AN ANALYSIS OF ECONOMIZATION BASED OPTIMIZATION OF FLEXIBLE PAVEMENT STRUCTURAL DESIGN

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This is to certify that the

Thesis titled

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This thesis is dedicated to my parents, siblings, mentors and friends

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ABSTRACT

Pakistan is a developing country with many budget constraints. Road construction industry acts as a backbone in the progress of any country yet road development needs huge budgets. In developing countries like Pakistan with all the budget constraints, it is important to introduce new cost saving techniques in road design and construction. Pavement thickness is a significant cost factor in any road project. This research work emphasizes on coming up with a cost effective design of flexible pavements by reducing the thickness of asphalt concrete layer without compromising on durability and longevity/service life of the pavement. For this purpose, multilayer pavement approach has been used to make the pavement economical by placing high quality material on top while low quality material on the bottom (base and sub base). Base and sub-base layer thickness will also be increased in order to reduce the asphalt concrete layer thickness; therefore, the strength requirements will be fulfilled. With a focus on ride quality as a major aspect of AC, the multilayer pavement approach can be made more economical by keeping its thickness to a minimum. AASHTO 1993 design guidelines for structural design of pavements are dependent on the application of load by different vehicles directly on pavement layers, depending upon their respective stiffness. Therefore, according to AASHTO design, it can be established that a reduction in thickness of the asphalt concrete layer will directly increase the thickness of base and sub base layers, hence adjusting the respective stresses and stiffness. To make the pavement structure more economical, this technique is very practical in a developing country like Pakistan where very high quality granular material is abundantly available. Due to its high construction cost, the thickness of asphalt concrete layer is a hot issue among highway agencies and engineering consultancies all over the world as asphalt concrete is an expensive material. If there is an appreciable impact on cost with no compromise on stability from the actual AASHTO design, then the AC thickness may be reconsidered and subsequently new local parameters of pavement thickness design may be set to save budget for highway construction using local materials. The existing design practices in road construction in Pakistan are reviewed and suitable recommendations are also provided. Optimization of flexible pavement thickness is carried out by using linear integer programming technique and for this purpose linear integer program solver (LiPs) software was used in order to find the optimal flexible pavement thickness configuration. AASHTO design method was used for pavement thickness design. Iterations were done to reach the desirable values of pavement thickness. Flexible pavement structural design analysis was carried out by using KENPAVE software and in the end a brief comparison is presented between AASHTO flexible pavement structural design and optimize flexible pavement structural design analysis results in order to draw out the final decision matrix. This study will also be a good viable solution for conservation of local materials and to provide a platform for future research and studies into making pavement thickness design economical.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF EQUATIONS

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
LiPs	Linear Integer Program Solver
NCHRP	National Cooperative Highway Research Program
MFPD	Michigan Flexible pavement structural design
SN	Structural Number
ESALs	Equivalent Single Axle Loads
CBR	California Bearing Ratio
NHA	National Highways Authority
CSR	Composite Schedule of Rates
AC	Asphaltic Concrete

1. INTRODUCTION

1.1. BACKGROUND

Pakistan is a developing country with many budget constraints. Road construction industry acts as a backbone in country development but it also has a very high budget requirement. In developing countries like Pakistan with all budget constraints it is very important to minimize the overall budget requirements for road construction by introducing advance techniques in road design and construction. The major cost factor in a road project is pavement thickness. Full depth and multilayer both approaches have been using in the flexible pavement structural design. Multilayer approach is economical compare to full depth in a country like Pakistan where binding agent "asphalt" bearing highest cost among pavement materials. This research work emphasizes on cost effective design of flexible pavements by reducing the thickness of asphalt concrete layer without compromising the durability and longevity/service life of pavement. For this reason, multilayer pavement approach is used to make pavement economical by keeping high quality material on top while low quality material on bottom (base and sub base). Base and sub base layer thickness will be increased in order to reduce the asphalt concrete layer thickness; therefore, the strength requirements will be adequate. Keeping in view the riding quality as a major aspect of AC, the multilayer pavement approach can be made more economical by keeping its thickness to minimum.

AASHTO 1993 design guidelines for structural design of pavements are dependent on application of load by different vehicles directly on pavement layers, depending upon their respective stiffness's. Therefore, according to AASHTO design it can be concluded that reduction in thickness of asphalt concrete layer will directly increases the thickness of base and sub base layers, hence adjusting the respective stresses and stiffness's. To make the pavement structure more economical this technique seems to be considerable in a developing countries like Pakistan where very high quality granular material is abundantly available. The thickness of asphalt concrete layer due to its high construction cost is a hot issue among highway agencies and engineering consultancies all over the world as asphalt concrete is an expensive material. A good technique to resolve this issue may be the nonlinear analysis of the pavement using different pavement analysis and design software's with the reduction in AC thickness up to minimum, and its impact on cost and pavement stability. If there is appreciable impact on cost with no compromise on stability resulted from actual AASHTO design then the claim of high cost, from highway agencies, due to AC thickness may be sincerely consider and subsequently new local parameters of pavement thickness design may set to save the budget for highway construction using local material. The existing design practices in road construction in Pakistan will be reviewed and suitable recommendations will be provided to concerned departments.

Pavement analysis and design will be carried out by using different software's which includes MFPD (Michigan Flexible pavement structural design), MICH PAVE, BISAR and KEN Layer software's. Iterations will be done to reach the desirable values of pavement thickness. AASHTO design method will be used for pavement thickness design. A brief comparison will also be carried out between AASHTO design method and British Road Note.

The study will also be a good viable solution for conservation of local material and provide a plate form for any pursuit at local and national level to make pavement thickness design economical.

1.2. PROBLEM STATEMENT

Pakistan is a developing country with many budget constraints and road construction required enormous amount of budget. Most of the cost is incurred in asphalt layer construction as asphalt is an expensive item. This research study optimizes the thickness of asphalt concrete layer and hence reducing the overall cost of pavement construction while keeping in view the pavement strength, longevity, durability and stability as per traffic loading and environmental conditions of Pakistan. The major cost factor in any flexible pavement construction project is Asphaltic Concrete (AC) Layer thickness, since asphalt is the most expensive pavement material. Flexible pavement design in Pakistan is mainly based on AASHTO procedure but designers mostly make adjustments in pavement layer thicknesses without appropriately standardized and logically developed methodology.

The pavement layer constructed by using asphalt concrete which is an expensive construction material and also the major cost factor in a road project is pavement thickness. As highways and road covers a huge portion of land thus their construction cost is far more than any other civil engineering projects. The main reason for selection of this topic is to determine the cost effective pavement design techniques in order to economize the pavement construction in Pakistan. Besides providing a recommendation on reduction in asphalt layer thickness this research study also includes a review of existing flexible pavement structural design practices in Pakistan. Most of the firms in Pakistan follows the AASHTO design procedures but most of the times the pavement is overdesigned, the agencies in Pakistan then reviews the design and decreases the asphalt layer thickness in order to have an economical design but no rationale or design procedure is explained and provided in the design reports by which the layer thicknesses are changed. Therefore, there is a need to formulate a logical and analytical methodology for optimization of pavement layer thicknesses especially the AC Layer because of the obvious reasons. With this perspective, this research work emphasizes on cost effective design of flexible pavements by reducing the thickness of AC Layer and correspondingly adjusting the Base and Sub-Base layer thicknesses without compromising the strength, durability and longevity/service life of pavement as per AASHTO requirement with respect to traffic loading & environmental conditions in Pakistan. By keeping in view the discrepancies in design this research study is based on logical reduction in thickness by optimizing the pavement structural design also this study will also be a good viable solution for conservation of local material available for road construction in Pakistan by ultimately increasing the base and sub-base layer thicknesses with reduction in asphaltic concrete layer thickness so that the pavement structure strength requirements can’t be compromised. This research also provides a plate form for any pursuit at local and national level to make pavement thickness design economical.

1.3. RESEARCH OBJECTIVES

The prime objectives of this research are as under

1. To synthesize/review the existing design practices of flexible pavement structural design for higher category national highways/motorways and lower category non-national highways.

2. Determine/Analysis of optimal/cost effective pavement structural design which includes asphalt concrete layer thickness for national and non-national highways without compromising pavement strength, longevity, durability and stability requirements with respect to traffic loading and environmental conditions in Pakistan.

3. To proffer recommendations in the form of decision matrix for economizing the pavement structural design with respect to traffic loading and existing environmental conditions in Pakistan.

1.4. SIGNIFICANCE OF STUDY

By carrying out this research the pavement surface layer thickness will be reduced hence reducing the cost incurred in construction. Existing design practices in road construction in Pakistan will be reviewed and recommendations will be provided to relevant departments. The thickness of asphalt concrete layer will be reduced while making use of high quality granular material which is abundantly available in Pakistan. Construction practices in highway industry of Pakistan will be reviewed and enhanced according to the local needs and environment. AASHTO design methods are based on USA climate and standards, by doing this research work the existing guidelines, parameters and standards will be modified according to the environment of Pakistan

1.5. THESIS OVERVIEW

This thesis has been organized into six chapters.

Chapter 1 is ‘Introduction.’ It includes introduction to the research, problem statement, and study objectives and significance. It provides a general overview to the research.

Chapter 2 is ‘Literature Review.’ It explains the previous studies done concerning the research providing essential information and guidelines for design and analysis of flexible pavements. Different methods are explained in this chapter. Also this chapter contains previous studies concerning the optimization of flexible pavements.

Chapter 3 is ‘. This chapter contains the review of existing flexible pavement structural design practices in Pakistan and provides some conclusion about the discrepancies in current practices.

Chapter 4 is ‘Methodology’ of research. It explains how the research is conducted to obtain our research design.

Chapter 5 is ‘Analysis of AAHSTO and Optimize Pavement Structural Design’ it covers the analysis of data after being collected, modeling and results according to our research objectives. It also discusses in detail how our objectives are achieved from using our analyzed data. It also provides a procedure of analysis using KENPAVE software.

Chapter 6 is ‘Results and Discussions.’ This chapter includes all the results and discussions of results,

Finally, Chapter 7 is ‘Conclusions and Recommendations.’ Final conclusions and recommendations have been summarized in this chapter.

2. LITERATURE REVIEW

2.1. BACKGROUND

This chapter discusses the past work done related to the research being carried out. It entails a discussion on the AASHTO 1993 flexible pavement structural design method, optimization of flexible pavement structural design, software analysis, stress and strains in flexible pavement structure and distress models.

2.2. Past Studies on AASHTO Flexible pavement structural design

The pavement design process is the technique of developing a combination of top layers of different materials in most economical manner to cater for the total axle load over the design life of a highway. In other words, this is an art through which the stresses as induced in the top layers of a highway due to movement of heavy wheel load is disseminated and minimized to safe level through selection of different type and appropriate thickness of pavement layers.

The American Association of State Highway and Transportation Officials (AASHTO) design method for flexible as well as rigid pavements is the most widely used design method all across the globe. The AASHTO guide for design of pavement structures (AASHTO 1993) is based on many parameters. These (AASHTO, 1993) parameters account for traffic loading, environmental conditions, soil properties, material properties, drainage conditions, reliability concept and performance trends. The flexible pavement structural design in Pakistan is based on AASHTO 1993 guidelines with some minor changes. Flexible pavement structural design is carried out using

AASHTO 1993 method which is based on tests and studies that AASHTO carried out in Ottawa and Illinois in between year 1958 and 1960.

Based on these tests results the first manual for pavement design was published in 1961 by AASHTO after which first revision was carried out in 1972 and second in 1981. With the passage of time few modifications are made into the manual under NCHRP 20-7/24 project they presented 1986 AASHTO pavement design manual (Ghanizadeh, 2016). In 1972 interim design guide researchers have made first step to extend the empirical relationships developed at the AASHO road test to broaden the range of materials and environmental conditions. The 1972 guide broadens the range of materials and environmental conditions. The 1986 revision added more features to 1972 guide. The main revision was carried out to incorporate the better characterization of sub grade and unbound material, this guide incorporates the pavement drainage concept. This guide also presented better consideration of environmental effects and reliability factor. In 1986 revision the sub grade was for the first time characterized by its resilient modulus (M_r). This resilient modulus is the fundamental engineering property. The structural layer coefficients for unbound layer material are also related to resilient modulus. Drainage quality was also incorporated in structural number equation.

$$a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \qquad \text{Equation 2-1}$$

In above structural number equation m_2 and m_3 are the drainage coefficients for base and sub base layer. The values for drainage coefficients are defined and are based on quality of drainage and period of exposure to moisture levels near saturation.

Table 2-1: Recommended Values of Drainage Coefficient for unbound base and subbases in flexible pavement (Huang 1993)

Percent of Time Pavement structure is exposed to moisture levels approaching saturation				
Quality of Drainage	< 1%	1–5%	5 –25%	>25%
Excellent	1.40 – 1.35	1.35 –1.30	1.30 –1.20	1.20
Good	1.35 – 1.25	1.25 –1.15	1.15–1.00	1.00
Fair	1.25 – 1.15	1.15 –1.05	1.00 –0.80	0.80
Poor	1.15 – 1.05	1.05–0.80	0.80 –0.60	0.60
Very Poor	1.05 – 0.95	0.95–0.75	0.75–0.40	0.40

Reliability concept was also introduced in 1986 AASHTO guide. Reliability accounts for effects of uncertainty and variability in design. In 1993 and 1986 AASHTO guide there are few changes but the design equation given by both guides is same (Regis L. Carvalho 2006).

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.5-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07 \qquad \text{Equation 2-2}$$

In which

i. The estimated future traffic in term of ESAL, for the design period, W18
ii. The Reliability Level, R
iii. Standard Normal Deviate Value, Z_R
iv. The overall Standard Deviation, So
v. The road bed soil Resilient Modulus,M_R
vi. The design serviceability loss, PSI= P_o–Pt

The solution for the above equation is same in 1993 and 1986 guide. The equation is solved for structural number (SN) and after that the layer thicknesses are computed. Different combinations of thicknesses can be found using this equation. In order to determine the optimal final design additional design constraints which include cost must also be considered (Carvalho, 2006).

Few assumptions are also made in the design. From the AASHO Road Test, equations were developed which related loss in serviceability, traffic, and pavement thickness. Because they were developed for the specific conditions of the AASHO Road Test, these equations have some significant limitations:

- The equations were developed based on the specific pavement materials and roadbed soil present at the AASHO Road Test.
- The equations were developed based on the environment at the AASHO Road Test only.
- The equations are based on an accelerated two-year testing period rather than a longer, more typical 20+ year pavement life. Therefore, environmental factors were difficult if not impossible to extrapolate out to a longer period.
- The loads used to develop the equations were operating vehicles with identical axle loads and configurations, as opposed to mixed traffic.

In order to apply the equations developed as a result of the AASHO Road Test, some basic assumptions are needed:

- The characterization of subgrade support may be extended to other subgrade soils by an abstract soil support scale.
- Loading can be applied to mixed traffic by use of ESALs.
- Material characterizations may be applied to other surfaces, bases, and subbases by assigning appropriate layer coefficients.
- The accelerated testing done at the AASHO Road Test (2-year period) can be extended to a longer design period.

When using the 1993 AASHTO Guide empirical equation or any other empirical equation, it is extremely important to know the equation's limitations and basic assumptions. Otherwise, it is quite easy to use an equation with conditions and materials for which it was never intended. This can lead to invalid results at the least and incorrect results at the worst (Pavement Interactive Website).

2.2.1 AASHTO 1993 Design Guide

In 1993 guide whole procedure from top layer to bottom layer is mentioned in order to compute the thicknesses of pavement layers (Carvalho, 2006). The first step is to calculate the structural number required to protect the base. This structural number is computed by using resilient modulus of base (Carvalho, 2006). This will give the thickness of asphalt layer which includes asphaltic base and wearing course layer. Only the structural layer coefficient of asphalt wearing and base course layer is different

$$D_1 \geq \frac{SN_1}{a_1} \qquad \text{Equation 2-3}$$

Step two is to calculate SN2 required protecting the subgrade, using equation 2.4 below with the subgrade effective resilient modulus as MR. The thickness of the base is computed as:

$$D_2 \geq \frac{SN_2 - a_1 D_1}{a_2 m_2} \qquad \text{Equation 2-4}$$

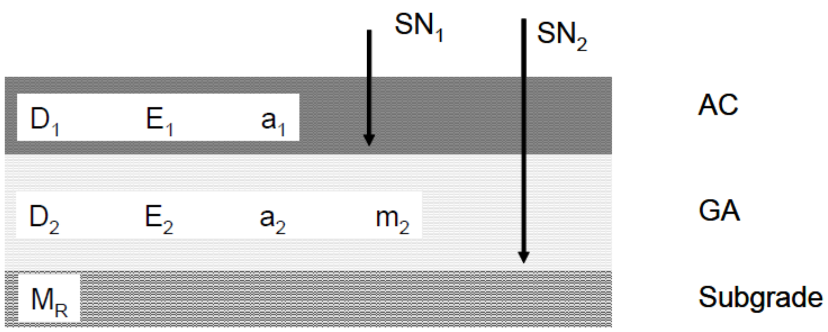


Figure 2-1 General procedure for computing thickness (Carvalho, 2006)

Design period and serviceability loss. Serviceability loss is defined as the difference between initial and terminal serviceability. Initial serviceability is the condition immediately after pavement construction. The conventional value is 4.2 (the average initial serviceability value at the AASHO Road Test). Terminal serviceability is the value at which the pavement is no longer capable of providing adequate service and major rehabilitation is required. Most state agencies have their own specification, although the 1993 AASHTO Guide recommends a terminal PSI of 2.5 for major highways and 2.0 for low volume roads, unless otherwise specified (Huang, 1993).

The other input variables are separated into three groups: (a) traffic, (b) material properties, and (c) environmental effects.

2.2.1.1 Material properties

The fundamental material property in the 1993 AASHTO Guide is the resilient modulus. Since the framework was constructed based upon structural layer coefficients, empirical relationships were developed to correlate resilient modulus with structural layer coefficient. Figure 2.2 below summarizes the relationship for the layer coefficient a1 for asphalt concrete (Carvalho, 2006).

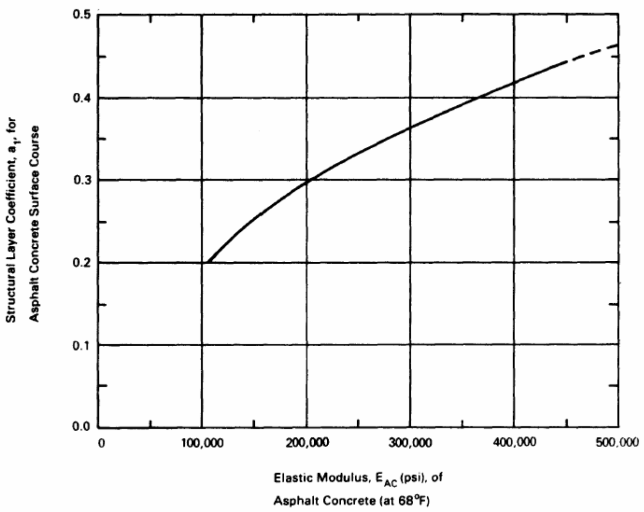


Figure 2-2: Chart for estimating layer coefficient for asphalt concrete based on elastic (Huang, 1993)

The layer coefficients in the AASHO Road Test were assumed equal to 0.44 for asphalt concrete, which corresponds to a MR = 450,000 psi; 0.14 for the granular base, corresponding to MR = 30,000 psi; and 0.11 for the subbase, equivalent to MR = 15,000 psi. The subgrade is characterized solely by its resilient modulus. There are also several correlations between MR and other soil properties that can be found in the literature. Most of them relate MR to CBR or R-Value (Heukelom and Klomp, 1962; Asphalt Institute, 1982; Van Til et al., 1972 – after Huang, 1993; NCHRP, 2004).

2.2.1.2 Environmental Effects

Environmental effects other than swelling and frost heave are accounted for in two input parameters in the 1993 AASHTO Guide, the seasonally-adjusted subgrade resilient modulus and the drainage coefficient mi applied to the structural number. It is recommended that an effective subgrade resilient modulus be used to represent the effect of seasonal variations, especially for moisture-sensitive fine-grained soils or for locations with significant freeze-thaw cycles (AASHTO, 1993). The effective resilient modulus is the equivalent modulus that would result in the same damage to the pavement as if seasonal modulus were used. The relative damage u_r is described by the following empirical relationship:

$$u_r = 1.18 \times 10^8 M_r^{-2.32} \qquad \text{Equation 2-5}$$

2.2.1.3 Traffic Studies

Vehicle and load distributions grouped by axle type are used to transform mixed traffic into a unified traffic parameter that can be used in the design equation. The mixed traffic is converted into one parameter called the Equivalent Single Axle Load (ESAL). ESAL is defined as the number of 18-kip single axles that causes the same pavement damage was caused by the actual mixed axle load and axle configuration traffic. The damage associated with the equivalent axle can be defined in numerous ways; in the 1993 AASHTO Guide it is defined in terms of serviceability. The 18-kip single axle load was chosen because it was the maximum legal load permitted in many states at the time of the AASHO Road Test by Zhang et al.(2000). Traffic studies are intended to provide necessary input data for determination of the magnitude and pattern of the traffic load for the project highway through the design period. This entails collection, verification and analysis of the traffic data. From the collected data, the projected traffic for the design life of the subject highway is determined. Traffic volume is converted into equivalent single axle load (ESALs). In order to determine the cumulative axle load damage that a pavement will sustain during its design life, it is necessary to express the total number of heavy vehicles that will use the road during the design period in terms of the cumulative number of Equivalent Single Axle Load (ESALs).

The damage caused by vehicles to a road depends on the axle loads and wheel configuration of the vehicles. It is, therefore, important to determine the axle loads of heavy commercial vehicles in the projected traffic mix that is likely to use proposed alignment. For pavement design purposes the damaging power of axles is related to a standard axle of8.16 tonnes (18000Ibs) using equivalence factors which have been derived from empirical studies. In order to determine the cumulative ESALs over the design period the following procedure has been adopted.

- AADT is determined.
- Direction Factor value has been selected.
- Annual traffic in design lane is determined.
- Equivalent Single Axle Load is estimated.

2.2.1.4 Reliability (R)

Design reliability refers to the degree of certainty that a given design alternative will last for the entire design period. This concept was first introduced in 1986 AASHTO Guide to account for the effects of uncertainty and variability in the design inputs. Reliability is defined as the probability that the design pavement will achieve its design life with serviceability higher than or equal to the specified terminal serviceability. Although the reliability factor is applied directly to traffic in the design equation, it does not imply that traffic is the only source of uncertainty. Table 2.2 suggests appropriate levels of reliability for various highway classes.

There is some guidance on how reliability is considered. High volume and high speed highways have higher reliability factors than minor roads and local routes. The standard deviation (S0) and reliability factor (ZR) parameters in the design equation are respectively defined as the standard deviation of uncertainties and the area under a normal distribution curve for $p <$ reliability. The parameter ZR can be retrieved from Table 2-3. The 1993 AASHTO Guide recommends values for So between 0.35 and 0.45 for flexible pavements (Huang 1993).

Table 2-2: Suggested levels of reliability for various highway classes (AASHTO, 1993).

Functional classification	**Recommended level of reliability**	
	Urban	**Rural**
Interstate and freeways	85-99.9	80-99.9
Principal arterials	80-99	75-95
Collectors	80-95	75-95
Locals	50-80	50-80

Table 2-3: ZR values for various levels of reliability (Huang, 1993)

Reliability	Z_R	**Reliability**	Z_R
50	0.000	93	-1.476
60	-0.253	94	-1.555
70	-0.524	95	-1.645
75	-0.674	96	-1.751
80	-0.841	97	-1.881
85	-1.037	98	-2.054
90	-1.282	99	-2.327
91	-1.340	99.9	-3.090
92	-1.405	99.99	-3.750

2.2.1.5 Standard Deviation(So)

The reliability factor is a function of the overall standard deviation that accounts for standard variation in materials and construction, the probable variation in the traffic prediction and the normal variation in pavement performance for a given design traffic application. The recommended value of standard deviation for total variation in material properties and in traffic estimation is given in AASHTO 1993 guide.

2.2.1.6 Performance Criteria

The serviceability of a pavement is defined as its ability to serve the type of traffic that uses the facility. Initial and terminal service ability indices have been established to compute the total change in serviceability that will be used in the design equations. Serviceability loss is defined as the difference between initial and terminal serviceability. Initial serviceability is the condition immediately after pavement construction. The conventional value is 4.2 (the average initial serviceability value at the AASHO Road Test). Terminal serviceability is the value at which the pavement is no longer capable of providing adequate service and major rehabilitation is required. Most state agencies have their own specification, although the 1993 AASHTO Guide recommends a terminal PSI of 2.5 for major highways and 2.0 for low volume roads, unless otherwise specified.

Initial Serviceability Index (Po)

The initial serviceability index is a function of pavement design and construction quality. For flexible pavement design typical value, as recommended by AASHTO Road Test, is 4.2.

Terminal Serviceability Index (Pt).

The terminal serviceability index is the lowest index that will be tolerated before rehabilitation, resurfacing or reconstruction, becomes necessary and it generally varies with the importance or functional classification of the pavement.

2.2.1.7 Resilient Modulus MR

The basis for material characterization in the AASHTO Guide 1993 is Elastic or Resilient Modulus (MR). In the absence of necessary equipment required to determine resilient modulus of unbound layers, following correlation between CBR and MR. The subgrade is characterized solely by its resilient modulus. There are also several correlations between M_R and other soil properties that can be found in the literature. Most of them relate M_R to CBR or R-Value (Heukelom and Klomp, 1962; Asphalt Institute, 1982; Van Til et al., 1972 – after Huang, 1993; NCHRP, 2004). One of the basic equation is given below

$$M_R = 2555\ (CBR)^{0.64} \qquad \text{Equation 2-6}$$

2.3 Synthesis on Optimization of Flexible pavement structural design:

Pavement construction is one of the costliest parts of transportation infrastructures. Incommensurate design and construction of pavements, in addition to the loss of the initial investment, would impose indirect costs to the road users and reduce road safety (Ghanizadeh, 2016). An optimizing technique was required for this purpose. Optimization model has been developed for this purpose, after this an optimize pavement layer thickness was determined for secondary rural roads, major rural roads and freeways. The optimum thickness was based on recommended price in highway Code (Ghanizadeh, 2016). By increasing the strength of sub grade soil in terms of resilient modulus the sub base layer may be removed from the pavement structure (Ghanizadeh, 2016).

Researchers in this field proposed different optimization models for optimum design of flexible pavements. Mixed integer programming model was proposed by (Rouphail, 2006) This model helps in identifying the thickness, type and number of road paving materials that are required to meet the structural strength at minimum initial construction cost. Mamlouk et al used project level optimization approach to minimize the construction cost of pavement within an analysis period. An optimization program which was a supplement to DNPS86 pavement design computer program was introduced by Nicholls. This program helps in finding the minimum cost combination of pavement layer thicknesses. Optimization model for design of flexible pavements based on fatigue and rutting performance was proposed by Mu-yu and Shao-yi. They used genetic algorithms (GAs) to solve the optimization model. Abaza and Abu-Eisheh represented an optimum approach for the design of flexible pavements based on AASHTO method which utilized the anticipated performance of pavement and its life-cycle cost. They showed that pavements should be designed for higher terminal serviceability index values than currently recommended. Ouyang and Madanat presented a mixed-integer nonlinear programming for optimal highway pavement rehabilitation planning which minimizes the life-cycle cost during design period. Fakhri and Ghanizadeh developed an optimization model to determine the optimum structure and thickness of pavement layers, based on the AASHTO method. The proposed model, in the form of a linear programming model, could determine the optimum configuration of pavement layers as well as optimum thickness of pavement layers. It could only consider the optimum structure of pavements consisting of asphalt, granular base, and granular subbase layers. Proposed model did not consider the treated base layers in pavement structure. Also, by employing this model, thickness of layers was determined as real numbers not integer numbers which should be revised for application in construction stage. Sanchez-Silva et al. present a model for reliability cost-based optimization of asphalt pavement structures based on both economic and operation considerations. The proposed model considered the fatigue damage caused on the asphalt surface and the degradation of granular materials caused by repetitive loading cycles. They showed that the reliability based design optimization combined with a long-term maintenance policy of pavements produces appropriate integral designs. Rajbongshi and Das presented a simple methodology to assist a pavement designer in selecting an optimal pavement design thickness which is cost effective yet does not compromise the reliability of the pavement design. They developed pavement design charts as an illustrative example to explain how the proposed methodology can be considered as an improvement over the deterministic design. Santos and Ferreira proposed a pavement design optimization model, called OPTIPAV, which considers pavement performance, construction costs, maintenance and rehabilitation costs, user costs, the residual value of the pavement at the end of the project analysis period, and preventive maintenance and rehabilitation interventions.

2.3 Past Studies on Optimization Techniques:

Modern pavement design optimization techniques provide a scientific basis for decision-makers to increase serviceability, efficiency, and performance, and to decrease initial construction cost. The optimum objective of optimization techniques is to maximize benefit, minimize cost, or minimize the cost-benefit ratio. Throughout the optimization process, the best alternative strategies for pavement construction and rehabilitation can be selected based on some specified criteria. Different techniques for optimization are 1) linear programming 2) integer programming 3) nonlinear programming and 4) dynamic programming Holsapple et al. (1994). Optimization techniques are the tools that give the best conceivable solutions in the decision-making process for any type of pavement design project. Linear integer programming is the widely used optimization technique because of its simplicity.

Optimization techniques can provide tools that are capable of giving the best possible solutions in the decision-making process of many engineering applications. Linear programming is widely used optimization technique because of its simplicity. In the real world, however, most physical and mechanical phenomena cannot be modelled by linear functions.

The American Association of State Highway and Transportation Officials (AASHTO) and mechanistic pavement models, for example, are nonlinear functions. Moreover, the pavement project management is a multistage decision process.

2.4 Linear Integer Programming:

Linear integer programming technique has been used by many researchers for optimization of pavement design. A classical optimization model, using a mixed integer-linear programming algorithm is utilized to formulate the minimum-cost problem. In the AASHO Interim Guide, individual layer thicknesses can be calculated in one of two ways. The first method utilizes the required weighted structural number, SN, over the subgrade. Alternatively, if the soil support values for the base and sub base courses are readily available to the designer, then the required structural number above each layer may be calculated (Rouphail,1985). Linear integer programing is the mathematical way to solve the problem. An optimization model contains objective function, constraints and decision variables. Integer programming problem is a mathematical optimization or feasibility program in which some or all of the variables are restricted to be integers. In many settings the term refers to integer linear programming (ILP), in which the objective function and the constraints (other than the integer constraints) are linear.

Sometime the fractional solutions of any problem are not realistic so in order to solve a problem an optimization model is required (AMP Chapter 09).

Objective Function

Maximize or Minimize $\sum_{j=1}^{n} c_j x_j$

Constraints

$\sum_{j=1}^{n} a_{ij} x_j = b_i$ (i = 1, 2…………………..m)

$x_j \geq 0$ (j= 1, 2…………………………n)

x_j Is integer for some or all j= 1, 2…………………………n

The problem mentioned above is called as integer programming problem (linear). When some of the variables are restricted to be integer the same problem is then called as mixed integer program. The objective function in this problem is the main mathematical model used for optimization. It contains the cost and the function which is to be optimized. Integer-programming models arise in practically every area of application of mathematical programming (AMP Chapter 09). Linear programming model consists of one objective which is a linear equation that must be maximized or minimized.

The objective function purpose is defined as to maximize or to minimize any possible solution. For pavement design optimization the objective function is used to provide the best possible layer thicknesses in order to minimize the cost. After the objective function there are number of linear inequalities which are called as constraints.

The disparities can be <=, >= or = since all numbers are real values and most of the time it is required that one or more variables must be whole number. It is impractical to simply explain the model as is and afterward round to the closest arrangement. Issues with whole number variables are called whole number or discrete programming issues. On the off chance that all variables are whole number it is known as an unadulterated whole number programming issue, else it is a blended whole number programming issue. An special case of integer variables are binary variables. These are variables that can just take 0 or 1 as worth. They are utilized as often as possible to program end conditions. Double variables are characterized as number variables with a most extreme (upper bound) of 1 on them.

2.5 Linear Program Solver (LiPS):

Linear Program Solver (LiPS) is an optimization package intended for solving linear, integer and goal programming problems. LiPS solver is based on the efficient implementation of the modified simplex method. LiPS provide not only an answer, but a detailed solution process as a sequence of simplex tables, so you can use it in studying (teaching) linear programming. LiPS provide the procedures of sensitivity analysis, which enable us to study the behaviour of the model when you change its parameters, including: analysis of changes in the right sides of constraints, analysis of changes in the coefficients of the objective function, the analysis of changes in the column / row of the technology matrix. Such information may be extremely useful in the practical application of LP models. LiPS provide the methods of the goal programming, including the lexicographic and weighted GP methods. Goal programming methods are intended for solving multi-objective optimization problems.

LiPS computer program is intended for solving linear, integer and goal programming problems. The main features of the LiPS are it is based on the efficient implementation of the modified simplex method that solves the large scale problems.

2.6 Past Studies on Flexible Pavement Stresses and Strains

Stresses are the response of loading on the pavement. Material containing subgrade and environment condition are also responsible for stresses in the pavement. The top most paved surface of such type of pavement is flexible, that is extremely dependent on the underlying layers. Due to flexible, pavement is free to move. In such type of pavements following stresses are the most common and are extremely effective.

1. Vertical stresses
2. Shear stress
3. Radial stress

Vertical stress effects the pavement by compressing the pavement material. When pavement compresses, then material in a pavement gets crushed and as a result rutting become visible on the top horizontal pavement.

Rutting is the depression in the surface of wheel path. Along the sides of the rutting, pavement may uplift (due to shear). These ruts are very clear in the pavement after rain when ruts filled up with water.

Shear stress occurs in the pavement when load is more than the capacity of the pavement. When load approaches the critical point, then as a result movement occurs in the base layer and that movement is responsible for the shear stress in the top pavement when tension occurs at the bottom of layers due to seepage, removal of material from particular layer or by any other mean. As a result, fatigue cracking occurs in the pavement due to wear and tear of loads. That cracking leads to radial stresses in the pavement.

Flexible pavement consists of one or more layer of asphalt concrete with base and sub base layers. Mostly the base is consisting of unbound layer or granular layer. The design of the thickness of flexible pavements for any purpose is based on the calculation of stresses and strains, occurring within the structure due to the traffic loadings, and the comparison with the allowable stresses and strains.

Usually the linear elastic multi-layer theory is used to calculate the occurring stresses and strains. This however implies that the actual material behaviour is simplified to a great extent because most road building materials don’t behave linear elastic. Unbound materials behave strongly stress dependent and asphalt mixes are viscoelastic materials. Nevertheless, the assumption of linear elastic material behaviour is in most cases justified and that is certainly the case if the occurring stresses and strains in the structure are rather limited.

For the purpose of thickness design, the traffic loading is the main input. Apart from traffic loading, elastic modulus of each layer or pavement has to be known. The amount of load transfer to the pavement layers depends upon the bending stiffness of the respective layers. The bending stiffness depends on elastic modulus (E) and pavement layer thickness. As pavement structure is a three dimensional system hence the poison ratio of subsequent layer is also an important factor. It is important to know whether the subsequent pavement layers are fully bonded (which implies that the horizontal displacements just above and just below the interface are equal) or that they can move relatively to each other in the horizontal direction. Pavement stress-strain analysis is an ideal tool for analytical modelling of pavement behaviour and thus, constitutes an integral part of pavement design and performance evaluation. It is the fundamental basis for the mechanistic design theory by Lubinda et al. (2000).

For a three-layer pavement, several charts and tables have been developed by Peattie, Jones and Fox (Witczak, 1975) to determine the stresses, strains, and deflections. Peattie (1962) developed graphical solutions for vertical stress in three-layer systems. Jones (1962) presented solutions for horizontal stresses in a tabular form. Both of these solutions were based upon a Poisson's ratio of 0.5 for all layers. Different compute programs were developed in order to simplify the calculation of stresses and strains in flexible pavement. These computer programs allow greater flexibility in accommodating material properties and multiple loads.

The input required for using these computer programs are as under

- Material properties of each layer
- Modulus of elasticity
- Poisson's ratio
- Thickness of each pavement layer
- Loading conditions (2 of 3 listed below)
- Magnitude of load
- Radius of load
- Contact pressure
- Number of loads
- Location of load(s) on the surface (x,y coordinates)
- Location of analysis points for output (x,y,z coordinates)

2.7 Synthesis on Fatigue and rutting in flexible pavements:

Highway engineers design flexible pavements after carrying out stresses and strains analysis. Mechanistic method of flexible pavement structural design contains distress models which are mainly fatigue cracking and rutting. These models then used to find the design life of pavements. Rutting and fatigue cracking considered as most important distress models due to high severity and density levels by Ahmed Ebrahim et al. (2012).

Traffic Loads on the surface of pavement produce stresses and strains. Two strains are believed to be critical for design of pavement. Strains due to cracking and rutting are considered as most critical for the design of asphalt pavements (Gedafa, 2006). These strains are horizontal tensile strain (ε_t) at the bottom of asphalt layer and vertical compressive strain (ε_v) at the top of sub grade. The type of failure depends on these two strains, if the vertical compressive strain (ε_v) is more than horizontal tensile strain (ε_t) then the permanent deformation is occurring on the surface in flexible pavement structure, this permanent deformation also called as rutting. If horizontal tensile strain (ε_t) is excessive than cracking in surface layer occurs and this pavement distress is called as fatigue by Ahmed Ebrahim et al. (2012).

2.7.1 Fatigue Cracking Model:

Miner's (1945) provides cumulative damage concept, which has been widely used to predict the fatigue cracking in flexible pavement. The allowable numbers of load repetitions are related to tensile strain at the bottom of asphalt layer. Amount of damage occurs expressed in terms of damage ratio. Damage ratio is the ratio between predicted and allowable number of load repetitions. Damage occurs when sum of damage ratios reaches one (Huang, 2004). The allowable number of load repetitions (N_f) can be computed using equation 2.6 given below.

$$N_f = f_1 \, (\varepsilon_t)^{-f2} \, (E^{-f3}) \qquad \text{Equation 2-6}$$

Where ε_t is the tensile stain at the bottom of asphalt layer, E is the elastic modulus of asphalt layer and f_1 , f_2 , f_3 are constants which are obtained from calibrations they are also called as regression coefficients. N_f is the allowable number of load repetitions, they prevent the fatigue cracking from reaching certain limit. Generally, the limit is defined as 10 to 20 percent of the pavement surface area by Ahmed Ebrahim et al. (2012).

2.7.2 Rutting Model:

Many procedures have been used by researchers to limit the rutting. The most important one are to limit the vertical compressive strain on top of the subgrade, and to limit the total accumulated permanent deformation on the pavement surface based on the permanent deformation properties of each individual layer. In the Asphalt Institute and Shell design methods, the allowable number of load repetitions (*Nd*) to limit rutting is related to the vertical compressive strain (ε_v) on top of the subgrade Muniandy et al. (2013). Equation 2.7 given below gives the allowable number of load repetitions to limit rutting.

$Nd = f4\ (\varepsilon v) - f5$ Equation 2-7

Where: *f4 and f5* are calibrated values using predicted performance and field observation, ε_v is the vertical compressive strain at the top of subgrade. Under heavy traffic with thicker asphalt concrete layer, most of the permanent deformation occurs in the asphalt layer, rather than in the subgrade. Because rutting is caused by the accumulation of permanent deformation over all layers, it is more reasonable to determine the permanent deformation in each layer and sum up the results. The coefficients for rutting and cracking used by different institutions are given in Table 2.4 (Huang, 2004)

Table 2-4: Coefficients for rutting and cracking distress models (Huang, 2004)

No	Distress Models	*f1*	*f2*	*f3*	*f4*	*f5*	Source
1	Asphalt Institute (AI) Model	0.0796	3.291	0.854	$1.365*10^{-9}$	4.477	Asphalt Institute
2	Shell Model (95% reliability)	0.0685	5.671	2.363	$1.05*10^{-7}$	4.0	Shell
3	U.K. Transport & Road Research Laboratory (85% reliability)	$1.66*10^{-10}$	4.32	0	$6.18*10^{-8}$	3.95	Powell et al.
4	Belgian Road Research Center	$4.92*10^{-14}$	4.76	0	$3.05*10^{-9}$	4.35	Verstraetenet al.

2.8 Literature on KENPAVE Computer Program:

KENPAVE software was developed by Huang in 1993 (Huang, 2004). It is a Microsoft-Windows based version that combines the old KENPAVE flexible pavement software and Kenslabs rigid pavement software. This software allows the use of linear elastic, nonlinear, and viscoelastic properties of the materials for the different layers. The software can handle up to 19 layers and performs damage analysis. The interface between the different layers can be specified as either unbounded or fully bonded. KENLAYER can be applied to layer systems under single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently, linear elastic, nonlinear elastic, or viscoelastic. Damage analysis can be made by dividing each year into a maximum of 12 periods, each with a different set of material properties. Each period can have a maximum of 12 load groups, either single or multiple. The damage caused by fatigue cracking and permanent deformation in each period over all load groups is summed up to evaluate the design life (Huang, 2004).

2.8.1 Input Parameters in KENLAYER Computer Program:

For the purpose of analysis in KENPAVE computer program many input parameters are required. . The parameters can be inputted both in SI and U.S. customary units. Some of the input parameters for linear elastic analysis are traffic load, material properties, thickness of each layer, number of periods, number of load groups etc.

2.8.2 Output Parameters of KENLAYER:

For a single and multiple load groups, a maximum of nine and ten responses can be obtained, respectively. Only the vertical compressive strain on the surface of subgrade and the radial (Tangential) tensile strains at the bottom of asphalt layer are used for damage analysis.

2.8.3 Methodology:

For the purpose of this research work KENPAVE computer program is used to carry out the analysis of pavement performance. Fatigue cracking and rutting are two distress models considered in KENPAVE program. These two models are considered at the bottom of asphalt layer (bottom of asphaltic base layer) and at the top of sub grade. KENPAVE computer program is used to predict the performance of new pavement. In this research work it is assumed that new pavement is being design and executed.

Pavement performance can be predicted by doing performance modelling. Pavement performance modelling can be of two types

1) **Empirical**

In empirical approach measured or estimated variables which include deflection, accumulated traffic loads and environmental conditions are related to loss in serviceability.

2) **Mechanistic Empirical**

In mechanistic empirical approach which is used in this research, pavement response to traffic load which are calculated response and includes pavement layer stresses and strains combined with traffic loads, environmental conditions and loss of serviceability are the measures of deterioration through regression analysis.

2.8.4 Tensile and Compressive Strains using KENPAVE:

KENPAVE is a mechanistic - empiric design (M-E), and its performance prediction is based in the most critical of two analysis: (1) tensile strain in the bottom of the bounded layers (HMA, I presume) to predict fatigue cracking, or (2) compressive strain in top of the subgrade to predict general rutting of the structure. These two criterion are also compared just like service life criterion.

The values of both strains which results in fatigue and rutting were compared by permissible values using the Heukelom and Klomp Model. It was found that both vertical and horizontal strains values are less than the permissible values for both design alternatives at each value of ESALs.

2.9 Heukelom and Klomp (1962)

Heukelom and Klomp (1962) suggested a relationship between number of load repetetetions to failure and strain in asphalt concrete. The model is given as

$$N_f = 10^{-X} \qquad \text{Equation 2-8}$$

Where N_f is the number of load repartitions to failure while X is the function which is given as

$$X = 5 \log \varepsilon_t + 2.665 log_{10}\left(\frac{E}{14.22}\right) + 0.392 \qquad \text{Equation 2-9}$$

Where ε_t is the horizontal tensile strain at the bottom of asphalt layer and E is the elastic modulus of asphalt concrete.

The second part of model is the function of allowable horizontal tensile strain at the bottom of asphalt concrete layer.

$$€_{AC} = 10^{-A} \qquad \text{Equation 2-00}$$

Where $€_{AC}$ is the allowable tensile strain at the bottom of asphalt layer and A is given by model as below:

$$A = (log_{10} N_i + 2.665 log_{10}\left(\frac{E}{14.22}\right) + 0.392)/5 \qquad \text{Equation 2-11}$$

Where $N_i$number of actual load repetitions and E is the elastic modulus of asphalt concrete. By using the above model value of allowable tensile strains were calculated against each value of load repetition i.e. ESAL. These values are than considered as a threshold values for analysis and decision making.

For compressive strain at the top of sub grade following Heukelom and Klomp (1962) equation was used to find out the permissible or allowable compressive strains at the top of sub grade at each value of ESAL.

$$€_{SUBG} = 10^{-A} \qquad \text{Equation 2-12}$$

Where $€_{SUBG}$ is allowable vertical compressive strain at the top of sub grade and A is given by the model as follow

$$A = 0.1408 log_{10} N_i + 2.408 \qquad \text{Equation 2-13}$$

By using the above model value of allowable vertical compressive strains were calculated against each value of load repetition i.e. ESAL.

2.10 Summary and gap analysis of Literature Review:

1. According to (Ghanizadeh, 2016) in his research paper the pavement construction is one of the costliest parts of transportation infrastructures. Incommensurate design and construction of pavements, in addition to the loss of the initial investment, would impose indirect costs to the road users and reduce road safety. An Optimization model has been developed for this purpose, by (Ghanizadeh, 2016) from which the optimize pavement layer thickness was determined for secondary rural roads, major rural roads and freeways. The optimum thickness was based on recommended price in highway Code. His methodology proposed that by increasing the strength of sub grade soil in terms of resilient modulus the sub base layer may be removed from the pavement structure hence reducing the construction cost. Therefore,

in his research studies he actually varies the subgrade resilient modulus strength at different ESALs values which ultimately predicts the pavement structural layer thicknesses. He proposed many alternatives regarding pavement structural configuration.

2. Mixed integer programming model was first introduced by (Rouphail, 1985) in his research. This model helps in identifying the thickness, type and number of road paving materials that are required to meet the structural strength at minimum initial construction cost.
3. Abaza et.al. (2003) represented an optimum approach for the design of flexible pavements based on AASHTO method which utilized the anticipated performance of pavement and its life-cycle cost. They showed that pavements should be designed for higher terminal serviceability index values than currently recommended. But my research study is based on levels of serviceability based on AASHTO.
4. (Fakhri, 2016) developed an optimization model to determine the optimum structure and thickness of pavement layers, based on the AASHTO. The proposed model, in the form of a linear programming model, could determine the optimum configuration of pavement layers as well as optimum thickness of pavement layers. The problem with this research was that it could only consider the optimum structure of pavements consisting of asphalt, granular base, and granular subbase layers and also the proposed model did not consider the treated base layers in pavement structure. Also, by employing this model, thickness of layers was determined as real numbers not integer numbers which should be revised for application in construction stage.

5. Sanchez-Silva et al. (2005) present a model for reliability cost-based optimization of asphalt pavement structures based on both economic and operation considerations. The proposed model considered the fatigue damage caused on the asphalt surface and the degradation of granular materials caused by repetitive loading cycles. They showed that the reliability based design optimization combined with a long-term maintenance policy of pavements produces appropriate integral designs. The research study aims in developing optimization models based on fatigue damage only however in my research work both fatigue and rutting damages are considered.
6. Das et al. (2015) presented a simple methodology to assist a pavement designer in selecting an optimal pavement design thickness which is cost effective yet does not compromise the reliability of the pavement design. They developed pavement design charts as an illustrative example to explain how the proposed methodology can be considered as an improvement over the deterministic design. Research work proposed by them is based on reliability concept, but my research work is based on pavement structural strength.
7. Santos et al. (2015) proposed a pavement design optimization model, called OPTIPAV, which considers pavement performance, construction costs, maintenance and rehabilitation costs, user costs, the residual value of the pavement at the end of the project analysis period, and preventive maintenance and rehabilitation interventions.

3. An Analysis of Pavement Design Practices in Pakistan

3.1. INTRODUCTION

In Pakistan the majority of pavements is of flexible type. The road and surface failure of the flexible pavement has become the most important and attention diverting problem, in Pakistan which may be due to the low quality of materials used, less and inadequate experience of the technical staff, and errors that occur during the designing of the pavement structure. The design of thickness of the flexible pavement has taken the backbone place in determining the overall performance and providing high level of serviceability of the pavement structure for the heavy traffic loads under the adverse climatic conditions, during the expected design period. Flexible pavement structural design reports are acquired from different construction firms in Pakistan. The purpose of these reports is to study and carrying out brief analysis on design procedure currently being followed in Pakistan road industry. Many of the parameters are same except few which are used according to the construction requirements. In Pakistan mostly AASHTO 1993 procedure is followed for flexible pavement structural design however few of the firms are using Road Note for comparison purposes.

3.2. ANALYSIS ON DESIGN PRACTICES

Pavement thickness design is carried out using AASHTO 1993 procedure; the parameters used for this purpose are usually taken with respect to the conditions. After studying different reports its imperative that many firms are using empirical methods to design the flexible pavement, the thickness of asphalt layer is reduced empirically rather than carrying out an analysis using any software. Mostly the software's are used for analysis purpose only thereafter reducing the required asphalt thickness. A design period of 10 years is adopted which is a normal practice to calculate the traffic load and eventually carrying out pavement design. Equivalent axle load factors from NTRC survey are used for calculations of ESALS by consultants. These axle load factors are obtained from the latest study with a comprehensive sample base. ESALS are determined by using AASHTO procedure.

The basis for material characterization in AASHTO 1993 guide is elastic or resilient modulus. Different values of resilient modulus are used depending upon the nature of sub grade soil, however in the absence of necessary equipment required to determine the resilient modulus of sub grade various correlations between CBR and MR are used by different consultants in Pakistan.

Several studies have claimed that traffic is a controversial parameter in the 1993 AASHTO Guide. The fact that the guide relies on a single value (i.e. ESAL) to represent the overall traffic spectrum is questionable Schwartz et al. (2007). Zhang et al. (2000) have found that the ESAL, used to quantify damage equivalency in terms of serviceability or even deflections in the 1993 AASHTO Guide, is not enough to represent the complex failure modes of flexible pavements.

Today it is widely accepted that load equivalency factor is not a sufficient technique for incorporating mixed traffic into design equations. In addition, the trucks used during the AASHO Road Test were modest in comparison to the trucks utilized in the oil industry today. The models developed and modified from the Road Test relate key pavement properties and traffic to performance but do not consider the range of climatic effects that can also contribute to pavement distress. In addition, the performance index used in the 1993 AASHTO Design Guide relies on an empirical assessment of the overall pavement surface quality. The pavement serviceability index (PSI) is the evaluation users give about the road surface condition, as defined during the AASHO Road Test. PSI cannot be measured and therefore it was correlated to ride quality and other smoothness indices in research done during the period of the mid-1980's to mid-1990's. Currently, distresses measured directly on the pavement surface are more accepted as performance measures. They provide a better representation of failure mechanisms and can be modeled directly using site-specific characteristics. To address some of the limitations of its original design guide, AASHTO in 2004 published the Mechanistic-Empirical Pavement Design Guide (MEPDG). This new design procedure incorporates mechanistic principles, including calculations of pavement stress, strain and deformation responses using site-specific climatic, material, and traffic characteristics. It replaces the 1993 guide's subjective-based performance index, PSI, with objective distress models for various modes of pavement failure and allows calibration of the distress models in order to allow the design method to represent each region's unique conditions. The new guide is a significant departure from traditional pavement design procedures and its implementation requires road agencies to overcome some challenges. However, in the current climate of increasingly urgent infrastructure needs and shrinking funding, it is important for agencies to identify cost effective and structurally adequate pavements that serve stakeholders for their full design life. It is also important for an agency to know, before undertaking a change in design method, whether MEPDG-designed pavements will indeed show performance benefits over their empirically-designed counterparts.

The 1993 AASHTO Guide is the latest version of the AASHTO Interim Pavement Design Guide, originally released in 1961. The evolution of the AASHTO Guide is outlined, followed by a description of the current design equation and input variables. At the end of this section, a summary of recent evaluation studies of the AASHTO guide is also presented.

3.3 Summary of Flexible Pavement Structural Design Reports

In Pakistan, the road and surface failure of the flexible pavement has become the most important and attention diverting problem, which may be due to the low quality of materials used, less and inadequate experience of the technical staff, and errors that occur during the designing of the pavement structure. The design of thickness of the flexible pavement has taken the backbone place in determining the overall performance and providing high level of serviceability of the pavement structure for the heavy traffic loads under the adverse climatic conditions, during the expected design period. A brief summary on the design process followed by agencies in Pakistan is given below.

3.3.1 Layer Coefficients:

Pavement design reports have been reviewed and compared according to material properties and different design parameters used. Layer coefficients used in different design reports are based on AASHTO design guide 1993. Some of the reports used AASHTO guide relationships in order to calculate the layer coefficient and elastic modulus. Elastic modulus is computed with the layer coefficient. The elastic modulus for granular sub-base has been taken with respect to CBR value. Table 3-1 given below compares the value used in different pavement design reports.

Table 3-1 Structural Layer Coefficient Values used in Different Design Reports

Design Report	Layer	Layer Coefficient/Cm
Tarnol Interchange Project **Republic Engineering Corporation (Pvt) Ltd Year 2010**	Asphalt Wearing Course	0.157
	Asphalt Base Course	N/A
	Granular Base Course	0.067
	Granular Sub-base	0.045
Dualization & Improvement of SOHAWA-CHAKWAL ROAD **Associated Consulting Engineers –ACE (Pvt) Ltd Year 2013**	Asphalt Wearing Course	0.157
	Asphalt Base Course	N/A
	Granular Base Course	0.067
	Granular Sub-base	0.047
Construction of Hasanabdal-Havelian Section of E-35 **National Engineering Services Pakistan (Pvt) Ltd Year 2009**	Asphalt Wearing Course	0.173
	Asphalt Base Course	0.142
	Granular Base Course	0.055
	Granular Sub-base	0.043
4 Lane Expressway Starting from Korai on Indus Highway N-55 and Terminating at Hakla **National Engineering Services Pakistan (Pvt) Ltd Year 2016**	Asphalt Wearing Course	0.165
	Asphalt Base Course	0.165
	Granular Base Course	0.051
	Granular Sub-base	0.047
YAKMACH to KHARAN Road **Associated Consulting Engineering-TES (Pvt) Ltd Year 2015**	Asphalt Wearing Course	0.173
	Asphalt Base Course	N/A
	Granular Base Course	0.047
	Granular Sub-base	0.047

3.3.2 Drainage Coefficient:

Drainage coefficient value is taken as 1 in all design reports studied.

3.3.3 Design ESALS:

ESALS are based on annual daily traffic predicted during design service life of pavement. All design reports are based on 10 years of service life. ESALs are calculated by using truck factors from NTRC report. The damage caused by vehicle to a road depends on the axle loads and wheel configuration of heavy commercial vehicles in the projected traffic mix that is likely to use the proposed alignment. The damage caused by vehicles to a road depends on the axle loads and wheel configuration of the vehicles. It is, therefore, important to determine the axle loads of heavy commercial vehicles in the projected traffic mix that is likely to use proposed alignment.

For pavement design purposes the damaging power of axles is related to a standard axle of 8.16 tones(18000Ibs) using equivalence factors which have been derived from empirical studies. Equivalent Axle Load Factors (EALF) are determined separately, for different types of axle configurations. The EALF obtained by NTRC Survey have been adopted in all pavement design reports in Pakistan, being the latest study with a comprehensive sample base. The pavement design procedure is based on the cumulative number of expected Equivalent Single-Axle Loads during the design period. The projected Cumulative ESALs as computed over the design life of 10 years for road design in Pakistan.

3.3. CONCLUSIONS

Following conclusions are made after brief study of pavement design reports from different design firms of Pakistan. The overall summary has been given in table 3-2

Table 3-2 Summary of Flexible Pavement Design in Pakistan

Parameter	Min Value	Max Value	Average Value
ESAL	2 Millions	48 Millions	25 Millions
Reliability Level (R)	90%	95%	92.5%
Standard Deviation (So)	0.40	0.50	0.45
Initial Serviceability (Pi)	4.2	4.5	4.35
Terminal Serviceability (Pt)	2	2.5	2.25
Effective Roadbed Resilient Modulus (Mr)	10000 Psi	15000 Psi	12500 Psi
Elastic Modulus for Asphaltic Wearing Course (E_{AWC})	420000 Psi	450000 Psi	435000 Psi
Elastic Modulus for Asphaltic Base Course (E_{ABC})	325000 Psi	350000 Psi	337500 Psi
Elastic Modulus for Granular Base (E_{GB})	30000 Psi	42000 Psi	36000 Psi
Elastic Modulus for Granular Sub-Base (E_{GSB})	18000 Psi	22000 Psi	20000 Psi
Layer Coefficient for Asphaltic Wearing Course (a_{AWC})	0.42/inch	0.44/inch	0.43/inch
Layer Coefficient for Asphaltic Base Course(a_{ABC})	0.36/inch	0.42/inch	0.39/inch
Layer Coefficient for Granular Base (a_{GB})	0.13/inch	0.14/inch	0.135/inch
Layer Coefficient for Granular Sub-Base (a_{GSB})	0.11/inch	0.12/inch	0.115/inch
Drainage Coefficient for Granular Base (m_{GB})	1	1	1
Drainage Coefficient for Granular Sub-Base (m_{GSB})	1	1	1
Service Life	10 Years	10 Years	10 Years

Following discrepancies in pavement design are noted while studying pavement design reports in Pakistan.

1. No proper procedure is explained while reducing the asphalt layer thickness to make the design more cost effective. Reduction in asphalt layer thickness has no rationale, just a reduced thickness is used for design without providing any rationale or procedure.

2. Software analysis reports are not attached with the most design reports; design of pavement is generally empirical.

3. Pavement design is termed as feasible/economical but no proper explanations or methodology is explained.

4. CBR method is used to calculate the subgrade resilient modulus, the properties of soil are mentioned but lab results are not attached with the reports.

5. Road note 31 is used for design by some firms but no proper procedure is written in the report. Only AASHTO procedure is explained.

6. Design Reports content is generally the same and repeated in every report. No proper consideration is given to environmental effect in some design reports.

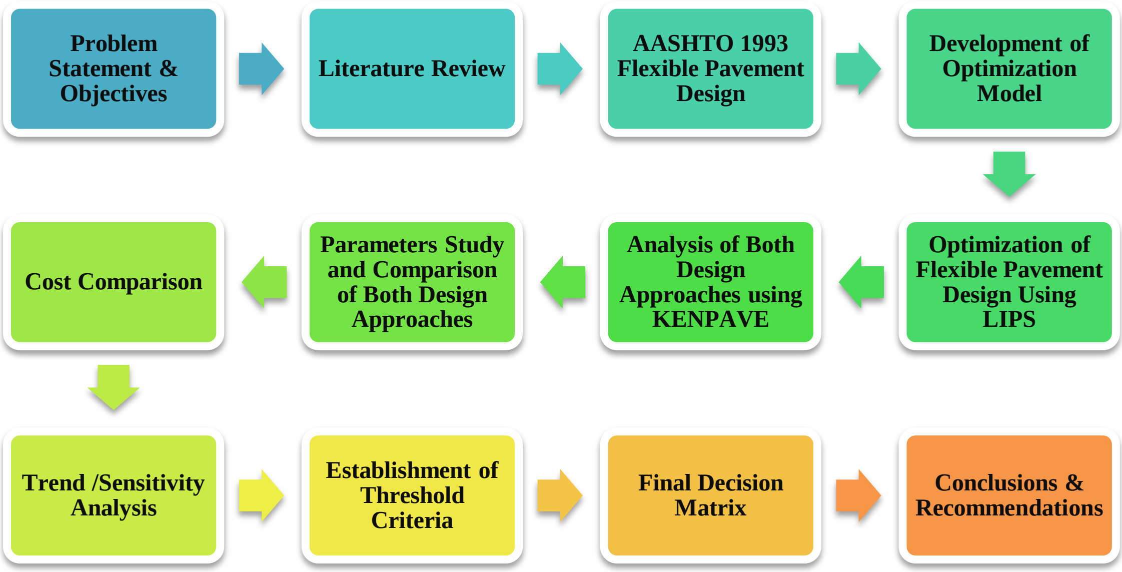
4. METHODOLOGY

4.1. INTRODUCTION

This AASHTO 1993 design guidelines for structural design of pavements are dependent on application of load by different vehicles directly on pavement layers, depending upon their respective stiffness's. Therefore, according to AASHTO design it can be concluded that reduction in thickness of asphalt concrete layer will directly increases the thickness of base and sub base layers, hence adjusting the respective stresses and stiffness's. To make the pavement structure more economical this technique seems to be considerable in a developing country like Pakistan where very high quality granular material is abundantly available. The thickness of asphalt concrete layer due to its high construction cost is a hot issue among highway agencies and engineering consultancies all over the world as asphalt concrete is an expensive material. A good technique to resolve this issue may be the nonlinear analysis of the pavement using different pavement analysis and design software's with the reduction in AC thickness up to minimum, and its impact on cost and pavement stability. If there is appreciable impact on cost with no compromise on stability resulted from actual AASHTO design then the claim of high cost, from highway agencies, due to AC thickness may be sincerely consider and subsequently new local parameters of pavement thickness design may set to save the budget for highway construction using local material. The existing design practices in road construction in Pakistan will be reviewed and suitable recommendations will be provided to concerned departments.

Pavement analysis and design will be carried out by using different software's which includes MFPD (Michigan Flexible pavement structural design), MICH PAVE, BISAR and KENLayer software's. Iterations will be done to reach the desirable values of pavement thickness. AASHTO design method will be used for pavement thickness design. KENLAYER software was used to carry out the analysis on pavement design. Little or no attention has been made on engineering economy while designing a flexible pavement. This in place cost of flexible pavement is generally out of scope of structural design method.

4.1.1 Research Methodolgy:



4.2. OPTIMIZATION OF FLEXIBLE PAVEMENT STRUCTURAL DESIGN

An optimization system was introduced in this research work and the purpose of this system is to develop a practical and realistic model for the optimal selection flexible pavement thickness design. The main aim is to reduce or optimize the asphalt layer thickness keeping in view the AASHTO minimum pavement thickness criteria. For the purpose of optimization, a linear integer programming concept is used. LIPS computer program is used for the purpose of developing a mathematical model for optimization of flexible pavement layer thickness. The design model consists of an objective function and constraining equations. The total flexible pavement construction cost is described by the objective function and a minimum cost solution is obtained for each combination of material cost, design requirements and environmental conditions.

The constraining equations in the model describe the boundary conditions by which the design of flexible pavement is subjected. These boundary conditions are obtained from AASHTO flexible pavement structural design guide and incorporate in the mathematical model. The model is solved by a linear integer programming technique using the LIPS computer program for any flexible pavement structural design situation.

4.2.1 Flexible pavement structural design LIPS code:

Equation 2-2 shows the basic equation based on AASHTO 1993 flexible pavement structural design guide to be used in mathematical model.

In the equation 2-2 all parameter are known except the structural number (SN) which is calculated using the AASHTO nomograph. Various measures of traffic conditions, soil support, pavement performance and environmental effects are combined into a single design parameter defined as a structural number. A nomograph for flexible pavement has been prepared by the AASHTO to quantify the structural requirements (AASHTO, 1993). The above equation was developed from the nomograph to be used in computer program for design procedure. The whole process is an iterative procedure. After finding the SN value, thickness of each layer of flexible pavement is found by converting the SN to real thickness of the constituent layer. These layers' thickness should satisfy the equation 2-2.

In AASHTO design method the quality of asphalt, base and sub base layer is governing by their structural later coefficients. AASHTO design method gives correlation charts for calculation of structural layer coefficients. Moreover, resilient modulus can be calculated using these correlations charts. In order to transform the structural number of different pavement layers into respected pavement layer thicknesses, following layer coefficients have been adopted.

Asphaltic Wearing Course, a1	=	0.44 /inch	(0.173/cm)
Asphaltic Base Course, a1	=	0.36/inch	(0.141/cm)
Base Course ,a2	=	0.14 /inch	(0.055/cm)
Granular Subase, a3	=	0.11 /inch	(0.043/cm)

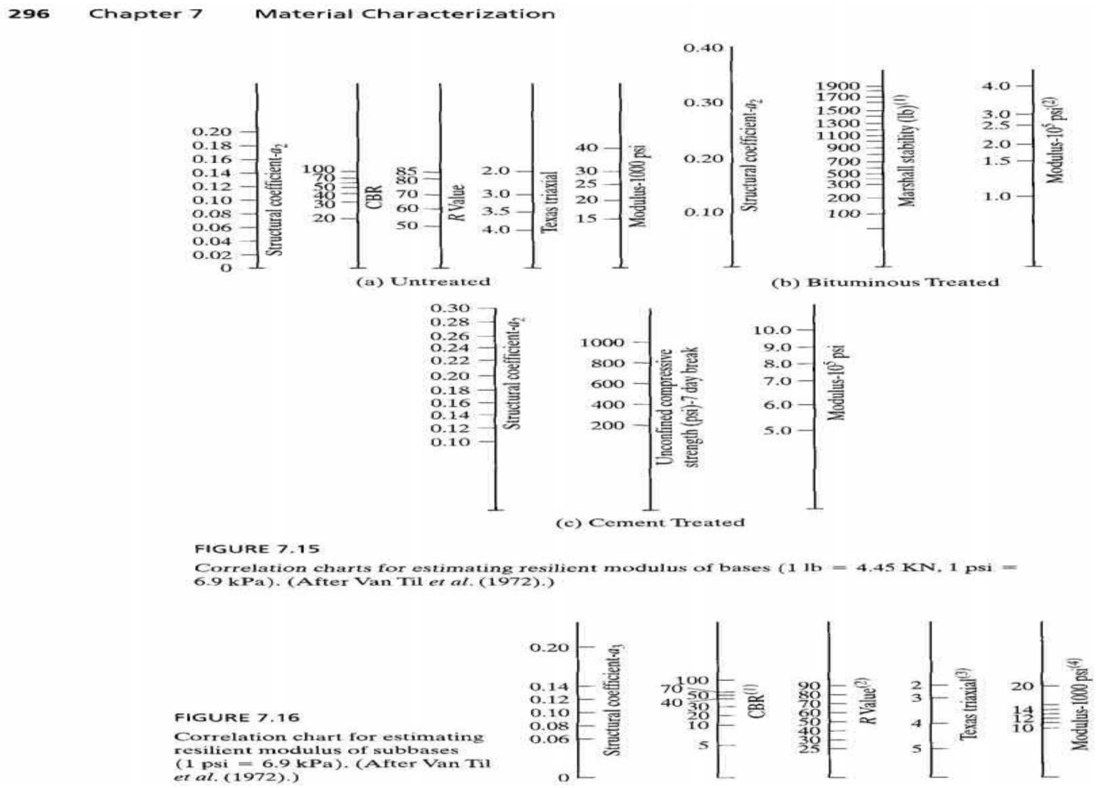


Figure 4-1 Correlation charts (Huang 1993 Chapter 7)

Other design parameters used for calculation of SN are shown in table 4-1.

Table 4-1 Design Parameters for Flexible pavement structural design

SR	Reliability	Standard Normal Deviate (Zr)	So	Pi	Pt
1	90	-1.282	0.45	4.2	2.5

During the design of flexible pavement, not only the equation 4.1 has to be satisfied but also the thickness of each layer should be such that the total compressive stress applied on the lower layers be reduced to tolerable stress of these layers. For this purpose, equation, 4-1 should also be satisfied.

$$a_1 . d_1 \geq SN_1$$

$$a_1 . d_1 + a_2 . m_2 . d_2 \geq SN_2 \qquad \text{Equation 4-1}$$

$$a_1 . d_1 + a_2 . m_2 . d_2 + a_3 . m_3 d_3 . \geq SN_3$$

In equation above SN_1, SN_2 and SN_3 are the structural number of granular base, granular sub base and sub grade layer. Values of structural numbers are found by using equation 3.1 or AASHTO nomograph. The only difference is while calculating SN_1 resilient modulus of granular base is used similarly while calculating SN_2 resilient modulus of sub base is used and resilient modulus of sub grade is used to calculate the SN_3. The thickness of asphalt concrete layer and granular base layer should not be taken less than those given in table 4.2. For granular subbase the minimum thickness is considered as 20 cm

TABLE 4.2 Minimum Thickness for Asphalt Surface and Aggregate Base

Traffic (ESAL)	Asphalt concrete	Aggregate base
Less than 50,000	1	4
50,001—150,000	2	4
150,001—500,000	2.5	4
500,001—2,000,000	3	6
2,000,001—7,000,000	3.5	6
Greater than 7,000,000	4	6
Source . After AASHTO (1986).		

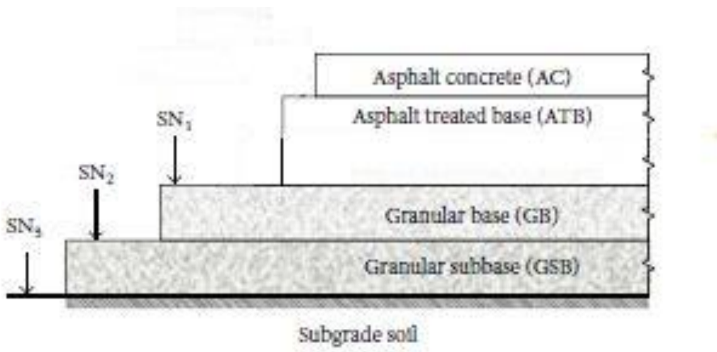


Figure 4-2: Assumed structure of pavement for optimization (Carvalho, 2006)

4.2.2 Optimal Design of Flexible Pavement LIPS CODE

Considering a five-layer flexible pavement system as shown in figure 4-2, the linear integer programming model to find the optimum structural configuration and optimum thickness of asphalt concrete, base and sub base layers can be written as follows. In the figure above the AC is the asphalt wearing course while the ATB represents the asphalt base course.

Objective Function:

Min C

$$= \frac{C_{AC}.d_{AC}}{100} + \frac{C_{AB}.d_{AB}}{100} + \frac{C_{GB}.d_{GB}}{100} + \frac{C_{GSB}.d_{GSB}}{100} \qquad \text{Equation 4-2}$$

Where

C = Construction Cost of one square meter of flexible pavement

C_{AC} = Construction Cost of asphalt wearing course material in cubic meters

C_{AB} = Construction Cost of asphalt base course material in cubic meters

C_{GB} = Construction Cost of granular base material in cubic meters

C_{GSB} = Construction Cost of granular sub base material in cubic meters

The constraint use in the mathematical model is as follows

Constraints

$a_{AC}\,.d_{AC} + \; a_{AB}\,.d_{AB} \geq SN_1$

$a_{AC}\,.d_{AC} + a_{AB}\,.d_{AB} + a_{GB}\,.m_{GB}\,.d_{GB} \geq SN_2$ $\longrightarrow$ Equation 4-3

$a_{AC}\,.d_{AC} + a_{AB}\,.d_{AB} + a_{GB}\,.m_{GB}\,.d_{GB} + \; a_{GSB}\,.m_{GSB}\,.d_{GSB} \geq SN_3$

$d_{AC} \;\geq$ Min d_{AC}

$d_{AB} \;\geq$Mind_{AB} $\longrightarrow$ Equation 4-4

$d_{GB} \;\geq$ Min d_{GB}

$d_{GSB} \;\geq$ Min d_{GSB}

Decision Variables

d_{AC} , d_{AB}, d_{GB},d_{GSB}=integer $\longrightarrow$ Equation 4-5

Where

d_{AC} = Thickness of asphalt wearing course layer (cm)

d_{AB} = Thickness of asphalt base course layer (cm)

d_{GB} = Thickness of granular base courser layer (cm)

d_{GSB} = Thickness of granular sub base course layer (cm)

a_{AC} = Layer coefficient of asphalt wearing course layer

a_{AB} = Layer coefficient of asphalt base course layer

a_{GB} = Layer coefficient of granular base layer

a_{GSB} = Layer coefficient of granular sub base layer

m_{GB} = Drainage coefficient of granular base layer

m_{GSB} = Drainage coefficient of granular sub base layer

Min = Minimum Thickness

Constraints shown by equation 4.5 are related to maximum allowable compressive stress on granular base, subbase and subgrade layers while constraint shown by equation 4.6 are related to minimum constraint thickness of respective flexible pavement layers which are to be taken from AASHTO 1993 guide. The objective of this mathematical model is to minimize the total cost of pavement system.

4.2.3 Optimized Flexible Pavement Structural Design

The development of this mathematical program and system provides the optimum flexible pavement structural design. The model while doing linear integer programming selects the flexible pavement cross -section which minimizes the total construction cost. The flexible pavement cross- section also fulfils the design objectives for the least total in place cost. This cost effectiveness approach provides an optimal, practical and economical solution to the problem of designing flexible pavement.

Each optimization is carried out for different ESALS. The ranges of ESALS are taken from AASHTO 1993 guide. Five values of ESALS are taken in each range, other parameters like elastic modulus, structural layer coefficients, drainage coefficients of pavement layers, standard deviation, reliability value and initial and terminal serviceability is taken as constant. These design parameter values are mentioned in table 4.1, section 4.2.1 of this chapter and table 4-3.

Table 4-3: Design Parameters for flexible pavement structural design

Layer Description	**Layer Coefficient, a_i**	**Drainage Coefficient, m_i**	**Elastic Modulus, psi**
Asphalt Wearing Course Layer	0.44	1.00	450,000
Asphalt Base Course	0.36	1.00	350,000
Granular Base	0.14	1.00	42,000
Granular Subbase	0.11	1.00	22,000

The resilient modulus for sub grade is taken as 13000 psi for pavement design. The numbers of equivalent single axle loads (ESALs) was assumed from 10000 to 50000000 ESALs with a proper increment between ranges provided by AASHTO. Structural number for each value of ESALs was calculated by using AASHTO nomograph. For each value of ESALs thickness of asphalt wearing course, asphalt base course, granular base and granular sub base was calculated using the nomograph.

By running the optimization model in program, an optimize thickness of each layer was obtained. After obtaining the thickness an analysis was run on KENLAYER computer program in order to calculate the tensile and compressive stresses and strains. These stresses and strains are than used to calculate the fatigue and rutting values in term of damage ratios.

This optimization model determines the optimum thickness of each pavement layer, using the NHA composite schedule of rates 2014 (NHA CSR 2014) the total construction cost of each optimum configuration of flexible pavement is determined. Table 4-4 shows the costs of each pavement layer based on the material used in construction.

Table 4-4 Construction Cost for Different Materials

Costs as per NHA CSR 2014 (Rawalpindi)					
Sr #	**Description**	**Unit**	**Cost (Rs)**	**Unit**	**Cost (Rs)**
1	ASPHALTIC CONCRETE FOR WEARING COURSE (CLASS A)	CM	19367.03	SM per cm	193.67
2	ASPHALTIC BASE COURSE PLANT MIX (CLASS A)	CM	18395.54	SM per cm	183.95
3	GRANULAR BASE	CM	2127.3	SM per cm	21.27
4	GRANULAR SUB-BASE	CM	1776.13	SM per cm	17.76

4.3 Flexible Pavement Structural Design Based on AASHTO 1993

Flexible pavement structural design depends upon two basic parameters; traffic and soil strength. The pavement design has been carried using AASHTO 1993 approach. The AASHTO procedure uses Structural Numbers and Serviceability Indices, layer thicknesses, environment, drainage and reliability.

4.3.1 Traffic Studies

Traffic studies are intended to provide necessary input data for determination of the magnitude and pattern of the traffic load for the project highway through the design period. This entails collection, verification and analysis of the traffic data. From the collected data, the projected traffic for the design life of the subject highway is determined.

4.3.2 Flexible Pavement Structural Design Life

Design life is the number of years reckoned from the completion of pavement construction and application of traffic load until the time when major maintenance is required so that it can continue to carry traffic satisfactorily for further period. A design period of 10 years has been adopted, which is a normal practice.

4.3.3 ESALs for Flexible Pavement Structural Design

The pavement design procedure is based on the cumulative number of expected Equivalent Single-Axle Loads during the design period. For the purpose of this study a range or ESALs are taken based on AASHTO design guide. The ranges are same as used in optimization of flexible pavement structural design.

4.3.4 Flexible Pavement Structural Design using AASHTO 1993

The pavement design process is the technique of developing a combination of top layers of different materials in most economical manner to cater for the total axle load over the design life of a highway. In other words, this is an art through which the stresses as induced in the top layers of a highway due to movement of heavy wheel load is disseminated and minimized to safe level through selection of different type and appropriate thickness of pavement layers. The AASHTOGuide1993 for Design of Pavement Structure has been used to compute the pavement thicknesses.

The AASHTO Guide for Pavement Design 1993 outlines this procedure for determination of flexible pavement thickness by solving AASHTO equations manually, by using different nomographs or by using the computer software.

For the purpose of this studies an excel sheet was developed for design of flexible pavement based on AASHTO 1993 guidelines, this excel sheets uses the nomograph for computation of flexible pavement layers' thickness based on structural number. Beside using the excel sheet some computer software are also used that includes, Michigian flexible pavement structural design software (MFPDS) for checking purpose. The excel sheet also uses equation 4.1 for pavement layer thickness computations. General design variables used in flexible pavement structural design have been discussed as under

4.3.4.1 Reliability (R)

Design reliability refers to the degree of certainty that a given design alternative will last for the entire design period. A design reliability level of 90% has been adopted for pavement design of the project road.

4.3.4.2 Standard Deviation(So)

The reliability factor is a function of the overall standard deviation that accounts for standard variation in materials and construction, the probable variation in the traffic prediction and the normal variation in pavement performance for a given design traffic application. The recommended value of standard deviation for total variation in material properties and in traffic estimation for flexible pavement is 0.45 and has been adopted for pavement design of the expressway.

4.3.4.3 Standard Normal Deviation (Z_R)

The value corresponding to reliability (R) of 90% is -1.282 which has been adopted in the design based on the recommended values of standard normal deviation (ZR) by AASHTO Guide 1993.

4.3.4.4 Performance Criteria

The serviceability of a pavement is defined as its ability to serve the type of traffic that uses the facility. Initial and terminal serviceability indices have been established to compute the total change in serviceability that will be used in the design equations.

Initial Serviceability Index (P_o)

The initial serviceability index is a function of pavement design and construction quality. For flexible pavement design typical value, as recommended by AASHTO Road Test, is 4.2 which has been adopted.

Terminal Serviceability Index (P_t).

The terminal serviceability index is the lowest index that will be tolerated before rehabilitation, resurfacing or reconstruction, becomes necessary and it generally varies with the importance or functional classification of the pavement. Recommended value of terminal serviceability index is2.5 for the expressway.

4.3.4.5 Resilient Modulus M_R

The basis for material characterization in the AASHTO Guide 1993 is Elastic or Resilient Modulus (MR). In the absence of necessary equipment required to determine resilient modulus of unbound layers, following correlation between CBR and MR.

$$M_R = 2555\ (CBR)^{0.64}$$

Where;

CBR=California Bearing Ratio in percentage.

M_R = Resilient modulus in psi

4.3.4.6 Computation of Required Pavement Thickness

The structure number(SN) requirement as determined through adopting the design parameters as discussed above is balanced by providing adequate pavement structure. Under AASHTO design procedure equation 4.2provides the means for converting the structural number into actual thicknesses of surfacing, base and subbase materials.

4.3.4.7 Design and Analysis of Flexible Pavement Structure

After calculations of required structural number, the required data is used to calculate the design layer thicknesses. For each value of ESALs an adequate pavement structure is design. In order to verify the thickness obtained from AASHTO 1993 procedure mechanistic approach is used. KENLAYER software used for this purpose. KENLAYER is a linear/nonlinear finite element computer program developed by Kentucky State University. This software has been used for design and analysis of flexible pavement structures.

Each layer in pavement is divided into discrete elements. The stress state in each element is calculated using theory of elasticity, assuming adjacent elements are dependent upon each other and they are connected at the nodes. The formulation of finite elements allows the modulus of any particular element to be a function of the stress level within the element. However, for the purpose of this study a linear analysis technique was adopted.

KENLAYER uses ASPHALT INSTITUTE distress models for the determination of layer thicknesses. This software is used to carry out the analysis of the design pavement structure obtained from AASHTO 1993 method. For each pavement configuration value of compressive and tensile strains are obtained. The critical strain and stress in flexible pavement structural design occurs at some point. The horizontal tensile strain at the bottom of asphalt layer and vertical compressive strain at the top of subgrade is the critical one. The value of these two strains are obtained after carrying out the analysis using KENLAYER. After calculating the strains, service life of each pavement configuration for each value of ESAL is obtained followed by the damage ratio for fatigue and rutting values.

All the results are than compared within the range of ESALs. Graphs are plotted for increase of decrease in the value of analysis parameters.

4.4 Cost Analysis

Cost analysis was carried out for comparison between design and optimize pavement configuration. For the purpose NHA composite schedule of rates (CSR) 2014 was used. Table 4-4 shows the costs of each pavement layer based on the material used in construction. Cost is calculated for square meter per cm of pavement structure. NHA CSR 2014 provides material cost as per cubic meter but for this research studies as cost is related to thickness of pavement layers hence the calculated cost is based on square meter per cm of pavement. The cost given in NHA CSR 2014 is divided by

100 to convert into cm. Table 4-4 gives the cost as per material used for pavement construction also Figure 4-3 illustrates the idea.

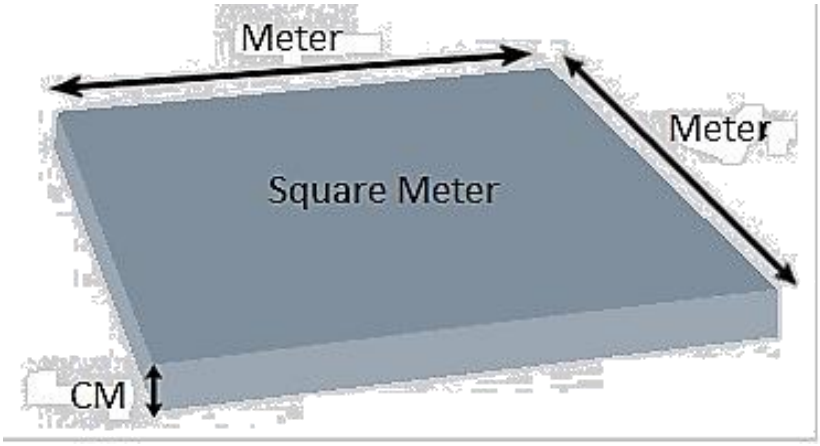


Figure 4-3 Cost Calculation as per NHA CSR 2014

4.5 Flexible Pavement Analysis using KENPAVE:

KENPAVE software was developed by Huang, 1993 (Huang, 2004). It is a Microsoft-Windows based version that combines the old Kenlayer flexible pavement software and Ken slabs rigid pavement software. This software allows the use of linear elastic, nonlinear, and viscoelastic properties of the materials for the different layers. The software can handle up to 19 layers and performs damage analysis. The interface between the different layers can be specified as either unbounded or fully bonded. KENLAYER can be applied to layer systems under single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently, linear elastic, nonlinear elastic, or viscoelastic. Damage analysis can be made by dividing each year into a maximum of 12 periods, each with a different set of material properties. Each period can have a maximum of 12 load groups, either single or multiple. The damage caused by fatigue cracking and permanent deformation in each period over all load groups is summed up to evaluate the design life (Huang, 2004). KENPAVE uses Asphalt institute distress models for the determination of layer thicknesses

4.5.1 Input Parameters in KENPAVE Computer Program:

For the purpose of analysis in KENPAVE computer program many input parameters are required. The parameters can be inputted both in SI and U.S. customary units. Some of the input parameters for linear elastic analysis are traffic load, material properties, thickness of each layer, number of periods, number of load groups etc.

4.5.2 Output Parameters of KENPAVE:

For a single and multiple load groups, a maximum of nine and ten responses can be obtained, respectively. Only the vertical compressive strain on the surface of subgrade and the radial (Tangential) tensile strains at the bottom of asphalt layer are used for damage analysis.

An analysis file is shown below:

INPUT FILE NAME -C:\KENPAVE\10000 red.DAT

NUMBER OF PROBLEMS TO BE SOLVED = 1

TITLE : Thesis

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=2, SO DAMAGE ANALYSIS WITH DETAILED PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa ,unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 2 2 10.16 15.24
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750 3 24.000 0.000 4 24.000 6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 10000

```
DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2  ARE: 0.414  3.291  0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5  ARE:  1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO.  1  LOAD GROUP NO.  1

 POINT    VERTICAL   VERTICAL   VERTICAL     MAJOR       MINOR  INTERMEDIATE
                               PRINCIPAL    PRINCIAL  P. STRESS
  NO.    COORDINATE    DISP.     STRESS      STRESS      STRESS  (HORIZONTAL
                       (STRAIN)    (STRAIN)    (STRAIN)  P. STRAIN)
  1      4.00000    0.05694     441.675     441.675    -891.795    -887.555
         (STRAIN)            4.411E-04  4.411E-04  -3.049E-04  -3.049E-04
  1     29.40010    0.03462      63.293      63.299       1.192       2.715
         (STRAIN)            6.865E-04  6.866E-04  -3.181E-04  -3.181E-04

  2      4.00000    0.05318     370.343     378.735    -810.542    -740.198
         (STRAIN)            3.772E-04  3.819E-04  -2.835E-04  -2.835E-04
  2     29.40010    0.03420      59.086      60.500       1.178       3.140
         (STRAIN)            6.304E-04  6.533E-04  -3.064E-04  -3.064E-04

  3      4.00000    0.03044       8.245     250.942       0.110       1.717
         (STRAIN)           -3.205E-05  1.037E-04  -3.660E-05  -3.660E-05
  3     29.40010    0.02737      29.396      39.391       0.902       2.346
         (STRAIN)            2.615E-04  4.232E-04  -1.995E-04  -1.766E-04

  4      4.00000    0.03003       6.504     231.156       0.631       4.307
         (STRAIN)           -3.060E-05  9.507E-05  -3.389E-05  -5.314E-05
  4     29.40010    0.02731      28.017      38.062       0.919       2.767
         (STRAIN)            2.437E-04  4.061E-04  -1.947E-04  -1.921E-04

AT BOTTOM OF LAYER  2   TENSILE STRAIN =  -3.049E-04
ALLOWABLE LOAD REPETITIONS =   5.457E+05  DAMAGE RATIO =   1.832E-02

AT TOP OF LAYER  5   COMPRESSIVE STRAIN =   6.865E-04
ALLOWABLE LOAD REPETITIONS =   1.984E+05  DAMAGE RATIO =   5.041E-02

******************************
* SUMMARY OF DAMAGE ANALYSIS *
******************************
AT BOTTOM OF LAYER  2     SUM OF DAMAGE RATIO = 1.832E-02
AT TOP OF LAYER  5        SUM OF DAMAGE RATIO = 5.041E-02

MAXIMUM DAMAGE RATO =   5.041E-02   DESIGN LIFE IN YEARS = 19.84
```

Different type of materials can be selected in this software. The material type can be input in form of numbers, 1 when all layers are linear elastic, 2 when some layers are nonlinear elastic and the remaining, if any, are linear elastic, 3 when some layers are viscoelastic and the remaining, if any, are linear elastic, 4 when some layers are nonlinear elastic, some are viscoelastic, and the remaining, if any, are linear elastic. For the purpose of this study 1 is selected because it is assumed that all layers are linear elastic.

NDAMA (damage analysis): 0 no damage analysis, 1 damage analysis with summary printout, and 2 damage analysis with more detailed printout. When a large number of periods or load groups are used, the use of 2 may result in a large volume of printout and is therefore not recommended. Both 0 and 1 can be input according to the requirements. For this research 0 and 1 is selected as both detailed and no damage analysis is required for analysis purpose.

Each year can be divided into a maximum of 12 periods for damage analysis. Even without damage analysis, NPY can be used to find the effect of layer moduli on pavement responses by assigning different moduli for each period. One period per year is selected for this study purpose as resilient modulus value is already known, which is based on seasonal variations.

Number of layers, maximum 19): The default NL is 3 which you probably would like to change, as indicated in red. As total number of layers to be analyzed are 5 so NL=5.

NLBT (number of layers with damage analysis based on the tensile strain at the bottom of asphalt layer). In most cases, NLBT =1. If NLBT is more than 1, damage ratios at NLBT locations will be compared and the maximum ratio determined. AS number of layer for tensile strain is one i.e. at the bottom of asphalt base course layer, so 1 is taken as the value in software.

NLTC (number of layers with damage analysis based on the vertical compressive strain at the top of subgrade or other unbonded layers). In most cases, NLTC =1. If NLTC is more than 1, damage ratios at NLTC locations will be compared and the maximum ratio determined. Similarly, vertical compressive strain is required only at the top of sub-grade layer so NLTC=1 in this study.

Next the thickness of each layer is required which is already design and optimize. Analysis has been carried out for both design and optimizes flexible pavement thicknesses. Analysis results are computed for each ESALs value. Elastic modulus and poison ratios of each layer remains constant throughout the analysis.

For the purpose of analysis, it is assumed that load is applied on the pavement as single axle load with two contact areas i.e. single axle with two tires. Spacing between both tires is assumed to be 120 cm with contact radius of 11.3 cm and tire pressure of 552 kPa. Due to asymmetry results on one side are equal to the other side. Results are required at four points in pavement cross-section mentioned below.

Table 4-5 Loading Points on KENPAVE

Point	XPT (x coordinates of points to be analyzed) cm	YPT (y coordinates of points to be analyzed) cm
1	0	0
2	0	6.75
3	24	0
4	24	6.75

Next the layer number for bottom tension and top compression is required, which in this research study are layer 2 and layer 5. The volume of traffic is input in the form of ESALs. After giving required inputs to the software the analysis was run.

After running analysis, the software provides with tensile strain at the bottom of asphalt layer, compressive strain at the top of sub-grade, damage ratio for fatigue and rutting and also the sum of damage ratios. The software also predicts the design life of pavement structure.

Chapter 5

5. Analysis of Basic AASHTO & Optimized Flexible Pavement Designs

5.1 AASHTO Flexible Pavement Structural Design:

For the purpose of pavement design, AASHTO 1993 design guide procedure was implemented. Using the AASHTO 1993 equation, required SN for each layer was computed and layer thicknesses was calculated by trial and error method. A flexible pavement distributes the traffic loads through a system of pavement components to sub-grade. These components are the pavement layers that are generally identified as surface layer, base and sub-base layers. For the purpose of this research work, the surface layer comprises of asphalt wearing course layer and asphalt base course layer. The differences between two layers are the change in their elastic modulus and structural layer coefficients. Different acceptable cross sections were design for different traffic loadings in terms of ESALs.

Design parameters for flexible pavement structural design are mentioned in table 4.3. The resilient modulus for sub grade is taken as 13000 psi for pavement design. The numbers of equivalent single axle loads (ESALs) was assumed from 10000 to 50000000 ESALs with a proper increment between ranges provided by AASHTO. Structural number for each value of ESALs was calculated by using AASHTO nomograph. For each value of ESALs thickness of asphalt wearing course, asphalt base course, granular base and granular sub base was calculated using the nomograph. AASHTO Guide 1993 for Design of Pavement Structure has been used to compute the pavement thicknesses. AASHTO Guide for Pavement Design 1993 outlines this procedure for determination of flexible pavement thickness by solving AASHTO equations manually, by using different nomographs or by using the computer software. For the purpose of this studies an excel sheet was developed for design of flexible pavement based on AASHTO 1993 guidelines, this excel sheets uses the nomograph for computation of flexible pavement layers' thickness based on structural number. Beside using the excel sheet some computer software are also used that includes, Michigian flexible pavement structural design software (MFPDS) for checking purpose. The excel sheet is based on equation 4.1 for pavement layer thickness computations.

5.1.1 Flexible Pavement Structural Design Using Excel Sheet:

An excel sheet meant for designing flexible pavement is used, few changes are made according to this research. The typical cross section of pavement to be design is shown in figure 5-1.

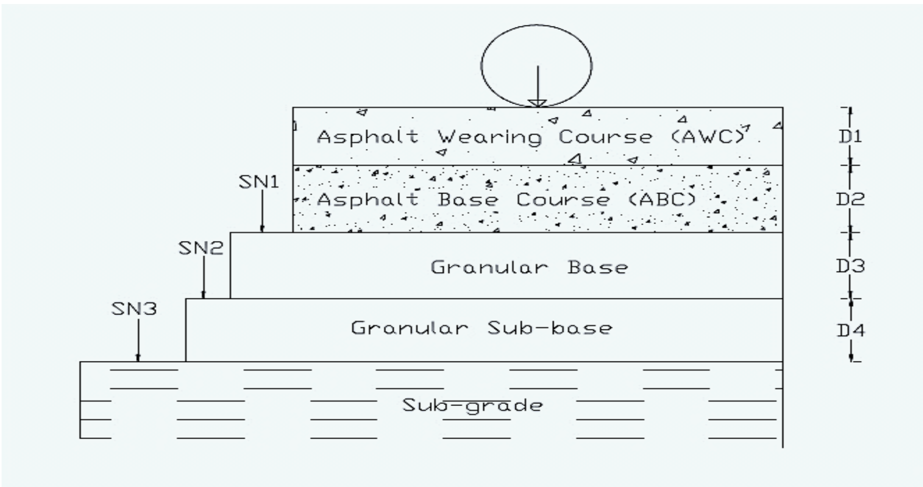


Figure 5.1 Assumed Pavement Cross –section

The top layer that comes in contact with traffic is called as asphalt wearing course layer. It may be composed of one or several different HMA sublayers. It provides characteristics such as friction, smoothness, noise control, rut and shoving resistance and drainage. In addition, it serves to prevent the entrance of excessive quantities of surface water into the underlying base, sub-base and sub-grade (NAPA, 2014). This top structural layer of material is sometimes subdivided into two layers:

Wearing Course. This is the layer in direct contact with traffic loads. It is meant to take the brunt of traffic wear and can be removed and replaced as it becomes worn. A properly designed (and funded) preservation program should be able to identify pavement surface distress while it is still confined to the wearing course. This way, the wearing course can be rehabilitated before distress propagates into the underlying intermediate/binder course. Wearing course structural properties are mentioned in table 5.1.

Asphalt Base Course. This layer provides the bulk of the HMA structure. Its chief purpose is to distribute load. The structural properties used for design purposes are shown in table 5-1.

Table 5-1 Flexible pavement structural design Parameters

Layer Description	**Layer Coefficient, a_i**	**Drainage Coefficient, m_i**	**Elastic Modulus, psi**
Asphalt Wearing Course Layer	0.44	1.00	450,000
Asphalt Base Course	0.36	1.00	350,000
Granular Base	0.14	1.00	42,000
Granular Sub-base	0.11	1.00	22,000

These design parameters values are obtained from studying different flexible pavement structural design reports. These design reports are obtained from NESPAK and NHA Pakistan. These design parameters are based on environmental conditions of Rawalpindi/Islamabad city in Pakistan. The resilient modulus for sub grade is taken as 13000 psi for pavement design. This resilient modulus was obtained from CBR equation. The design of pavement is based on 13% CBR values of sub-grade at 95 % MDD AASHTO T-180. The basis for material characterization in the AASHTO Guide 1993 is Elastic or Resilient Modulus(MR).

In this research studies different pavement cross sections are designed based on ESALs, other design parameters which includes, reliability factor, reliability, standard deviation, initial and terminal serviceability, resilient modulus, structural layer coefficients, drainage coefficient and layer elastic modulus are kept constant. Table 5-2 showing design parameters used in AASHTO design equation.

Table 5-2 AASHTO Design Equation Parameters

Reliability	95%
Standard Deviation (S_0)	0.45
Standard Normal Deviation (Z_r)	-1.645
Initial Serviceability Index (P_o)	4.2
Terminal Serviceability Index (P_t)	2.5
Resilient Modulus (M_r)	13000 Psi or 913.99 Kg/Cm2

Using the above mentioned values, pavement layer thicknesses are computed with respect to ESALs ranging from 10000 to 70000000. During the design of flexible pavement, not only the equation 4.1 has to be satisfied but also the thickness of each layer should be such that the total compressive stress applied on the lower layers is reduced to tolerable stress of these layers. For this purpose, equation, 4.3 should also be satisfied.

Values of structural numbers are found by using equation 4.1 or AASHTO nomograph. The only difference is while calculating SN_1 resilient modulus of granular base is used similarly while calculating SN_2 resilient modulus of sub base is used and resilient modulus of sub grade is used to calculate the SN_3. The thickness of asphalt concrete layer and granular base layer should not be taken less than those given in table 4-2. For granular sub-base the minimum thickness is considered as 20 cm. There are many combinations of layer thicknesses that can be adopted to achieve a given structural number. These are, however several design, construction and cost constraints that maybe applied to reduce the number of possible layer thickness combinations and to avoid the possibility of constructing an impractical design.

Following thickness combinations are evaluated for each value of ESALs calculated. AWC stands for asphalt wearing course, ABC for asphalt base course. Table 5-3 showing the results.

Table 5-3 Results after AASHTO Design

ESAL	**Asphalt Wearing Course Thickness**	**Asphalt Base Course Thickness**	**Granular Base Course Thickness Design**	**Granular Subbase Design**
	(cm)			
10000	2.54	5.08	10.16	15.24
20000	2.54	5.08	10.16	15.24
30000	2.54	5.08	11	15.24
40000	2.54	5.08	11	15.24
50000	2.54	5.08	11	15.24
55000	4	5.08	11	15.24
75000	4	5.08	11	15.24
100000	5.08	5.08	12	20.32
125000	5.08	5.08	12	20.32
150000	5.08	5.08	12	20.32
175000	5.08	7.62	15.24	20.32
200000	5.08	8	15.24	20.32
300000	5.08	10.16	15.24	20.32
400000	5.08	10.16	15.24	20.32
500000	5.08	10.16	15.24	20.32
550000	5.08	10.16	15.24	20.32
800000	7.62	10.16	15.24	20.32
1000000	10.16	10.16	15.24	20.32

1500000	10.16	10.16	15.24	20.32
2000000	10.16	12.7	15.24	20.32
2500000	10.16	12.7	15.24	20.32
3000000	12	12.7	15.24	20.32
4500000	12.7	12.7	20.32	20.32
5500000	12.7	15.24	20.32	20.32
7000000	13	15.24	20.32	20.32
10000000	13.97	15.24	25.4	25.4
20000000	16.51	16.51	25.4	25.4
30000000	17.78	18.29	25.4	25.4
40000000	18.29	19.05	25.4	25.4
50000000	19.05	20.32	35.56	38.1
55000000	19.5	20	36	38
60000000	20	20	36	38
65000000	20	21	36	38
70000000	20	21.5	36	38

5.2 Analysis of Optimized Flexible Pavement Structural Design:

An optimization system was introduced in this research work and the purpose of this system is to develop a practical and realistic model for the optimal selection flexible pavement thickness design. The main aim is to reduce or optimize the asphalt layer thickness keeping in view the AASHTO minimum pavement thickness criteria. For the purpose of optimization, a linear integer-programming concept is used. LIPS computer program is used for developing a mathematical model for optimization of flexible pavement layer thickness. The design model consists of an objective function and constraining equations. The total flexible pavement construction cost is described by the objective function and a minimum cost solution is obtained for each combination of material cost, design requirements and environmental conditions.

Model presented in chapter 4 used for optimization. The objective of this optimal selection is to minimize the total cost of the pavement system. Model is run for each value of ESAL ranging from 10000 to 70000000. For each value of ESAL other design parameters as discussed in flexible pavement structural design section kept constant.

The objective function is

Min C

$$= \frac{C_{AC}.d_{AC}}{100} + \frac{C_{AB}.d_{AB}}{100} + \frac{C_{GB}.d_{GB}}{100} + \frac{C_{GSB}.d_{GSB}}{100}$$

In which C represent the cost and the main objective is to evaluate the pavement configuration with minimum cost.

To quantify the boundary conditions to which the optimal design of the flexible pavement components is subject, the following constraint equations are necessary to complete the realism of this design model

1. The selection of layer thicknesses must satisfy the structural number requirement.

$$a_1D_1 + a_2D_2 + a_3D_3m_3 + a_4D_4m_4 \geq \text{SN}$$

$a_i.$ = coefficient of relative strength of material 'i'

SN = structural number for design.

2. The total thickness of the flexible pavement must be at least equal or greater than minimum thickness requirement given by AASHTO 1993 guide.

$$d_{AWC} + d_{ABC} \geq Min\, d_{AC}$$
$$d_{GB} \geq Min\, d_{GB}$$
$$d_{GSB} \geq Min\, d_{GSB}$$
$$d_{GB} \geq d_{GB}\ (AASHTO\ Design)$$
$$d_{GSB} \geq Min\, d_{GSB}\ (AASHTO\ Design)$$

An upper limit of 20 in is set for thickness of the granular sub-base to conform to present construction practices in Pakistan.

The solution to this design model illustrates the cost-effectiveness evaluation that is permitted in this approach to the structural design of flexible pavements. The development of this procedure for the design of flexible pavement provides a direct determination of the optimal design. The resultant design model involves the selection of that pavement cross-section which minimizes the total cost of the pavement system for the selected unit costs of the pavement materials, for the specified values of the various design and environmental parameters, and for the prevailing construction practices. Each flexible pavement section fulfills the design objectives for the least total in-place cost. Therefore, this cost-effectiveness approach provides an optimal, practical, and economical solution to the problem of designing flexible pavements.

The main concept was to reduce the asphalt layer thickness as asphalt is the costly material. According to AASHTO design it can be concluded that reduction in thickness of asphalt concrete layer will directly increases the thickness of base and sub base layers, hence adjusting the respective stresses and stiffness's. To make the pavement structure more economical this technique seems to be considerable in a developing country like Pakistan where very high quality granular material is abundantly available. The thickness of asphalt concrete layer due to its high construction cost is a hot issue among highway agencies and engineering consultancies all over the world as asphalt concrete is an expensive material.

5.3 Linear Program Solver (LiPs) Program.

Based on the model presented above, a program has been written on LiPs software. The first line is the objective function in which D1, D2, D3 and D4 are defined as the pavement layer thicknesses. The value for cost is obtained from NHA CSR 2014 and table is presented in chapter 4. Each layer compromises of different material hence the construction cost for each layer is different. Cost is evaluated for a meter square per cm of pavement thickness.

Row 1 to Row 7 is the constraints based on AASHTO 1993 guide. From row 1 to row 3 in program the layer thicknesses are satisfying the SN. Each structural number is computed for each ESALs using the AASHTO design equation. From row 4 to row 7 minimum thickness requirement constraint has been modelled in the software. Int mean integer. This model is for 1 Million ESALs.

Objective Function:

min: 193.67*D1 + 183.85*D2 + 21.27*D3 +17.76*D4;

Constraints:

0.173*D1 >= 0.7 (SN1)

0.142*D2 >= 1.89 (SN2)

0.173*D1 + 0.142*D2 + 1*0.055*D3 >= 2.445 (SN3)

0.173*D1 + 0.142*D2 + 1*0.055*D3 + 1*0.043*D4 >= 3.01 (SN4)

D1 + D2 >= 7.62 (AASHTO Min Thickness Req)

D3 >= 15.24 (AASHTO Min Thickness Req)

D4 >= 20 (AASHTO Min Thickness Req)

D3 >= 21 (Basic Optimization Concept)

D4 >= 21 (Basic Optimization Concept)

int D1,D2,D3,D4;

int D1, D2, D3, D4;

The results obtained after linear integer programming, in which several iterations are carried out to reach an optimal configuration are

Table 5-4: Linear Integer Programming Results

Variable	Value	Obj. Cost	Reduced Cost
D1	2	193.67	0
D2	2	183.95	0
D3	11	21.27	0
D4	20	17.76	0
Constraint	**Value**	**RHS**	**Dual Price**
row1	1.087	1.004	1119.479769
row2	111.692	1.35	0
row3	312.552	2.17	0
BND.D1	3	2.54	0
BND.D2	4	2.54	24.98387283
BND.D3	11	10.16	21.27
BND.D4	20	20	17.76

The thickness of each layer is reduced according to the cost of the pavement layer. Hence the optimize pavement layer thickness are obtained. The optimal pavement configuration for each ESAL obtained from linear integer programming software is presented below in table 5-5

Table 5-5 Results of Linear Optimization

ESAL	Asphalt Wearing Course Thickness	Asphalt Base Course Thickness	Granular Base Course Thickness Design	Granular Subbase Design
	(cm)			
10000	2	2	11	20
20000	2	2	11	20
30000	2	4	15	20
40000	2	4	15	20
50000	2	4	15	20
55000	3	4	15	20
75000	3	4	15	20
100000	5	4	15	21
125000	4.5	5	15	21
150000	4.5	5	15	21
175000	5	6	20	21
200000	5	7	20	21
300000	6	7	20	21
400000	7	7	20	21
500000	7	7	20	21
550000	7	7	20	21

800000	7	10	20	21
1000000	8.5	10	20	25
1500000	9	10	20	25
2000000	10	10	25	25
2500000	10	10	25	25
3000000	11	11.5	25	25
4500000	12	12	25	25
5500000	12	13	25	25
7000000	13	14	25	25
10000000	13	14	30	30
20000000	15	15	30	30
30000000	16	17	30	30
40000000	17	18	30	30
50000000	18	19	30	30
55000000	17.5	20	40	40
60000000	18	20	40	40
65000000	18.5	20	40	40
70000000	18.5	20	40	40

5.4 Sensitivity Analysis:

Sensitivity analysis carried out to check the sensitivity of percentage change in different criterion used in analysis. For the purpose of analysis for this research study sensitivity of service life, tensile and compression strains, and damage ratio for fatigue and rutting has been checked. Two alternatives are considered i.e. AASHTO design and optimize flexible pavement design. Percentage change in criteria before and after optimization is calculated and plotted against ESALs. Sensitivity against ESAL has been checked afterwards. The criteria checked for sensitivity are service life of pavement, tensile strains at the bottom of asphaltic concrete layer, compressive strains at the top of sub-grade and damage ratios for fatigue and rutting. Percentage difference after carrying out optimization was plotted against ESALs.

5.4.1 Service Life:

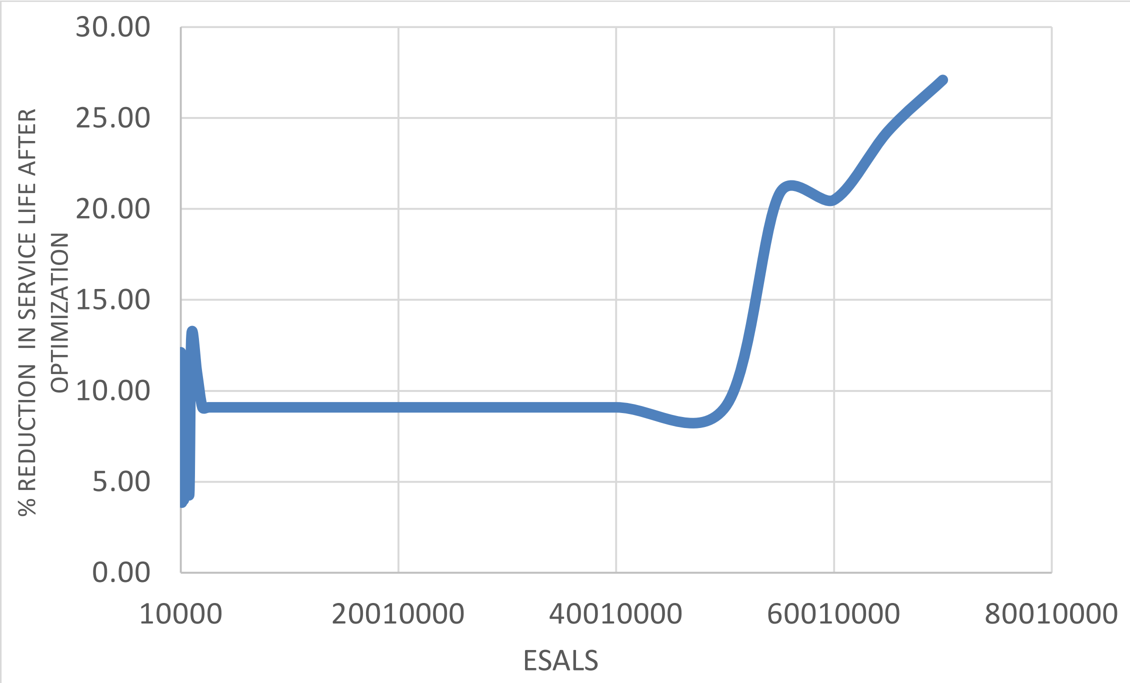


Figure 5-2 Trend line Showing Percentage Increase in Service Life with ESALs

5.4.1.1 Conclusions on Service Life Change:

For service life the above graph is against the percentage change in AASHTO and optimize design service life values against each value of ESAL. From the figure above it is concluded that up to 200000 ESALs the service life change is more sensitive i.e. more fluctuations in the value of percentage reduction in service life has been seen between ESALs values but after 300000 ESALs the trend line is smooth. This means that the service life change is more sensitive for less traffic as compared to heavy traffic. The value of percentage reduction is increases with increasing the traffic this may be because at higher level of traffic volume the pavement structural design should be such that it can accommodate that traffic for which the pavement layers' thickness kept more and more. Mostly the asphaltic concrete layer thickness increased in order to provide safe and efficient service to the traffic. But in case of optimize design the thickness can't be changed as the evaluated thickness is based on cost effectiveness and by increasing the thickness the purpose of optimization can't be achieved and no difference will remain between AASHTO and optimize design and hence the main purpose of this research study will be lost.

5.4.2 Tensile and Compressive Strains Sensitivity Analysis:

To check the sensitivity, a graph shown in figure 5-3 (a) plotted against the percentage change in AASHTO and optimized design tensile and compressive strains values against each value of ESAL.

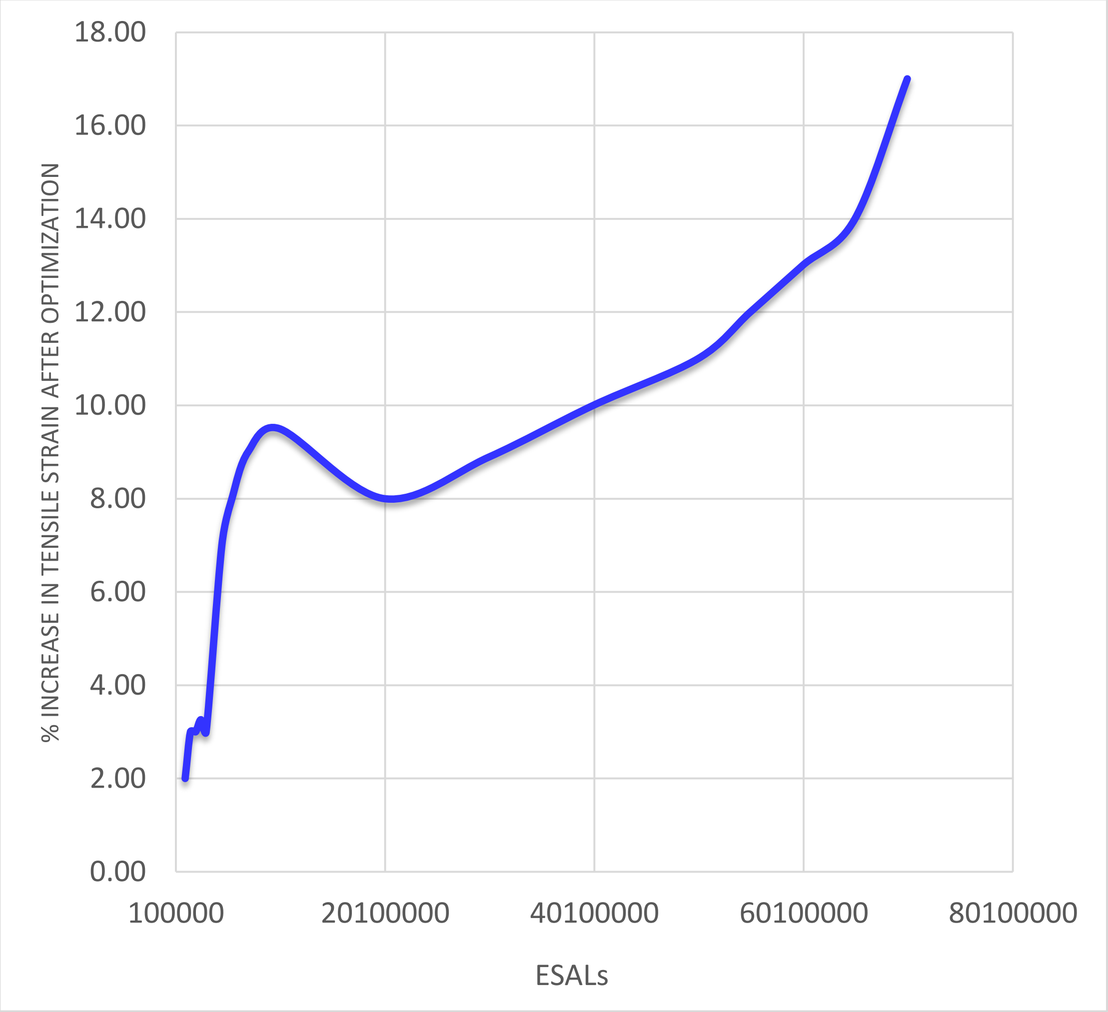


Figure 5-3 a Trend Line Showing Percentage Increase in Tensile Strains with ESALs

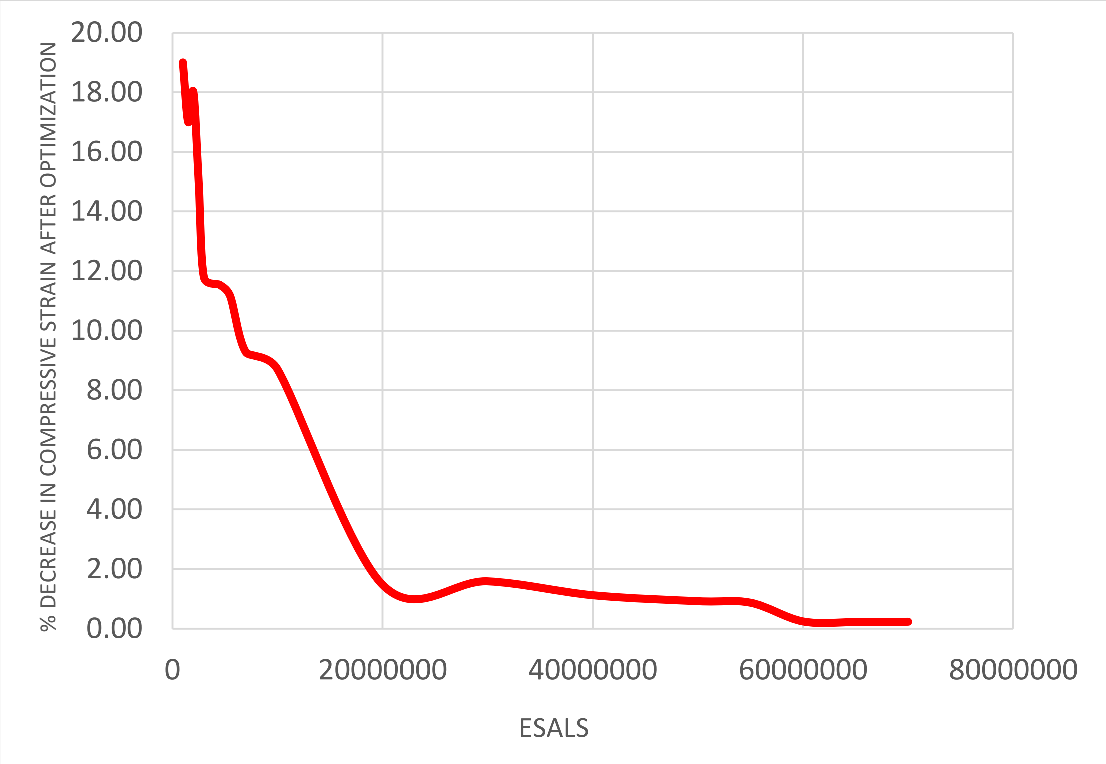


Figure 5-3 b Trend Line Showing Percentage Decrease in Compressive Strains with ESALs

5.4.2.1 Conclusions:

From the graph above it is concluded that up to 300000 ESALs more fluctuations in the percentage change values has been seen and after 300000 the trend line is smooth. The reason behind this fluctuation up to low volume of traffic is that at low volume of traffic there is a very nominal difference between pavement layer thicknesses hence the change is more fluctuated however at higher level of traffic the difference in pavement structure design also changes considerably and hence the percentage increase in tensile strains after optimization of flexible pavement design also increases with a smooth trend line. Same is the case with compressive strains at the top of subgrade as shown in the figure 5-3 (b). The compressive strains tend to reduce in optimized pavement structural design as the thickness of base and sub-base layer are more as compared to AASHTO design. Hence the percent change is decreasing.

5.4.3 Sensitivity Analysis of Damage Ratios (Fatigue and Rutting)

Damage caused by traffic loading in pavement. This damage caused when the damage ratio or sum of damage ratio exceeds the value 1. KENPAVE software has been used in this research work to calculate the damage ratio for fatigue and rutting for each ESAL value used for analysis purpose. A graph is plotted against the percentage increase or decrease in damage ratio after optimizing or reducing the asphaltic concrete pavement layer thickness. From the graphs the conclusions are made which are as under.

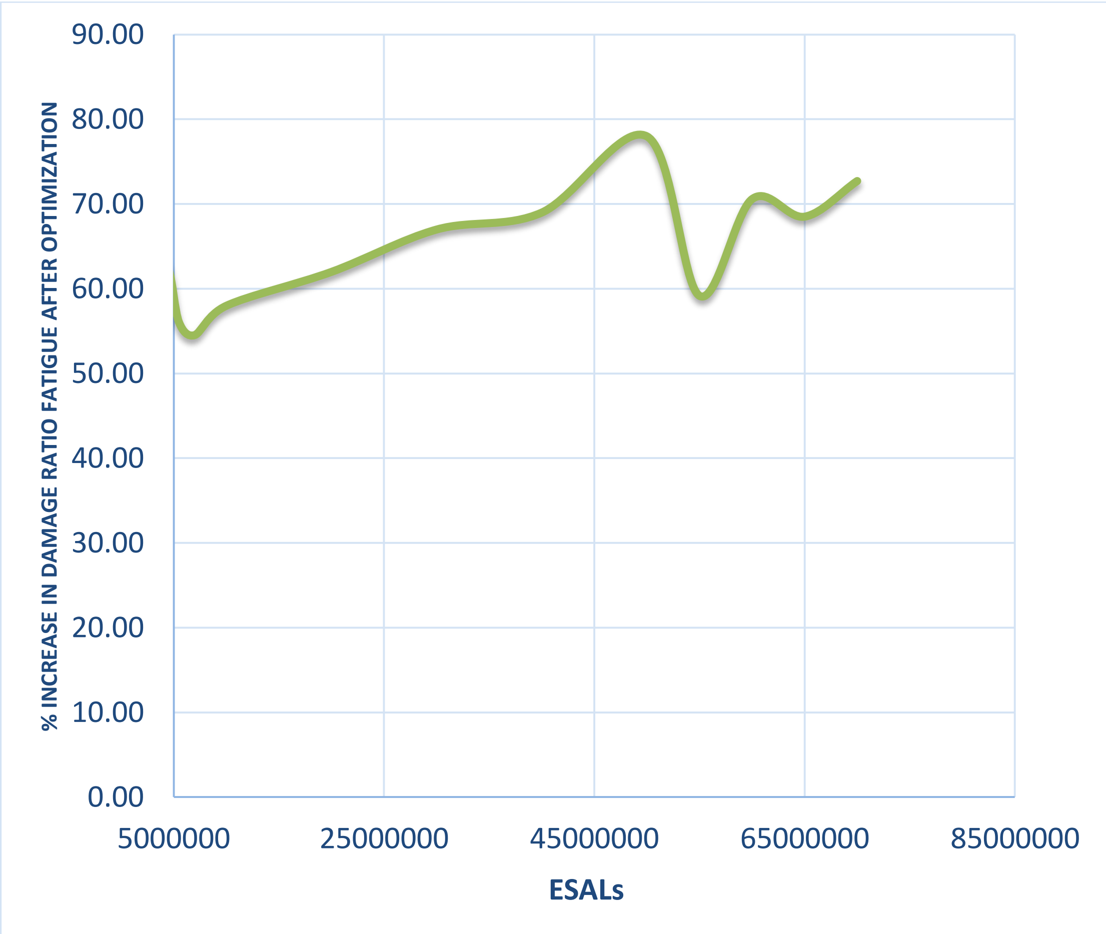


Figure 5-4 (a) Trend Line Showing Percentage Increase in Damage Ratio Fatigue with ESALs

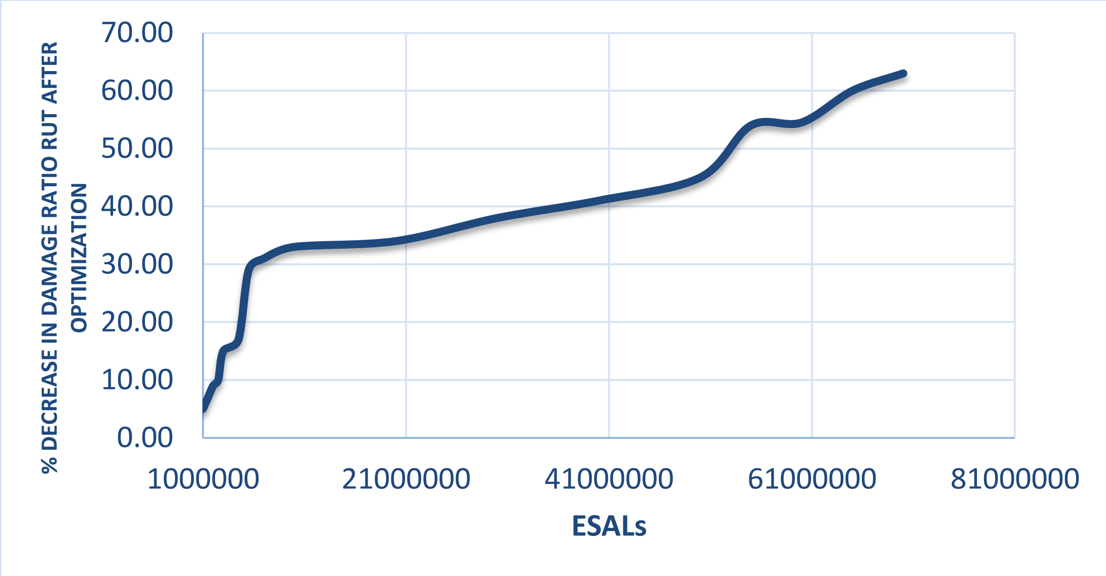


Figure 5-4 (b) Trend Line Showing Percentage Decrease in Damage Ratio Rut with ESALs

5.4.3.1 Conclusions:

From the figures above it is concluded that the increase in both damage ratios i.e. for fatigue and rutting the damage ratios are more sensitive up to 10 Million Esals and after that the trend line is smooth and can be seen in figures above. The main reason for variation in change up to 10 Million ESALs is same as discussed above for other parameters like service life and strains. The percent change in damage ratio for case of rutting is decreasing and the reason behind this behaviour is that in case of optimized pavement structural design the compressive strains at the top of sub-grade are less as compared to AASHTO design hence reduction in damage ratio rut is observed after optimization.

5.4 Developing a Decision Matrix:

For the purpose of determine the effectiveness of reduction of asphalt layer thickness i.e. optimization technique it is in need to develop a matrix, or trade off analysis has to be done. In trade off analysis strength parameters are compared against the cost savings i.e. the compromise in strength parameters are compared with the benefits in terms of cost savings. The variables used to evaluate the effectives of optimization technique are, service life, tensile strain at the bottom of asphalt layer, compressive strain at the top of subgrade layer, damage ratios fatigue and rut, asphalt layer thickness reduction and the cost savings.

Only the reduction in asphalt layer thickness and cost savings are terms as benefits of this optimization technique. Other strength parameters are compromised. In order to calculate the effectiveness few parameters are set. These parameters are as follows.

- If Service Life after reduction is less than 10 years, then original AASHTO design method is recommended.
- If damage ratio for fatigue and rut exceeded the value 1, then AASHTO design method is recommended.
- In case of cost savings optimize asphalt thickness design is recommended.
- For compressive strain at top of subgrade and tensile strains at the bottom of asphalt layer both design approaches have same trend lines but opposite response.

5.4.1 Criteria for Analysis and Comparison

Following criteria are considered for analysis and final development of decision matrix.

1. Service Life of Pavement
2. Tensile Strain at the bottom of Asphalt Layer
3. Compressive Strains at the top of Sub grade
4. Damage ratio for both fatigue and rutting
5. Construction cost of pavement

Graphs for each criterion were plotted; two types of graphs are plotted for that purpose, line and histograms.

5.4.2 Threshold Values Set for Comparison:

For each criterion a threshold value is taken from literature and from engineering judgment e.g. the threshold for service life is 10 years which is taken from design reviewing current pavement design practices in Pakistan. The threshold values are shown in table 5-6 below.

Table 5-6 Threshold Values for Criteria

Criterion	**Threshold Value**	**Source**
Service Life	10 Years	Pavement Design Reports (Pakistan)
Damage Ratio Fatigue	1	Daba S.Gedafa,2006
Damage Ratio Rut	1	Daba S.Gedafa,2006
Tensile Strain at Bottom of Asphalt Layer	$€_{AC} = 10^{-A}$ A = $(log_{10} N_i + 2.665 log_{10}\left(\frac{E}{14.22}\right) + 0.392)/5$	Heukelom & Klomp Model 1962
Compressive Strains at Top of Sub-Grade	$€_{SUBG} = 10^{-A}$ A = $0.1408 log_{10} N_i + 2.408$	Heukelom & Klomp Model 1962

This type of analysis can be also term as multi criteria decision making process as two alternatives i.e AASHTO flexible pavement structural design and optimize flexible pavement structural design are compared against the above mentioned criterions. By comparing each criterion result at each ESALs value with the threshold value the feasibility of both designs was concluded. This feasibility is presented in the form of matrix in which against each ESAL value the above six criterions are compared and the feasible method of design is mentioned.

In case of service life of pavement, a threshold value of 10 years was set keeping in view the design practices currently being followed by all consulting firms of Pakistan. KENPAVE software analysis results are taken and compared against the threshold value. Graphs are plotted to depict the result in graphical form. Keeping in view this threshold value the decision is made regarding which design process alternative is better at each value of ESALs.

5.4.2.1 Tensile and Compressive Strains Criteria:

KENPAVE is a mechanistic - empiric design (M-E), and its performance prediction is based in the most critical of two analysis: (1) tensile strain in the bottom of the bounded layers (HMA, I presume) to predict fatigue cracking, or (2) compressive strain in top of the subgrade to predict general rutting of the structure. These two criterion are also compared just like service life criterion. The values of both strains which results in fatigue and rutting were compared by permissible values using the Heukelom and Klomp Model. It was found that both vertical and horizontal strains values are less than the permissible values for both design alternatives at each value of ESALs.

5.4.2.1.1 Heukelom and Klomp (1962)

Heukelom and Klomp (1962) suggested a relationship between number of load repetetetions to failure and strain in asphalt concrete. The model is given as

$$N_f = 10^{-X}$$

Where N_f is the number of load repartitions to failure while X is the function which is given as

X = $5 \log \mathcal{E}_t + 2.665 log_{10}\left(\frac{E}{14.22}\right) + 0.392$

Where $\mathcal{E}_t$is the horizontal tensile strain at the bottom of asphalt layer and E is the elastic modulus of asphalt concrete.

The second part of model is the function of allowable horizontal tensile strain at the bottom of asphalt concrete layer.

$€_{AC}$ = 10^{-A}

Where $€_{AC}$ is the allowable tensile strain at the bottom of asphalt layer and A is given by model as below:

A = $(log_{10} N_i + 2.665 log_{10}\left(\frac{E}{14.22}\right) + 0.392)/5$

Where $N_i$number of actual load repetitions and E is the elastic modulus of asphalt concrete. By using the above model value of allowable tensile strains were calculated against each value of load repetition i.e. ESAL. These values are than considered as a threshold values for analysis and decision making. Results ate shown in form of tables in next chapter.

For compressive strain at the top of sub grade following Heukelom and Klomp (1962) equation was used to find out the permissible or allowable compressive strains at the top of sub grade at each value of ESAL.

$€_{SUBG}$ = 10^{-A}

Where $€_{SUBG}$ is allowable vertical compressive strain at the top of sub grade and A is given by the model as follow

A = $0.1408 log_{10} N_i + 2.408$

By using the above model value of allowable vertical compressive strains were calculated against each value of load repetition i.e. ESAL. These values are than considered as a threshold values for analysis and decision making. Results are shown in next chapter.

5.4.2.2 Rutting and Fatigue Damage Ratio Criteria:

Fatigue cracking is a phenomenon which occurs in pavements due to repeated applications of traffic loads. The fatigue criterion in mechanistic design approach is based on limiting the horizontal tensile strain on the underside of the asphalt bound layer due to repetitive loads on the pavement surface, if this strain is excessive, cracking (fatigue) of the layer will result. Various researchers have shown that the relationship between load repetitions to failure Nf and strain for asphalt concrete material is dependent on the horizontal tensile strain at the bottom of the asphalt bound layer and the elastic modulus of the asphalt concrete.

Rutting criterion is based on limiting the vertical compressive subgrade strain, if the maximum vertical compressive strain at the surface of the subgrade is less than a critical value, then rutting will not occur for a specific number of traffic loadings. The magnitude of rutting has been correlated with the amount of traffic and the vertical compressive strain level at the surface of the subgrade.

In mechanistic design, failure criterion is used to define the point at which failure occurs in a pavement by determining the incremental damage. The incremental damage is simply the number of a particular axle load expected during a given design period divided by the number of repetitions to failure. The incremental damage is summed for all axle loads to obtain the expected damage ratio over the life of the pavement. The damage ratio is given by:

$$D = \frac{N_i}{N_r}$$

Where N_i is the actual number of load repetitions while N_r is the allowable number of load repetitions. If D is less than a value of one, then the pavement can be expected to exceed its design life, if D is greater than one, the pavement is expected to fail prematurely. If this value is much less than one, the pavement is probably designed too conservatively. Hence the value of 1 is taken as threshold value for both fatigue and rutting damage ratios. If any of the damage ratio for any alternative exceed for given value of ESAL then the alternative is termed as non-feasible as compared to other alternative.

Chapter 6

6. Results and Discussions

6.1 Fatigue or Service Life:

Fatigue life of pavement has remarkably reduced with the optimization in asphalt concrete thickness for the purpose of economization compare to actual AASHTO design thickness.

Table 6-1Comparison of Service Life between AASHTO and Optimize Layer Thickness Design

ESAL	Design Service Life (AASHTO) (Years)	Service Life (Optimize Design) (Years)
10000	65.00	58.00
20000	33.00	29.00
30000	22.00	20.00
40000	16.00	15.00
50000	15.00	14.00
55000	14.00	13.00
75000	14.00	12.50
100000	13.50	12.50
125000	13.00	12.50
150000	13.00	12.00
175000	12.50	12.00
200000	12.50	12.00
300000	12.50	12.00
400000	12.00	11.00
500000	12.00	11.00
550000	12.00	11.00
800000	11.50	11.00
1000000	11.50	10.00
1500000	11.24	10.00
2000000	11.00	10.00
2500000	11.00	10.00
3000000	11.00	10.00
4500000	11.00	10.00
5500000	11.00	10.00
7000000	11.00	10.00
10000000	11.00	10.00
20000000	11.00	10.00
30000000	11.00	10.00
40000000	11.00	10.00
50000000	11.00	10.00
55000000	10.22	8.09
60000000	10.00	7.95
65000000	10.00	7.57
70000000	10.00	7.29

As from the table it is concluded that service life depends on the thickness of asphalt layer, service life of AASHTO design is more than the optimize design. From reading different pavement design reports from different consultant firms in Pakistan the design life of pavement was set as 10 years, and this is taken as the minimum service life pavement should have. So from the results it is concluded that AASHTO design thickness provides service life of 10 or more years as compared to optimize design. Graphical representation is shown in figure 6-1below

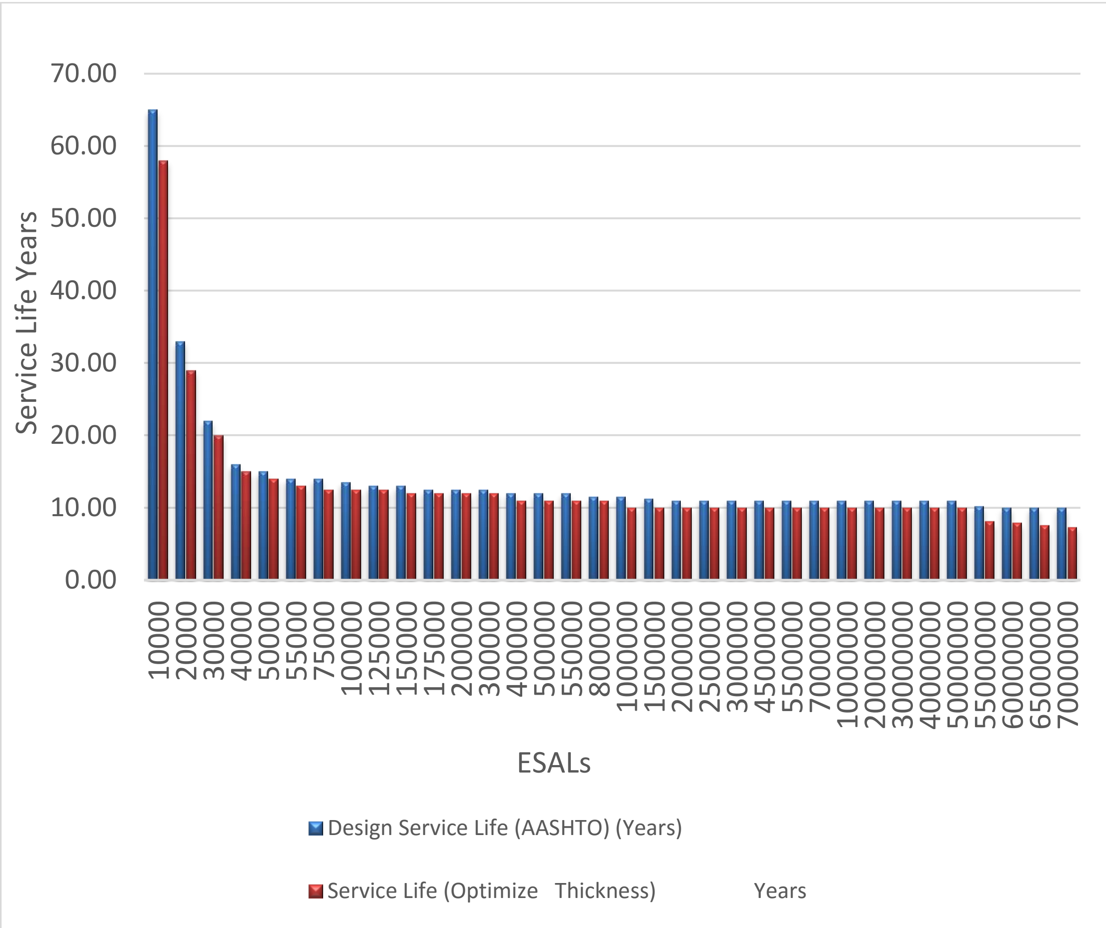


Figure 6-1 Comparison of Life with respect to ESAL

From the results above it is predicted that service life has decreased in case of optimize flexible pavement thickness configurations. These results depict that asphalt layer reduction has a remarkable effect on the service life of pavement. The trend is almost similar.

6.1.1 Matrix Development for Service Life Criterion

In case of service life of pavement, a threshold value of 10 years was set keeping in view the design practices currently being followed by all consulting firms of Pakistan. KENPAVE software analysis results are taken and compared against the threshold value. Graphs are plotted to depict the result in graphical form. Keeping in view this threshold value the decision is made regarding which design process alternative is better at each value of ESALs. Service Life Comparison with threshold is given in table 6-2.

Table 6-2 Service Life Comparison with threshold

ESALS	Design Service Life (AASHTO) (Years)	Service Life (Optimize Thickness) Years	Total Reduction in Service Life (Years)	Percentage Reduction (%)	Threshold Value (Years)	Status AASHTO	Status Optimize
10000	65.00	58.00	7.00	10.77	10	OK	OK
20000	33.00	29.00	4.00	12.12	10	OK	OK
30000	22.00	20.00	2.00	9.09	10	OK	OK
40000	16.00	15.00	1.00	6.25	10	OK	OK
50000	15.00	14.00	1.00	6.67	10	OK	OK
55000	14.00	13.00	1.00	7.14	10	OK	OK
75000	14.00	12.50	1.50	10.71	10	OK	OK
100000	13.50	12.50	1.00	7.41	10	OK	OK
125000	13.00	12.50	0.50	3.85	10	OK	OK
150000	13.00	12.00	1.00	7.69	10	OK	OK
175000	12.50	12.00	0.50	4.00	10	OK	OK
200000	12.50	12.00	0.50	4.00	10	OK	OK
300000	12.50	12.00	0.50	4.00	10	OK	OK
400000	12.00	11.00	1.00	8.33	10	OK	OK
500000	12.00	11.00	1.00	8.33	10	OK	OK
550000	12.00	11.00	1.00	8.33	10	OK	OK
800000	11.50	11.00	0.50	4.35	10	OK	OK
1000000	11.50	10.00	1.50	13.04	10	OK	OK

1500000	11.24	10.00	1.24	11.03	10	OK	OK
2000000	11.00	10.00	1.00	9.09	10	OK	OK
2500000	11.00	10.00	1.00	9.09	10	OK	OK
3000000	11.00	10.00	1.00	9.09	10	OK	OK
4500000	11.00	10.00	1.00	9.09	10	OK	OK
5500000	11.00	10.00	1.00	9.09	10	OK	OK
7000000	11.00	10.00	1.00	9.09	10	OK	OK
10000000	11.00	10.00	1.00	9.09	10	OK	OK
20000000	11.00	10.00	1.00	9.09	10	OK	OK
30000000	11.00	10.00	1.00	9.09	10	OK	OK
40000000	11.00	10.00	1.00	9.09	10	OK	OK
50000000	11.00	10.00	1.00	9.09	10	OK	OK
55000000	10.22	8.09	2.13	20.84	10	OK	Not OK
60000000	10.00	7.95	2.05	20.50	10	OK	Not OK
65000000	10.00	7.57	2.43	24.30	10	OK	Not OK
70000000	10.00	7.29	2.71	27.10	10	OK	Not OK

Table above showing the results of both alternatives. As the results are showing that after 50 Million ESALs the optimize flexible pavement structural design is not remains feasible as the service life is less than 10 years' threshold value, hence this concludes that for ESALS greater or equal than 50 Million AASHTO flexible pavement structural design alternative is OK and recommended. The main reason behind such a difference between the results that the KENPAVE software used different parameters to evaluate the service life and by the service life it means that this pavement will survive upto these years under the load and material properties. The AASHTO criterion is based on the degradation of serviceability (PSI) with traffic, which is an empirical performance model developed from data obtained in the AASHO Road Test (1958-1961) and modified with research up until 1998. It is heavy dependent on pavement roughness. KENPAVE is a mechanistic - empiric design (M-E), and its performance prediction is based in the most critical of two analysis: (1) tensile strain in the bottom of the bounded layers (HMA, I presume) to predict fatigue cracking, or (2) compressive strain in top of the subgrade to predict general rutting of the structure. In the M-E procedure the outcome depends on the transfer functions (a.k.a. fatigue laws) that you have used. There are quite of them in the literature i.e. Asphalt Institute, MEPDG national calibration, Shell bitumen, CSIR, French Pavement Design Guide, etc. The real benefit in M-E design is to use the fatigue and rutting relationships that represents your materials, construction practices and deterioration rates

6.2 Construction Cost:

Construction cost of actual flexible pavement structural design is more than the optimize thickness. The main governing factor for cost is the asphalt layer thickness as its cost is almost ten times more than the cost of base and sub-base material. The cost is computed for one-meter square per cm of flexible pavement. The table 6-3 is showing the comparison between the costs for AASHTO and Optimize Structural Design. All the costs are based on NHA CSR 2014.

Table 6-3 Comparison of Construction Cost between AASHTO and Optimize Layer Thickness Design

ESAL	Total Cost per meter square per cm (Design Thickness) Rs	Total Cost per meter square per cm (Optimize Thickness) Rs	Net Savings (Rs/Meter Square/Cm)
10000	1913.232	1344.480	568.751
20000	1913.232	1344.480	568.751
30000	1931.101	1797.483	133.618
40000	1931.101	1797.483	133.618
50000	1931.101	1797.483	133.618
55000	2213.860	1991.154	222.706
75000	2213.860	1991.154	222.706
100000	2534.524	2396.255	138.269
125000	2534.524	2483.376	51.149
150000	2534.524	2483.376	51.149
175000	3070.695	2870.531	200.164
200000	3140.598	3054.487	86.112
300000	3537.942	3248.157	289.785

400000	3537.942	3441.827	96.115
500000	3537.942	3441.827	96.115
550000	3537.942	3441.827	96.115
800000	4029.865	3993.693	36.171
1000000	4521.787	4355.244	166.543
1500000	4521.787	4008.047	513.741
2000000	4989.034	4752.115	236.919
2500000	4989.034	4752.115	236.919
3000000	5345.387	5221.718	123.669
4500000	5589.023	5507.366	81.657
5500000	6056.270	5691.321	364.949
7000000	6114.371	6068.947	45.424
10000000	6500.526	6264.119	236.407
20000000	7226.072	6835.415	390.657
30000000	7799.473	7396.996	402.478
40000000	8038.051	7774.621	263.430
50000000	8860.566	8478.771	381.795
55000000	8896.44	8629.71	266.73
60000000	8993.27	8726.55	266.73
65000000	9177.23	8823.38	353.85
70000000	9269.20	8823.38	445.82

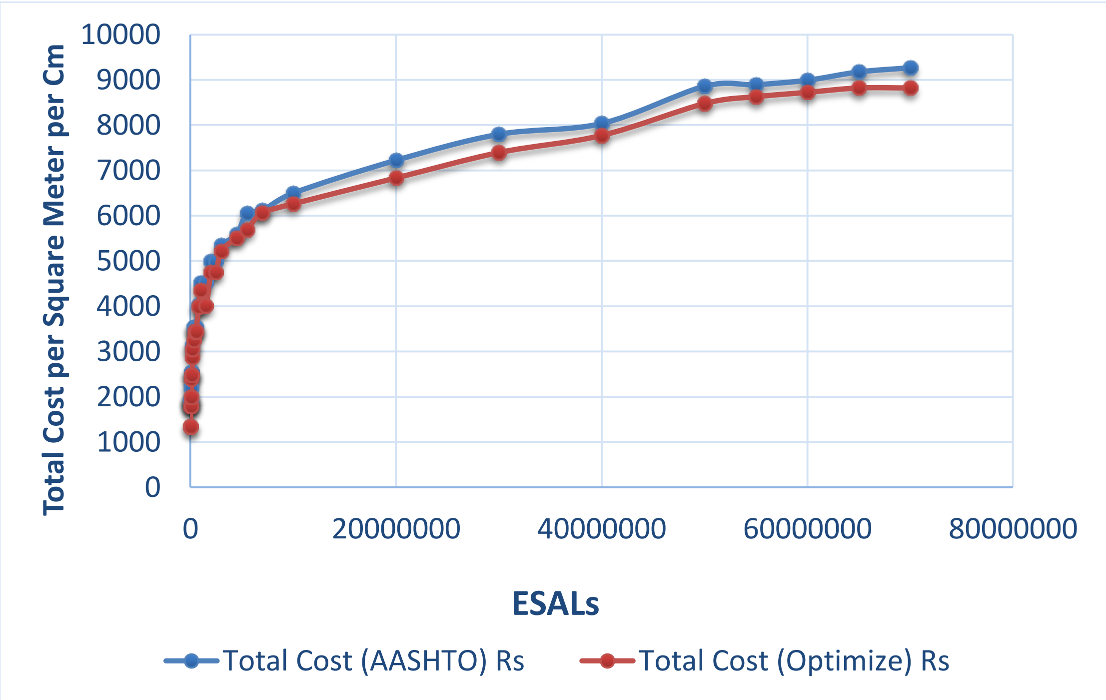


Figure 6-2(a) Comparison of Cost with respect to ESALs.

The cost incurred in case of AASHTO structural design construction is more as compared to optimize pavement design hence for cost the optimize design is recommended. Cost effectiveness of optimized structural design is elaborated thorugh figure 6-2 (b) below.

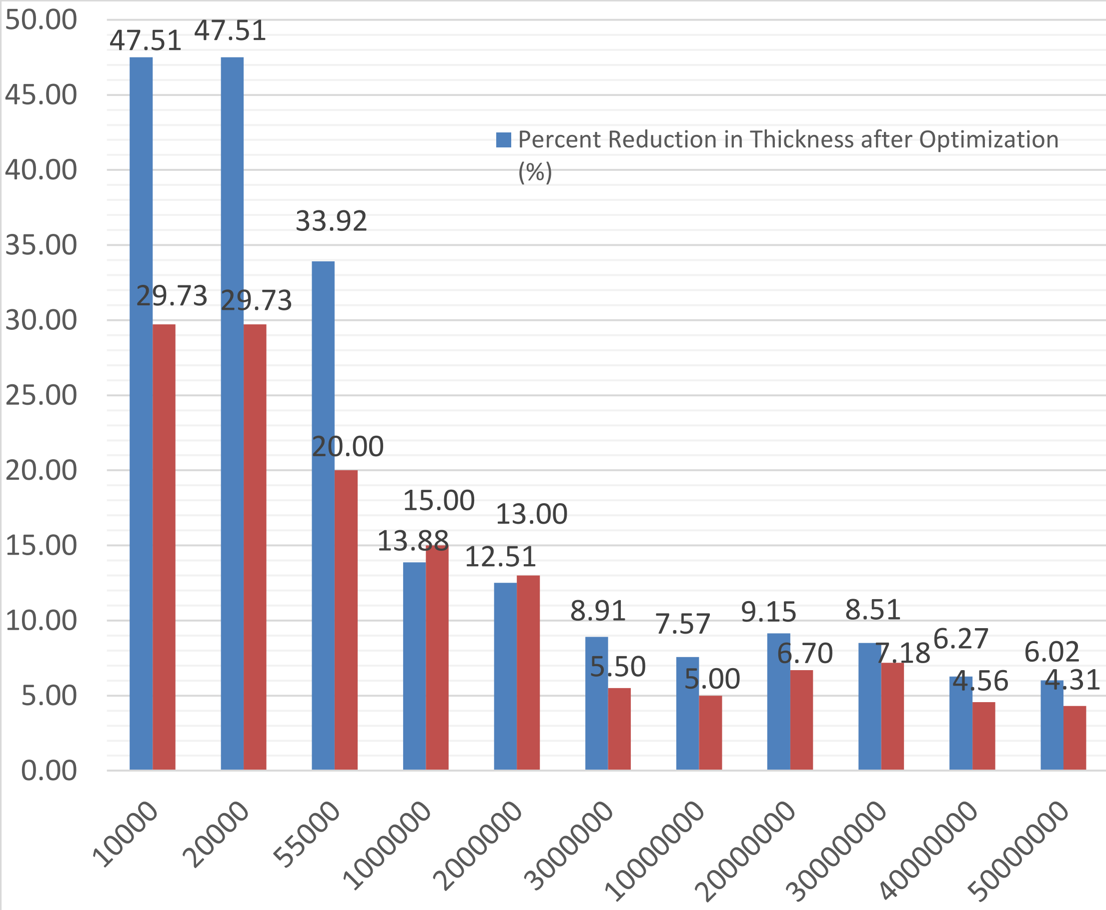


Figure 6-2 b Percent Savings in Cost after Optimization

Cost effectiveness due to optimized design is considerably higher for ESAL up to 1 Million as compared to ESAL beyond 1 Million. Cost effectiveness is calculated for each ESAL value from 10000 ESAL to 70 Million ESAL. The figure above shows that for lower traffic volume the method of optimization is more effective as cost savings are more. The reason behind this response is that for lower traffic volume the cushion for thickness reduction is more as pavement is well sustaining the strength requirements and the traffic load is also less hence detoriation rate is also minimal.

But for ESAL after 1 Million the traffic volume is considered as higher volume and rate of pavement detoriation also increases, hence the cushion for asphaltic concrete layer thickness is less in order to meet the strength requirements. Hence cost savings are also less.

6.3 Tensile Strain at the Bottom of Asphalt Layer:

It seems from results obtained that tensile strain at the bottom of asphalt base layer is reducing with increasing AC thickness. By reducing or optimizing the asphalt layer thicknesses the tensile strains at the bottom of asphalt layer is also increasing this depicts that the pavement will be less susceptible to surface cracking in case of AASHTO Design. It is intuitive from the literature that with increase in asphalt concrete layer thickness the tensile strain at the bottom of asphalt layer tends to reduce as shown in figure 6.3. Table 6-4 is showing the comparison

Table 6-4: Comparison of Tensile Strain at the Bottom of Asphalt Base layer between AASHTO and Optimize Layer Thickness Design

ESAL	**Tensile Strain at bottom of Asphalt Base Layer (10^{-4}), AASHTO Design**	**Tensile Strain at bottom of Asphalt Base Layer (10^{-4}), Optimize Layer Thickness**
10000	2.88	2.99
20000	2.88	2.99
30000	2.88	2.96
40000	2.88	2.96
50000	2.88	2.90
55000	2.58	2.83
75000	2.58	2.82
100000	2.35	2.40
125000	2.35	2.39
150000	2.20	2.27
175000	2.00	2.07
200000	1.87	2.02
300000	1.55	1.77
400000	1.55	1.64

500000	1.55	1.62
550000	1.55	1.60
800000	1.28	1.32
1000000	1.08	1.24
1500000	1.08	1.22
2000000	0.92	1.04
2500000	0.92	0.95
3000000	0.82	0.88
4500000	0.80	0.81
5500000	0.71	0.77
7000000	0.66	0.68
10000000	0.61	0.62
20000000	0.53	0.57
30000000	0.45	0.49
40000000	0.40	0.45
50000000	0.37	0.41
55000000	0.37	0.40
60000000	0.35	0.39
65000000	0.35	0.38
70000000	0.34	0.35

The results obtained for both scenarios depicts that as the ESALs value increasing the pavement design is also changing i.e. the layer thicknesses are increasing in order to counter the load from traffic. As layer thickness of asphalt concrete is increasing the tensile strain also decreasing. But if the comparison between the AASHTO and Optimize design has been made than it is obvious that in case of optimize design the asphalt concrete layer thickness is reduced which results in increase in tensile strain at the bottom of asphalt base layer. The result is also shown in figure 6-3 a and 6-3 b.

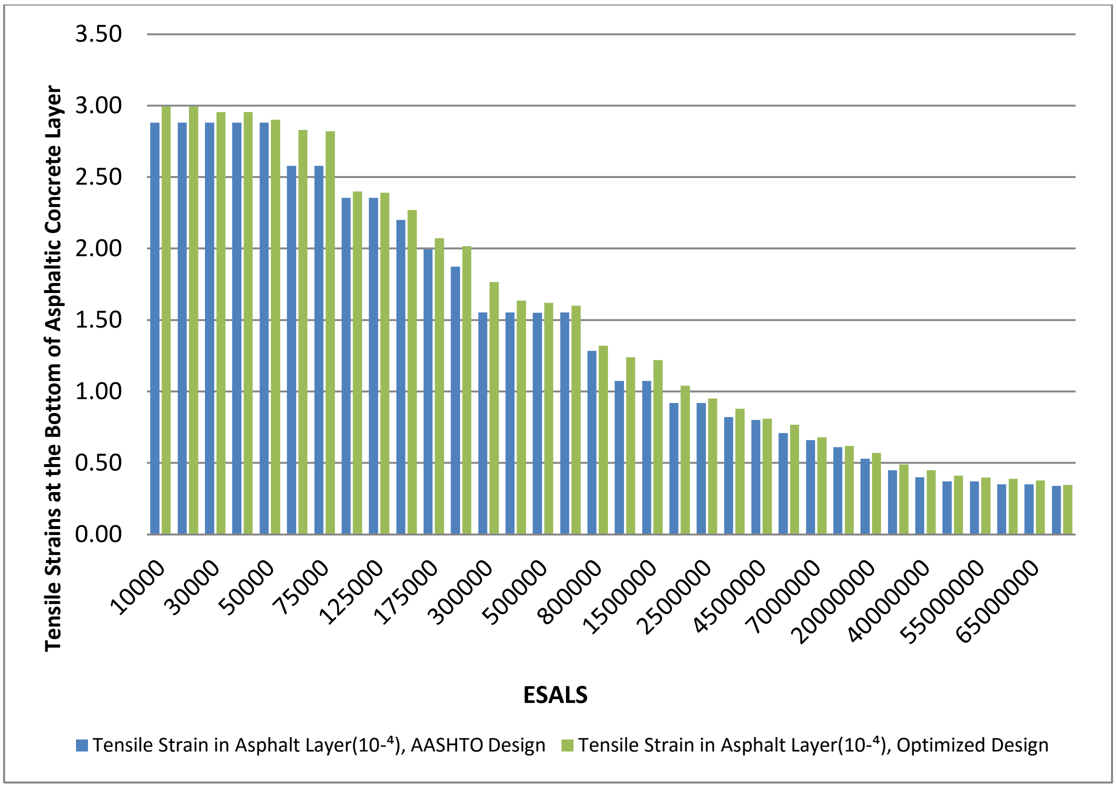


Figure 6-3 (a): Comparison of Tensile Strain with respect to ESALs.

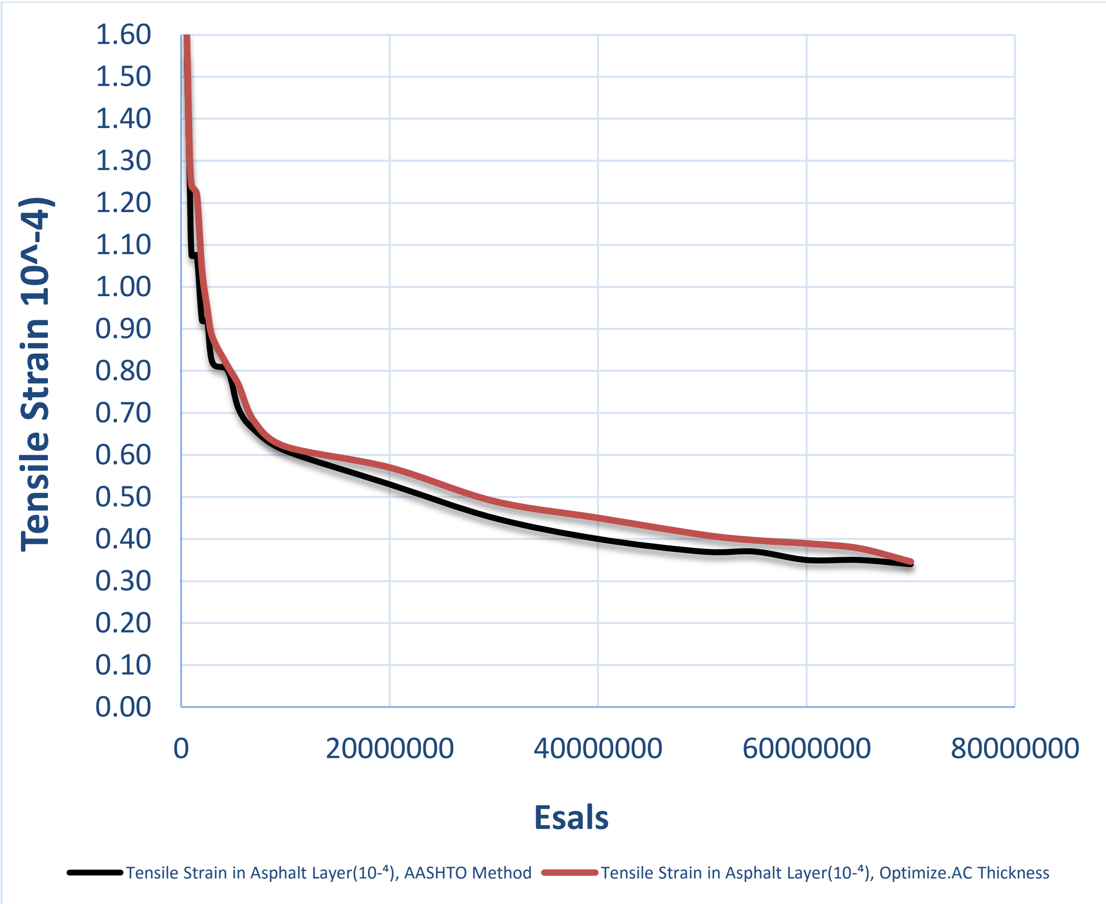


Figure 6-3 (b): Trend Line Comparison of Tensile Strain with respect to ESALs.

The comparison of tensile strains with the threshold values is given in table 6-5. As seen in the table all the tensile strains in both AASHTO and optimize scenarios are within the limits and are much less than the permissible values. Hence concluded that both design approaches have same response. However, in case of optimize design the tensile strains tends to increase.

Table 6-5 Comparison with respect to the threshold values

ESALS	Tensile Strain in Asphalt Layer(10^{-4}), AASHTO Method	Tensile Strain in Asphalt Layer(10^{-4}), Optimize. AC Thickness	Total Increase in Tensile Strain after optimization (10^-4)	Threshold Value (Heukelom and Klomp) 10^-4)	Status AASHTO	Status Optimize
10000	2.88	2.99	0.11	85.74	OK	OK
20000	2.88	2.99	0.11	74.64	OK	OK
30000	2.88	2.96	0.08	68.83	OK	OK
40000	2.88	2.96	0.08	64.98	OK	OK
50000	2.88	2.90	0.02	62.14	OK	OK
55000	2.58	2.83	0.25	60.97	OK	OK
75000	2.58	2.82	0.24	57.30	OK	OK
100000	2.35	2.40	0.05	54.10	OK	OK
125000	2.35	2.39	0.04	51.74	OK	OK
150000	2.20	2.27	0.07	49.89	OK	OK
175000	2.00	2.07	0.08	48.37	OK	OK
200000	1.87	2.02	0.14	47.10	OK	OK
300000	1.55	1.77	0.21	43.43	OK	OK
400000	1.55	1.64	0.08	41.00	OK	OK
500000	1.55	1.62	0.07	39.21	OK	OK
550000	1.55	1.60	0.05	38.47	OK	OK
800000	1.28	1.32	0.04	35.69	OK	OK
1000000	1.08	1.24	0.17	34.13	OK	OK
1500000	1.08	1.22	0.15	31.48	OK	OK
2000000	0.92	1.04	0.12	29.72	OK	OK
2500000	0.92	0.95	0.03	28.42	OK	OK
3000000	0.82	0.88	0.06	27.40	OK	OK
4500000	0.80	0.81	0.01	25.27	OK	OK
5500000	0.71	0.77	0.06	24.27	OK	OK

7000000	0.66	0.68	0.02	23.13	OK	OK
10000000	0.61	0.62	0.01	21.54	OK	OK
20000000	0.53	0.57	0.04	18.75	OK	OK
30000000	0.45	0.49	0.04	17.29	OK	OK
40000000	0.40	0.45	0.05	16.32	OK	OK
50000000	0.37	0.41	0.04	15.61	OK	OK
55000000	0.37	0.40	0.03	14.34	OK	OK
60000000	0.35	0.39	0.04	13.33	OK	OK
65000000	0.35	0.38	0.03	12.90	OK	OK
70000000	0.34	0.35	0.01	12.2	OK	OK

6.4 Compressive Strain at the Top of Subgrade:

The behaviour of pavement with respect to compressive strain is same as tensile strains, as mentioned in the table the compressive strain at the top of subgrade is more in case of optimize configuration of flexible pavement structural design as compared to AASHTO design pavement thicknesses. The reason for this behaviour is that as the asphalt concrete layer thickness tends to reduce after optimization the vertical compressive strains at the subgrade increase as stiffness of pavement layers tends to reduce. Table 6-6 shows the comparison in terms of values of compressive strain at the top of subgrade.

Table 6-6: Comparison of Compressive Strain at the Top of Subgrade between AASHTO and Optimize Layer Thickness Design

ESAL	**Compressive Strain (10^{-4})at top of Subgrade, AASHTO Method**	**Compressive Strain (10^{-4}) at top of Subgrade, Optimize.AC Thickness**
10000	4.71	5.27
20000	4.71	5.27
30000	4.53	3.68
40000	4.44	3.65
50000	4.40	3.60
55000	3.92	3.39
75000	3.92	3.39
100000	3.00	2.89
125000	3.01	2.36

150000	2.21	2.10
175000	2.54	2.08
200000	2.18	2.18
300000	1.85	1.68
400000	1.85	1.58
500000	1.85	1.50
550000	1.12	1.41
800000	1.12	1.47
1000000	1.09	1.30
1500000	1.09	1.28
2000000	1.06	1.08
2500000	1.23	1.09
3000000	1.12	0.92
4500000	1.04	0.92
5500000	1.01	0.90
7000000	0.91	0.83
10000000	0.77	0.703
20000000	0.68	0.67
30000000	0.63	0.62
40000000	0.49	0.58
50000000	0.46	0.55
55000000	0.46	0.46
60000000	0.45	0.45
65000000	0.45	0.45
70000000	0.44	0.44

Figure 6.4 a & b represent the relationship between compressive strains on the top of subgrade versus the ESALs. The figure shows that as the ESALs are increasing the compressive strains tends to reduce as the pavement layer thicknesses are also increasing in order to coup the increase in ESALs. If comes to the comparison between AASHTO and optimize pavement configuration the compressive strains in optimize pavement design are more than in AASHTO.

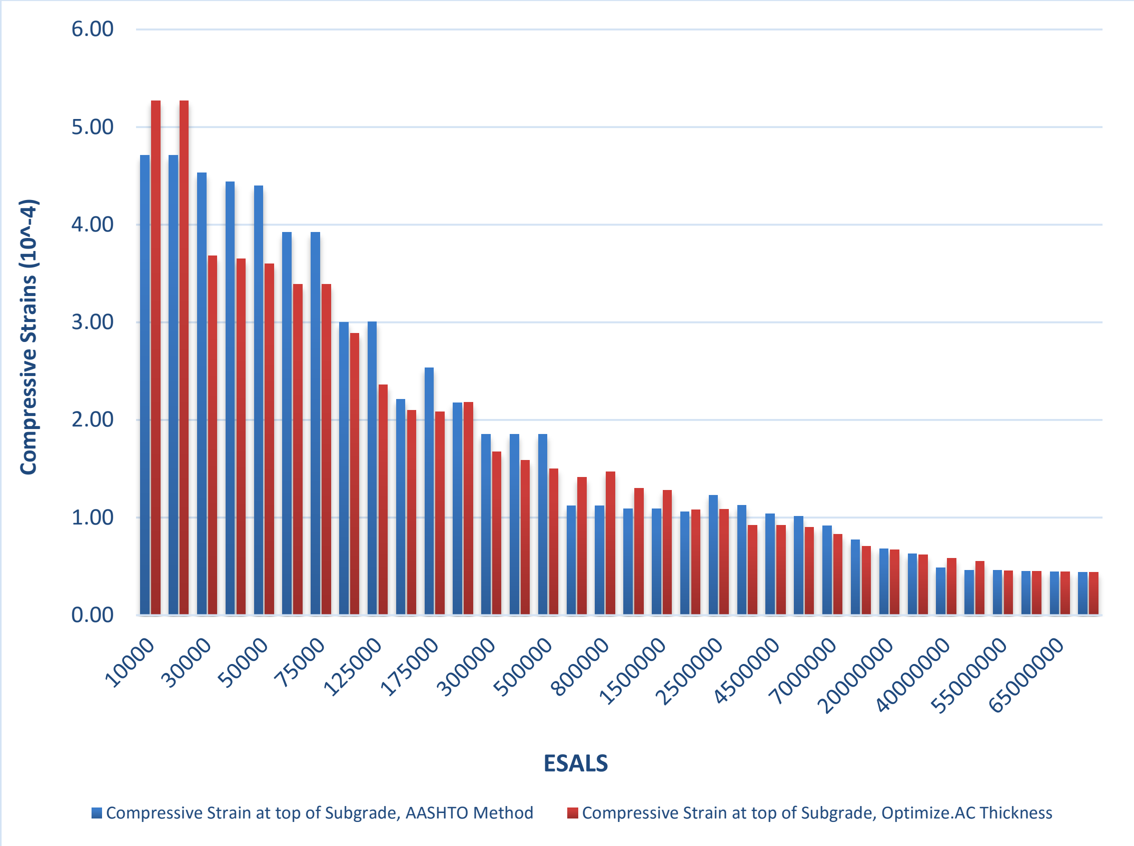


Figure 6-4 (a): Comparison of Compressive Strains with respect to ESALs.

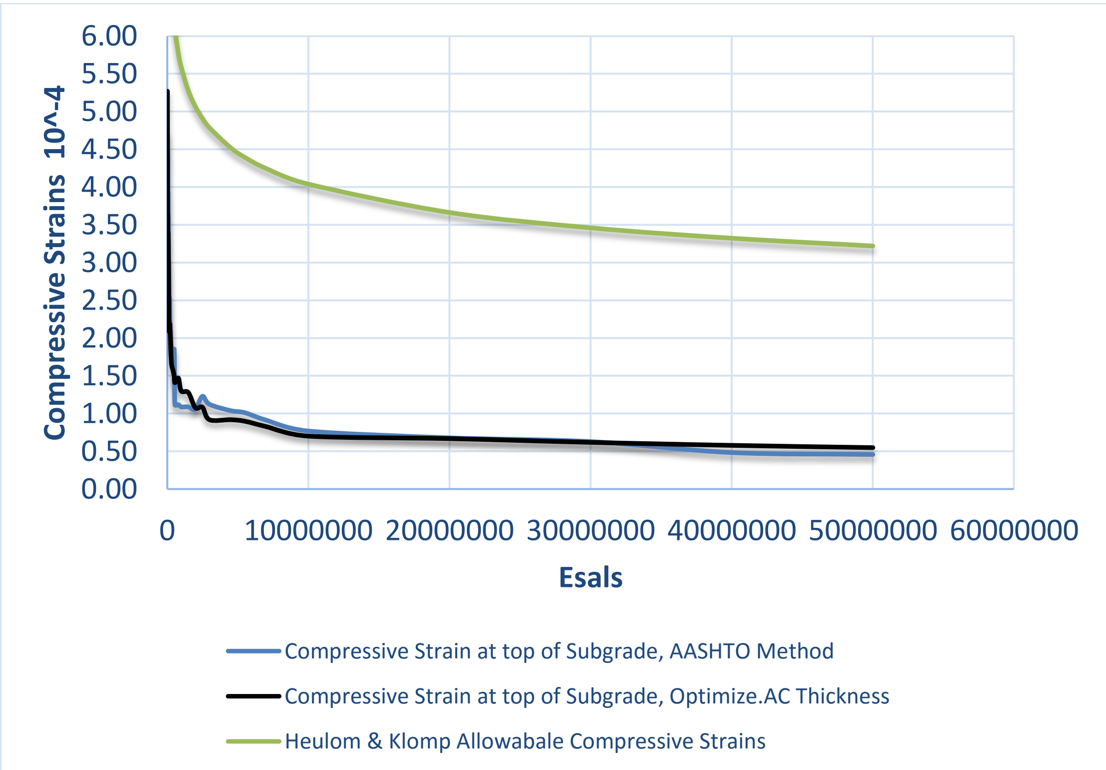


Figure 6-4 (b): Trend Line for comparison of Compressive Strains

Table 6-7 is showing the comparison with respect to the threshold values and with respect to this comparison it is also concluded that both the design approaches have same response.

Table 6-7 Comparison with respect to the threshold values

ESAL	Compressive Strain at top of Subgrade, AASHTO Method 10^-4	Compressive Strain at top of Subgrade, Optimize.AC Thickness 10^-4	Total Increase in Compressive Strain after optimization (10^-4)	Threshold Value (Heukelom and Klomp) 10^-4	Status AASHTO	Status Optimize
10000	4.71	5.27	0.56	10.69	OK	OK
20000	4.71	5.27	0.56	9.69	OK	OK
30000	4.53	3.68	-0.85	9.15	OK	OK
40000	4.44	3.65	-0.79	8.79	OK	OK
50000	4.40	3.60	-0.80	8.52	OK	OK
55000	3.92	3.39	-0.53	8.41	OK	OK
75000	3.92	3.39	-0.53	8.05	OK	OK
100000	3.00	2.89	-0.11	7.73	OK	OK
125000	3.01	2.36	-0.65	7.49	OK	OK
150000	2.21	2.10	-0.11	7.30	OK	OK
175000	2.54	2.08	-0.46	7.14	OK	OK
200000	2.18	2.18	0.00	7.01	OK	OK
300000	1.85	1.68	-0.18	6.62	OK	OK
400000	1.85	1.58	-0.27	6.36	OK	OK
500000	1.85	1.50	-0.35	6.16	OK	OK
550000	1.47	1.12	-0.35	6.08	OK	OK
800000	1.47	1.12	-0.35	5.77	OK	OK
1000000	1.30	1.09	-0.21	5.59	OK	OK
1500000	1.28	1.09	-0.19	5.28	OK	OK
2000000	1.09	1.06	-0.03	5.07	OK	OK
2500000	1.09	0.92	-0.17	4.91	OK	OK
3000000	1.12	0.92	-0.20	4.79	OK	OK
4500000	1.04	0.92	-0.12	4.52	OK	OK
5500000	1.01	0.90	-0.11	4.39	OK	OK
7000000	0.91	0.83	-0.08	4.25	OK	OK
10000000	0.77	0.703	-0.07	4.04	OK	OK
20000000	0.68	0.67	-0.01	3.66	OK	OK
30000000	0.63	0.62	-0.01	3.46	OK	OK
40000000	0.49	0.48	-0.01	3.32	OK	OK
50000000	0.46	0.45	-0.01	3.22	OK	OK

55000000	0.46	0.46	0.00	3.11	OK	OK
60000000	0.45	0.45	0.00	2.98	OK	OK
65000000	0.45	0.45	0.00	2.83	OK	OK
70000000	0.44	0.44	0.00	2.74	OK	OK

6.6 Conclusion remarks for both tensile and compressive strains:

The threshold values for tensile and compressive strains in pavement are given by Heukelom and Klomp (1962). By using the model value of allowable horizontal tensile strain at the bottom of asphalt layer and vertical compressive strains at top of sub grade were calculated against each value of load repetition i.e. ESAL. These values are than considered as a threshold values for analysis. By comparing the values obtained from analysis using Kenpave software with the threshold values given by Heukelom and Klomp (1962) model a decision will be made regarding the feasibility of AASHTO and optimize pavement design. The results are shown in form of tables. From the results it is concluded that both the design approaches are feasible. And with these two criterion the optimize design is recommended for all value of ESALs. The reason is that the optimize design is already cost effective hence as both design approaches are feasible so if the cost criterion is considered alongside with this criterion than it is concluded that optimize design approach is better than the AASHTO design approach keeping tensile and compressive strain criterion in question. The results are shown in table 6-5 and 6-7.

6.5 Damage Ratios Comparison:

As discussed earlier in literature review chapter that damage ratios are of two types, damage ratio rut and damage ratio fatigue. Damage ratio is the ratio between predicted and allowable number of load repetitions. Damage to pavement occurs when the sum of damage ratio reaches 1. The amount of damage caused is expressed in terms of damage ratio. In ESAL approach, all the axle loads have been converted into equivalent standard axle load for the design period. Asphalt Institute method is adopted while carrying put damage analysis for this study. Table 6-5 represent the value damage ratios for both fatigue and rutting. If any of the damage ratio for fatigue or rutting is less than a value of one, then the pavement can be expected to exceed its design life, if D is greater than one, the pavement is expected to fail prematurely. The governing failure is that which damage ratio exceeds 1.

Table 6-8: Comparison Damage Ratios between AASHTO and Optimize Layer Thickness Design

ESAL	Damage Ratio Fatigue (AASHTO)	Damage Ratio Fatigue (Optimize Design)	Damage Ratio Rut (AASHTO)	Damage Ratio Rut (Optimize Design)
10000	0.0150	0.0170	0.0093	0.0155
20000	0.0300	0.0340	0.0186	0.0311
30000	0.0440	0.0490	0.0236	0.0094
40000	0.0510	0.0695	0.0315	0.0125
50000	0.0520	0.0800	0.0460	0.0125
55000	0.0560	0.0957	0.0220	0.0118
75000	0.0620	0.1000	0.0309	0.0167
100000	0.0640	0.1042	0.0143	0.0203
125000	0.0690	0.1300	0.0156	0.0058
150000	0.0713	0.1400	0.0047	0.0035
175000	0.0742	0.1450	0.0016	0.0039
200000	0.0737	0.1460	0.0059	0.0060
300000	0.0790	0.1480	0.0043	0.0050
400000	0.0795	0.1490	0.0057	0.0027
500000	0.0800	0.1490	0.0018	0.0027
550000	0.0830	0.1490	0.0078	0.0023
800000	0.0852	0.1490	0.0058	0.0047
1000000	0.0870	0.1490	0.0059	0.0031
1500000	0.0890	0.1530	0.0060	0.0043
2000000	0.0898	0.1550	0.0045	0.0046
2500000	0.0880	0.1560	0.0057	0.0036
3000000	0.0970	0.1600	0.0046	0.0042
4500000	0.1010	0.1640	0.0048	0.0029

5500000	0.1070	0.1670	0.0052	0.0030
7000000	0.1100	0.1700	0.0050	0.0029
10000000	0.2300	0.2500	0.0028	0.0019
20000000	0.3200	0.3800	0.0034	0.0031
30000000	0.3900	0.4600	0.0034	0.0033
40000000	0.4000	0.5700	0.0014	0.0037
50000000	0.4300	0.6700	0.0014	0.0033
55000000	0.4900	0.7800	0.0016	0.0015
60000000	0.5100	0.8700	0.0016	0.0015
65000000	0.5400	0.9100	0.0016	0.0020
70000000	0.5500	0.9500	0.0015	0.0017

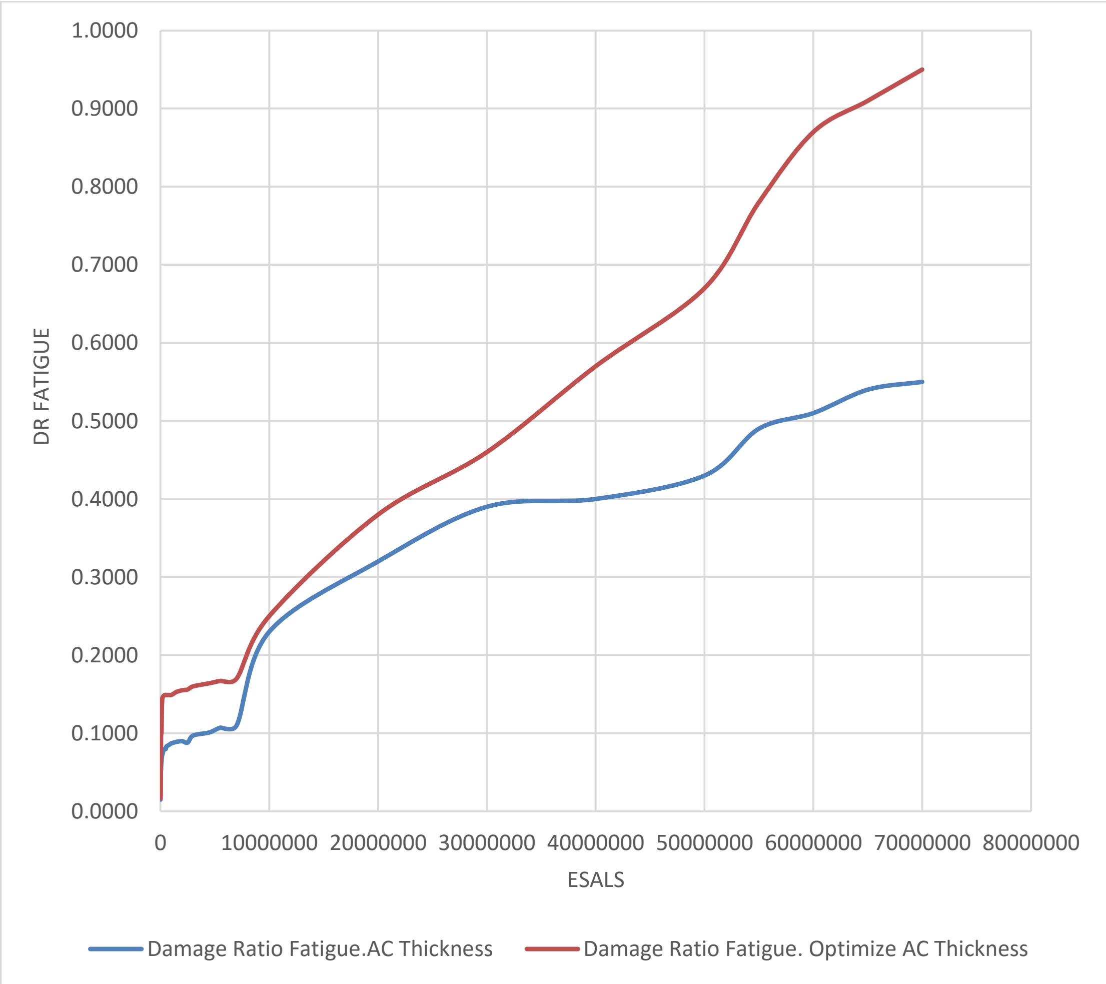


Figure 6-5 (a) Trend lines showing the comparison of damage ratio Fatigue rut and rutting

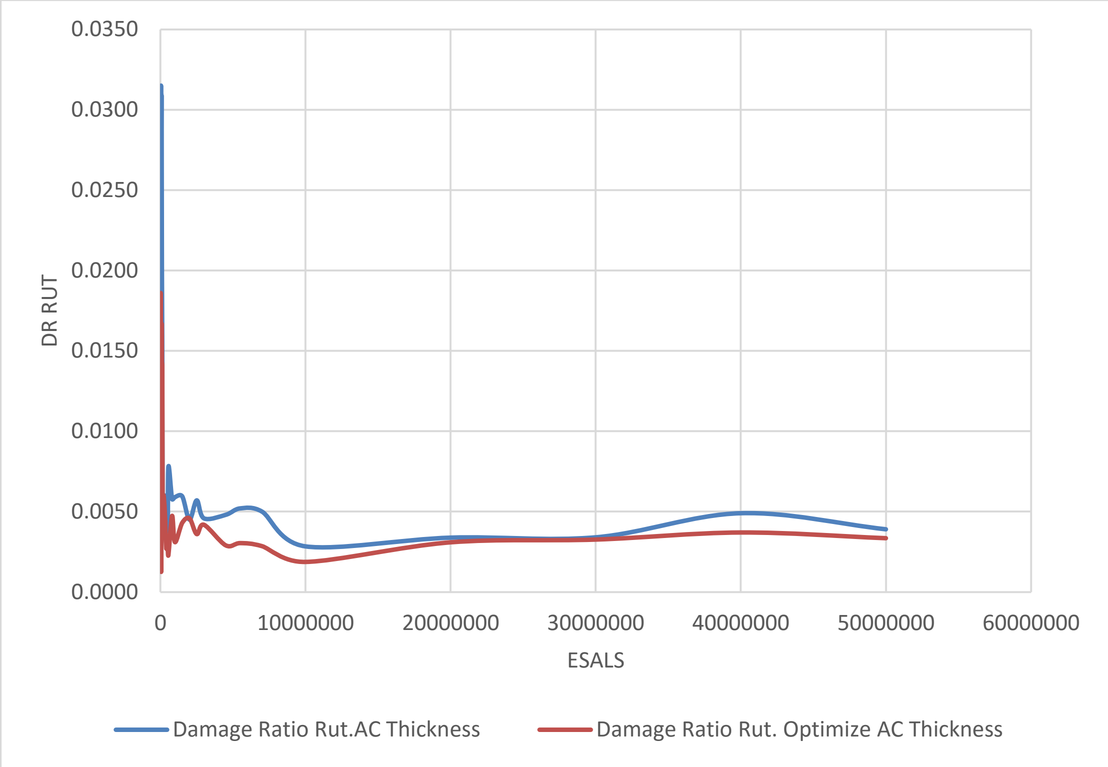


Figure 6-5 (b) Trend lines showing the comparison of damage ratio Fatigue rut and rutting

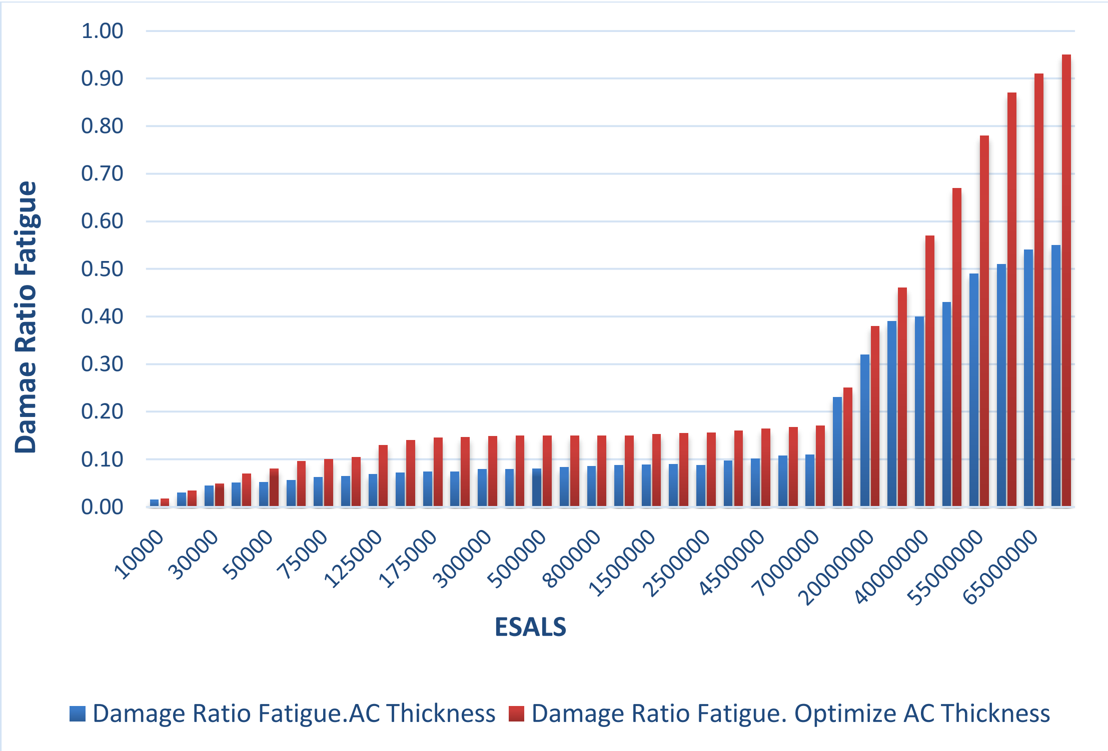


Figure 6-6 Bar Chart showing the comparison of damage ratio fatigue

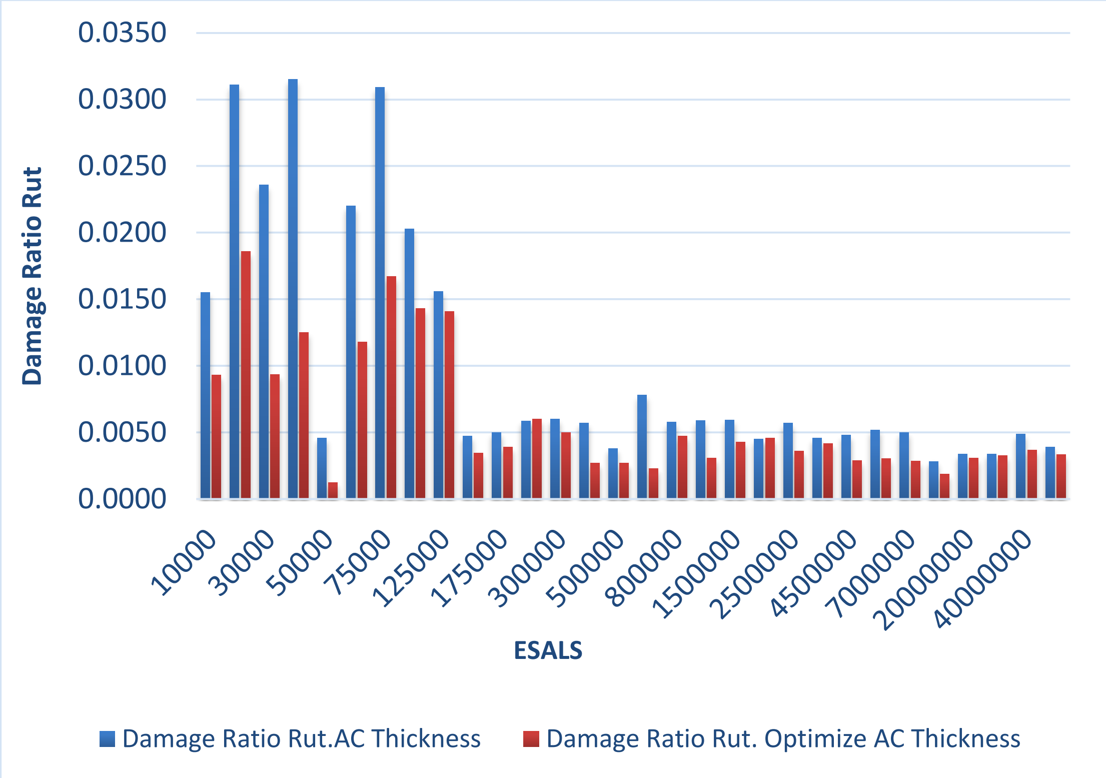


Figure 6-7 Bar Chart showing the comparison of damage ratio Rut

6.6.1 Conclusions:

The fatigue damage ratios are more as compared to the rutting damage ratios hence the fatigue is the governing failure for the pavement design. In case of fatigue the optimize design values for higher traffic volume is exceeding the permissible damage ratio value which is 1. However, the rutting damage ratios are less than permissible for both design approaches. Figure 6-5 to 6-7 are elaborating the results. As mentioned above if the damage ratio value exceeded one the pavement design failed. Results in table above shows that for the optimize pavement thicknesses the damage ratio sum is more than that of AASHTO design. It means that the optimize pavement thickness design is more susceptible to the damage as compared to AASHTO design but as far as the sum of damage ratio is less than one the design is fine. It is confirmed from the results that the optimize pavement design for ESALs 30000000,40000000 and 50000000 the damage ratio for fatigue exceeded one which means that damage has occurred and pavement fails with fatigue is the governing failure distress. As damage ratio is the ratio between predicted and allowable number of load repetitions the predicted load repetitions exceeds the allowable load repetitions limit hence the damage occurs. Figures below gives a graphical representation of the results.

6.3.1.1 Matrix Development for Damage Ratios Criterions:

If any of the damage ratio for any alternative exceed for given value of ESAL then the alternative is termed as non-feasible as compared to other alternative. From the results obtained by KENPAVE analysis it is concluded that pavement will have failed in case of optimize design for ESALs greater or equal to 30 Million. The damage ratio for rut failure criterion are much less as compared to fatigue failure criterion which depicts that the governing failure distress in pavement is fatigue. Detail results in form of tables are shown below. From the results above the optimize design method is not feasible for ESALs equal or greater than 30 Million as the damage ratio value for fatigue exceeds the threshold value of 1. For these ESALs hence the AASHTO design alternative is recommended. The results and comparison with the threshold values are shown in table 6-9 and 6-10. From the results it is concluded that both damage ratios value is within the threshold set. Hence the both AASHTO and optimized pavement structural design are recommended. But as optimized design is cost effective hence it is recommended and cost effective up to 70 Million ESALs.

Table 6-9 Comparison of Damage Ratio Fatigue with threshold value

ESALS	Damage Ratio Fatigue AASHTO	Damage Ratio Fatigue. Optimized	Threshold Value	Status AASHTO	Status Optimize
10000	0.02	0.02	1	OK	OK
20000	0.03	0.03	1	OK	OK
30000	0.04	0.05	1	OK	OK
40000	0.05	0.07	1	OK	OK
50000	0.05	0.08	1	OK	OK
55000	0.06	0.10	1	OK	OK
75000	0.06	0.10	1	OK	OK
100000	0.06	0.10	1	OK	OK
125000	0.07	0.13	1	OK	OK
150000	0.07	0.14	1	OK	OK
175000	0.07	0.15	1	OK	OK
200000	0.07	0.15	1	OK	OK
300000	0.08	0.15	1	OK	OK
400000	0.08	0.15	1	OK	OK
500000	0.08	0.15	1	OK	OK
550000	0.08	0.15	1	OK	OK
800000	0.09	0.15	1	OK	OK
1000000	0.09	0.15	1	OK	OK
1500000	0.09	0.15	1	OK	OK
2000000	0.09	0.16	1	OK	OK
2500000	0.09	0.16	1	OK	OK
3000000	0.10	0.16	1	OK	OK
4500000	0.10	0.16	1	OK	OK
5500000	0.11	0.17	1	OK	OK
7000000	0.11	0.17	1	OK	OK
10000000	0.23	0.25	1	OK	OK
20000000	0.32	0.38	1	OK	OK
30000000	0.39	0.46	1	OK	OK
40000000	0.40	0.57	1	OK	OK
50000000	0.43	0.67	1	OK	OK
55000000	0.49	0.78	1	OK	OK
60000000	0.51	0.87	1	OK	OK
65000000	0.54	0.91	1	OK	OK
70000000	0.55	0.95	1	OK	OK

Table 6-10 Comparison of Damage Ratio Rut with threshold value

ESALS	Damage Ratio Rut. AAHSTO	Damage Ratio Rut. Optimized	Threshold Value	Status AASHTO	Status Optimize
10000	0.0093	0.0155	1	OK	OK
20000	0.0186	0.0311	1	OK	OK
30000	0.0236	0.0094	1	OK	OK
40000	0.0315	0.0125	1	OK	OK
50000	0.0460	0.0125	1	OK	OK
55000	0.0220	0.0118	1	OK	OK
75000	0.0309	0.0167	1	OK	OK
100000	0.0143	0.0203	1	OK	OK
125000	0.0156	0.0058	1	OK	OK
150000	0.0047	0.0035	1	OK	OK
175000	0.0016	0.0039	1	OK	OK
200000	0.0059	0.0060	1	OK	OK
300000	0.0043	0.0050	1	OK	OK
400000	0.0057	0.0027	1	OK	OK
500000	0.0018	0.0027	1	OK	OK
550000	0.0078	0.0023	1	OK	OK
800000	0.0058	0.0047	1	OK	OK
1000000	0.0059	0.0031	1	OK	OK
1500000	0.0060	0.0043	1	OK	OK
2000000	0.0045	0.0046	1	OK	OK
2500000	0.0057	0.0036	1	OK	OK
3000000	0.0046	0.0042	1	OK	OK
4500000	0.0048	0.0029	1	OK	OK
5500000	0.0052	0.0030	1	OK	OK
7000000	0.0050	0.0029	1	OK	OK
10000000	0.0028	0.0019	1	OK	OK
20000000	0.0034	0.0031	1	OK	OK
30000000	0.0034	0.0033	1	OK	OK
40000000	0.0014	0.0037	1	OK	OK
50000000	0.0014	0.0033	1	OK	OK
55000000	0.0016	0.0015	1	OK	OK
60000000	0.0016	0.0015	1	OK	OK
65000000	0.0016	0.0020	1	OK	OK
70000000	0.0015	0.0017	1	OK	OK

7 Conclusions

Following conclusions are made after carrying out this research studies. The conclusions are given with respect to each criteria.

7.1 Service Life:

It is concluded that service life depends on the thickness of asphalt layer, service life of AASHTO pavement structural design is more than the optimize pavement structure design. The difference between service life for both design procedures is less for traffic level 5 Million ESALs. However, for ESALs more than 55 Million the difference between service life of AASHTO and optimize design increases up to 8 years which is alarming hence the optimize pavement structure design is not recommended for higher levels of traffic. Figure 7-1 shows the results.

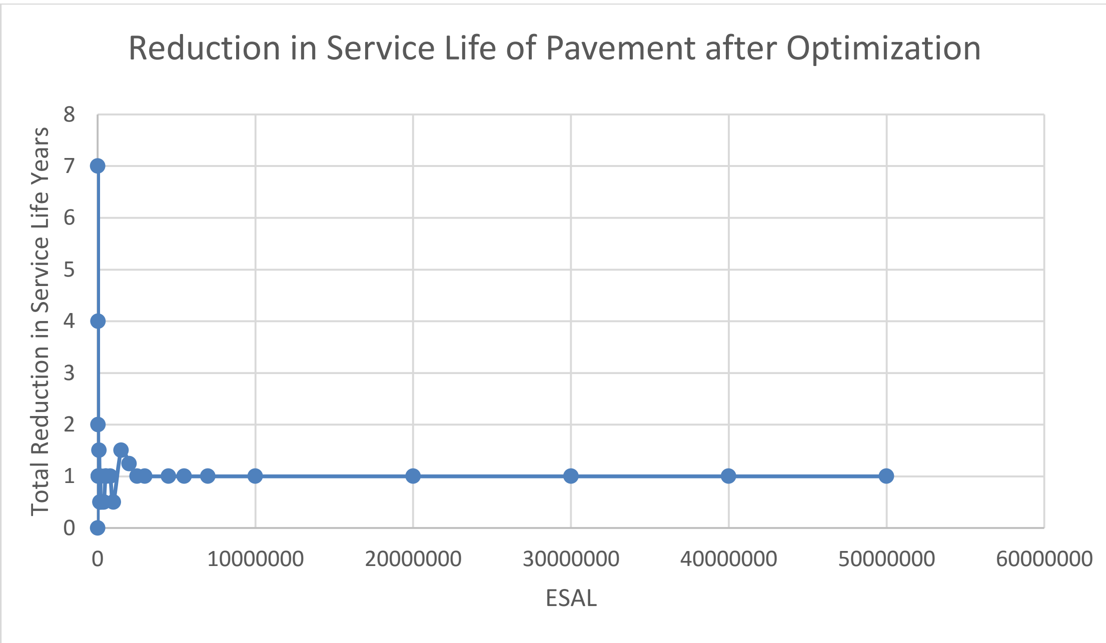


Figure 7-1 Trend Line of Reduction in Service Life

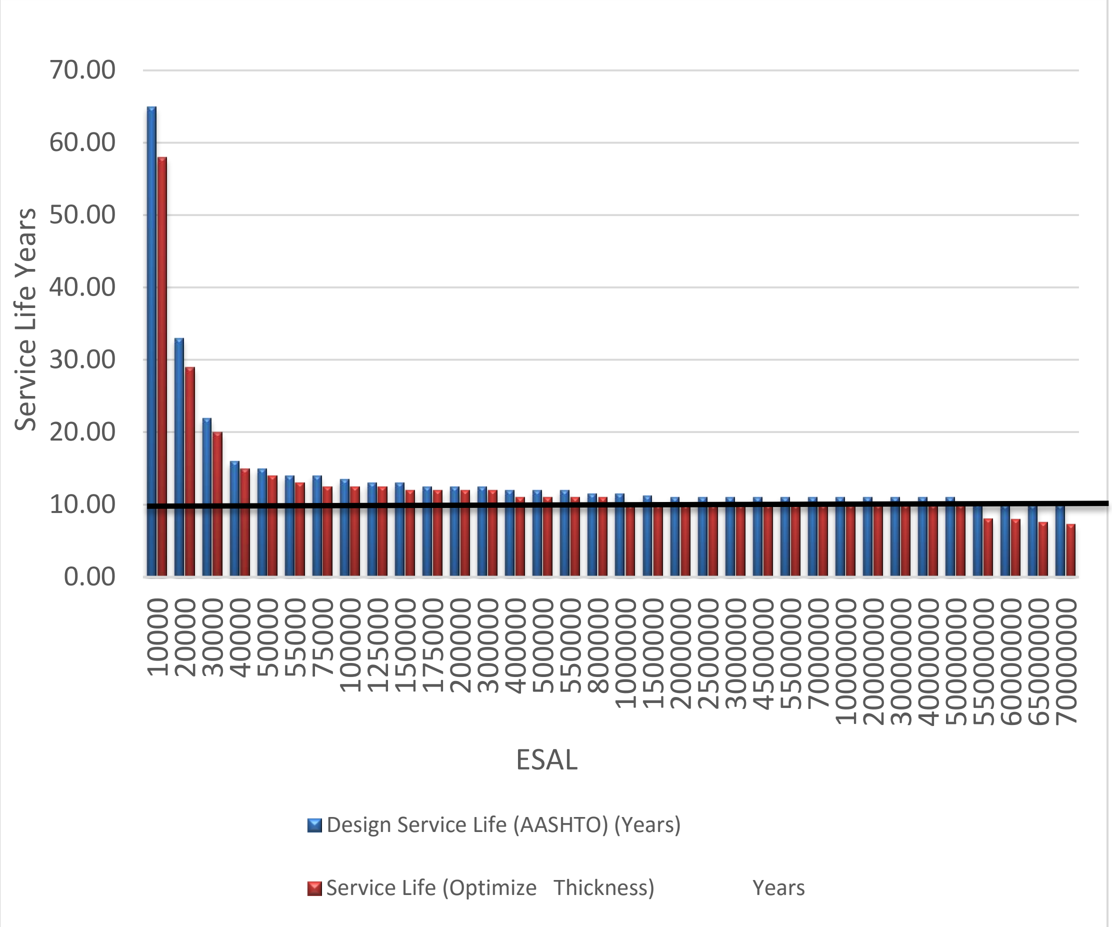


Figure 7-2 Bar Chart with respect to Threshold

From the results it is concluded that after 50 Million ESALs the optimize flexible pavement structural design does not remain feasible or serviceable as the service life is less than 10 year’s threshold value, hence this concludes that for ESALS greater or equal than 50 Million AASHTO flexible pavement structural design alternative is advisable and recommended while the optimized pavement structural design is not recommended. Figure 7-2 showing the results.

7.2 Tensile and Compressive Strains in Pavement Structure:

It seems from results obtained that tensile strain at the bottom of asphalt base layer is reducing with increasing AC thickness. By reducing or optimizing the asphalt layer thicknesses the tensile strains at the bottom of asphalt layer is also increasing this depicts that the pavement will be less susceptible to surface cracking in case of AASHTO Design. It is intuitive from the literature that with increase in asphalt concrete layer thickness the tensile strain at the bottom of asphalt layer tends to reduce. Hence in case of optimize design the asphalt layer thickness is less as compare to the AASHTO design that is why the tensile strains are more in case of optimize layer thicknesses. Figure 7-3 showing the trend line of change in tensile strains. For higher level of traffic more than 2 Million ESALs the increase in tensile strains is more.

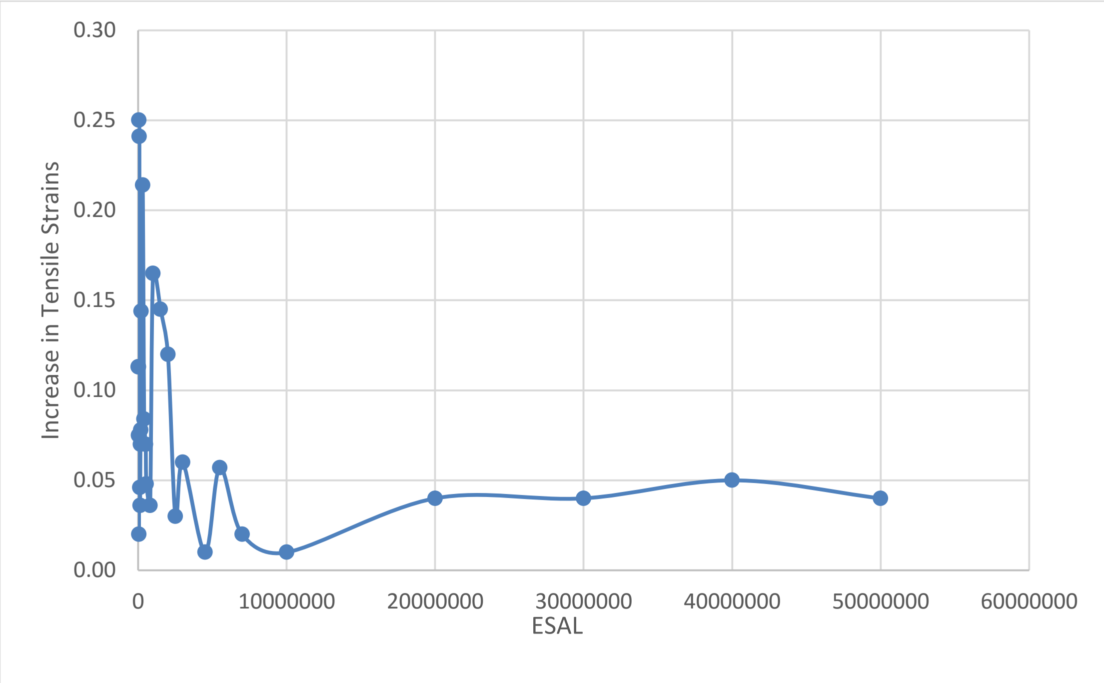


Figure 7-3 Trend Line of Increase in Tensile Strains at Bottom of Asphaltic Base Layer

For compressive strains at the top of sub-grade as the ESALs are increasing the compressive strains tends to reduce as the pavement layer thicknesses are also increasing in order to coup the increase in ESALs. If comes to the comparison between AASHTO and optimize pavement configuration the compressive strains in optimize pavement design are less than in AASHTO. The response is opposite in tensile strains response and can be shown in figure 7-4. Both tensile and compressive strains increase considerably for higher level of traffic however, for compressive strains at the top of sub-grade. The reason behind this anomaly will be based on pavement layer thickness above sub-grade. The compressive strain for optimized design is less as the pavement thickness for base and subbase layer is more in optimized structure design as compared to AASHTO design and also from the literature it is concluded that as the pavement layer thickness increases the strains in pavement structure tends to reduce.

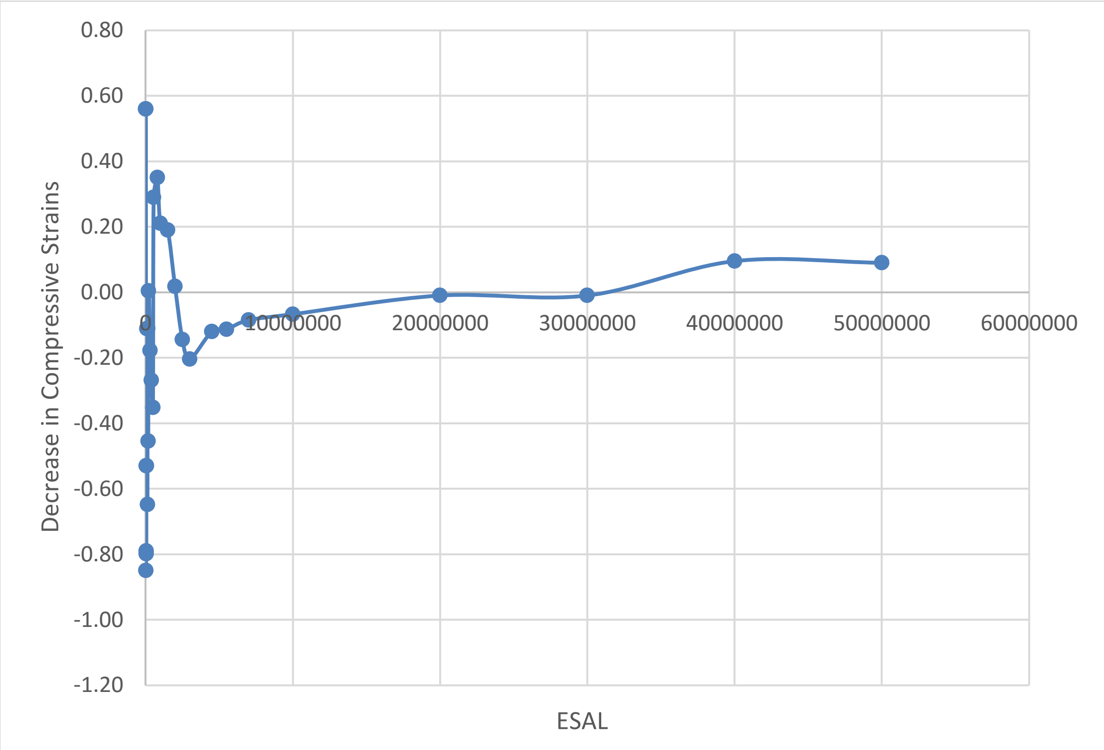


Figure 7-4 Trend Line of Decrease in Compressive Strains at Top of Sub-grade

From figure above it can be concluded that for less volume of traffic the compressive strains in optimized pavement design are slightly more as compared to AASHTO design as base and sub base layer thicknesses for both design approaches are having less difference. The base and sub base layer thickness for optimized design are slightly more as compared to AASHTO design. But for higher volume of traffic the compressive strains in optimized design are less as compared to AASHTO as base and sub base layer thicknesses are more. It is concluded from the results that for both the strains criteria the values of compressive and tensile strains are within the permissible limit given by heukelom and klomp model (1962). Hence it is concluded that for tis criteria both AASHTO and optimize pavement structure design have same response and can be recommended. But considering the cost effectiveness of optimize design it will be more beneficial to recommend the optimize pavement structure design. Hence for this criterion it is recommended to use optimize flexible pavement structural design. **(OATDR).**

7.3 Fatigue and Rutting Damage Ratios:

Damage to pavement occurs when the sum of damage ratio reaches 1. The amount of damage caused is expressed in terms of damage ratio. If any of the damage ratio for fatigue or rutting is less than a value of one, then the pavement can be expected to exceed its design life, if D is greater than one, the pavement is expected to fail prematurely. The governing failure is that which damage ratio exceeds 1.

From the results obtained by KENPAVE analysis it is concluded that pavement have same response in case of AASHTO and optimized design however damage ratio fatigue is increasing after optimization while the damage ratio rut is decreasing. The reason behind this pavement response is that for fatigue cracking rhe responsible strain is tensile strain at the bottom of asphaltic concrete layer and after optimization of pavement structure the layer thickness of asphaltic concrete is reduced hence the strains occurring at the bottom of asphaltic layer increases this results in increase in damage ratio fatigue after optimization. In case of damage ratio rut the compressive strains at the top of subgrade layer is responsible. Now after optimization the base and sub-base layer thickness increased as to meet the strength requirements this results in decrease in compressive strains at the top of subgrade and hence the damage ratio rut also decreased after optimization. The damage ratio for rut failure criterion are much less as compared to fatigue failure criterion which depicts that the governing failure distress in pavement is fatigue. From results it is also concluded that damage ratios for both fatigue and rutting are more in case of optimize pavement structure as the pavement layer thickness are less as compared to AASHTO design. The change or increase in damage ratios for both fatigue and rutting is depicted in form of trend line and shown in figure 7-5. As both the criteria are meeting the threshold requirements hence it is recommended to use optimize flexible pavement structural design. **(OATDR).**

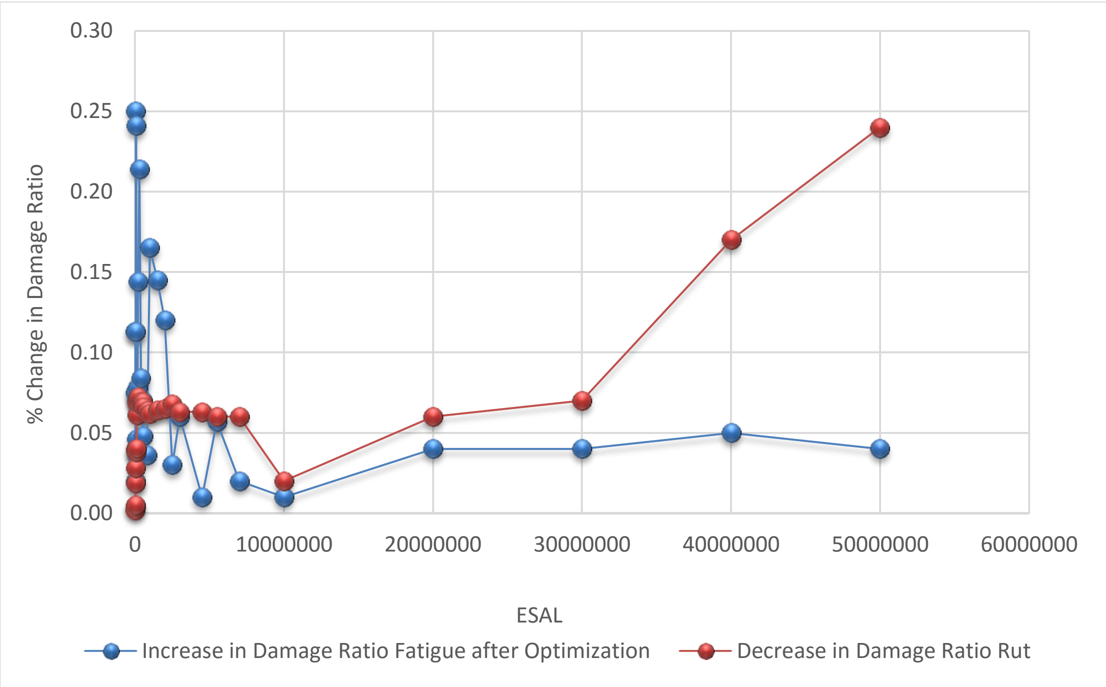


Figure 7-5 Trend Line of Increase/Decrease in Damage Ratios

For both AASHTO and optimized pavement structural design the damage ratios for both rutting and fatigue for all ESALs are less than permissible value of 1. Hence it is recommended to use both design approaches.

7.4 Final Decision Matrix:

From the conclusions and results of this research study the final decision matrix is developed keeping in view all the criterions. Keeping in view the thresholds value for each criterion the recommendations are made. Table 7-5 showing the final decision matrix.

From the decision it is concluded that from analysis it is clear that Optimize Asphalt Thickness Design Approach is traffic volume dependent. For traffic volume more than 50 Million the AASHTO design is recommended. Each criterion is checked against the permissible value. If one of the criteria failing to satisfy the design procedure than it is not recommended whether the design procedure is satisfying the other criterions. It can be seen for ESALs greater than 50 Million the optimize pavement structural design is satisfying the tensile, compressive strains, da mage ratio and cost criterion but it fails when comes to service life criterion hence the optimize design is not recommended and vote is given in favour of AASHTO pavement structural design.

As different design parameters and criterions are used to evaluate the design of pavement. Keeping in view the threshold values it is concluded that after ESALs greater than 50 Million due to increase in traffic loading the optimize pavement fails and have service life of less than 10 years which is the design practice followed in Pakistan, hence the optimized flexible pavement thickness method is not recommended. Table 7-1 shows the final decision matrix drawn out from results.

7.5 Synthesis of Conclusions:

A brief summary of conclusions is provided in this section

- Decision matrix revealed that Optimized Asphalt Thickness Design Approach is Traffic Volume Dependent.
- Optimized Asphalt Thickness Design Approach is not applicable for ESALs greater than 50 Million hence Basic AASHTO Thickness design is recommended.
- For ESALs 50 Million or More the Optimized Pavement Structural Design satisfies all the criteria less the Service Life Criterion
- Tensile Strains in Pavement structure were found to increase with decrease in AC Thickness (Optimization) but were significantly less than threshold strain values.
- Compressive Strains in Pavement structure were found to decrease with decrease in AC Layer Thickness (Optimization) but were significantly less than threshold strain values.
- Damage Ratio for fatigue in Pavement structure were found to increase with decrease in AC Layer Thickness (Optimization) but were significantly less than threshold strain values.
- Damage Ratio for rutting in Pavement structure were found to decrease with decrease in AC Layer Thickness (Optimization) but were significantly less than threshold strain values.
- Damage Ratio for Rutting Failure Criterion are much less as compared to Fatigue Failure Criterion which depicts that the governing failure distress in pavement is Fatigue.

- Cost effectiveness due to optimized design is considerably higher for ESAL up to 1 Million as compared to ESAL beyond 1 Million.
- Difference in pavement AC layer thickness between AASHTO & Optimized design is considerably higher for ESAL up to 1 Million as compared to ESAL beyond 1 Million.

Traffic Volume	Design Recommended
≤ 50 Million	Optimization Asphalt Thickness Design Recommended (OATDR)
> 50 Million	AASHTO Design Method Recommended (ADR)

7.6 Recommendations:

Keeping in view this research studies synthesis following recommendations are made for future research work.

1. For ESAL less than 50 Million Optimize Asphalt Design Approach is recommended based on cost effectiveness and without compromising the strength, durability and service life of flexible pavement as per AASHTO requirements.
2. For ESAL more than 50 Million Optimize Asphalt Design Approach is not recommended because the longevity or service life of pavement is compromising.
3. Comparison with Advanced Pavement Design Methods should also be Made

4. More Design Alternatives should be incorporated in Optimization Model for Future Studies
5. Pavement Response should be checked against Different Design Parameters

7.7 Contribution of this Research:

This research contributed in following ways

- Pioneer study on economization based optimization of flexible pavement structure in Pakistan. Previously no literature has been found on economization of flexible pavement structural design in Pakistan. This research work is considered as a pioneer study.
- Unique study considering the pavement Asphaltic Base course as separate layer in optimized design. There is a severe lack in international literature in which asphalt wearing course and asphalt base course layer has dealt separately for analysis and design. This research work provided consideration for both layers and both layers has been dealt separately for optimization and design.
- Pioneer study to develop a Decision Matrix for optimized pavement structure design for numerous traffic loading conditions and prevailing material requirements in Pakistan

Table 7-1 Final Decision Matrix

Traffic Volume(Mi)	**Nominated Esal**	**Criterias Considered for Research**					
		Cost	**C.Strain**	**T.Strain**	**Rutting**	**Fatigue**	**Service Life**
< =50000	10000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	20000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	30000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	40000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	50000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
50001 - 150000	55000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	75000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	100000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	125000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	150000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
150001 - 500000	175000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	200000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	300000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	400000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	500000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
500001 - 2000000	550000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	800000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	1000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	1500000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	2000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
2000001 -7000000	2500000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	3000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	4500000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	5500000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	7000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
> 7000000	10000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	20000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	30000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	40000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	50000000	OATDR	OATDR	OATDR	OATDR	OATDR	OATDR
	55000000	OATDR	OATDR	OATDR	OATDR	OATDR	ADR
	60000000	OATDR	OATDR	OATDR	OATDR	OATDR	ADR
	65000000	OATDR	OATDR	OATDR	OATDR	OATDR	ADR
	70000000	OATDR	OATDR	OATDR	OATDR	OATDR	ADR

OATDR - Optimize Asphalt Thickness Design Recommended

ADR - AASHTO Design Recommended

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APPENDIX A:

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1
Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C
THICKNESSES OF LAYERS (TH) ARE : 5.08 5.08 11 20.32
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED
FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04
LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120
RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750
NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5
LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 100000
DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854
DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477
DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1
AT BOTTOM OF LAYER 2 TENSILE STRAIN = -2.354E-04
ALLOWABLE LOAD REPETITIONS = 1.279E+06 DAMAGE RATIO = 6.40E-02

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 3.007E-04
ALLOWABLE LOAD REPETITIONS = 7.989E+06 DAMAGE RATIO = 2.03E-02

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 6.40E-02 DESIGN LIFE IN YEARS = 13.49

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 5 4 15 21
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04
LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120
RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750
NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1

NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5
LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 100000
DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854
DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477
DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1
AT BOTTOM OF LAYER 2 TENSILE STRAIN = -2.40E-04
ALLOWABLE LOAD REPETITIONS = 1.084E+06 DAMAGE RATIO = 1.42E-01
AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 2.889E-04
ALLOWABLE LOAD REPETITIONS = 9.559E+06 DAMAGE RATIO = 1.046E-02

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 1.42E-01 DESIGN LIFE IN YEARS = 12.51

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 10.16 12.7 15.24 20.32
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 2000000

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

AT BOTTOM OF LAYER 2 TENSILE STRAIN = -9.200E-05
ALLOWABLE LOAD REPETITIONS = 2.815E+07 DAMAGE RATIO = 8.98E-02

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 1.06E-04
ALLOWABLE LOAD REPETITIONS = 4.365E+08 DAMAGE RATIO = 4.582E-03

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 7.104E-02 DESIGN LIFE IN YEARS = 11.08

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 10 10 25 25
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 2000000

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

AT BOTTOM OF LAYER 2 TENSILE STRAIN = -1.044E-04
ALLOWABLE LOAD REPETITIONS = 1.857E+07 DAMAGE RATIO = 1.550E-01

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 1.058E-04
ALLOWABLE LOAD REPETITIONS = 7.894E+08 DAMAGE RATIO = 4.66E-03

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 1.550E-01 DESIGN LIFE IN YEARS = 10.28

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 13.97 15.24 25.4 25.4
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06

3 2.896E+05 4 1.517E+05 5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 1E+07

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

AT BOTTOM OF LAYER 2 TENSILE STRAIN = -6.111E-05
ALLOWABLE LOAD REPETITIONS = 1.082E+08 DAMAGE RATIO = 2.37E-01

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 7.73E-05
ALLOWABLE LOAD REPETITIONS = 3.526E+09 DAMAGE RATIO = 2.836E-03

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 2.37E-01 DESIGN LIFE IN YEARS = 10.92

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 14 14.5 30 30
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 1E+07

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

AT BOTTOM OF LAYER 2 TENSILE STRAIN = -6.209E-05
ALLOWABLE LOAD REPETITIONS = 1.027E+08 DAMAGE RATIO = 2.54E-01

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 7.035E-05
ALLOWABLE LOAD REPETITIONS = 5.332E+09 DAMAGE RATIO = 1.9E-03

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 2.54E-01 DESIGN LIFE IN YEARS = 10.27

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 19.05 20.32 35.56 38.1
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 5E+07

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

AT BOTTOM OF LAYER 2 TENSILE STRAIN = -3.730E-05
ALLOWABLE LOAD REPETITIONS = 5.491E+08 DAMAGE RATIO = 4.3E-01

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 4.638E-05
ALLOWABLE LOAD REPETITIONS = 3.445E+10 DAMAGE RATIO = 3.91E-03

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 4.3E-01 DESIGN LIFE IN YEARS = 10.98

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED

NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 18 19 30 30
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :   1 3.103E+06   2 2.413E+06
3 2.896E+05   4 1.517E+05   5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1   0.000   0.000 2   0.000   6.750
3 24.000   0.000 4 24.000   6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 5E+07

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

AT BOTTOM OF LAYER 2 TENSILE STRAIN = -4.194E-05
ALLOWABLE LOAD REPETITIONS = 3.735E+08 DAMAGE RATIO = 6.77E-01

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 4.589E-05
ALLOWABLE LOAD REPETITIONS = 1.494E+10 DAMAGE RATIO = 3.346E-03

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 6.77E-01 DESIGN LIFE IN YEARS = 10.07

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1
TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 20 20 40 40
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3

CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 7E+07

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

AT BOTTOM OF LAYER 2 TENSILE STRAIN = -3.399E-05
ALLOWABLE LOAD REPETITIONS = 6.228E+08 DAMAGE RATIO = 5.50E-01

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 4.413E-05
ALLOWABLE LOAD REPETITIONS = 4.767E+10 DAMAGE RATIO = 1.500E-03

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 5.50E-01 DESIGN LIFE IN YEARS = 10.01

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA=1, SO DAMAGE ANALYSIS WITH SUMMARY PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 1

TOLERANCE FOR INTEGRATION (DEL) -- = 0.001
NUMBER OF LAYERS (NL)------------- = 5
NUMBER OF Z COORDINATES (NZ)------ = 0
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 18.5 20 40 40
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.35 0.4 0.45 0.45
ALL INTERFACES ARE FULLY BONDED

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 3.103E+06 2 2.413E+06
3 2.896E+05 4 1.517E+05 5 8.963E+04

LOAD GROUP NO. 1 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 11.3
CONTACT PRESSURE (CP)------------- = 552
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 4
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 33.75
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 120

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 6.750
3 24.000 0.000 4 24.000 6.750

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 2
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 5

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 7E+07

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 2 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 5 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

AT BOTTOM OF LAYER 2 TENSILE STRAIN = -3.514E-05
ALLOWABLE LOAD REPETITIONS = 5.105E+08 DAMAGE RATIO = 9.48E-01

AT TOP OF LAYER 5 COMPRESSIVE STRAIN = 4.460E-05
ALLOWABLE LOAD REPETITIONS = 4.102E+10 DAMAGE RATIO = 1.107E-03

****
* SUMMARY OF DAMAGE ANALYSIS *
****

MAXIMUM DAMAGE RATO = 9.48E-01 DESIGN LIFE IN YEARS = 7.29

APPENDIX B:

ESAL RANGE	ESAL	Total Thickness AC AASHTO Design (cm)	Total Thickness AC Optimize (cm)	Design Service Life (AASHTO) (Years)	Service Life (Optimize Thickness) Years
< =50000	10000	7.62	4.00	65.00	58.00
	20000	7.62	4.00	33.00	29.00
	30000	7.62	6.00	22.00	20.00
	40000	7.62	6.00	16.00	15.00
	50000	7.62	6.00	15.00	14.00
50001 - 150000	55000	9.08	6.00	14.00	13.00
	75000	9.08	6.00	14.00	12.50
	100000	10.16	9.00	13.50	12.50
	125000	10.16	9.00	13.00	12.50
	150000	10.16	9.00	13.00	12.00
150001 - 500000	175000	12.70	11.00	12.50	12.00
	200000	12.70	11.00	12.50	12.00
	300000	15.24	13.00	12.50	12.00
	400000	15.24	14.00	12.00	11.00
	500000	15.24	14.00	12.00	11.00
500001 - 2000000	550000	15.24	14.00	12.00	11.00
	800000	17.78	17.00	11.50	11.00
	1000000	20.32	17.50	11.50	10.00
	1500000	20.32	19.00	11.24	10.00
	2000000	22.86	20.00	11.00	10.00
2000001 -7000000	2500000	22.86	20.00	11.00	10.00
	3000000	24.70	22.50	11.00	10.00
	4500000	25.40	24.00	11.00	10.00
	5500000	27.94	25.00	11.00	10.00
	7000000	28.24	27.00	11.00	10.00
> 7000000	10000000	29.21	27.00	11.00	10.00
	20000000	33.02	30.00	11.00	10.00
	30000000	36.07	33.00	11.00	10.00
	40000000	37.34	35.00	11.00	10.00
	50000000	39.37	37.00	11.00	10.00
	55000000	39.50	37.30	10.22	8.09
	60000000	40.00	38.00	10.00	7.95
	65000000	41.00	39.00	10.00	7.57
	70000000	41.50	40.00	10.00	7.29

Asphalt Wearing Course Thickness Design (cm)	Asphalt Wearing Course Thickness Optimize (cm)	Asphalt Base Course Thickness Design (cm)	Asphalt BaseCourse Thickness Optimize (cm)	Base Course Thickness Design(cm)	Base Course Thickness Optimize (cm)	Subbase Design Thickness (cm)	Subbase Optimize Thickness (cm)
2.54	2	5.08	2	10.16	11	15.24	20
2.54	2	5.08	2	10.16	11	15.24	20
2.54	2	5.08	4	11	15	15.24	20
2.54	2	5.08	4	11	15	15.24	20
2.54	2	5.08	4	11	15	15.24	20
4	3	5.08	4	11	15	15.24	20
4	3	5.08	4	11	15	15.24	20
5.08	5	5.08	4	12	15	20.32	21
5.08	4.5	5.08	5	12	15	20.32	21
5.08	4.5	5.08	5	12	15	20.32	21
5.08	5	7.62	6	15.24	20	20.32	21
5.08	5	8	7	15.24	20	20.32	21
5.08	6	10.16	7	15.24	20	20.32	21
5.08	7	10.16	7	15.24	20	20.32	21
5.08	7	10.16	7	15.24	20	20.32	21
5.08	7	10.16	7	15.24	20	20.32	21
7.62	7	10.16	10	15.24	20	20.32	21
10.16	8.5	10.16	10	15.24	20	20.32	25
10.16	9	10.16	10	15.24	20	20.32	25
10.16	10	12.7	10	15.24	25	20.32	25
10.16	10	12.7	10	15.24	25	20.32	25
12	11	12.7	11.5	15.24	25	20.32	25
12.7	12	12.7	12	20.32	25	20.32	25
12.7	12	15.24	13	20.32	25	20.32	25
13	13	15.24	14	20.32	25	20.32	25
13.97	13	15.24	14	25.4	30	25.4	30
16.51	15	16.51	15	25.4	30	25.4	30
17.78	16	18.29	17	25.4	30	25.4	30
18.29	17	19.05	18	25.4	30	25.4	30
19.05	18	20.32	19	35.56	37	38.1	40
19.50	17.50	20.00	20.00	36.00	40.00	38.00	40.00
20.00	18.00	20.00	20.00	36.00	40.00	38.00	40.00
20.00	18.50	21.00	20.00	36.00	40.00	38.00	40.00
20.00	18.50	21.50	20.00	36.00	40.00	38.00	40.00

Tensile Strain in Asphalt Layer(10^{-4}), AASHTO Design	Tensile Strain in Asphalt Layer(10^{-4}), Optimized Design	Compressive Strain at top of Subgrade, AASHTO Method	Compressive Strain at top of Subgrade, Optimize.AC Thickness
2.88	2.99	4.71	5.27
2.88	2.99	4.71	5.27
2.88	2.96	4.53	3.68
2.88	2.96	4.44	3.65
2.88	2.90	4.40	3.60
2.58	2.83	3.92	3.39
2.58	2.82	3.92	3.39
2.35	2.40	3.00	2.89
2.35	2.39	3.01	2.36
2.20	2.27	2.21	2.10
2.00	2.07	2.54	2.08
1.87	2.02	2.18	2.18
1.55	1.77	1.85	1.68
1.55	1.64	1.85	1.58
1.55	1.62	1.85	1.50
1.55	1.60	1.12	1.41
1.28	1.32	1.12	1.47
1.08	1.24	1.09	1.30
1.08	1.22	1.09	1.28
0.92	1.04	1.06	1.08
0.92	0.95	1.23	1.09
0.82	0.88	1.12	0.92
0.80	0.81	1.04	0.92
0.71	0.77	1.01	0.90
0.66	0.68	0.91	0.83
0.61	0.62	0.77	0.703
0.53	0.57	0.68	0.67
0.45	0.49	0.63	0.62
0.40	0.45	0.49	0.58
0.37	0.41	0.46	0.55
0.37	0.40	0.46	0.46
0.35	0.39	0.45	0.45
0.35	0.38	0.45	0.45
0.34	0.35	0.44	0.44

Damage Ratio Fatigue.AC Thickness	Damage Ratio Fatigue. Optimize AC Thickness	Damage Ratio Rut.AC Thickness	Damage Ratio Rut. Optimize AC Thickness
0.0150	0.0170	0.0093	0.0155
0.0300	0.0340	0.0186	0.0311
0.0440	0.0490	0.0236	0.0094
0.0510	0.0695	0.0315	0.0125
0.0520	0.0800	0.0460	0.0125
0.0560	0.0957	0.0220	0.0118
0.0620	0.1000	0.0309	0.0167
0.0640	0.1042	0.0143	0.0203
0.0690	0.1300	0.0156	0.0058
0.0713	0.1400	0.0047	0.0035
0.0742	0.1450	0.0016	0.0039
0.0737	0.1460	0.0059	0.0060
0.0790	0.1480	0.0043	0.0050
0.0795	0.1490	0.0057	0.0027
0.0800	0.1490	0.0018	0.0027
0.0830	0.1490	0.0078	0.0023
0.0852	0.1490	0.0058	0.0047
0.0870	0.1490	0.0059	0.0031
0.0890	0.1530	0.0060	0.0043
0.0898	0.1550	0.0045	0.0046
0.0880	0.1560	0.0057	0.0036
0.0970	0.1600	0.0046	0.0042
0.1010	0.1640	0.0048	0.0029
0.1070	0.1670	0.0052	0.0030
0.1100	0.1700	0.0050	0.0029
0.2300	0.2500	0.0028	0.0019
0.3200	0.3800	0.0034	0.0031
0.3900	0.4600	0.0034	0.0033
0.4000	0.5700	0.0014	0.0037
0.4300	0.6700	0.0014	0.0033
0.4900	0.7800	0.0016	0.0015
0.5100	0.8700	0.0016	0.0015
0.5400	0.9100	0.0016	0.0020
0.5500	0.9500	0.0015	0.0017

Total Reduction in Thickness (cm)	Total Reduction in Service Life After Thickness Reduction (Years)	Total Increase in Tensile Strain After Thickness Reduction (10^{-4})	Total Increase in Compressive Strain After Thickness Reduction (10^{-4})	Total Increase in Damage Ratio (Fatigue) After Thickness Reduction	Total Increase in Damage Ratio (Rutting) After Thickness Reduction
3.62	7.00	0.11	0.56	0.0020	-0.0062
3.62	4.00	0.11	0.56	0.0040	-0.0125
1.62	2.00	0.08	-0.85	0.0050	-0.0142
1.62	1.00	0.08	-0.79	0.0185	-0.0190
1.62	1.00	0.02	-0.80	0.0280	-0.0034
3.08	1.00	0.25	-0.53	0.0397	-0.0102
3.08	1.50	0.24	-0.53	0.0380	-0.0142
1.16	1.00	0.05	-0.11	0.0402	-0.0060
1.16	0.50	0.04	-0.65	0.0610	-0.0015
1.16	1.00	0.07	-0.11	0.0687	-0.0013
1.70	0.50	0.08	-0.46	0.0708	-0.0011
1.70	0.50	0.14	0.00	0.0723	0.0001
2.24	0.50	0.21	-0.18	0.0690	-0.0010
1.24	1.00	0.08	-0.27	0.0695	-0.0030
1.24	1.00	0.07	-0.35	0.0690	-0.0011
1.24	1.00	0.05	0.29	0.0660	-0.0055
0.78	0.50	0.04	0.35	0.0638	-0.0011
2.82	1.50	0.17	0.21	0.0620	-0.0028
1.32	1.24	0.15	0.19	0.0640	-0.0017
2.86	1.00	0.12	0.02	0.0652	0.0001
2.86	1.00	0.03	-0.15	0.0680	-0.0021
2.20	1.00	0.06	-0.20	0.0630	-0.0004
1.40	1.00	0.01	-0.12	0.0630	-0.0019
2.94	1.00	0.06	-0.11	0.0600	-0.0022
1.24	1.00	0.02	-0.08	0.0600	-0.0022
2.21	1.00	0.01	-0.07	0.0200	-0.0010
3.02	1.00	0.04	-0.01	0.0600	-0.0003
3.07	1.00	0.04	-0.01	0.0700	-0.0001
2.34	1.00	0.05	0.10	0.1700	-0.0012
2.37	1.00	0.04	0.09	0.2400	-0.0006
2.20	2.13	0.03	0.00	0.2900	-0.0001
2.00	2.05	0.04	0.00	0.3600	-0.0002
2.00	2.43	0.03	0.00	0.3700	-0.0004
1.50	2.71	0.01	0.00	0.4000	-0.0004

Asphalt Concrete Cost Design (Rs)	Asphalt Concrete Cost Optimize (Rs)	Base Course Cost Design (Rs)	Base Course Cost Optimize(Rs)	SuBBase Course Cost Design (Rs)	SuBBase Course Cost Optimize (Rs)	Total Cost (AASHTO) Rs	Total Cost (Optimize) Rs
1426.416	755.251	216.134	234.003	270.682	355.226	1913.232	1344.480
1426.416	755.251	216.134	234.003	270.682	355.226	1913.232	1344.480
1426.416	1123.162	234.003	319.095	270.682	355.226	1931.101	1797.483
1426.416	1123.162	234.003	319.095	270.682	355.226	1931.101	1797.483
1426.416	1123.162	234.003	319.095	270.682	355.226	1931.101	1797.483
1709.175	1316.833	234.003	319.095	270.682	355.226	2213.860	1991.154
1709.175	1316.833	234.003	319.095	270.682	355.226	2213.860	1991.154
1918.339	1704.173	255.276	319.095	360.910	372.987	2534.524	2396.255
1918.339	1791.293	255.276	319.095	360.910	372.987	2534.524	2483.376
1918.339	1791.293	255.276	319.095	360.910	372.987	2534.524	2483.376
2385.585	2072.084	324.201	425.460	360.910	372.987	3070.695	2870.531
2455.488	2256.039	324.201	425.460	360.910	372.987	3140.598	3054.487
2852.832	2449.710	324.201	425.460	360.910	372.987	3537.942	3248.157
2852.832	2643.380	324.201	425.460	360.910	372.987	3537.942	3441.827
2852.832	2643.380	324.201	425.460	360.910	372.987	3537.942	3441.827
2852.832	2643.380	324.201	425.460	360.910	372.987	3537.942	3441.827
3344.755	3195.246	324.201	425.460	360.910	372.987	4029.865	3993.693
3836.677	3485.752	324.201	425.460	360.910	444.033	4521.787	4355.244
3836.677	3582.587	324.201	425.460	360.910		4521.787	4008.047
4303.924	3776.257	324.201	531.825	360.910	444.033	4989.034	4752.115
4303.924	3776.257	324.201	531.825	360.910	444.033	4989.034	4752.115
4660.277	4245.860	324.201	531.825	360.910	444.033	5345.387	5221.718
4795.846	4531.508	432.267	531.825	360.910	444.033	5589.023	5507.366
5263.093	4715.464	432.267	531.825	360.910	444.033	6056.270	5691.321
5321.194	5093.090	432.267	531.825	360.910	444.033	6114.371	6068.947
5509.054	5093.090	540.334	638.190	451.137	532.839	6500.526	6264.119
6234.600	5664.386	540.334	638.190	451.137	532.839	7226.072	6835.415
6808.002	6225.967	540.334	638.190	451.137	532.839	7799.473	7396.996
7046.580	6603.592	540.334	638.190	451.137	532.839	8038.051	7774.621
7427.393	6981.218	756.468	787.101	676.706	710.452	8860.566	8478.771
7455.68	7068.34	765.83	850.92	674.93	710.45	8896.44	8629.71
7552.51	7165.17	765.83	850.92	674.93	710.45	8993.27	8726.55
7736.47	7262.01	765.83	850.92	674.93	710.45	9177.23	8823.38
7828.45	7262.01	765.83	850.92	674.93	710.45	9269.20	8823.38