364	20.3	2.475	3.32	95	5.25	34.39	3.37	56.98	11.92	2.60	18.61	0	4.128
48A	110348A414	21.6	2.431	5.04	53	5.18	31.00	4.22	59.60	10.42	2.56	14.92	0	1.270
48B	110348B415	21.6	2.462	3.83	53	5.26	35.47	2.97	56.29	10.42	2.56	14.92	0	3.810
49A	110349A418	21.6	2.486	2.89	84	5.31	33.20	2.75	58.74	10.50	3.42	23.50	1	5.715

Table 5.1 : Field / Lab data for test samples(continued)

Section Site	Sample Designation Number	AC Thick (cm)	Bulk Sp Gty	Air Voids %	Rec Pen	Percent by Weight				Service Life (years)	Yearly ESALS (million)	Cum ESALS (million)	Grade %	Rut Depth (cm)
						AC	Sand	Fine	Agg					
50A	220350A3153	6.7	2.269	11.40	33	4.37	34.48	4.93	56.21	4.83	0.16	0.91	0	0.635
51A	220351A	5.1	2.510	2.00	36	5.22	38.43	3.67	52.68	1.76	0.59	1.03	0	0.635
52A	220352A27215	12.1	2.390	6.63	35	4.22	28.98	3.85	62.95	3.50	1.27	3.42	0	1.270
52B	220352B27225	9.2	2.351	8.15	35	3.33	35.31	4.13	57.23	3.50	1.27	3.42	0	0.508
53A	220353A2061	10.8	2.431	5.05	68	5.19	42.18	4.72	47.91	3.08	0.51	1.25	0	0.635
54A	220454A1213	5.1	2.438	4.76	37	4.53	51.46	4.17	39.84	7.42	0.90	4.30	0	0.635
55A	110155A453	11.4	2.424	5.32	64	4.73	42.78	3.86	48.64	1.25	0.67	0.83	2	0.889
58A	110458A14	5.1	2.265	11.52	83	2.70	32.95	3.82	60.53	1.58	2.88	4.55	0	1.016
58C	110458C19	14.9	2.432	5.01	83	3.81	29.28	3.25	63.66	1.58	2.88	4.55	0	1.842
59A	110459A46	11.4	2.533	1.05	66	4.95	40.44	2.81	51.79	1.58	2.82	4.45	2	3.810
59B	110459B47	8.9	2.408	5.94	66	4.87	38.12	2.97	54.04	1.58	2.82	4.45	0	0.635
61A	330261A014	5.1	2.526	1.33	64	5.65	39.16	4.26	50.94	6.00	2.97	11.13	0	4.128
Average		**14.58**	**2.413**	**5.76**	**57**	**4.6**	**39.45**	**3.43**	**52.46**	**4.31**	**2.69**		**0.56**	

Table 5.2 : Descriptive Statistics of table 5.1

Variables	Mean	Std. Deviation
AC%	4.66	0.86
AV	5.70	2.79
AC thick	5.73	2.33
Agg	52.40	5.87
value of fine	3.48	0.89
Sand	39.46	5.37
Rec Pen	56.50	23.33
Grade	0.53	0.90
Cumulative ESALs	6.89	5.64
Rut depth	1.60	1.29

In the correlation matrix of Table 5.3, the positive correlation of ESAL to the AC layer thickness shows that highway agencies in Pakistan, are designing thicker pavements for higher ESALs. The correlation is not significant which depicts that this principle is not being followed in totality and that costs are playing a major role.

The positive correlation with the percent asphalt content, recovered asphalt penetration and negative correlation with the percent aggregate, substantiate the fact that in Pakistan, better materials are not being used for higher volume roads. Again, pavement decisions are being driven by cost, not by experiences. It has already been proven (16,19,21,22) that the performance of AC mixes depends on providing adequate aggregate interlock to distribute the wheel load. Increase in the percent aggregate, reduces the amount of sand and fine in the AC mix. This leads to a better aggregate face to face contact and reduces the ball bearing effect due to the presence of excessive sand and fine in the AC mix.

The above scenario implies that the statistical analysis between the dependent and independent variables must be conducted on the basis of engineering knowledge rather than on the basis of statistics alone. This point can be illustrated by examination of the values of the correlation coefficients between the dependent variables (rut depth) and the independent variables (material properties, traffic volume, and service life).

Such an examination leads to the following observations:

1. The rut depth increases with increasing the number of cumulative ESAL. This observation was expected and it is consistent with that reported in the literature. The reason for this is that pavement rutting is mainly caused by wheel loads and is accelerated by material and environmental factors. The correlation is significant.

Table 5.3 : Correlation matrix for the 75 extracted core locations.

		cum esal	ac	air voids	ac thick	sam gty	agg	sand	fines	grade	rec pen	rut dep
Pearson Corr	cum esals	1.00										
	ac %	0.48	1.00									
	air voids	-0.35	-0.57	1.00								
	ac thick	0.10	-0.08	0.12	1.00							
	sam gty	0.35	0.57	-1.00	-0.12	1.00						
	agg	-0.03	-0.39	0.04	0.09	-0.04	1.00					
	sand	-0.04	0.24	0.03	-0.08	-0.03	-0.98	1.00				
	fines	-0.05	0.12	0.07	-0.01	-0.07	-0.33	0.17	1.00			
	grade	-0.13	0.20	-0.31	0.11	0.31	-0.03	0.04	-0.24	1.00		
	rec pen	0.45	0.39	-0.34	-0.07	0.34	0.27	-0.33	-0.20	0.27	1.00	
	rut depth	0.54	0.51	-0.64	-0.03	0.64	0.11	-0.16	-0.27	0.37	0.60	1.00

2. The rut depth increases with increase in the longitudinal grade of the pavement. This is consistent with the observations (pavements on grade are experiencing higher ruts). The experience interpretation is that, higher grade causes lower truck speed. This implies that the pavement is being subjected to load for longer period of time. Since, the plastic deformation increases with time (creep), lower truck speed causes higher rut.

3. The rut depth increases with the increase in the recovered asphalt penetration. This observation is reasonable and it is consistent with the literature. Softer asphalts produce softer asphalt mixes with lower resistance to rutting. The correlation is partially significant.

4. The rut depth increases with an increase in the asphalt content in the mix. This was expected and it is consistent with the literature because higher AC contents cause more lubrication between the aggregate and thus results lower friction. The percent asphalt content has higher negative collinearity (0.57) with air voids which reflects that increase in AC % decreases air voids, thus resulting in rutting.

5. The rut depth increases with increasing percent of coarse aggregate and with decreasing percent sand contents in the mix. Both observations are not reasonable and they cannot be supported by engineering principles. The reason for the two observation is statistical in nature. Higher coarse aggregate contents in asphalt mixes has lower surface area and hence lower AC contents. It can be seen from table 5.3 that sand and aggregate contents have collinearities with AC content in the

order of 0.24 and 0.39 respectively. The implication of the discussion above is that the effect of the AC content on rut depth needs to be examined very carefully. The effect in the final statistical model will be affected by this collinearity.

6. The rut depth increases with decreasing percent fine (passing sieve number 200) content. This observation is not reasonable and it is not consistent with that reported in the literature. The herein the same as that reported in item 5 for the sand and coarse aggregate contents.

7. Finally, the rut depth increases with increasing bulk specific gravity. This observation is partially reasonable. This quantity is directly used to calculate the air voids. It is also evident from figure 5.3 that air voids have highest negative correlation with specific gravity (more specific gravity less of air voids). The air voids have parabolic/polynomial relationship with rut depth. Upto some level of air voids rut depth has negative and then positive correlation. Moreover AC % also has significant negative correlation with air voids which means that higher AC %, lower air voids.

The above observations indicate that collinearities exist among some independent variables. Stratification of the data to resolve the collinearity problem is not possible because of the nature of the field experiment. Hence, some other solution must be found before balanced statistical and engineering analysis can be conducted.

In order to alleviate the problem and to analyze the rut depth as a function of the material variables without a significant influence by the independent variables on each other, the following corrective measures were adopted.

1. The collinearity between the sand and aggregate contents. This can be eliminated by expressing the percent coarse aggregate contents in terms of the fine and sand contents as follows:

   $$AGG = 100 - (Fine + Sand)$$

   where AGG, Fine, and Sand are the percent aggregate, fine, and sand contents, respectively.

2. The collinearity between the air voids (AV) and the percent AC content (AC). The reason for this degree of collinearity was explained earlier. Unfortunately, because of the nature of this dependency, this collinearity problem cannot be resolved by expressing one variable in terms of the other. Thus, only one of the variables will be included in the model.

Some of the collinearity problems cannot be resolved prior to the statistical analysis. Consequently, several statistical analysis were conducted whereby various transformations were used to express each independent variable. It was found that the elimination of certain variable from the pool of variables did not cause any significant bias in the resulting model. Therefore, those variables were not included in the final analysis.

Based on the results of the statistical analyses presented in Tables 5.3 and the above discussion, and after the elimination of some variables from the pool , statistical analysis was conducted to obtain a regression equation relating the dependent and independent variables. This analysis is presented in the next section.

5.3.4 Regression Equation

Prior to the commencement of the statistical analysis, the dependent variable (Rut Depth) was plotted against each of the independent variables. In view of the plots, the trend between the dependent and each of the independent variables was observed. The observed trend of each independent variable was then modeled by using several transformations (e.g. linear, logarithmic, cubical, exponential, parabolic and polynomial). The form that yielded the lowest PIN value, the highest coefficient of correlation, highest significance level and lowest standard error in the presence of the other independent variables was selected for inclusion in the model.

Table 5.4 provides a list of the values of the correlation coefficients for all variables that satisfied the variable inclusion criteria (PIN = 0.05) in the final model. The variables included in the model in order of their significance are:

1. the percent air voids (AV) of the AC mix which is included in a polynomial form as AV and AV^2;
2. penetration of recovered asphalt in the form of $(\text{REC PEN})^{0.5}$;
3. gradient of the road section in the form of $(\text{GRADE})^{2.25}$;
4. cumulative ESALs (cum esal) that has been experienced by the pavement since construction or rehabilitation in the form of $(\text{ESAL})^{0.75}$.

The degree of collinearity between the independent variables listed in Table 5.4 were examined. It is evident from the results, that the terms "AV^2 & AV^3 have the highiest collinearity. Because of this high collinearity, the term AV^3 was eliminated from the final model. Table 5.5 provides a summary of the values of the coefficients of correlation and standard errors for each step of stepwise method. Table 5.6 provides, for each independent variable, a list of the six eigenvalues and condition indices and the percent of variance attributed to each eigenvalue. The results indicate that the different terms of air void, (AV and AV^2) have high variance proportion values for the 6th eigen value thus indicating significant multicollinearity. A similar conclusion can also be made based on the tolerance and VIF values listed in Table 5.7. These diagnostics indicate that a significant degree of collinearity exist only among the various polynomial terms of the same independent variable (the percent air void AV). Note that the presence of such collinearity does not affect the estimated parameter of the other independent variables.

The results of the regression analysis are listed in Table 5.7. All the variables that are included in the resulting equation satisfy the specified criterion for the variable inclusion. It can be seen from the Table that all the variables have a "T" significance value of less than 0.05. Hence, they are statistically significant variables. The results of the regression analysis listed in Table 5.8 indicates that the standard error (S.E.) of the resulting model is 0.203, the value of the coefficient of determination (R^2) is 0.852 and the adjusted R^2 is 0.840.

The numbers listed under the column designated "B" in Table 5.8 are the regression constants for the indicated variables. Table 5.8 also provides a list of the regression constants and the values of the R^2 and the standard error obtained from each step of the stepwise method of the SPSS computer program. This percentage increases, as it should be expected, with the inclusion of more variables in the model. The final model can be expressed as follows:

Table 5.4 : Correlation matrix for the final variables.

		rut depth	av	av^2	av^3	$esal^{.75}$	$grade^{2.25}$	rec $pen^{.5}$
Pearson Correlation	rut depth	1.00						
	av	-0.68	1.00					
	av^2	-0.51	0.96	1.00				
	av^3	-0.51	0.96	1.00	1.00			
	$esal^{.75}$	0.53	-0.37	-0.30	-0.30	1.00		
	$grade^{2.25}$	0.47	-0.32	-0.26	-0.26	-0.05	1.00	
	rec $pen^{.5}$	0.58	-0.32	-0.25	-0.25	0.43	0.25	1.00

Table 5.5: Model Summary

Model	R	R Square	Adjusted R Square	Std. Error Estimate (cm)	Durbin-Watson
1	0.680	0.462	0.454	0.955	
2	0.846	0.716	0.708	0.698	
3	0.898	0.806	0.797	0.582	
4	0.908	0.824	0.813	0.558	
5	0.923	0.852	0.840	0.516	2.056

Table 5.6 : **Collinearity diagnostics (eigenvalue, condition indices and the proportion of the variance attributable to the eigenvalue).**

Model	Dimension	Eigen value	Cond Index	Variance Proportions					
				Constant	av	av^2	rec $pen^{.5}$	$grade^{2.25}$	$esal^{.75}$
1	1	1.90	1	0.050	0.050				
	2	0.10	4.345	0.950	0.950				
2	1	2.74	1.000	0.008	0.002	0.004			
	2	0.25	3.310	0.167	0.001	0.057			
	3	0.01	16.901	0.824	0.997	0.939			
3	1	3.57	1.000	0.001	0.001	0.002	0.002		
	2	0.40	3.007	0.008	0.003	0.039	0.026		
	3	0.02	12.436	0.203	0.123	0.203	0.619		
	4	0.01	22.212	0.788	0.873	0.755	0.352		
4	1	3.73	1.000	0.001	0.001	0.002	0.002	0.010	
	2	0.94	1.998	0.000	0.001	0.004	0.000	0.593	
	3	0.30	3.528	0.011	0.002	0.040	0.031	0.329	
	4	0.02	12.930	0.210	0.111	0.180	0.672	0.033	
	5	0.01	23.097	0.779	0.886	0.774	0.294	0.035	
5	1	4.40	1.000	0.001	0.001	0.001	0.001	0.007	0.007
	2	0.94	2.159	0.000	0.001	0.005	0.000	0.494	0.002
	3	0.52	2.915	0.000	0.002	0.015	0.002	0.226	0.164
	4	0.11	6.273	0.036	0.000	0.049	0.049	0.146	0.661
	5	0.02	14.825	0.200	0.089	0.127	0.801	0.066	0.114
	6	0.006	25.657	0.762	0.908	0.802	0.147	0.062	0.050

$$\textbf{RUT DEPTH} = 2.350 - 0.830\ AV + 0.0506(AV)^2 + 0.193(REC\ PEN)^{.5} + 0.0936(GRADE)^{2.25} + 0.102(ESAL)^{.75} \qquad (5.4)$$

$R^2 = .852$, SE = .203, F = 76. 864 , $F_{(sig)} = .000$

Based on the statistics of the model (equation 5.4), the following observations were noted:

1. The null hypothesis (no linear relationship between the independent variables and rut depth) was rejected and it was concluded that at a probability $F_{(sig)}$ of zero, 85.2 percent of the variation of the dependent variable (rut depth) is explained by the independent variables included in the equation.
2. The largest value of "t" significance is .0007. This implies that all the variables in the model are statistically significant. In view of the 95% confidence interval (default value), t significance of the variables be lower than 0.05.

In addition, the following statistics were calculated to study the values of the residuals and the predicted rut depth values and are listed in Table 5.9.

1. PRED = the unstandardized predicted values;
2. RESID = the unstandardized residual values;
3. ZPRED = the standardized predicted values;
4. ZRESID = the standardized residual values; and
5. D.W.S. = the Durbin Watson statistics.

Table 5.7 : Results of regression analysis and Collinearity diagnostics TOL and VIF.

Models		Unstand Coefficients		Stand Coefficients	T	T sig	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	Constant	3.396	0.256		13.28	5.1E-18		
	av	-0.315	0.040	-0.680	-7.80	1.5E-11	1.000	1.00
2	Constant	5.295	0.304		17.43	5.5E-18		
	av	-1.111	0.105	-2.395	-10.62	5.5E-18	0.080	12.56
	av^2	0.065	0.008	1.788	7.93	8.0E-12	0.080	12.56
3	Constant	2.827	0.506		5.59	4.2E-07		
	av	-0.972	0.091	-2.096	-10.73	5.9E-18	0.074	13.56
	av^2	0.058	0.007	1.582	8.26	1.2E-12	0.077	13.03
	rec $pen^{.5}$	0.269	0.048	0.321	5.64	3.5E-07	0.867	1.15
4	Constant	2.657	0.490		5.42	8.3E-07		
	av	-0.915	0.090	-1.973	-10.22	6.3E-18	0.069	14.39
	av^2	0.055	0.007	1.497	8.03	5.2E-12	0.074	13.43
	rec $pen^{.5}$	0.253	0.046	0.302	5.48	6.6E-07	0.853	1.17
	$grade^{2.25}$	0.067	0.026	0.146	2.63	1.0E-02	0.847	1.18
5	Constant	2.35	0.461		5.10	3.1E-06		
	av	-0.83	0.086	-1.789	-9.62	1.4E-16	0.064	15.62
	av^2	0.050	0.006	1.378	7.84	1.6E-11	0.072	13.94
	rec $pen^{.5}$	0.193	0.046	0.230	4.19	8.4E-05	0.734	1.36
	$grade^{2.25}$	0.093	0.025	0.203	3.78	3.3E-04	0.770	1.30
	$esal^{.75}$	0.102	0.029	0.203	3.54	7.4E-04	0.675	1.48

The Durbin-Watson statistics (2.056) of equation 5.4 listed in Table 5.5 signifies a satisfactory non autocorrelation between the residuals. If variables in the model have autocorrelation, the values as well as the Standard errors of the parameters estimates are affected. Values of the parameters will be greater than normal and SSE will be under estimated. Prediction power of the model will be poor. Acceptable level of Durbin Wastin is from 1.50 to 2.50 (67).

The analysis of residual statistics shown in table 5.9 indicates that this model is capable of predicting a rut depth from 0.153 inch to 5.097 centimeters. The maximum standard error of the predicted values of maximum rut depth is 0.295 which is reasonable. The process of standardization includes averaging of out lying point inclusion of influential data. In table 5.9, the minimum and maximum values indicate the spread of the values and standard deviation helps in understanding the nature of the dispersion of the predicted values.

5.3.4.1 Shortcomings of the Model

Although the rut model, presented in equation 5.4, has a relatively high R^2 value (0.852), but its engineering interpretation is problematic. For example, the model indicates that pavement rutting is possible even without any traffic load (ESALs = 0). The reason for this could be inaccuracy of the ESAL data which was obtained from NTRC and NHA of Pakistan. It should be noted that other forms of the rut prediction equations were also tried. For example, several forms where, the ESALs was a multiplier or in logarithmic functions, were used. Unfortunately, the resulting statistical equations were very poor with R^2 values of less than 0.3.

Table 5.8: Regression matrix for depth of rutting.

No.	Constants	Regression Coefficients					Statistics	
		AV	AV^2	Rec $Pen^{.5}$	$Grade^{2.25}$	$ESAL^{.75}$	R^2	S.E
1	3.396	-0.315					0.462	0.955
2	5.295	-1.111	0.0656				0.716	0.698
3	2.827	-0.972	0.0580	0.269			0.806	0.582
4	2.657	-0.915	0.0549	0.253	0.0672		0.824	0.558
5	2.350	-0.830	0.0506	0.193	0.0936	0.102	0.852	0.516

Table 5.9 : Residual statistics of equation 5.4

Statistics	Minimum	Maximum	Mean	Std. Deviation	N
Predicted Value	0.060	2.007	0.630	0.470	73
Std. Predicted Value	-1.214	2.931	0.000	1.000	73
Standard Error Pred Value	0.080	0.296	0.141	0.045	73
Adjusted Predicted Value	0.133	5.005	1.605	1.185	73
Residual	-1.543	1.246	0.000	0.498	73
Std. Residual	-2.989	2.414	0.000	0.965	73

Further, the inaccuracy of the ESALs data can also be depicted from table 5.1. It can be seen that the ESALs number varies substantially from one point to another on the same road. For example sections 44A & 45A are both located on N-5 highway within 2 to 3 km distance from each other. The ESALs for section 44A and 45A are 9.8 and 19.71 respectively.

5.4.5 Sensitivity Analysis and Engineering Interpretation

The statistical model presented by equation 5.4 was used to predict the average rut depths over the selected sites. Figure 5.1 depicts the values of the average predicted depths of rutting versus the average observed ones. The straight line in the Figure represents the percent errors between the predicted and the observed depths of rutting. It can be seen that the error in the predicted rut values is substantially for low rut & relatively accurate with high rut values. Since high pavement rutting is typically associated with higher ESALs, the error of the low rut values may indicate inaccuracy in the ESALs data. Examination of figure 5.2 indicates that 2/3 of the predicted rut depths are within 40 percent error of the observed values. This indicates that the statistical model is relatively accurate. One additional point should be made herein is that, the rut depth of a pavement structure is a function of several variables including the stiffness and thickness of the various pavement layers, the properties of the materials in the AC mix, the traffic volume, load, and environmental factors. In this study, only the asphalt mix properties and the traffic load and volume data are used.

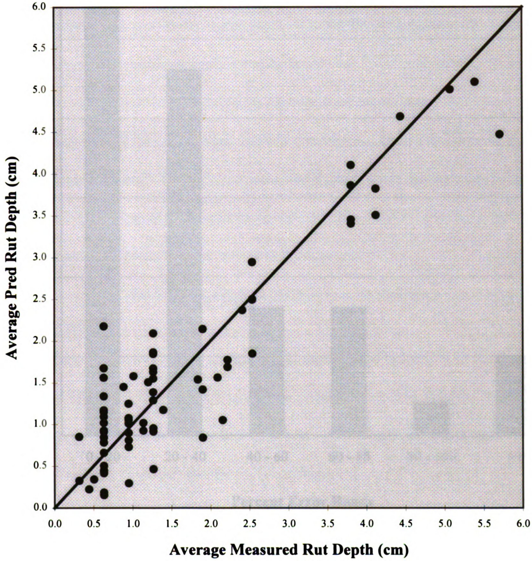

Figure 5.1 : Average measured versus the predicted rut depth at 73 sites.

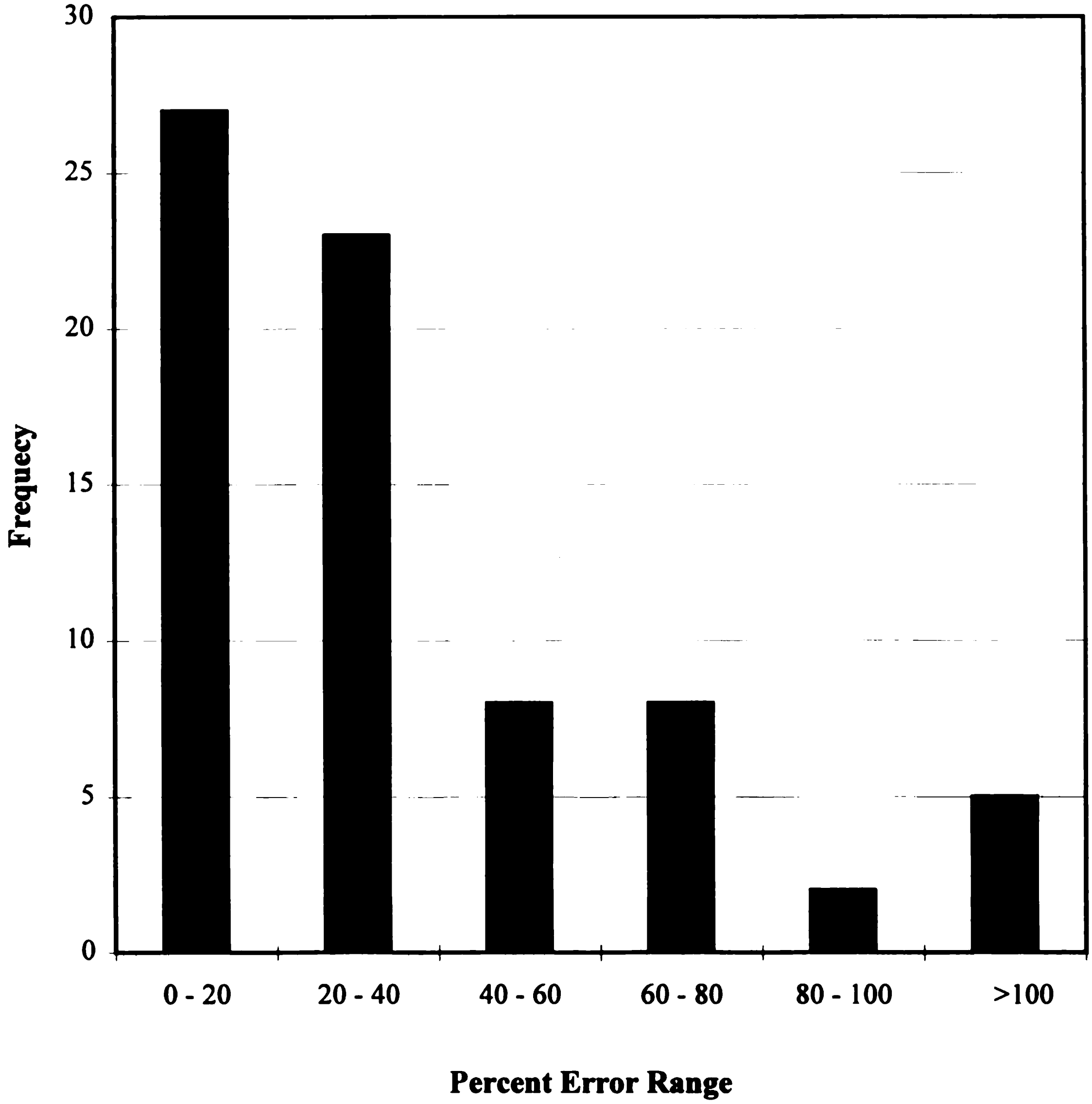

Figure 5.2: Percent difference between the average observed and the predicted rut depths for 73 sites.

Consequently, one should not expect the statistical model presented in equation 5.4 to fully explain the variations in the rut depths.

A sensitivity analysis of equation 5.4 was performed to determine the effect of each variable in the equation on the predicted rut depth at the 73 core locations. The sensitivity of the model to each variable was analyzed such that:

1. The values of three variables in the equation were held constant.
2. The values of the one constant variable were changed from a low, to a medium, to a high value and values of other two variables were kept constant over the values, within the range of values of that variable. Since most of the sites have zero longitudinal grade, so its average was taken as zero.
3. The value of the fourth variable in question was varied from one end of its range to the other.

Results of the sensitivity analysis are presented and discussed below.

Air Voids - Figure 5.3 depicts the sensitivity of the predicted rut depth to the percent air voids for average values of grade (since most of the data points have zero grade, so average is replaced by zero value) and Rec Pen (57) and three levels of ESALs (1.5, 8, 15).

Examination of the figure reveals that:

1. Decreasing the percent air voids below three percent level causes significant increase in the rut depth.
2. The rut depth increases as the percent air voids increases above eight percent level.
3. The rut depth increases with increasing the ESALs.

The result presented in figure 5.3 and three observations stated above should be viewed with extreme caution. The reason is that the air voids is highly correlated to the percent asphalt content as it can be seen from table 5.3. This correlation or collinearity tends to distort the true effect of the air voids on rut depth.

Field data suggests that the pavement constructed with air voids of less than three percent, experiences excessive rutting. On the other hand, pavements constructed with 3 to 6 percent air voids show low rutting potential. Pavements constructed with air voids higher than 6 percent will have higher rutting potential. The rut prediction model shows an optimum air voids of 8 percent. This represents significant departure from the field observations of optimum air voids of about 4.5 percent. Once again, the reason for such distraction is the collinearity between the AC content and the air voids & visa versa. Since AC contents cause an increase in the rut potential, the true effect of air voids on rutting cannot be obtained from the given data.

Recovered Asphalt Penetration - Figure 5.4 depicts the sensitivity of the predicted rut depth and the recovered asphalt penetration (Rec Pen) for average values of grade (zero) and air voids (5.76) and three levels of ESALS. It can be seen that the rut depth increases with an increase of recovered asphalt penetration. In view of this observation, lower grade asphalt should be used to retard rutting. However, low grade asphalts have high temperature cracking potential. Therefore, the selection of proper asphalt grade should be made after balancing the rutting and temperature cracking potentials. In this regard the new SHRP specifications concerning AC grades are recommended to be used in Pakistan.

In addition, a significant collinearity between ESALs and asphalt penetration exists (see table 5.3). Higher ESALs implies older pavements and longer oxidation time which causes lower recovered asphalt penetrations. Unfortunately, such collinearity can be resolved but by substantially increasing the sample size so that the data can be stratified. This is possible if and only if, a complete pavement management system data bank exists. This not the case in Pakistan.

ESALs - Figure 5.5 depicts the effects of ESALs on rut depth for average value of Gradient(zero) and air voids (5.76) three levels of Rec Pen. It is evident from the figure that rut depth increases with increasing number of 18 kip ESALs. The rate of increase decreases with increasing ESALs. This signifies that rutting due to densification is significant after opening the pavement to traffic. The rate of rutting decreases as the AC layer is subjected to a higher degree of densification (compaction) due to traffic.

Gradient - Figure 5.6 depicts the sensitivity of the predicted rut depth to the gradient(grade) of the road section with average values of Rec Pen (57 and air voids (5.76) and three levels of ESALs.. The rut depth increases slightly in the initial stage of the gradient and then rises sharply and substantially after 1.5 percent of grade. These observations were expected because higher grades retard the speed of vehicles. Lower speed induces longer loading time and higher rut. Excessive overloading is the normal practice in Pakistan. The effect of overloading is enhanced many fold, in areas where roads also have a longitudinal grade.

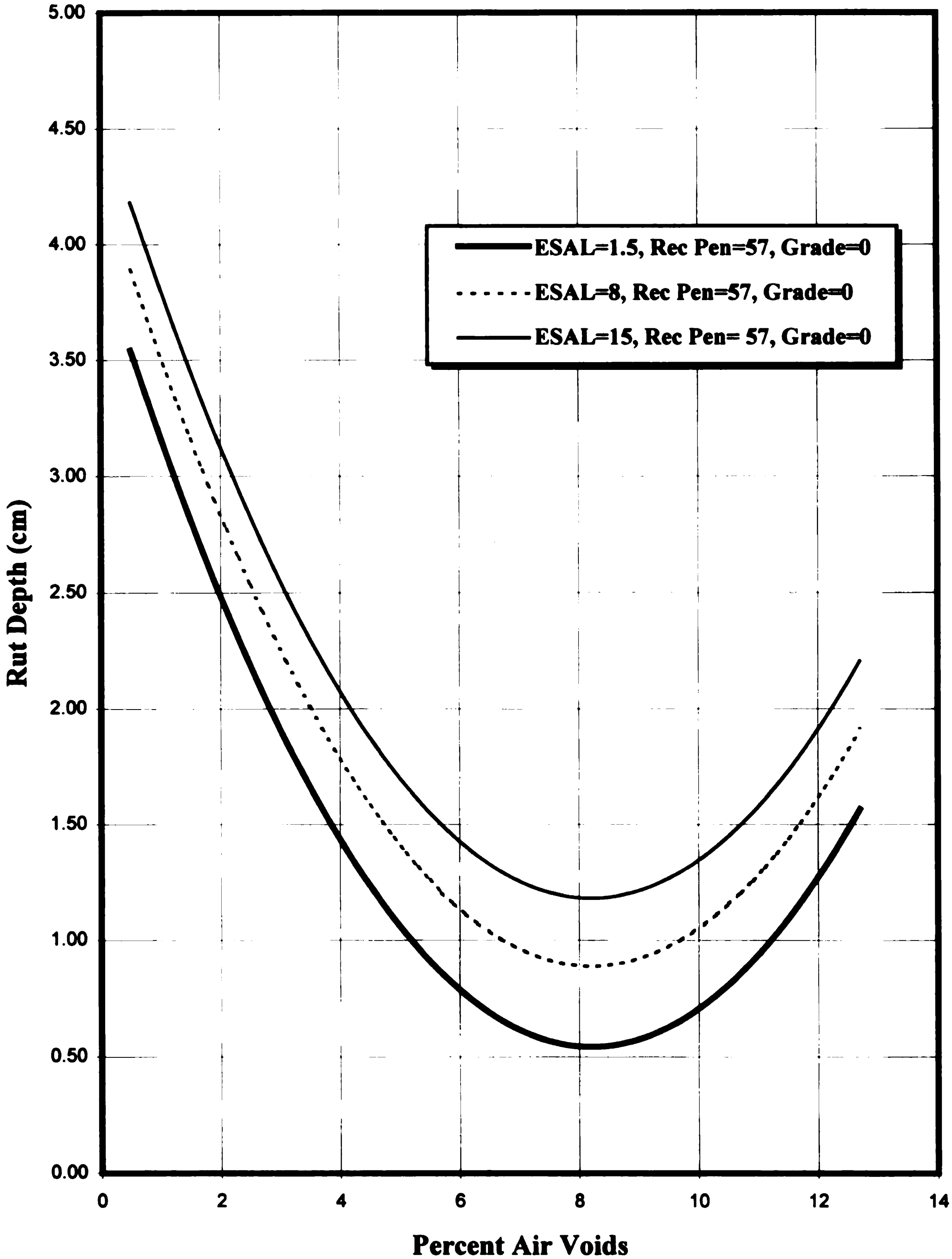

Figure 5.3: Rut depth as a function of air voids for average values of Rec Pen, Grade and three levels of ESALS (millions).

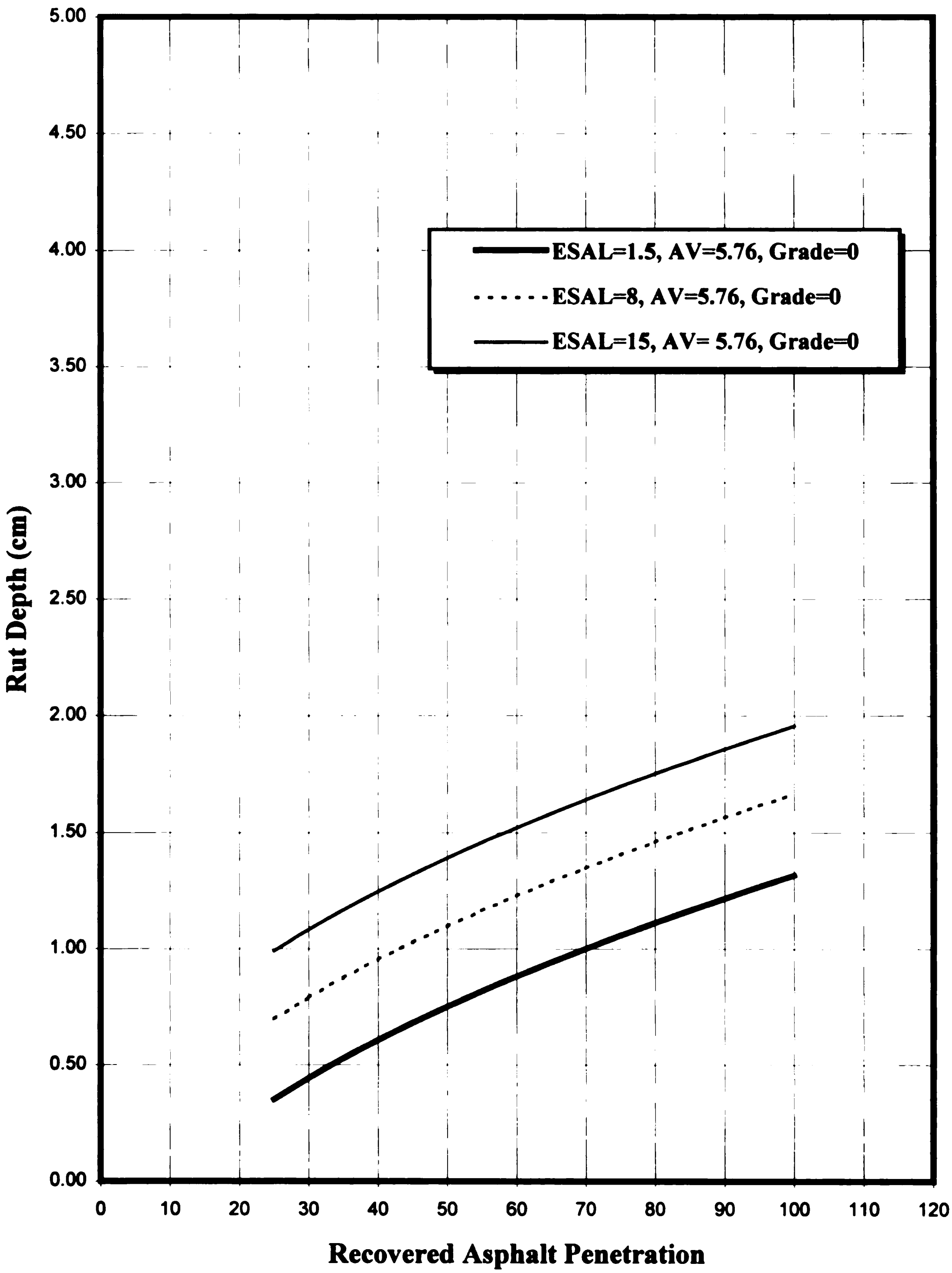

Figure 5.4: Rut depth as a function of Rec Pen for average values of air voids, Grade and three levels of ESALS(millions).

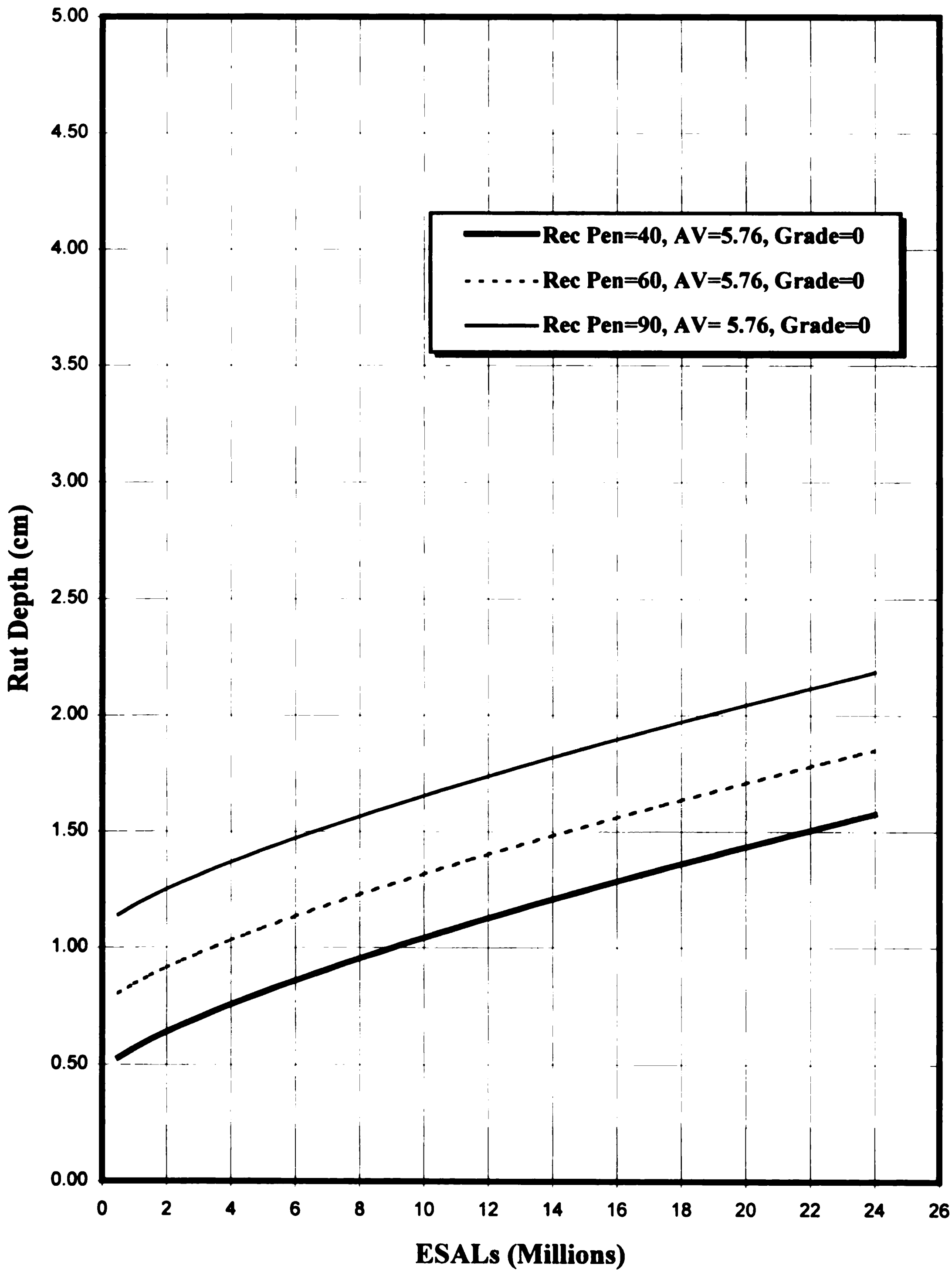

Figure 5.5: Rut depth as a function of ESALs for average values of air voids, Grade and three levels of Rec Pen.

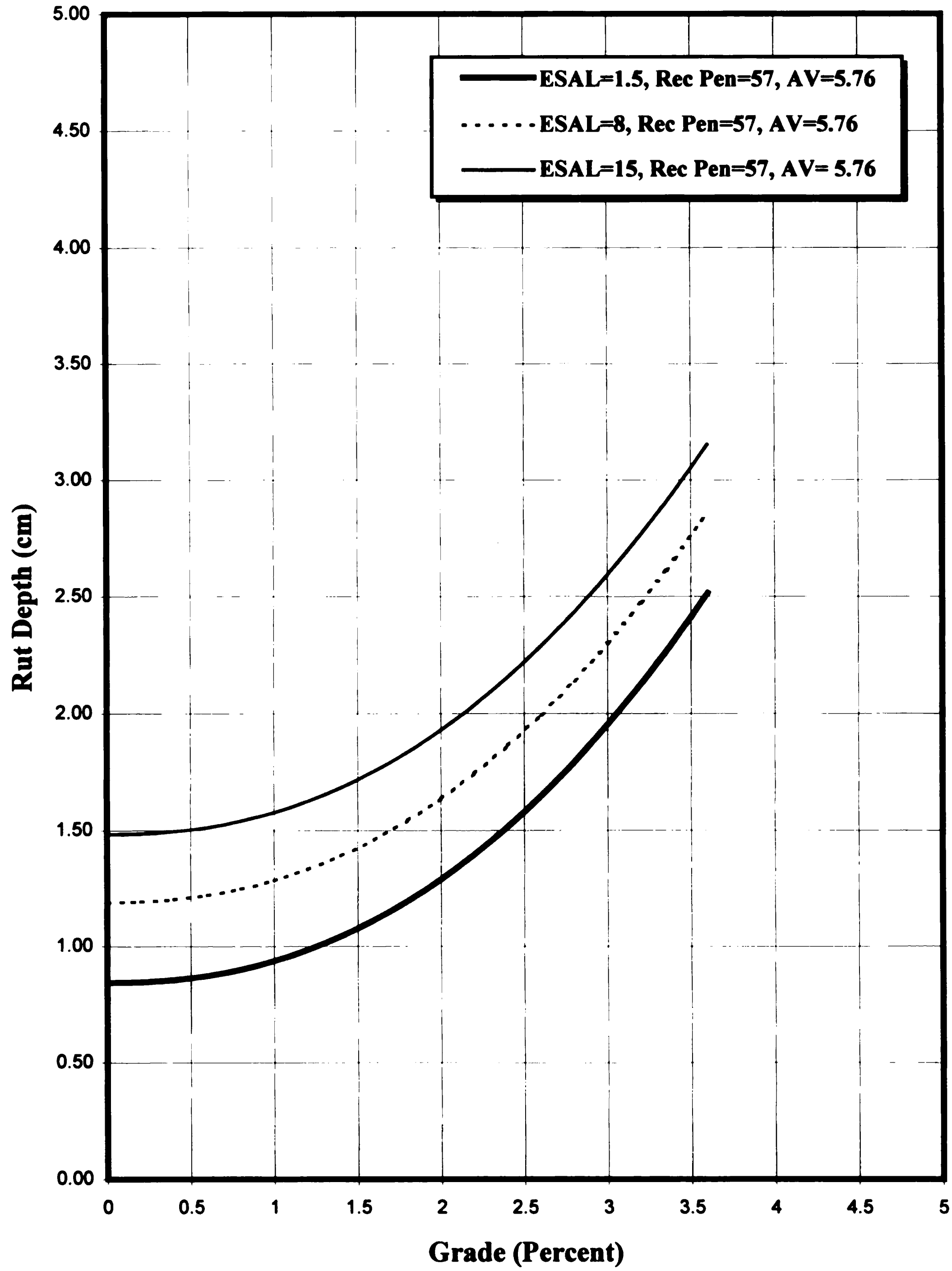

Figure 5.6: Rut depth as a function of grade for average values of Rec pen, air voids and three levels of ESALS.

CHAPTER 6

SUMMARY, CONCLUSION AND RECOMMENDATIONS

6.1 SUMMARY

The effects of the asphalt mix variables on pavement rutting were investigated in this study. Full depths AC cores were obtained from selected pavement sections. The properties of the AC mixes were determined as described in chapter 4.

A rut prediction model was developed which is based on the asphalt mix properties, cumulative 18 kip ESALs and gradient of the road sections. Sensitivity analysis of the model was conducted to determine the behavior and influence of each variable on pavement rutting. It is shown that the most significant variables affecting pavement rutting are air voids, recovered asphalt penetration, cumulative ESALs and gradient of the road sections.

6.2 CONCLUSIONS

Based on the test data, the analysis and literature on the subject, the following conclusions were drawn:

1. Rut resistance of the AC mixes can be maximized by specifying a maximum of 3 to 5 percent air voids.

2. Asphalt viscosity/penetration has shown definite impact on rutting. Softer asphalt grade are most susceptible to rutting.

3. The number of cumulative 18 kip ESALs effect pavement rutting. Sections experiencing higher volumes of traffic/loads in a short span of service life have shown extensive amount of rutting.

4. Gradient of the road sections (longitudinal slope) causes higher rut. Road grade impedes vehicles speed which causes longer loading time.

5. Significant variations in the thickness of the AC layers and AC mix properties (air voids etc.) were observed within one pavement section.

6.3 RECOMMENDATIONS

Based on the findings and conclusions of the study, the following recommendations are made:

1. To improve the rut resistance of AC mix, the air voids percentage between 3 to 5 percent be specified.

2. Measures be taken to rationalize and enforce the axle load limit to avoid excessive damage to roads.

3. Gradient specification or grade be reduced if economically feasible.

4. Quality assurance/quality control practices regarding pavement construction and AC mix manufacturing operations should be modified and enforced as lot of variation in pavement thicknesses and mix properties were found within the same section of road.

5. Fresh axle load and traffic volume surveys be undertaken to determine the accurate and updated records of truck factor, growth rates and traffic counts, which are required in pavement design. It will be more appropriate if weigh-in-motion devices are installed to monitor traffic volume and loads. Traffic record for newly constructed/rehabilitated roads should be maintained and pavement performance be monitored regularly.

5. The rut model presented in this thesis be expanded to include the other engineering properties of the AC mixes (e.g. elastic and plastic) and the properties of the other pavement layers and subgrade.

REFERENCES

1. Yoder E.J. and M. W. Witczak, "Principles of pavement design".A Wiley-Interscience Publication, John Wiley and Sons, Inc,.1975.

2. American Association of State, Highway, and Transportation Officials. "AASHTO Guide for design of pavement structures". Washington, D. C., 1986.

3. Jack Morris. "The prediction of permanent deformation and structural design method for flexible pavements". Department of civil Engineering, University of waterloo, Waterloo Ontario Canada.

4. Mukhtar H., "Reduction of pavement rutting and fatigue cracking". P.h.d Thesis 1993, Department of civil Engineering, Michigan State University USA.

5. Baladi G. Y.," "Integrated material and structural design method for flexible pavements," Vol. 1: Technical Report, FHWA, 1988.

6. Majidzadeh Kamran and Measorvic S, "A constituative model for asphalt concrete, application to rutting." Paper # 929386, 71th meeting TRB 1992.

7. Report 5, Highway Research Board Special Report 61E, " The ASSHO Road Test," 1962.

8. G. A. Huber and G. H. Heiman. "Effect of asphalt concrete parameter on rutting performance. A field investigation". AAPT. VOL. 56, 1987.

9. Jack Morris. Ralph C. G. Hass, Park Reilly, and Edward T. Hignell, "Permanent deformation in asphalt pavement can be predicted". APT. 1974, VOL. 43.

10. Parker F and Brown E R, "Effects of aggregate properties on flexible pavements rutting in ALABAMA." Paper # 910760, 70th meeting TRB 1991.

11. Harold Von Quintus and Thomas W. Kennedy, "AAMAS Mixture properties related to pavement performance". APT. Vol 58 1989.

12. Lee , K. W.and Al-Dhalaan, M. A., "Rutting, Asphalt Mix-Design, and Proposed Test Road in Saudi Arabia", Implications of Aggregate in the Design, Construction, and Performance of Flexible Pavements, ASTM STP 1016, H. G. Schreuders and C. R. Marek, Eds., American Society for Testing Materials, Philadelphia, 1989.

13. Hughes C. S. and G. W. Maupin, Jr., "Experimental bituminous mixes to minimize pavement rutting," APT Vol. 56, 1987.

14. Hsien H, Chen, Kurt M. Marshek, and Chhotel L, Saraf, "Effect of truck tire contact pressure distribution on the design of flexible pavement: A three dimensional finite element approach," Transportation Research Record (TRR), Transportation Research Board (TRB), 1095, 1986.

15. Baqur S. A." Analysis of Tire Pressures in Pakistan." MS Thesis Military College of Engineering Risalpur, 1993.

16. Bonquist R. R . "Effects of tire pressure on flexible pavement response and performance," Draft Report, Federal Highway Administration, Washington, D. C., 1989.

17. Harry A. Smith, "Truck tire characteristics and asphalt concrete rutting," Paper No. 910035, Transportation Research Board (TRB) 70TH Annual Meeting, 1991.

18. Richard. D. Barksdale, "Practical application of fatigue and rutting tests on bituminous based mixes". APT VOL.47,1978.

19. L.E Santucci and R. J. Schmidt, "The effects of asphalt properties on the fatigue resistance of asphalt paving mixtures". Asphalt Paving Technology VOL. 38, 1969.

20. Miller C. Ford, Jr. Pavement densification related to asphalt mix characteristics". TRR.1178, 1988.

21. Jon a Epps and Carl L. Monismith," Influence of mix variables on flexural fatigue properties of AC. "APT VOL. 38.

22. E. R. Brown, "Mix design and construction of asphalt concrete to support high tire pressures". Presented at AASHTO/FHWA Symposium on high pressure truck tires, Austin, Texas Feb. 1987.

23. Carpenter H. Samuel, " Permanent deformation: Field Evaluation" Paper # 930768, Transportation Research Board ,72nd meeting, 1993.

24. Lister W. N and Addis R. R, "Field observations of rutting and their practical implications" ,Transport and Research Laboratory, U.K. Department of the Environment Crowthorne, England.

25. "Essential physical properties of Sand, Gravel, Slag and Broken Stone for use in Bituminous Pavements". Better Roads and Streets. VOL. 6, No. 3.

26. "Mix design methods for asphalt concrete". Asphalt institute manual series NO. 2(MS-2).

27. Hicks, R.G. and David C. ESCH, "State of the Art on rutting in asphalt concrete". Alaska DOT and PF Fairbanks, AK 99701.

28. E R Brown and Charlas E. Bassett, " Effects of maximum aggregate size on rutting potential and other properties of asphalt aggregate mixtures". TRR. 1259, 1990.

29. Kamyar Mahboub and David L. Allen, "Characterization of rutting potential of large stone asphalt mixes in Kentucky ". TRR 1259 1990.

30. Moreland Herrin and W.H. Goetz, "Effect of aggregate shape on stability of bituminous mixes". Joint Highways Research Projects, Purdue University. HRB. Proceedings 33rd Annual Meeting 1954.

31. Mohammed Osamn Khalifa and Moreland Herrin, "The behavior of asphaltic concrete constructed with large sized aggregate". APT Vol.39.

32. Kandhal P S, Stephen A Cross and E R Brown, "Heavy duty asphalt pavements in Pennsylavinia: Evaluation for rutting."

33. W.S. Mendenhall, Jr., Chairman, "Ad hoc task force on asphalt pavement rutting and stripping". August 14, 1987.

34. Joe W. Button, Dario Perdomo and Robert L. Lytton, " Influence of aggregate on rutting in asphalt concrete pavements". TRR 1259 1990.

35. M.A. Young and G. Y. Baladi, "Repeated load triaxial testing, state of the art". Michigan State University. Department of Civil Engineering.

36. Ronald E. Reese and Joseph L. California Desert Test Road - A step closer to performance based specifications published in Asphalt paving technology Vol 62, 1993.

37. Neil C. Kruts, Raj Siddharthan and Mary Stroup-Gradiner, "Investigation of rutting potential using static creep testing on polymer modified AC mixture". Paper # 91-0705, 70th meeting TRB 1991.

38. E. R. Brown and Stephen A. Cross, "A study of in-place rutting of asphalt pavements". APT. VOL.58 1989.

39. M.J. Hensley and Rita B. Leahy, "Asphalt concrete mixtures as related to pavement rutting: Case study". TRR. 1217 1989.

40. Dale S. Decker and Joseph L. Goodrich, " Asphalt cement properties related to pavement performance". APT VOL. 58 1989.

41. Collins J H, Bouldin M G Gells R and Berker A, " Improved performance of paving asphalt by Polymer modifications".

42. R.D. Barksdale, "Compressive stress pulse time in flexible pavements for use in dynamic testing". Highways Research Board (HRB). Highways Research Record (HRR) NO. 345, 1971.

43. R.D. Barksdale, "Laboratory Evaluation of rutting in base course material". Pro. 3rd International Conference on the structural design of asphalt pavements, London 1972.

44. W. Heukelon and A.J.G. Klomp, "Consideration of calculated strain at various depths in connection with the stability of asphalt pavements". Pro. 2nd International Conference on the structural design of asphalt pavement, Ann Arbor 1967.

44. J.E. Romain, "Rut depth prediction in asphalt pavement" Pro. 3rd International Conference on the structural design of asphalt pavement, London 1972.

46. Richard D. Barksdale and Gerald A.Lenards, "Predicting performance of bituminous surfaced pavements". Pro. 2nd International Conference on the structural design of asphalt pavement, Ann Arbor 1967.

47. Soussou, J.E and Moavenzadeh, F. "Statistical Characteristics of fatigue damage accumulation in flexible pavements". American Society of Testing and Material special technical papers No. 561 1973.

48. Rita B. Leahy and Matthew W. Witczak. "The influence of test conditions and asphalt concrete mix parameters on permanent deformation co-efficient Alfa and Mu,". APT Vol 60, 1991

49. Baladi G. Y., "Fatigue life and permanent deformation characteristics of asphalt concrete mixes" , TRR, TRB, 1227, 1989.

50. Cardoso S H and Witczak WM,"Pavement deformation for flexible pavements."

51. Matthews J M and Pandey B B, " Performance of flexible pavements."

52. Traffic Volume Data 1995 by NTRC Islamabad.

53. Traffic count NWFP 1992.

54. Traffic Survey Punjab 1992.

55. ALMEC Corporation Pacific Consultants International, "The Study on National Transportation Plan in the Islamic Republic of Pakistan". Final Report Volume II, NTRC (Growth Factors), February 1995.

56. Iqbal Zafar, "Axle load survey to establish latest truck factors between Taxila and Lahore" MS Thesis 1993.

57. Axle load study on National Highways 1995 by NTRC Islamabad.

58. Axle load studies in Punjab by Road research and material testing institute Lahore.

59. Tresca, H "Memoire Sur L' ecoulement Des Corps Solides, "Mem. Pres. Par div. Savants 18, 1868.

60. Venant B. de Saint, "Memoire sur L'establishment des Equantions Differentiales des Mouvements Interiors opers Dans lrd Corps Solids Ductiles au Dela des Limits au L'elasticite Pourrait les Ramener a Leur Premier Etat, "C.R. Acad. Sci. (Paris), 70,1870.\

61. Levy M., "Memories sue L'equations Generales des Mouvements interieurs des Corps Solides Ductiles au Dela des Limits ou L'elasticite Pourrait les Ramener a Leur Premier Etat," C.R. Acad. Sci. (Paris), 70, 1870.

62. W. Prager, "An Introduction to Plasticity," Addison-Wesley Publishing Company, Inc., London, England. 1959.

63. ERES Consultants, Inc."Pavement design principles and practices" Champaign Illinois, National Highway Institute, Washington, D.C., 1987.

64. Majidzadeh, U. and G. Ilves.Flexible pavement overlay design procedures, volume I, Evaluation and modification of the design methods" Final report FHWA-RD-82a32. 1981.

65. John O. Rawlings, "Applied regression analysis, a research tool". Wadsworthand Brooks/Cole, Statistics/Probability Series 1989.

66. David A. Belsley, Edwin Kuh, Roy E. Welsch, "Regression diagnostics, identifying influential data and sources of collinearity". Wiley series in probability and mathematical statistics.

67. Marija J. Noruses/SPSS Inc, "SPSS statistical data analysis".