Study on Reliability of Pavement Quality due to Variable Material Properties and Environmental Conditions

Thesis Submitted in Partial Fulfilment of the Requirement for the Award of the Degree

OF

Master of Civil Engineering

IN

Soil Mechanics and Foundation Engineering

By

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EXECUTIVE SUMMARY

This report documents the research that incorporated reliability analysis into the existing mechanistic-empirical (M-E) flexible pavement design method based on IRC 37:2012 for India.

M-E methods are gaining more acceptances and greater use both nationally and internationally since they are more robust than traditional empirically-based design methods. For example, M-E methods can adapt to new design conditions (e.g., heavier loads, new pavement materials) by relying primarily upon mechanistic pavement modelling. Empirically-based procedures, however, are limited to the original test conditions encountered during procedure development.

The empirical method, mentioned above, assumed that the input parameters used for design were deterministic, but virtually every design parameter has some associated variability. Consequently, for the M-E procedure to be complete there must be an accounting for the inherent variability within the design process. Reliability analysis allows for a rational accounting of the variability in the design parameters. Other advantages of using reliability include the calibration of new design methods, developing rational design specifications, optimizing resources, and assessing the damage and remaining life of the pavement structure.

The project began with a comprehensive literature review that examined pavement design in the context of reliability analysis. The following topics were investigated in the literature review: definitions of reliability, methods of reliability analysis, reliability in the 1993 American Association of State Highway and Transportation Officials (AASHTO) Design Guide and variability of the design input parameters.

Once the literature review was complete, it was possible to proceed with the incorporation of reliability into the M-E method. The first task was to characterize the variability of the design input parameters. This was accomplished by synthesizing data from the literature. The second task was to formulate the reliability analysis scheme. It was found that Monte Carlo simulation was a straightforward means of incorporating reliability into the existing M-E framework. Consequently, our conventional method of design was modified to accommodate Monte Carlo simulation and reliability analysis.

After the input parameters had been statistically characterized, it was necessary to perform a sensitivity analysis to achieve the third objective of this project. The result of the sensitivity analysis are summarized below,

a. Fatigue variability is most affected by the inputs closer to the pavement surface.
b. The number of Monte Carlo cycles that should be used in design is 7,500.
c. Temperature is the most important parameter which causes the variation of the input parameters to a great extent.

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SYMBOLS AND ABBREVIATIONS USED

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society of Testing and Materials
BC	Bituminous Concrete
BM	Bituminous Macadam
CBR	California Bearing Ratio
CPVD	Commercial Vehicles per Day
COV	Coefficient of Variation
DBM	Dense Bituminous Macadam
F_R	Reliability Design Factor
GPR	Ground Penetrating Radar
IRC	Indian Road Congress
M_R	Resilient Modulus
MSA	Million Standard Axles
MORTH	Ministry of Road Transport and Highways
N_f	Cumulative Number of Repetitions for Fatigue Failure
N_t	Actual Number of ESALs, a structure can withstand
N_T	Actual Number of ESALs that will be applied for the design period
SAMI	Stress Absorbing Membrane Interlayer

RAP	Reclaimed Asphalt Pavement
V_a	Volume of Air Voids
V_b	Volume of Bitumen
VDF	Vehicle Damage Factor
WMM	Wet Mix Macadam
w_T	Predicted ESALs that will be applied to the pavement for the design period
W_t	Predicted ESALs that a structure can withstand
$Z_{\frac{\alpha}{2}}$	Associated *z*-statistics from standard normal table
ϵ_a	Axial Strain
ϵ_t	Horizontal Tensile Strain
ϵ_v	Vertical Subgrade Strain
ϑ	Poisson's Ratio
ε	Acceptable Level of Error

CHAPTER – 1

INTRODUCTION

Indian Road Network of 42 lakh km is 2nd largest in the world. Majority of the pavements are flexible type. With the rapid socio-economic development in India, there has been tremendous growth in industrialization of the country. This has resulted in a spurt of freight and passenger transport movement and increase in demand for better quality of road and transport system. In late seventies/eighties India also awakened to the importance of the multiplier effects in economy of Highway Development for the over-all benefit of the Country and took up comprehensive Projects with borrowed and internal investment of large amounts for the Development of Highways.

Flexible pavements are those having negligible flexural strength and are flexible in structural actions under the loads. The design of flexible pavement is based on load distributing characteristics of the component layers. Flexible pavements do possess some flexural strength which is however negligible. A typical flexible pavement consists of four components namely - Soil subgrade, Sub base, Base course, Surface course.

Flexible Pavements are widely used despite some doubts regarding their economics under different conditions. Two most important parameters that govern the pavement design are soil sub-grade and traffic loading. The Indian guidelines (IRC 37) for the design of flexible pavements use soil sub-grade strength in terms of California Bearing Ratio and traffic loading in terms of million standard axles (msa).

The Pavement designs given in the edition IRC: 37-1984 were applicable to design traffic up to only 30 million standard axles (msa). The earlier code is empirical in nature which has limitations regarding applicability and extrapolation. IRC: 37-2001 follows analytical designs and developed new set of designs up to 150 msa which is a conventional method for designing single or bi-layered flexible pavements in India.

IRC 37:2012 incorporates some of the new and alternate materials in the current design practices. A designer can use his sound engineering judgment consistent with local environment using a semi-mechanistic approach for design of pavements, but failed to give any idea about the stochastic variation of the material properties which can make the design unreliable to some extent.

In this paper, we are mainly concerned with incorporation of Reliability concept into our so called empirical conventional design method.

The role of reliability in pavement design is to quantify the probability that a pavement structure will perform, as intended, for the duration of its design life. Many of the parameters associated with pavement design and construction exhibit natural variability. Therefore, in order for a thickness design methodology to be complete, there must be an accounting of variability within the process. Reliability analysis allows for a rational accounting of the variability in the design parameters. Other advantages of using reliability include the calibration of new design methods, developing rational design specifications, optimizing resources, and assessing the damage and remaining life of the pavement.

Material properties, initial layer thicknesses, and load configurations are entered into a load-displacement model that calculates stresses and strains at critical locations (IITPAVE software can be used for this part). The calculated stresses and strains are used to compute the number of allowable loads until failure, while the number of expected loads for each particular condition must also be determined. This process is then iterated for each seasonal condition and load configuration.

The component missing from the above methodology is reliability. There is statistical variation in the input parameters. Consequently, there is variability in the calculated stresses and strains that lead to variations in the number of allowable loads. There is also variability in the number of expected loads during the design period. Finally, there is variability in regard to the transfer functions that predict pavement life.

In this paper, the predicted pavement life is a function of the pavement strain, and bituminous layer stiffness, which is a function of the surrounding temperature. Since the temperature is variable, it follows that the pavement life would also exhibit variability. The reliability may then be interpreted as the probability of the pavement structure exceeding some level of predicted pavement life. In actual pavement design, there are many more input parameters to consider which all contribute to the stochastic nature of the design methodology. Therefore, the incorporation of reliability in flexible pavement thickness design is the focus of this research project.

CHAPTER – 2

LITERATURE REVIEW

2.1. General:

Reliability concept is very new in India, especially in pavement design. But all over the world, Reliability analysis is gaining its acceptance globally as a major parameter for checking the quality of the pavement. As Mechanistic – Empirical approach is widely used for designing flexible pavement; Reliability concept can add a new dimension on it. However very small numbers of research works are available on this reliability analysis, some of the literatures what we reviewed are being topic-wisely stated below.

2.2. Definition of Reliability:

There are a number of definitions concerning the term "reliability." In the general sense, reliability implies trustworthiness or dependability.

When applied to structural pavement design, the *AASHTO Guide* defines reliability in this way (1993).

"The reliability of a pavement design-performance process is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period."

Other definitions of reliability include, *"...the probability that:"*

"... serviceability will be maintained at adequate levels from a user's point of view, throughout the design life of the facility" (Lemer A.C. and Moavenzadeh F., 1971).

"... the load applications a pavement can withstand in reaching a specified minimum serviceability level is not exceeded by the number of load

applications that are actually applied to the pavement" (Kher R.K. and Darter M.I., 1973).

"... the pavement system will perform its intended function over its design life (or time) and under the conditions (or environment) encountered during operation" (Darter M.I. and Hudson W.R., 1973).

"... any particular type of distress (or combinations of distress manifestations) will remain below or within the permissible level during the design life" (AASHTO Guide, 1993).

"... a pavement as designed will withstand the actual number of load applications on it during a selected design life while maintaining its structural integrity" (Kulkarni Ram B., 1994).

Some of the above definitions imply pavement failure as a serviceability loss (e.g., loss of ride quality). Serviceability is often viewed as a subjective measure of pavement performance, while more objective measures include evaluating the fatigue cracking and rut depth of the pavement. For example, pavement failure may be defined as a specific amount of rutting or fatigue cracking. Regardless of the type of failure, it is critical to establish a failure threshold since in the strictest sense, reliability is expressed as,

R = 1- P (Failure)

As cited by *Kulkarni (1994)*, a common basis for comparison between pavement types is essential in estimation of life cycle costs. Since flexible and rigid pavements exhibit different types of distress, defining failure by distress could lead to erroneous conclusions. Therefore, *Kulkarni* defines reliability in terms of traffic that is the same regardless of pavement type,

"To provide uniformity, pavement design reliability is defined as the probability that the pavement's traffic load capacity exceeds the cumulative traffic loading on the pavement during a selected design life."

The above statement may be expressed mathematically as

R = P (N > n)

Where "N" is the traffic load capacity of the pavement structure and "n" is the actual number of load applications. This definition is consistent with definitions in structural mechanics where reliability is the probability that the resistance provided by the structure is greater than the load effects. A graphical representation of the equation is shown in *Figure 2.1*. The shaded area in the figure concerns reliability but is not the explicit representation of reliability. *Kulkarni* explains that the figure simply illustrates how the two distributions may overlap and moving them further apart increases the reliability.

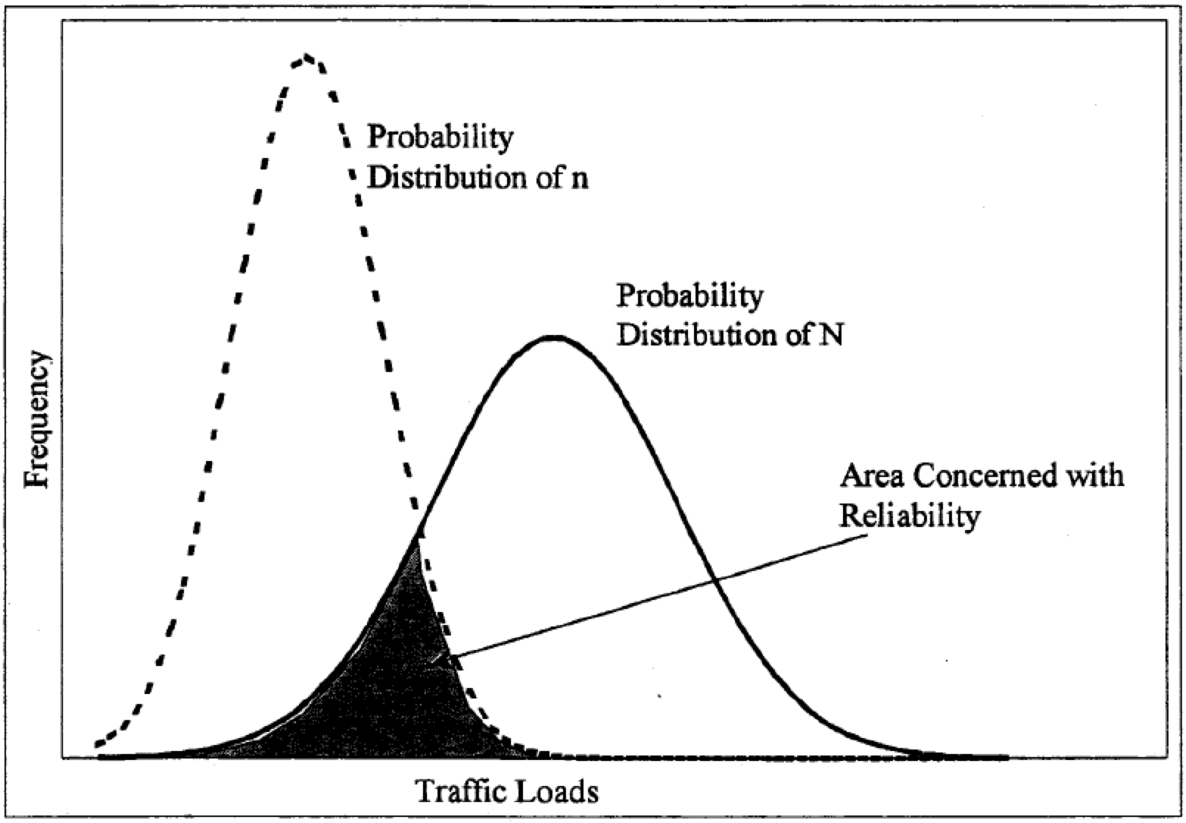


Figure 2.1 – Probability Distribution of N and n

It is important to keep in mind that N and n are random variables. Consequently, in order to quantify reliability, the distribution of each must be determined. Typically, N and n are determined from functions that utilize relevant input parameters. The variability of the input parameters causes N and n to exhibit variation. For example, say that N may be expressed as a function of three input parameters x_1, x_2, and x_3. Since each of the x values exhibit variability, N must also be variable. Therefore, the characterization of N and n are critical to reliability.

2.3. Reliability Analysis:

In order to accurately quantify reliability, reliability analysis must be performed. The objective of reliability analysis, as described by *Harr (1987)*, is to determine the probability distribution of a function's output given a number of variable input parameters.

There are three general categories of reliability analysis cited by *Harr* and each yields a representation of the output probability distribution,

i. First Order, Second Moment (FOSM) method.
ii. Point-Estimate method (PEM).
iii. Exact method.

2.3.1. FOSM Method:

First-Order, Second Moment (FOSM) methods are based on the truncation of the Taylor series expansion of the function in question *(Harr Milton E., 1987)*. In contrast to exact methods, the inputs and outputs are expressed as expected values and standard deviations rather than entire distributions. While this reduces the reliance upon computers, the mathematical requirements in terms of the derivatives associated with the Taylor series may be prohibitively complicated. This is especially true when dealing with a load-displacement model including iterative steps and complicated functions that calculate pavement response.

2.3.2. Point-Estimate Method:

The second category presented by *Harr (1987)* was the point-estimate method (PEM), which was originally developed by *Rosenblueth (1975)*. This method avoids the issue of obtaining derivatives required by the FOSM method and most calculations may be handled by a pocket calculator. The method relies upon knowing the mean, variance, and skewness of each input parameter. Two points, drawn from each of the input distributions, are then selectively chosen about the mean and their associated probabilities are determined. The parameters are then transformed by the function and the mean and variance of the function are determined. Other researchers, including *Van Cauwelaert (1988 and 1994)* expanded the original work of Rosenblueth to include a three-point estimation for better accuracy.

A disadvantage of this method is that although the selected points are transformed by the function, the probabilities are not, they are simply associated with each point. *Eckmann (1987)* demonstrated, however, that if the transforming function could be modelled by a polynomial with a certain degree (generally second or third order) then the Rosenblueth method provides accurate results.

2.3.3. Exact Method:

The final category cited by *Harr (1987)*, exact methods, includes numerical integration and Monte Carlo simulation. *Harr* uses the term "exact" in the context that entire distributions are input into the function rather than representative points. Therefore, the probability distributions of all the component variables must be initially specified. Briefly, the Monte Carlo method involves artificially reproducing each input distribution, entering the values into the function, and obtaining the output distribution.

The primary advantage of an exact method is that the complete probability distribution of the dependent random variable is determined. This proves especially useful when examining the "tails" of the distribution. Additionally, Monte Carlo simulation is straightforward, involving nothing more than generation of random numbers and transforming these into particular distributions. The relative simplicity of the method is the primary motivation for using Monte Carlo simulation in this project.

As described by *Harr*, numerous Monte Carlo cycles may be required for an accurate approximation of the true distribution. In order to determine the number of required cycles, M, he proposed using a normal approximation of a binomial distribution. When generating the output distribution consisting of M cycles there are "P" correct and "(1-P)" incorrect values. With a large M, this binomial distribution may be approximated as normal. This approach leads to an equation that estimates the number of required cycles given an acceptable level of error and number of input parameters.

$$M = \left(\frac{Z_{\frac{\alpha}{2}}^{2}}{4\varepsilon^{2}}\right)^{m} \qquad \textit{Equation 2.1}$$

Where M= Number of required cycles,

ε = Acceptable level of error,

$Z_{\frac{\alpha}{2}}$ = Associated z-statistics from standard normal table,

m = Number of input parameters to function.

This equation will escalate the number of required cycles very quickly. For example, assuming only one parameter (m=l) and 99% confidence (ε = 0.01, $Z_{\alpha/2}$ = 2.58), the required number of cycles is 16,641. This number of cycles would already be somewhat prohibitive, and with the addition of more variables (increasing m), the number would currently be unreasonable for a personal computer. Another method of estimating the precision of a Monte Carlo distribution involves checking the repeatability of the results for a given number of cycles, M *(Mann Nancy R., Schafer Ray E., and Singpurwalla Nozer D. (1974))*.

2.4. Reliability in the 1993 AASHTO Pavement Design Method:

2.4.1. Method of Reliability Analysis in the AASHTO Guide:

An examination of either the flexible or rigid pavement design algorithms in the *AASHTO Guide (1993)* reveals that reliability is accounted for in the procedure. For design purposes, the reliability term is straightforward to use and guidance is given in selecting the relevant parameters. However, it is useful to investigate the origins of reliability in the AASHTO method because it lends insight into the general formulation of reliability in pavement design.

In the AASHTO guide, pavement section designs are governed by a performance equation. It is inherently assumed that the equation is an explicit mathematical formula that predicts the number of equivalent single axle loads (ESALs) the pavement can endure before reaching the specified minimum serviceability. When designed, the pavement must be able to withstand the expected applied traffic multiplied by a reliability design factor, F_R ($F_R \geq 1$). Mathematically speaking, predicted performance may be expressed as,

$$W_t = F_R \times W_T$$

Or, $\log W_t = \log F_R + \log w_T$

Where W_t is the predicted number of ESALs the structure can withstand, F_R is the reliability design factor, and w_T is the predicted ESALs that will be applied to the pavement for the design period.

It is the "reliability design factor" that accounts for the sources of uncertainty in the AASHTO procedure, which is often referred to as the "positive spacing factor" between $\log W_t$ and $\log w_T$,

$$\log F_R = (\log W_t - \log w_T) \geq 0$$

F_R is the only probability component that the designer "controls" by selecting a design level of reliability. Basically, F_R provides some probabilistic assurance that the actual pavement performance exceeds the actual design period traffic. The other sources of deviation (±δ) in the design procedure are,

i. *Prediction error in the design period traffic*

$(\log w_T - \log N_T) = \pm\delta\ (N_T, w_T)$

ii. *Prediction error in pavement performance*

$(\log N_t - \log W_t) = \pm\delta\ (W_t, N_t)$

This is the difference between the actual number of ESALs and the predicted ESALs the structure can withstand before reaching terminal serviceability.

iii. *Overall deviation of actual section performance from actual design period traffic, expressed as the algebraic and geometric sum of the above equations*

$(\log N_t - \log N_T) = \pm\ \delta_o$

This implies the difference between the actual numbers of ESALs the pavement can withstand before reaching terminal serviceability and the actual number of ESALs applied to the pavement structure.

<u>*Figure 2.2*</u> illustrates the above deviations in terms of present serviceability index.

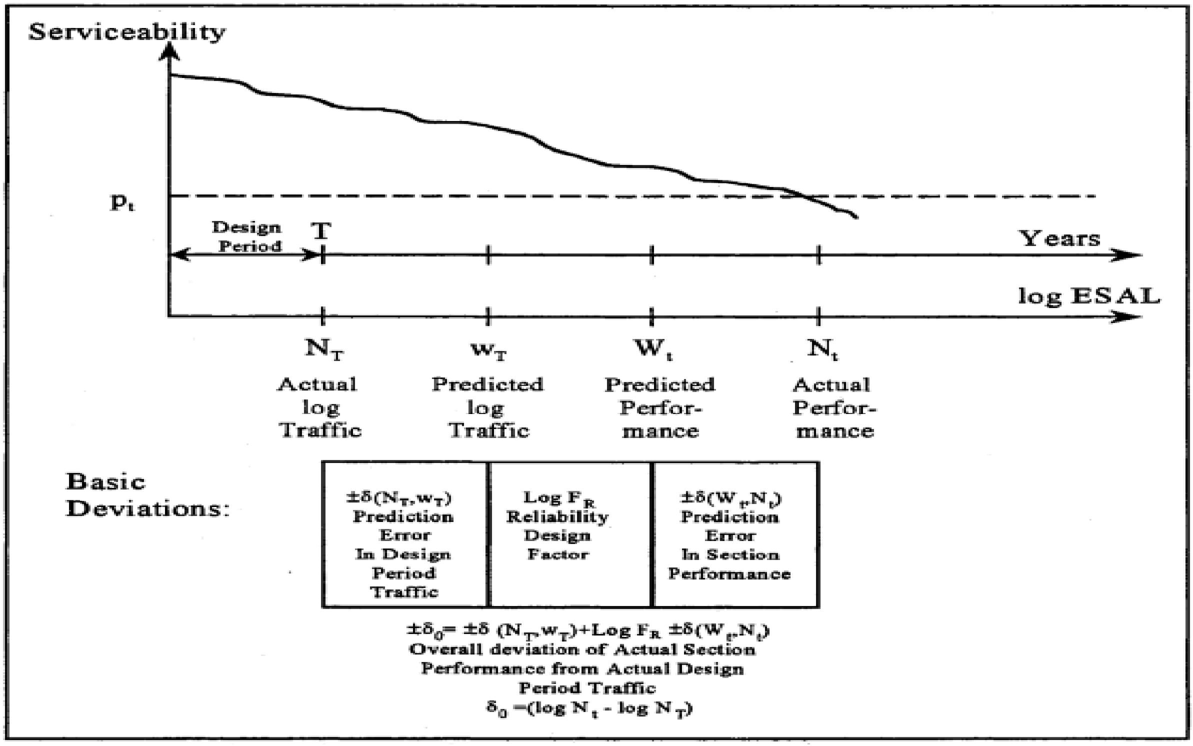


Figure 2.2 – Basic Deviations of AASHTO Reliability

AASHTO assumes that the set of all possible outcomes for each of the chance deviations would produce a normal probability distribution. Therefore the overall deviation, δ_0, would also have a normal probability distribution with a mean as the sum of the three deviate means and variance as the sum of the three deviate variances. Furthermore, assuming no bias in the prediction procedure, the set of all possible chance deviations will have an average value of zero.

$$\overline{\delta_o} = \bar{\delta}\,(N_T, w_t) + logF_R + \bar{\delta}\,(W_t, N_t)$$

Or, $\overline{\delta_o} = 0 + logF_R + 0 = logF_R$

The variance of the above distribution is then the sum of the squares of the variances from each source of chance variation. Since the reliability factor, F_R, is not random, its variance is zero. Mathematically,

$$S_o^2 = S_w^2 + 0 + S_N^2$$

Thus a normal random variable (δ_0) has been defined. This variable is a performance predictor that encompasses the error in predicting the design period traffic, the reliability design factor, and the error in predicting pavement performance. The mean of the distribution is the log of the reliability factor, while the variance is the sum of the squares of traffic prediction variance and pavement performance prediction variance. The area of concern associated with this distribution is when δ_0 equals zero. At this point the number of actual ESALs applied equals the number of allowable ESALs and failure is imminent. *Figure 2.3* illustrates the above concepts.

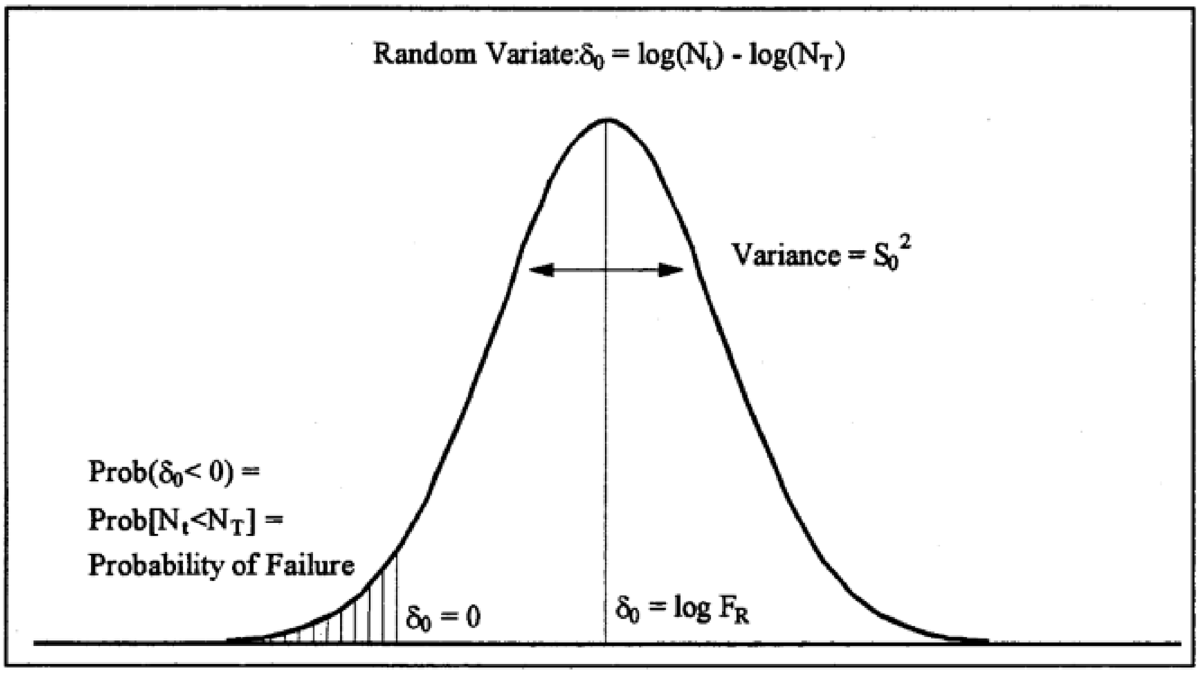


Figure 2.3 – Definition of AASHTO Reliability

The final steps in the AASHTO reliability method involve the transformation of δ_o to the standard unit normal variable. The equations below outline the process.

$$Z = \frac{\delta_o - \overline{\delta_o}}{S_0} = \frac{\delta_0 - logF_R}{S_0}$$

Taking δ_0 at zero, the point of failure, Equation becomes,

$$Z_R = \frac{-logF_R}{S_0}$$

$$F_R = 10^{-Z_R \times S_0}$$

Where Z_R may be obtained from a standard normal table for a specified level of reliability, and F_R is incorporated into the design equations as a positive spacing factor. Therefore, accurate predictions of reliability depend primarily on the selection of S_o. AASHTO provides a range of values and an appendix to help select S_o, while it is emphasized that the designer should develop an S_o to suit the given design conditions. Consequently, the problem involves determining the variability of the associated input parameters followed by a statistical exercise to determine the design reliability.

2.4.2. Levels of Reliability in the AASHTO Guide:

The design monograph in the *AASHTO Guide (1993)* allows the designer to select a level of reliability. However, a decision must be made regarding the appropriate level of reliability for a given design scenario. The AASHTO guide recognizes that different classifications of roads require different levels of reliability and states,

"Generally, as the volume of traffic, difficulty of diverting traffic, and public expectation of availability increases, the risk of not performing to expectations must be minimized. This is accomplished by selecting higher levels of reliability."

The recommended levels of reliability, shown in *Table 2.1*, are the result of a survey of the AASHTO Pavement Design Task force. The wide range of recommended design reliability suggests that there is currently some debate regarding this matter and that the design engineer's judgement must be employed on a case-by-case basis.

Table 2.1 – Suggested levels of Reliability for various functional classifications

Functional Classification	*Recommended Level of Reliability*	
	Urban	*Rural*
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

2.5. Design Input Parameters:

In order to incorporate reliability into the IRC-37:2012 based design process, the relevant input parameters needed to be statistically characterized. The following subsections discuss information found in the literature regarding the variability of *layer modulus*, *Poisson's ratio*, *layer thickness*, and *load characteristics*.

2.5.1. Layer Modulus/ Resilient Modulus:

Resilient Modulus is the stiffness of a material that may be defined, in the strictest sense, as the slope of the stress-strain curve that results when either load or displacement are applied to the material in its elastic range. The materials of concern for flexible pavement design are asphalt concrete and granular base. But in this paper we have considered only the upper-most layer, i.e. Asphalt concrete or bituminous layer as our concern because variation of temperature is maximum at the upper-most layer.

Asphalt Concrete:

Although much research has been devoted to the determination of asphalt concrete resilient modulus, less has been focused upon measuring the variability of this parameter.

Even *IRC 37:2012* states Resilient modulus is the measure of its elastic behaviour determined from recoverable deformation in the laboratory tests. It gives some empirical relationship of Resilient Modulus with its effective CBR value, but fails to give a clear idea about the variability of the modulus with the temperature.

Several studies evaluated the variation associated with measured resilient modulus (MR) by diametral testing. *Al-Sugair* and *Almudaiheem (1992)* tested laboratory-prepared asphalt concrete samples at different load levels, moisture conditions (wet or dry) and mix designs. They found the coefficient of variation to range between 7% and 16%, although they recommended using between 10% and 20% in practice. *Brown* and *Foo (1991)* conducted a similar study on asphalt concrete using different mix designs and found the COV to range between 6% and 10%.

Another study *(Stroup-Gardiner Mary and Newcomb David E. (1997)* also investigated the variability of asphalt concrete resilient modulus as tested by ASTM D4123. Asphalt materials from a sample highway, including mix design samples, specimens obtained from behind-the-paver, and cores taken from the in place pavement were tested at - 18^0C, 1°C, 25^0C and 40^0C. Although the COV did not vary significantly between the three types of samples, it did vary with temperature as shown in the second column of *Table 2.2*. An important finding of this research was that a log transformation of the data was necessary to achieve a normally distributed data set. In other words, *Stroup-Gardiner* and *Newcomb* suggested that the resilient modulus of asphalt concrete be treated as a lognormal random variable. Their final report *(1997)* contained statistical information of the un-transformed modulus data (*Table 2.2, column 3*). It should be pointed out that the values in the third column represented different mix designs from column two. Additionally, the range of the COV for column three (un-transformed data) was 5% to 15%. As in India, the temperature variation is quite large, the variation of

Resilient Modulus with temperature is far more important than the variation with moisture contents.

Table 2.2 – COV of Asphalt Concrete Modulus with temperature

Temperature (°C)	*COV (Log-Transformed)*	*COV (Un-transformed)*
-18	3.5%	11.2%
1	2.0%	8.9%
25	2.0%	10.6%
40	6.0%	Unable to test

2.5.2. Poisson's Ratio:

Poisson's ratio (ϑ) is the ratio of transverse strain (ϵ_t) to axial strain (ϵ_a) when a material is axially loaded.

$$\vartheta = \frac{\epsilon_t}{\epsilon_a}$$

Although Poisson's ratio is likely a random variable that potentially could be described by a particular distribution, it has traditionally been difficult to measure and the values have usually been assumed. *Yoder* and *Witczak (1975)* cited that, for most pavement materials, the influence of many factors on ϑ is generally small. Although for asphalt concrete they do report a change in ϑ with temperature, where ϑ varies between 0.25 at cooler temperatures (4^0C) to 0.5 at warmer temperatures (60^0C), with a typical value of 0.35.

IRC 37:2012 does not state anything about the relation of fatigue life with the Poisson's Ratio. So in this case we will neglect the variation of Poisson's Ratio in our design methodology.

2.5.3. Layer Thickness:

The purpose of flexible pavement design is to determine the thickness of each pavement layer to withstand the traffic and environmental conditions during the design period. Ideally, the design thickness would be a deterministic parameter, but construction inherently causes layer thickness to be variable.

As construction conditions and procedures cannot be always controlled, the layer thickness can vary randomly. So it is hard to find out the exact varying behaviour, though when Ground-penetrating radar (GPR) was used in *Kansas (1994)* to determine the overall structural thickness of pavements, the thickness was found to be normally distributed for a major part of the data.

2.5.4. Load Characteristics:

In a broad sense, the type of traffic applied to the pavement structure defines load characteristics. The traffic, in turn, is comprised of a wide array of vehicle types which designers must somehow translate into values suitable for design. In past empirically based thickness design methods the concept of load equivalency was used to quantify the load effects. For example, the damage done by one type of axle would be converted into an equivalent number of standard axles to cause the same amount of damage. This approach becomes inaccurate when the loading configurations or material characteristics differ from those used to establish the empirical database.

However *IRC 37:2012* based design introduces a Vehicle Damage Factor which is a multiplier to convert the number of commercial vehicles of different axle loads and axle configuration into the number of repetitions of standard axle load of magnitude 80 kN. The VDF varies with the vehicle axle configuration and axle loading. Finally design traffic is evaluated in terms of cumulative number of standard axles which should be the initial input for a design methodology.

2.6. Summary of Literature Review:

Some simple examples of literatures of how the variability of the inputs can be related to the variability of the output, and ultimately Reliability, were depicted above. In those studies, the predicted pavement life is a function of the pavement strain, which is the function of the bituminous stiffness, which varies with the temperature. Since the bituminous temperature is variable, it follows that the pavement life would also exhibit variability. The reliability may then be interpreted as the probability of the pavement structure exceeding some level of predicted pavement life. In actual pavement design, there are many more input parameters to consider which all contribute to the stochastic nature of the design methodology. Therefore, the way of incorporation of reliability in flexible pavement design, which is the key focus of this research project, can be predicted from the above literature studies.

CHAPTER – 3

OBJECTIVE AND SCOPE OF WORK

3.1. Objective:

The main objective of this research was to develop a rational method of incorporating reliability into the existing Empirical pavement thickness design framework for India and assessing the effects of the design parameters and their statistical variability on pavement design reliability along with a critical review on the method proposed on IRC 37:2012.

3.2. Scope of Work:

Few major scopes of our project paper are as follows,

1. A literature review was conducted to investigate issues in reliability-based pavement design. Information was obtained regarding Monte Carlo simulation and other reliability analysis techniques, pseudo-random number generation and statistical characterization of the relevant design variables.
2. Our empirical method of design was enhanced to incorporate Monte Carlo simulation in the design process to formulate a reliability model.
3. For the purpose of this project, it is important to find out the statistical variation pattern of various factors like layer thickness and Resilient Modulus.
4. This allowed for an in-depth sensitivity analysis regarding the relative stochastic effects of each input parameter's variability on the design reliability.

CHAPTER – 4

DESIGN METHODOLOGY

4.1. Conventional Method of Design Based on IRC 37:2012:

Flexible Pavements are widely used despite some doubts regarding their economics under different conditions. Two most important parameters that govern the pavement design are soil sub-grade and traffic loading. The Indian guidelines for the design of flexible pavements use soil sub-grade strength in terms of California Bearing Ratio and traffic loading in terms of million standard axles (msa).

4.1.1. Empirical design of IRC 37:2012:

The recommended method considers design traffic of the cumulative number of standard axles used as axle load spectrum for heavy traffic. The following parameters are required to calculate the load spectrum.

i. Initial traffic after construction in terms of number of commercial vehicles per day (CVPD) -

Assessment of the present day average traffic should be based on seven-day-24-hour count made in accordance with IRC 9-1972.

ii. Traffic growth rate during the design life in percentage –

Traffic forecasting is done for next 15 or 20 years, which one is the design period of the pavement. Required traffic growth rate based on past studies is essential, however if the data for the annual growth rate of commercial vehicles is not available or if it is less than 5 per cent, a growth rate of 5 per cent should be used (IRC: SP 84-2009) in calculation.

Traffic prediction, $P_n = P_0 (1+r)^n$

Where, P_n = Traffic in n^{th} year.

P_0 = Traffic flow in the base year

n = Number of years

r = Traffic growth rate

iii. Design life in number of years –

It is recommended that pavements for National Highways and State Highways should be designed for a minimum life of 15 years. Expressways and Urban Roads may be designed for a longer life of 20 years or higher using innovative design adopting high fatigue bituminous mixes. In the light of experience in India and abroad, very high volume roads with design traffic greater than 200 msa and perpetual pavements can also be designed. For other categories of roads, a design life of 10 to 15 years may be adopted.

iv. Spectrum of axle loads and Vehicle Damage Factor (VDF) –

VDF is arrived carefully by carrying out specific axle load surveys on the existing roads. Minimum sample size for survey is taken 10 per cent for commercial vehicles per day more than 6000. Each direction can have different pavement thickness for a divided highway which is depend upon loading pattern. VDF is evaluated direction wise since on some sections, there may be significant difference in axle loading in two directions of traffic.

For rolling or plain terrain VDF may adopt 4.5 since the commercial vehicles per day are more than 1500.

v. Distribution of commercial traffic over the carriageway –

Distribution of commercial traffic in each direction and in each lane is required for determining the total equivalent standard axle load applications to be considered in the design. For dual carriage roads; the design of dual three-lane carriageway 60 per cent distribution of commercial traffic over the carriageway is adopted.

Finally CVPD, VDF, growth rate, design life are given as inputs in order to get the cumulative million standards axles is stated below,

$$N = \frac{365 \times [(1+r)^n - 1] \times A \times D \times F}{r}$$

N = Cumulative number of standard axles to be catered for in the design in terms of msa.

A = Initial traffic in the year of completion of construction in terms of the number of Commercial Vehicles per Day (CVPD).

D = Lane distribution factor (as explained in para 4.5.1 of IRC 37:2012).

F = Vehicle Damage Factor.

n = Design life in years.

r = Annual growth rate of commercial vehicles in decimal.

4.1.2. Pavement Composition:

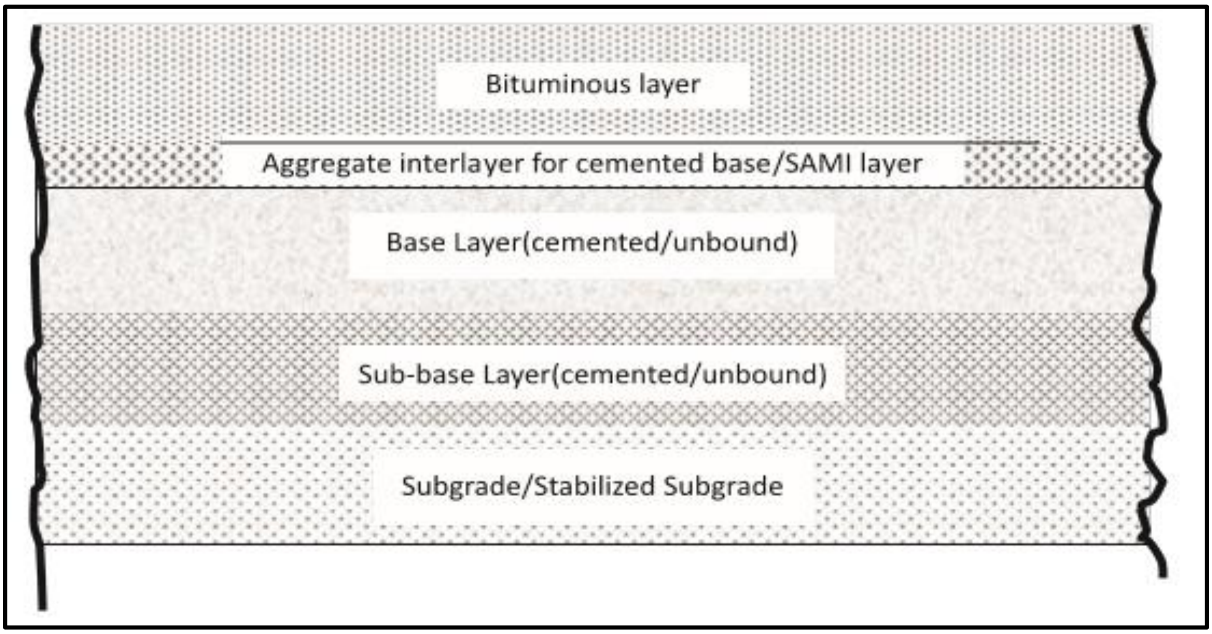


Figure 4.1 – Different layers of Bituminous Flexible Pavement

4.1.3. Different Design Procedures introduced in IRC 37: 2012:

Previous versions of IRC 37 were a bit rigid in its design procedure of flexible pavement as only some design catalogues, based on empirical formulas, were available depending on the different CBR values and the load spectrum. But IRC 37:2012 suggests the following methods of design which are a bit flexible and trustworthy, though needs to be full proofed.

i. <u>Using IITPAVE</u>:

Any combination of traffic and pavement layer composition can be tried using IITPAVE. The designer will have full freedom in the choice of pavement materials and layer thickness. The traffic volume, number of layers, the layer thickness of individual layers and the layer properties are the user specified inputs in the Program, which gives strains at critical locations as outputs. The adequacy of design is checked by the Program by comparing these strains with the allowable strains as predicted by the fatigue and rutting models, in-built in the Program. A satisfactory pavement design is achieved through iterative process by varying layer thicknesses or, if necessary, by changing the pavement layer materials.

ii. <u>Using Design Catalogues</u>:

These Guidelines provide a Design catalogue giving pavement compositions for various combinations of traffic, layer configuration and assumed material properties. If the designer chooses to use any of these combinations and is satisfied that the layer properties assumed in the design catalogue can be achieved in the field, the design can be straightway adopted from the relevant design charts given in the catalogue.

iii. <u>Material Properties</u>:

Regardless of the design procedure, it is essential that the material properties are adopted only after conducting relevant tests on the materials. Where all test facilities are not available, at least those tests must be carried out, which can validate the assumed design properties. In Annex XI of IRC 37:2012, the type of tests required as well as the range of values for material properties are given based on typical testing and experience in other countries. The values as suggested may be adopted for pavement design as default but not without validation by subjecting the materials to such tests which can be easily carried out in any laboratory to validate the assumed design values.

4.2. Incorporation of Reliability Concept into the Conventional Method of Design:

The challenge of incorporating reliability into mechanistic-empirical (M-E) design was handled in several phases. The first phase involved gathering information regarding the design input parameters and associated variability. These values formed the basis of design inputs for the mechanistic load-displacement model. The second phase required the development of a computer program that included Monte Carlo simulation, a mechanistic load-displacement model, and reliability analysis. The next phase involved a sensitivity analysis to determine the number of required Monte Carlo cycles and to develop a better understanding of the input's effect on output reliability. Finally, example designs were performed to draw comparisons between the reliability based M-E design procedure and current empirical methods.

4.2.1. Phase 1: Input data characterisation:

As discussed in the previous chapter, four input parameters and their variability play a major role in designing the flexible pavement.

Layer Modulus:

The resilient modulus of all the materials (asphalt concrete, granular base, sub-grade) was taken to be log-normally distributed. This conclusion was taken directly from data obtained through the literature review. Since the input variability for a particular design would depend upon those specific conditions, this report was intended only to quantify the range of practical variability for each of the materials. This was done by first identifying the characteristic shape of the distribution followed by establishing a practical range of coefficient of variation (COV). COV may be defined as the ratio of the standard deviation to the mean value.

In India, temperature variation plays a major role in most of the parts. Resilient Modulus of bituminous layer varies drastically with temperature as stated in IRC 37:2012 (Shown in *Table 4.1*). As temperature variation affects the uppermost layer of the pavement to its maximum, only resilient modulus of bituminous layer is taken into account for this project.

Mix type	Temperature °C				
	20	25	30	35	40
BC and DBM for VG10 bitumen	2300	2000	1450	1000	800
BC and DBM for VG30 bitumen	3500	3000	2500	1700	1250
BC and DBM for VG40 bitumen	6000	5000	4000	3000	2000
BC and DBM for Modified Bitumen (IRC: SP: 53-2010)	5700	3800	2400	1650	1300
BM with VG 10 bitumen	500 MPa at 35°C				
BM with VG 30 bitumen	700 MPa at 35°C				
WMM/RAP treated with 3 per cent bitumen emulsion/ foamed bitumen (2 per cent residual bitumen and 1 per cent cementatious material).	600 MPa at 35°C (laboratory values vary from 700 to 1200 MPa for water saturated samples).				

Table 4.1 – Resilient Modulus of Bituminous mixes

Information regarding bituminous moduli and variability was taken directly from the literature review. Based upon the synthesis of information, practical range of modulus COV is 10% to 40%. But in this report, baseline variability is being taken as 30%.

Poisson's Ratio:

As discovered in the literature review, the influence of many factors on Poisson's ratio is generally small. Consequently, it was decided to fix the Poisson's ratios for the three general types of materials to be consistent with that found in the literature review. So, no variability of Poisson's Ratio is considered in this report.

Usually Poisson's Ratios are taken as following,

Asphalt Concrete – 0.35

Base (Granular Soil) – 0.40

Subgrade (Cohesive Soil) – 0.45

Pavement Layer thickness:

As discussed in literature review, pavement layer thickness cannot be kept exactly the same as design thickness during construction. Some of the literature studies state thickness is normally distributed.

In this report, baseline coefficient of variation (COV) of bituminous layer and Base layer are taken as 5% and 15% respectively, at the same time we checked the result by replacing COV by the absolute value of variation of the design thickness (e.g. ± 5 mm, ± 10 mm, ± 15 mm and ± 20 mm).

4.2.2. Phase 2: Monte Carlo Simulation and Reliability Formulation:

We already developed a computer program which determines the required thickness for a load spectrum and CBR value of the soil sample based on the design catalogues given in IRC 37:2012. But the process was mechanical-empirical based design, so it was only necessary to incorporate Monte Carlo simulation into the existing software. The following subsections discuss the technical details of Monte Carlo simulation and the explicit formulation of reliability in this project.

Monte Carlo Simulation:

The process used in this research for utilizing the probability distributions of the input design parameters to determine the distributions of pavement lives was Monte Carlo simulation. As described by *Hart* (1982) and referenced by *Galambos* (1989), Monte Carlo simulation is a straightforward method of randomly combining each of a function's input variables and producing a distribution of output. The general process follows,

1. Define "x" as a random variable with some known cumulative distribution function F_x = Probability ($X \leq x$) = P ($X \leq x$).
2. Define "u" as a standard uniform variate with cumulative distribution function, F_u. By the definition of a standard uniform variate, F_u = Probability ($U \leq u$) = u.
3. A random number, u, is thus generated between 0 and 1.
4. By the definition in step 2, $F_u = u = F_x$.
5. Then x is found so that $F_x = P (X \leq x)$ is true for the value u.
6. The process of generating random numbers and finding the x-values is repeated for each input variable, until a set of input variables has been achieved.
7. The set of input variables is entered into the function.

The output of the function is stored and this constitutes one Monte Carlo cycle. The critical steps in the process (steps 3, 4, and 5) are illustrated in *Figure 4.2*. New sets of inputs are generated and run through the function until the required number of cycles has been achieved.

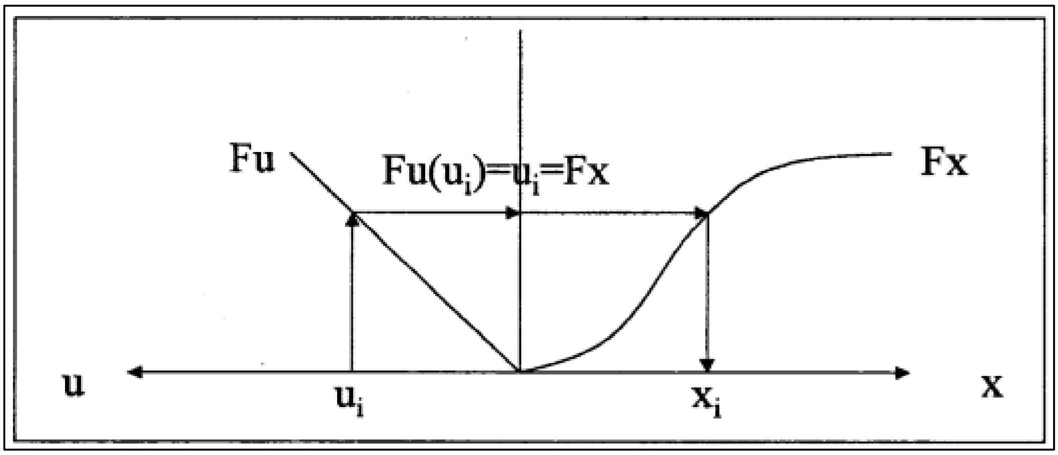


Figure 4.2 – Steps 3, 4 & 5 of Monte Carlo Simulation

When a distribution is characterized by a well-known function (e.g., normal or lognormal), as seen in the literature review regarding some of the variables, it is possible to work directly with equations to artificially generate the distribution. *Box* and *Muller (*1958) have shown that if U_1 and U_2 are two independent standard uniform variates, then

$$S_1 = \sqrt{-2\ln U_1} \times \cos 2\pi U_2 \qquad \textit{Equation 4.1}$$
$$S_2 = \sqrt{-2\ln U_1} \times \sin 2\pi U_2$$

are pair of statistically independent standard normal variates. Therefore, a pair of random numbers from a normal distribution (N (μ,σ)) may be obtained by,

$$x_1 = \mu + \sigma S_1 \qquad \textit{Equation 4.2}$$
$$x_2 = \mu + \sigma S_2$$

In other words, to generate a normally distributed random variable with some mean and standard deviation, two standard uniform random numbers are generated. The numbers are then transformed by *Equation 4.1* to standard normal values. The final step (*Equation 4.2*) uses the standard normal values and transforms them to the desired normal distribution. The benefit of *Equations 4.1 and 4.2* lies in the fact that two "x" values may be generated from two random numbers, which improves computing efficiency.

Lognormal random variables are generated in much the same fashion. For a log-normal variable (x) the following hold true when the mean (μ_x) and standard deviation (σ_x) are known.

Y = ln X

$$\sigma_y = \sqrt{Variance\ of\ Y} = \sqrt{\ln\left[\left(\frac{\sigma_x}{\mu_x}\right)^2 + 1\right]}$$ *Equation 4.3*

$$\mu_y = mean\ of\ Y = \ln \mu_x - \frac{\sigma_y^2}{2}$$

Therefore, the mean and standard deviation of ln (X) are first determined from the mean and standard deviation of X. The results from *Equation 4.1* then generate two standard normal values. Finally, two x values (log-normally distributed) are calculated by,

$$x_1 = e^{\mu_y + \sigma_y S_1}$$
$$x_2 = e^{\mu_y + \sigma_y S_2}$$ *Equation 4.4*

The above concept was incorporated into this project, but our main objective will be to formulate one single software which will incorporate this simulation and reliability concept in it in near future.

Reliability Formulation:

In this project, the definition of reliability is consistent with that proposed by *Kulkarni* (1994). Specifically, reliability is the probability that the number of allowable traffic loads exceeds the number of applied traffic loads. Mathematically speaking,

R = P [N > n] *Equation 4.5*

Where, N = number of allowable loads until either fatigue or rutting failure

n = number of load repetitions during life of pavement.

As previously discussed, the distribution of N may be determined from the mechanistic pavement model and transfer functions via Monte Carlo simulation. However, the traffic demand (n) is typically a more difficult number to quantify and relies primarily upon established traffic forecasting procedures. Therefore, for the purposes of this project, "n" was considered to be a deterministic design parameter. In other words, the value n was taken to be some pre-determined service life that the pavement must accommodate. *Equation 4.5* then quantifies the probability that the pavement structure will exceed the demand. *Figure 4.3* is a graphical representation of *Equation 4.5* where N is stochastic parameter and n is a deterministic parameter. *Figure 4.4* illustrates the reliability-based design procedure.

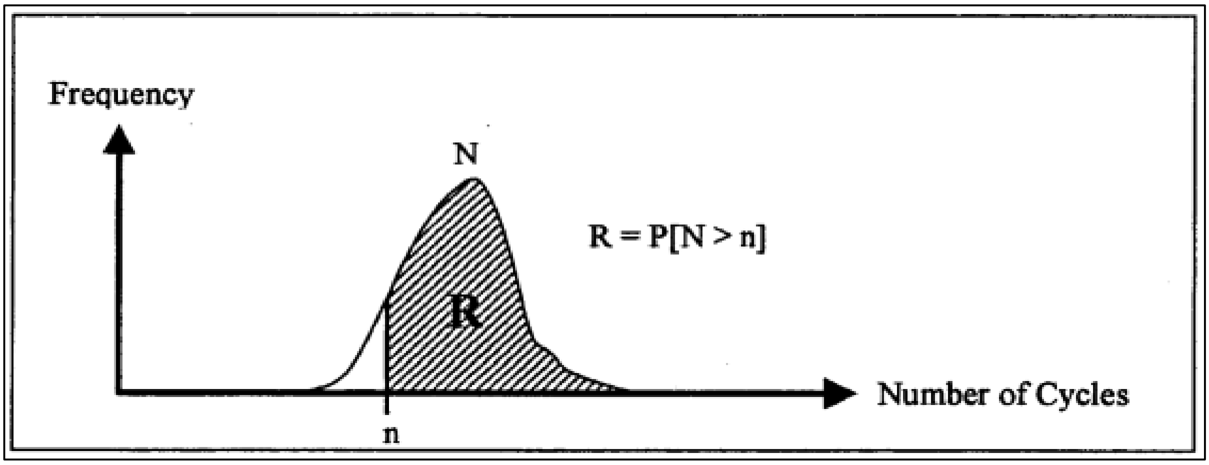


Figure 4.3 – Reliability where "N" is Stochastic and "n" is Deterministic

The definition of reliability is relatively straightforward. If the distribution of N pictured in *Figure 4.3*, were always described by the same characteristic function (e.g. normal curve) then the integration required to calculate the reliability would also be straightforward. If, however, the shape of the distribution of N were unpredictable then a numerical integration scheme would probably need to be employed.

4.2.3. Phase 3: Incorporation of Fatigue Life model into IRC 37:2012:

With every load repetition, the tensile strain developed at the bottom of the bituminous layer develops micro cracks, which go on widening and expanding till the load repetitions are large enough for the cracks to propagate to the surface over an area of the surface that is unacceptable from the point of view of long term serviceability of the pavement. The phenomenon is called fatigue of the bituminous layer and the number of load repetitions in terms of standard axles that cause fatigue denotes the fatigue life of the pavement.

Fatigue model has been calibrated in the R-56 (54) studies using the pavement performance data collected during the R-6 (57) and R-19 (58) studies sponsored by MORTH. The general fatigue equation for the conventional bituminous mixes designed by Marshall Method are given below,

$$N_f = 0.5161 \times C \times 10^{-4} \times \left(1/\epsilon_t\right)^{3.89} \times \left(1/M_R\right)^{0.854} \qquad \text{\underline{\textit{Equation 4.6}}}$$

N_f = Fatigue life in number of standard axles

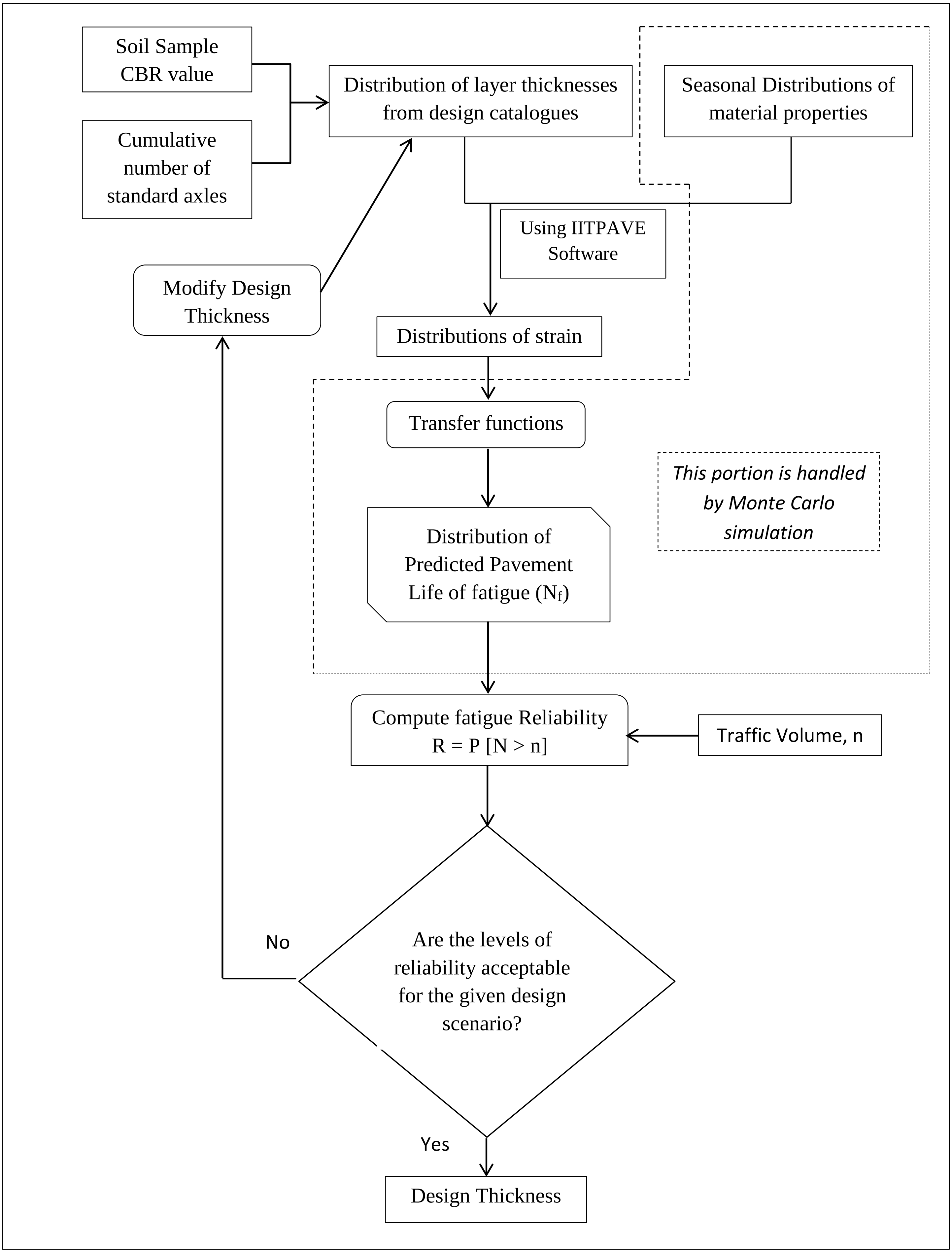
ϵ_t = Maximum Tensile strain at the bottom of the bituminous layer, and

M_R = Resilient modulus of the bituminous layer

$C = 10^M$, Where, $M = 4.84\left(\frac{V_b}{V_a+V_b} - 0.69\right)$

Many well-known fatigue models also include the above approach to take into account the effect of volume of bitumen and air voids in the bituminous mix. *Equation 4.6* would demonstrate that slight changes in volume air voids (V_a) and volume of bitumen (V_b) will have huge impact on the fatigue life. For example, if bitumen content is increased by 0.5 per cent to 0.6 per cent above the optimum bitumen content given by Marshall test and air void is reduced to the minimum acceptable level of 3 per cent and volume of bitumen increased to the level of 13 per cent, the fatigue life would be increased by about three times. The recommendation in these guidelines is to target low air voids and higher bitumen constant for the lower layer to obtain fatigue resistant mix.

Figure 4.4 – Reliability Based Design Procedure



CHAPTER – 5

RESULTS AND DISCUSSIONS

5.1. General Procedure of total analytical design:

As described in the previous chapter, each of the parameters associated with M-E design is stochastic in nature. Consequently, it is imperative that reliability analysis is incorporated into the design procedure. While the means for doing so were presented in Chapter 4, it is important for the designer to understand the interaction between input variability and output reliability. Additionally, the output distribution must be characterized so that reliability may be quantified. So, the detailed results and procedures are listed below,

Step 1: We are taking three different cumulative standard axles load values in our design, i.e. 50 msa, 100 msa and 150 msa.

Step 2: Assuming CBR value of 8%, the design thicknesses are calculated from the Design Catalogues of IRC 37:2012 (In this case, Plate 6 Catalogue).

Step 3: Thicknesses which are found from the catalogues, are as follows,

For 50 msa traffic,	*For 100 msa traffic,*	*For 150 msa traffic,*
BC = 40 mm	BC = 50 mm	BC = 50 mm
DBM = 100 mm	DBM = 115 mm	DBM = 135 mm
Granular Base = 250 mm	Granular Base = 250 mm	Granular Base = 250 mm
Granular Sub base = 200 mm	Granular Sub Base = 200 mm	Granular Sub Base = 200 mm

[Total Bituminous layer = (BC + DBM) thickness]

Step 4: The Stress analysis software IITPAVE has been used for the computation of stresses and strains in flexible pavements. Tensile strain, ϵ_t, at the bottom of the bituminous layer is considered as critical parameter for pavement design to limit cracking due to fatigue. For the simplicity of the project, we are considering an empirical relation of tensile strain with the thicknesses of the layers, given by *E.O.Ekwulo* and *D.B.Eme (2013)*, which is as follows,

$$\epsilon_t = -A \ln(T) + B \qquad \text{Equation 5.1}$$

Where, A and B = Constants, depending upon the load repetitions and CBR value.

[Here, a suitable value of A and B are chosen from the reference of the paper by E.O. Ekwulo and D.B.Eme, which is as follows,

A= 52.35 and B = 435.32]

T = Total thickness of Bituminous layers and Granular Base layer.

[For Example, for 50 msa traffic,

Bituminous layer thickness = 40 +100 = 140 mm

Granular Base thickness = 250 mm

Then T = (140 + 250) mm = 290 mm.]

Step 5: Resilient Modulus of different grades of Bitumen varies with temperature drastically, which is given in *Table 4.1*. As discussed earlier in this paper, temperature variation effect is maximum in the top most layer of the pavement, i.e. in the bituminous layer. So we will consider the variability of only bituminous layer with temperature variation. As discussed in Literature Review chapter, Resilient modulus of bitumen behaves like a Log-Normal variable. The baseline variability (in terms of COV) of the Resilient modulus is assumed in this paper is 30%.

Step 6: Pavement layer thicknesses vary during construction stage. Once casted, thickness does not vary significantly with temperature variation and other parameters. So, here we assumed some absolute values of variation of thickness for this project, i.e. ± 5mm, ±10 mm, ± 15mm and ± 20mm.

Step 7: As discussed in previous chapter, IRC 37:2012 has introduced a fatigue model equation (*Equation 4.6*) which takes bituminous mix design in account. The fatigue model suggested in the code is as follows,

$$N_f = 0.5161 \times C \times 10^{-4} \times \left(1/\epsilon_t\right)^{3.89} \times \left(1/M_R\right)^{0.854}$$

Where, C depends on the mix design of bitumen. The expression for C is given below,

$C = 10^M$

Where, $M = 4.84\left(\frac{V_b}{V_a+V_b} - 0.69\right)$

In our design mix, volume of air voids (V_a) is taken as 4% and volume of bitumen (V_b) is taken as 11% which give the value of "M" as 0.2097 and thus "C" becomes 1.6208.

Step 8: The above expression clearly shows fatigue life (in axles) depends on several factors, like mix design, tensile strain and also Resilient Modulus. Among these factors, mix design is done once before pavement design. So, C values remain constant throughout the life. But thickness is supposed to be a normal variable, which will lead to change the ϵ_t value, therefor N_f value.

Step 9: One of the main objective is to determine a reasonable number of Monte Carlo cycles the designer should use. It was found that the distribution converged after approximately 5,000 Monte Carlo cycles. But for better result we have used 7,500 cycles in this report.

Step 9: By applying Monte Carlo Simulation, we are now able to find out the different N_f values by changing its dependent variables within the limits which were assumed earlier in this chapter. By the theory of Reliability,

$R = P\,[N_f \geq N]$, where N is the design life (expressed in standard axles).

So, we can find out Reliability of a particular pavement for different temperature conditions, allowing different limits of thickness.

5.2. Outcomes of different Case Studies:

5.2.1. Variation of Reliability with change in Layer thickness:

Figure 5.1- Variation of Reliability with temperature and thickness for 50 MSA

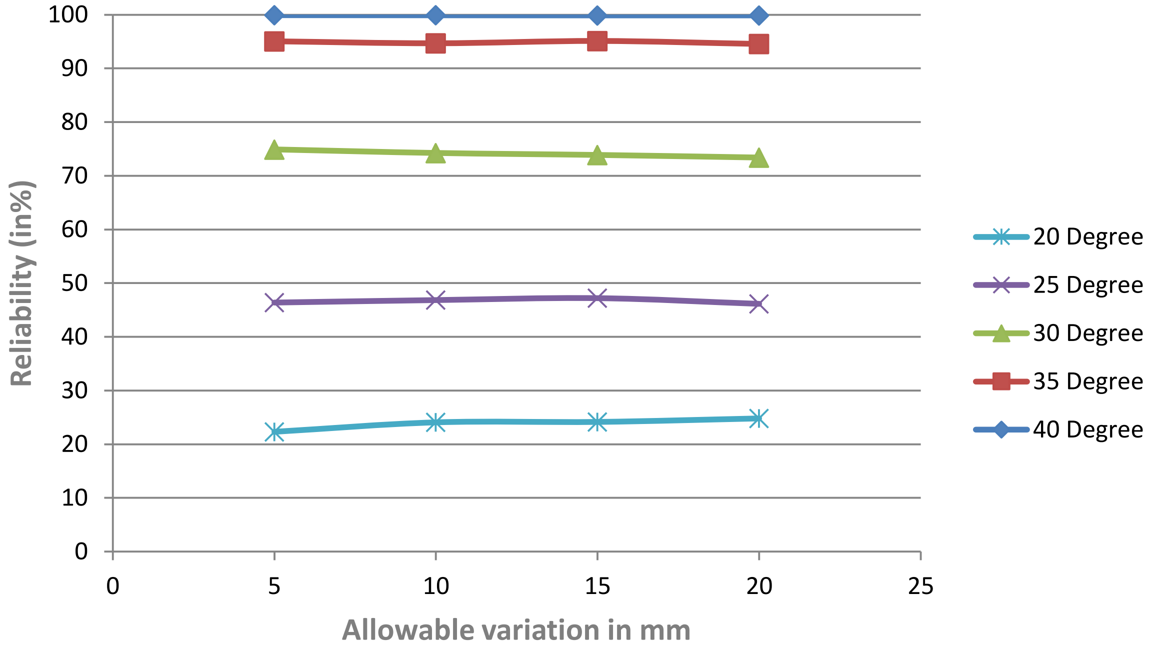


Figure 5.2- Variation of Reliability with temperature and thickness for 100 MSA

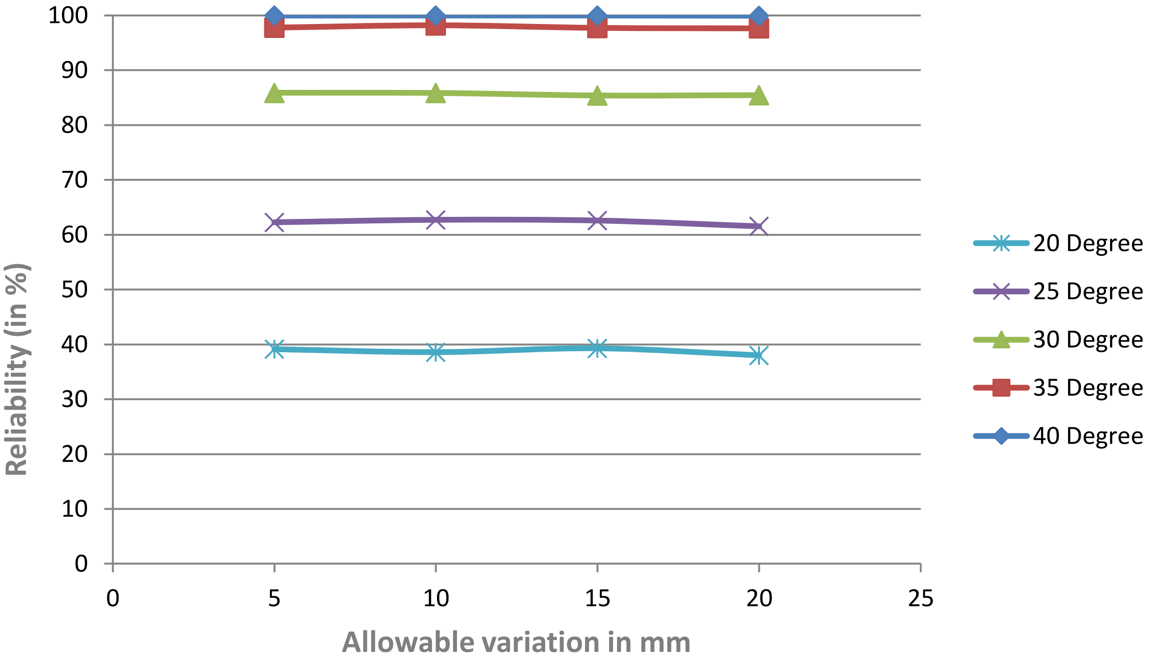
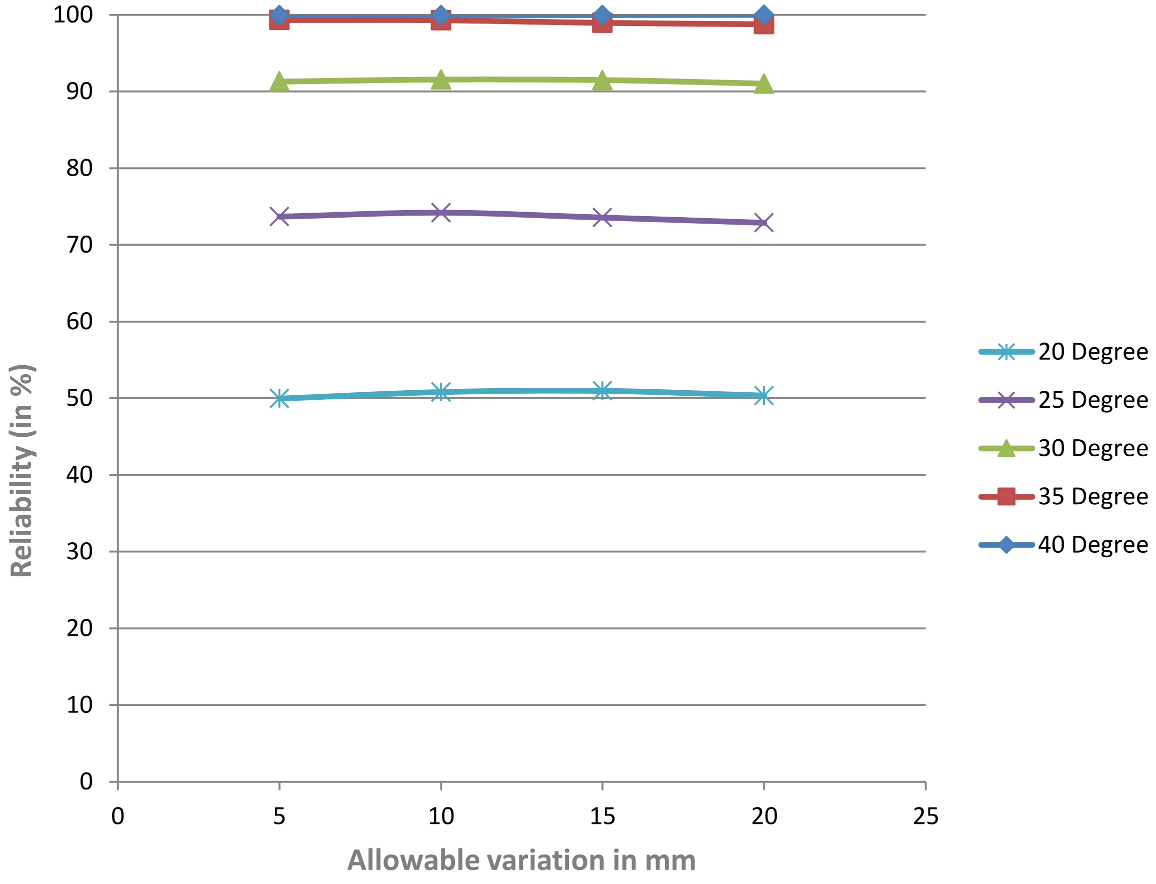


Figure 5.3- Variation of Reliability with temperature and thickness for 150 MSA



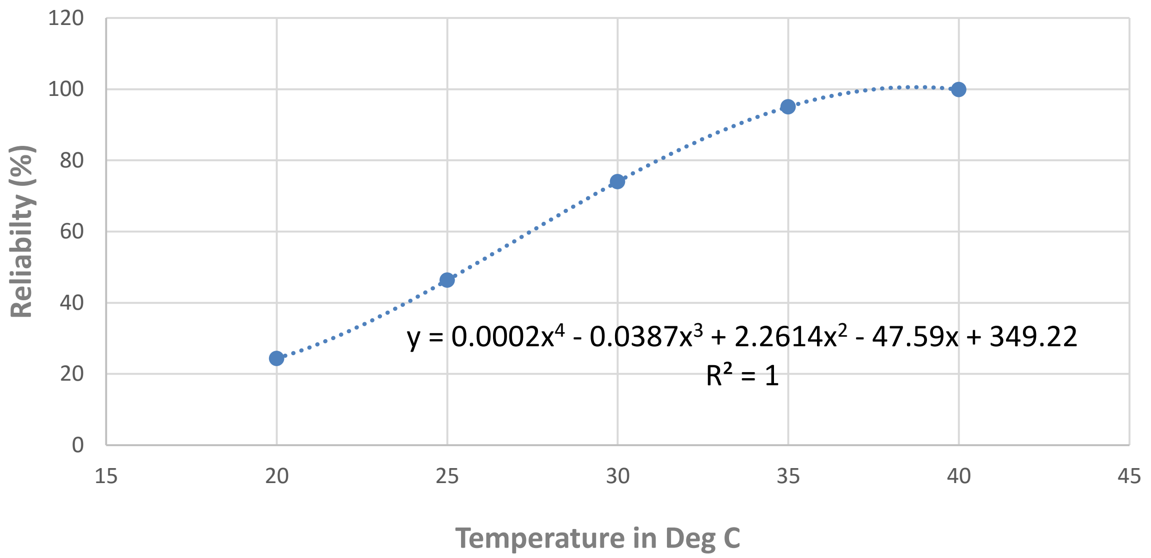
From the above three figures, it is clear that variation of thickness even up to 20 mm does not affect Reliability much. But, temperature variation makes a vast difference in Reliability values. So, the temperature criterion is much more important parameter to be considered than the thickness variation in the pavement design.

5.2.2. Variation of Reliability during Seasonal variation:

We have shown that seasonal variation is more significant factor than any others. We have categorised this variation into two parts- (1) When the error in thickness during construction is ± 5 mm and (2) when the error is ± 20 mm. As Reliability does not alter much with thickness varying even up to 20 mm, neglecting ± 10 mm and ± 15 mm error in thickness will not distract in finding the pattern of variation.

5.2.2.1. While Allowable Error in thickness is ± 5 mm:

Figure 5.4- Variation of Reliability with Temperature for variation in thickness as ± 5 mm for 50 MSA



From the above figure, it is clear that Reliability vary drastically with temperature with a polynomial relation of order 4. For 20^0C temperature a pavement with 50 msa axle spectrum is showing a Reliability of approximate 24.12% (approximate term is used as it can change a little for a next set of data cycles in Monte Carlo Simulation), whether the same pavement displays a Reliability of approximate 74.91% in 30^0C.

The same pattern is revealed when the design axle load spectrum is 100 msa and 150 msa (shown in <u>*Figure 5.5*</u> *and* <u>*Figure 5.6*</u>).

Figure 5.5 displays a clear picture of Reliability variation for 100 msa load spectrum, where Reliability comes approximately 39.13%, 62.27%, 85.89% and 97.77% for temperatures of 20^0C, 25^0C, 30^0C and 35^0C.

Figure 5.5- Variation of Reliability with Temperature for variation in thickness as ± 5 mm for 100 MSA

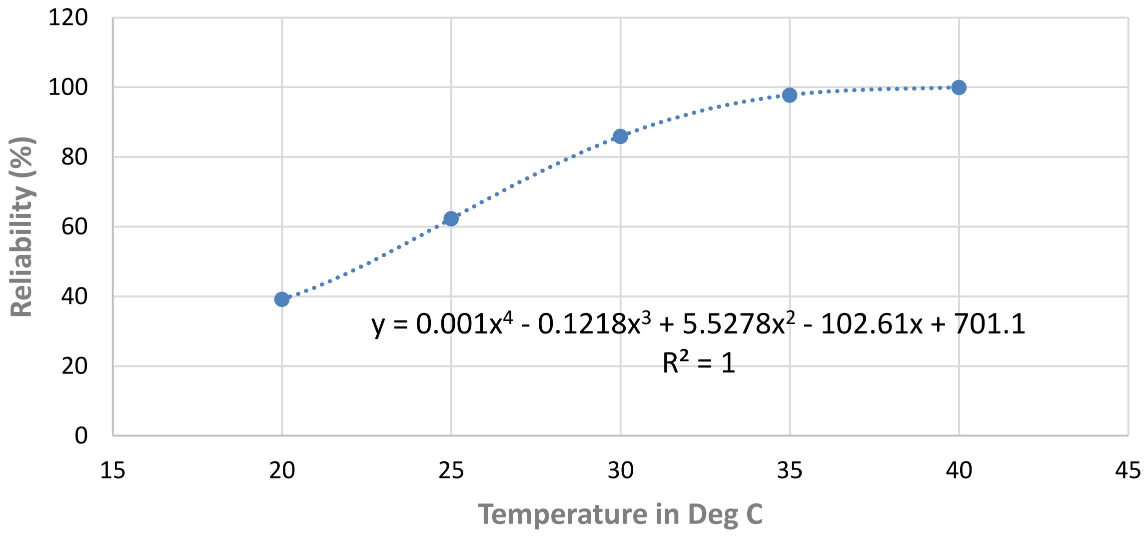
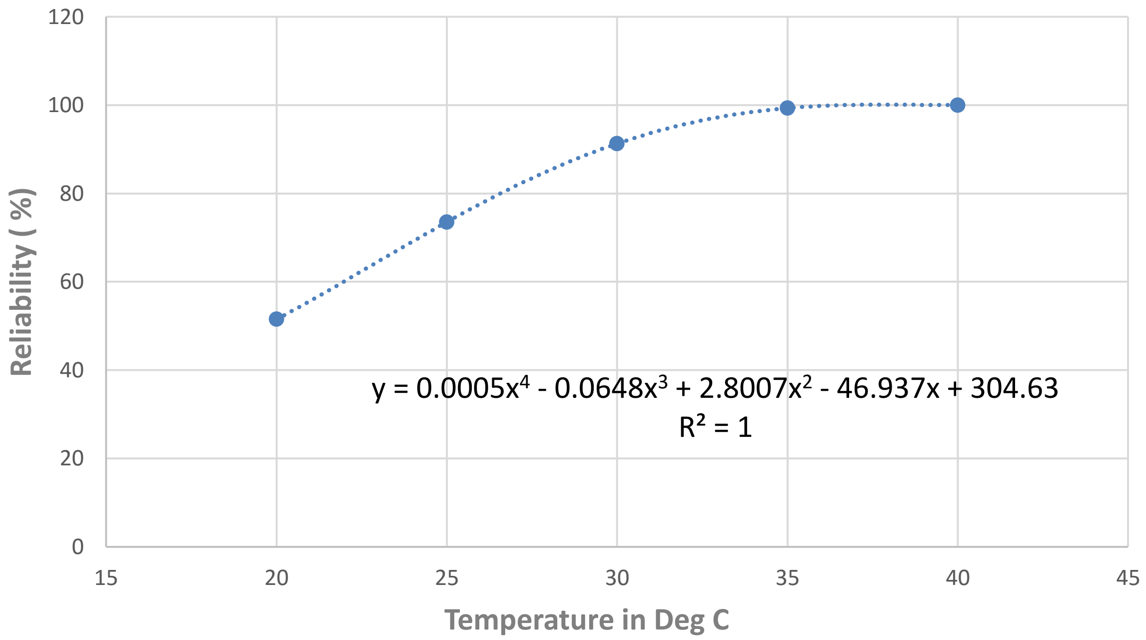


Figure 5.6- Variation of Reliability with Temperature for variation in thickness as ± 5 mm for 150 MSA



Above figure proves the fact that "Reliability" term alters with temperature with a polynomial degree of 4, which we found in *Figure 5.4* and *Figure 5.5*, i.e.

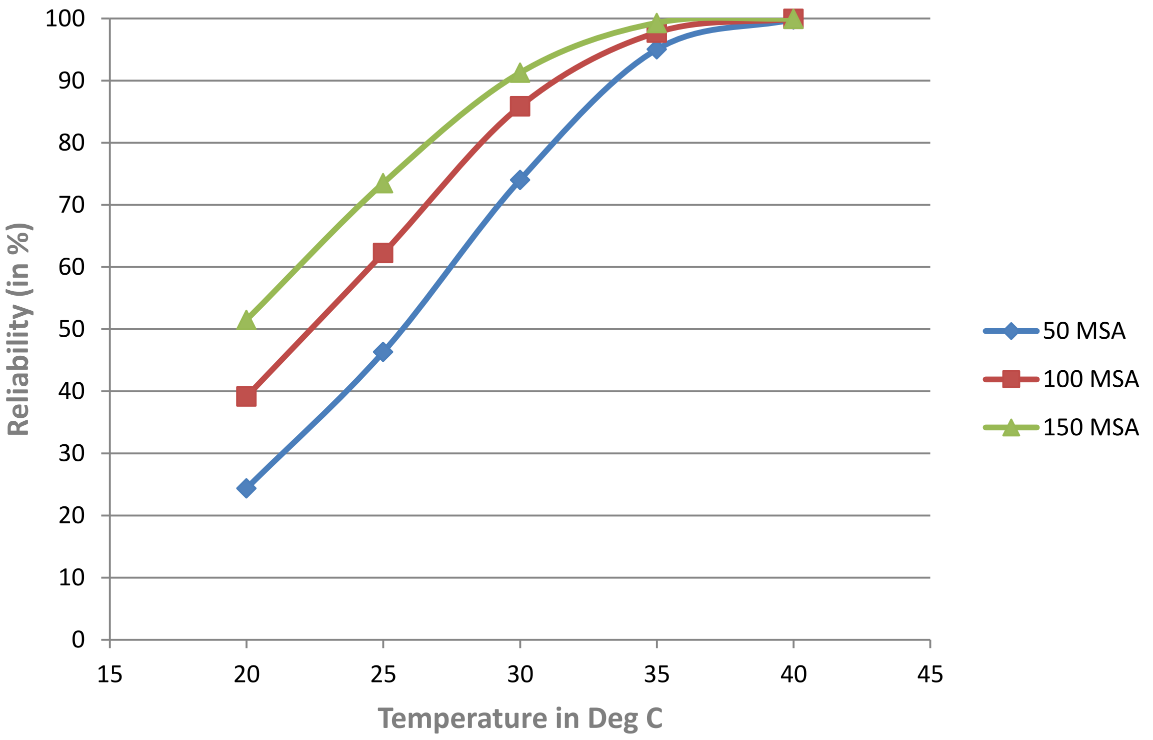
$$R = AT^4 - BT^3 + CT^2 - DT + E, \quad \text{Equation 5.2}$$

Where R = Reliability in percentage

A, B, C, D, E = Constants

T = Temperature in Degree Celsius.

Figure 5.7- Comparative Variation of Reliability with Temperature and MSA values for ± 5 mm thickness variation



It may be concluded apparently that Design Catalogues in IRC 37:2012 hold good for summer in India, but it fails exceptionally during winter or in low temperature regions. Reason for this variability may be described from the nature of bitumen. In low temperature bitumen gets stiffened due to its high Resilient Modulus, which leads bitumen to act as brittle material. So fatigue cracks are developed easily due to repetitive loading, making fatigue life abridged. So Reliability reduces dramatically in low temperature regions.

5.2.2.2. While Allowable Error in thickness is ± 20 mm:

When we allow the design thickness to vary 20 mm up and down during construction stage, the results are said to be almost same with the above figures. It again clearly states the fact that design thickness variation does not take a major role in variation of Reliability.

Figure 5.8- Variation of Reliability with Temperature for variation in thickness as ±20 mm for 50 MSA

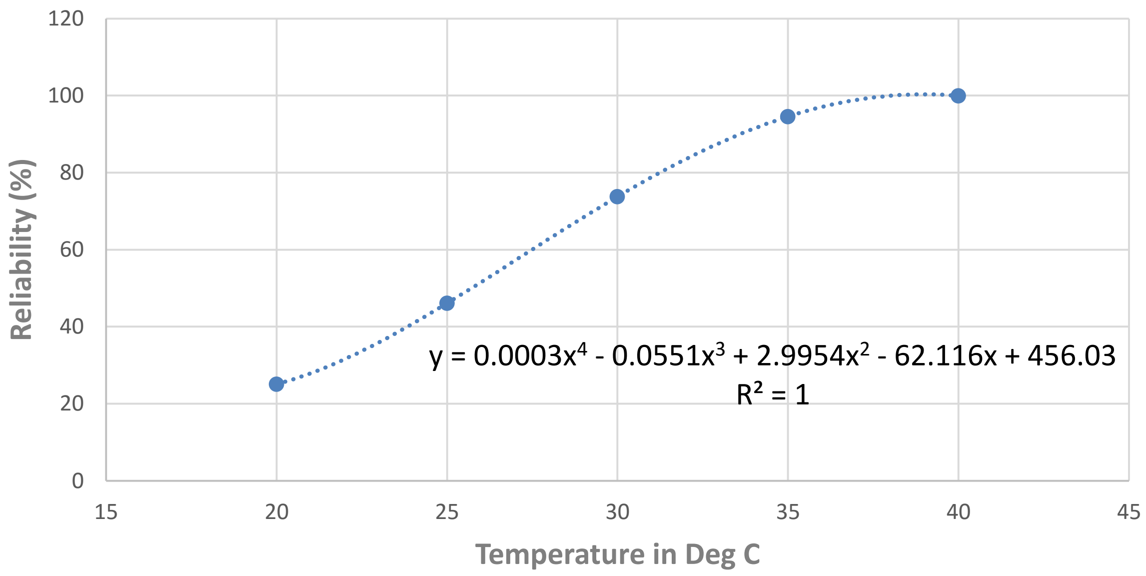


Figure 5.9- Variation of Reliability with Temperature for variation in thickness as ±20 mm for 100 MSA

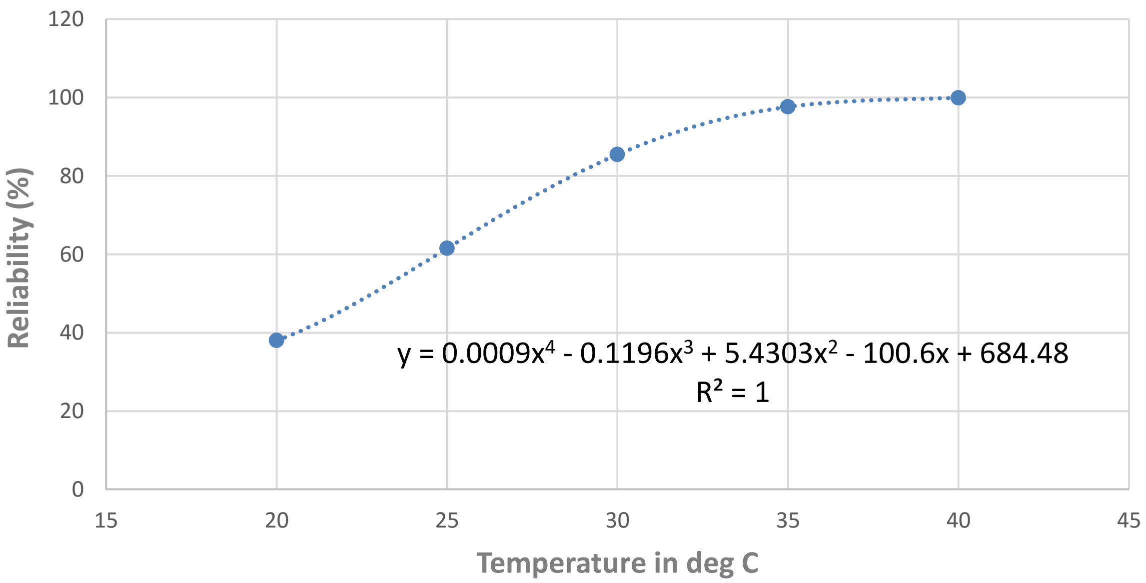


Figure 5.10- Variation of Reliability with Temperature for variation in thickness as ±20 mm for 150 MSA

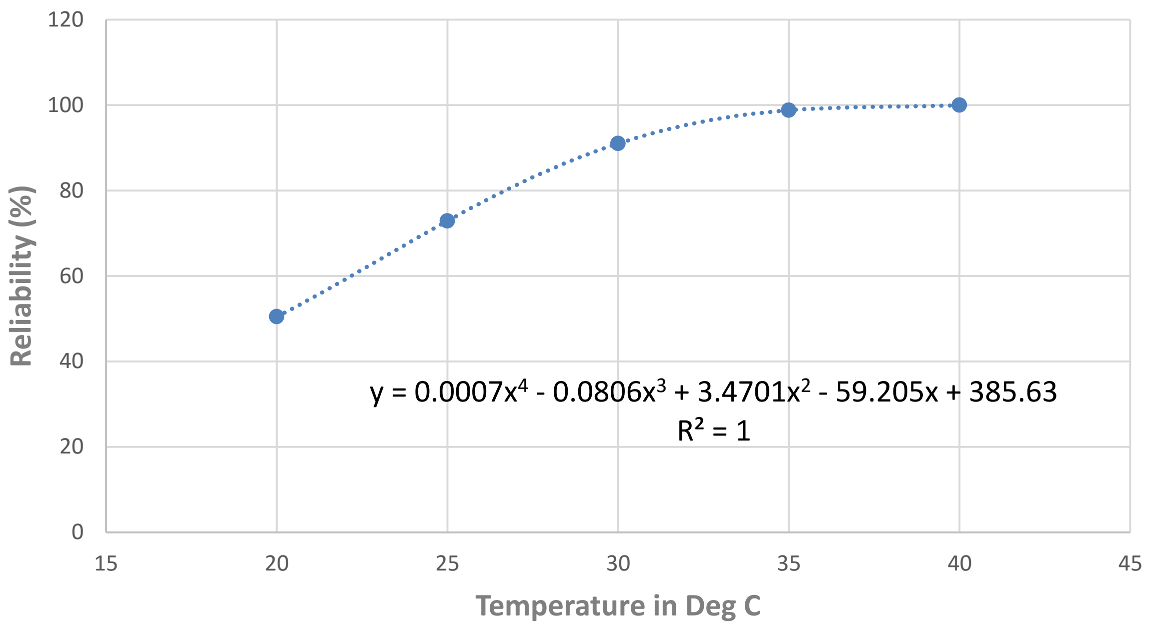
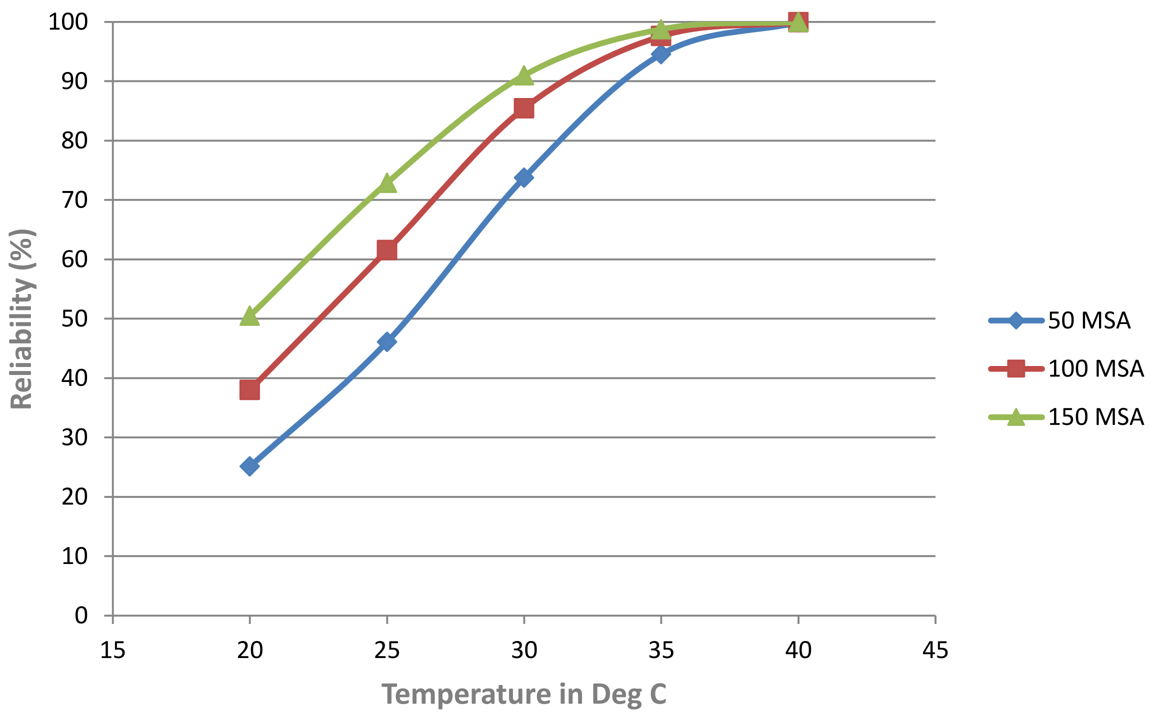


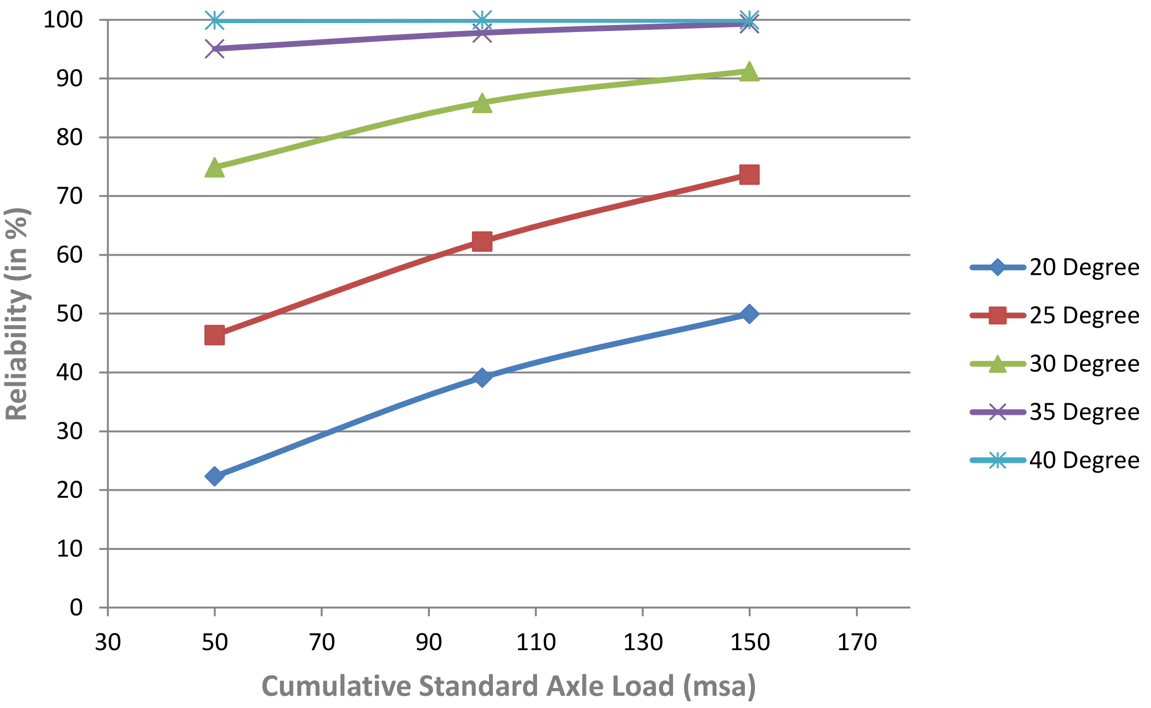
Figure 5.11- Comparative Variation of Reliability with Temperature and MSA values for ±20 mm thickness variation



5.2.3. Variation of Reliability with Load spectrum (msa):

If we plot reliability with cumulative standard axles (msa) by keeping temperature constant, we will see that design catalogues with higher msa values are more dependent for designing the pavement. Though at 35-40°C temperature all design catalogues hold good for a pavement design, the lower temperature catalogues are of main concern.

Figure 5.12- Variation of Reliability with MSA values at different Temperatures



For 20 Degree Celsius temperature, Reliability of a pavement designed from the same design catalogue differs for approximately 14% with a variation of 100 msa cumulative standard axle load. Higher the msa value, greater the Reliability. Not only for 20 Degree Celsius, has this relation continued even up to 30 Degree.

So somehow the design catalogues given in IRC 37:2012 hold good only for a higher msa values in summer. So, the catalogues need to be revised for winter season or low temperature regions along with lower msa values as design load spectrum.

CHAPTER – 6

CONCLUSION

6.1. Conclusions and Recommendations:

The primary objective of this research was to develop a rational means of accounting for the variability of the Mechanistic-Empirical (M-E) input design parameters. This was accomplished by incorporating Monte Carlo simulation and reliability analysis into the existing empirical design methodology.

Based upon the research findings presented in this report, the following conclusions and recommendations may be made,

1. Monte Carlo simulation is an effective means of incorporating reliability analysis into the M-E design process for flexible pavements.
2. The resulting distributions of fatigue life and rutting life, obtained from Monte Carlo simulation using the input distributions, are governed by an “extreme value type I function”.
3. For most practical design scenarios, the number of Monte Carlo cycles should be set at minimum 5,000. However, the designer may use his or her judgement to adjust the number of cycles accordingly. In this paper, reliability was used to vary within 2-3% boundary when number of Monte Carlo cycles was set to be 5,000. So in this project, the number of cycles was set to be 7,500 and it was proved to be a better assumption as Reliability values were almost converged.
4. Reliability may be defined as the probability that the allowable number of loads exceeds the expected actual number of loads (R = P [N > n]). This definition is consistent with other definitions of reliability.

5. For the purposes of design, data from the literature showed that,
   i. Layer moduli are log-normally distributed.
   ii. Layer thicknesses are normally distributed.
6. Generally speaking, the input parameters having the greatest influence on the fatigue performance variability are bituminous modulus and thickness. As a whole, we have seen that change in bituminous modulus can force to vary reliability drastically than other parameters like thickness variation. Results shown in the Chapter 5 apparently portray that drop in 5^0 C temperatures can lower the reliability value up to 25% for any load spectrum and Reliability is coming to vary with temperature by a polynomial relation of order 4 (*Equation 5.2*) which is as follows. But this needs to be examined and verified in the field.

   $$R = AT^4 - BT^3 + CT^2 - DT + E$$

   If a highway is linking two states like Himachal Pradesh and Maharashtra, the temperature variation will cause fatigue failure in colder region (i.e. Himachal Pradesh) if we provide design thickness from the reference of IRC 37:2012 as it does not give a provision to change thickness with the temperature.

   The code of IRC is concerned in determining the thickness of layers, but change in thickness to some amount during construction does not affect much in reliability. So our main point of concern should be the temperature criteria, not the thickness criteria.
7. Indian maps are advised to split into some temperature zones for flexible pavement design as we have seen enough that temperature variation can affect reliability analysis drastically. So it is recommended that design catalogues should have been different for different temperature zones, otherwise pavement may fail in fatigue criteria in colder regions.

Though AASHTO guidelines introduced “Reliability” term in 1993, the concept even now is quite new in India. So, some research works are accomplished on Reliability based on the AASHTO guidelines. IRC 37:2012 (the latest version of IRC 37) gives the design guidelines based on Empirical-Semi Mechanical approach.

Until this design procedure is used on a wider basis it will be difficult to make firm recommendations regarding acceptable levels of design reliability. For now, the recommendations made by AASHTO (discussed in Chapter 2) may be used as points of reference. However, the designer must ultimately make the informed decision based upon the available information regarding the design scenario.

It is important to reemphasize the needs of continually refine this design procedure. One of the fundamental principles of M-E design is that it must be calibrated to local conditions. This is clearly evident in the fatigue performance predictions. Further investigations and calibration are required to fine-tune the design procedure. However, with the framework in place, the calibration process should be relatively straightforward and changes may be implemented without great difficulty.

6.2. Future Scopes of Study:

In this project, a Reliability concept is introduced, and is incorporated into the conventional method of design based on IRC 37:2012. To make a complete analytical model, it is necessary to include the stochastic behaviour of the pavement variables. As temperature, load, thickness etc. cannot illustrate any deterministic nature, the variables were linked with a known distribution (Normal or Log-normal) to make it stochastic in nature. As it is a brand new concept in India, it needs to be studied thoroughly in future.

6.2.1. Develop a Complete Software Package:

This project suggests three different parts of study to incorporate an analytical model into IRC 37:2012.

1. A computer model is prepared for determining the design thicknesses of layers of the pavement based on the Design Catalogues supplied by IRC 37:2012, by taking CBR value of the soil and load spectrum as inputs.
2. IITPAVE software provides us the tensile strain parameter which is one of the major contributors in fatigue life. Though, we have used an empirical formula to make it easy for calculation, IITPAVE value needs to be incorporated into the actual design.

The results from the above two steps are required for Monte Carlo simulation to fit in the stochastic nature of the variables and finally to find out Reliability.

Practically, this whole process is vast and will take immense time for the above three steps to be done separately. So, the only solution is to develop a complete package of software that will contain the above steps, which can make a revolution in Indian Code of practise.

6.2.2. Modification of IRC 37:2012:

When every country is trying to find out some analytical models of pavement fatigue characteristics, empirical relation based design method of IRC 37:2012 is out-dated and not reliable. So this paper suggests some modification over IRC 37:2012 which are as follows,

1. *Temperature Variation*:

   We have seen in the previous chapter how the temperature affects Reliability of the pavement. But IRC 37:2012 has no provision of considering temperature aspect into the design procedure. The temperature variation in different states of India at a certain time is very common due to its large stretch over North-South direction.

Again, in this project temperature fluctuation has not been considered for a pavement. We have taken five temperatures as data and corresponding MSA values with Reliability.

Similarly, the seasonal variation of some of the states in India is quite large, even up to 35 degree. So, an optimised pavement thickness is to be considered for design. Even winter does not prevail over 3 months in India. So, a weighted average of temperature with number of days should be the criteria for the design temperature. But all these things need to be first investigated and examined for at least couple of years and then should reach to the decision.

2. *Variation of Input parameters*:

Input parameters like Resilient Modulus, Poisson's Ratio, Layer thickness etc. need to be statistically characterised for finding out Reliability. So IRC 37 will have to introduce detailed description about these parameters and their variation characteristics. Though Poisson's ratio does not change significantly with temperature, for thorough analysis it is to be taken into account.

6.2.3. Rutting Life Prediction:

Rutting is the permanent deformation in pavement usually occurring longitudinally along the wheel path. The rutting may partly be caused by deformation in the subgrade and other non-bituminous layers which would reflect to the overlying layers to take a deformed shape.

Only fatigue life is considered in this project. Rutting life is to be calculated in near future to determine the exact reliability. Usually fatigue is a primary criterion for flexible pavement, but for poor subgrade, rutting acts as a major criterion. The bituminous mixes also may undergo rutting due to secondary compaction and shear deformation under heavy traffic load and higher temperature. Excessive rutting greatly reduces the serviceability of the pavement and therefore, it has to be limited to a certain reasonable value.

IRC 37:2012 gives two equations of rutting model for 80% and 90% reliability levels, but failed to suggest a general equation of rutting model. The two equations are given below,

$$N = 4.1656 \times 10^{-8} \times \left[\frac{1}{\epsilon_v}\right]^{4.5337}$$ *Equation 6.1*

$$N = 1.41 \times 10^{-8} \times \left[\frac{1}{\epsilon_v}\right]^{4.5337}$$

As discussed above, sometimes rutting failure acts as major failure criteria. So it is important to incorporate rutting model into this Reliability analysis to find a better result.

6.2.4. Other Sub Base and Base materials:

This project only deals with the pavement which consist bituminous layer with Granular Base and Granular Sub base. But IRC 37:2012 suggested design catalogues also for the pavement of following layers,

1. Cementitious Base and Cementitious Sub base of aggregate interlayer for crack relief. Upper 100 mm of the cementitious sub base is the drainage layer.
2. Cementitious base and sub base with SAMI at the interface of base and the bituminous layer.
3. Foamed bitumen/bitumen emulsion treated RAP or fresh aggregates over 250 mm cementitious sub base.
4. Cementitious base and granular sub base with crack relief layer of aggregate layer above the cementitious base.

So, the above categories of base and sub base layers will also have to analyse for Reliability prediction in near future.

6.2.5. Top-Down Fatigue cracking:

Fatigue cracking is one of the dominant distress modes in bituminous pavements. The conventional approach assumes cracks to initiate at the bottom of bituminous layer and to propagate further upward to the surface. However, core samples from multiple site investigations have revealed cracks that occur in the reverse direction. This failure phenomenon, called “top-down fatigue cracking” was recognized recently as a failure mode and it was shown to be prevalent in many parts of the world. The mechanism and failure process of top down fatigue cracking is not fully understood and established, thus making it difficult to consider this failure mode effectively in the design process.

Research regarding top-down fatigue cracking has been on-going through the years. In the last decade, Hot Mix Asphalt Fracture Mechanics (HMAFM) based Mechanistic-Empirical analysis models were established that could predict crack initiation time and crack propagation rate (*Birgisson, Wang,* & *Roque, 2006*). Though these newly developed models have shown their capacity in delivering results which are in a good agreement with the observed performance in the field, still more work is needed to enhance, calibrate and validate these models. Moreover, as shown in recent research, the morphology of asphalt mixtures which plays a critical role in governing and controlling mixture aging properties and consequently the mixture long term fracture characteristic is missing in these models.

Thus a thorough investigation and research is needed to develop, calibrate and validate a mechanics-based top down fatigue cracking initiation prediction framework which is based on fundamental material behaviour and that encompasses key factors and design parameters.

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