A STUDY ON THE PERFORMANCE OF FLEXIBLE PAVEMENTS ON MATURE SOIL SUBGRADES

Thesis

Submitted for the award of the degree of

DOCTOR OF PHILOSOPHY

in Faculty of Engineering

by

SREEDEVI B.G

DIVISION OF CIVIL ENGINEERING, SCHOOL OF ENGINEERING

COCHIN UNIVERSITY OF SCIENCE AND TECHNOLOGY

KOCHI, 682022 - INDIA

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CERTIFICATE

This is to certify that the thesis entitled ***'A STUDY ON THE PERFORMANCE OF FLEXIBLE PAVEMENTS ON MATURE SOIL SUBGRADES'****, submitted to Cochin University of Science and Technology, Kochi for the award of the degree of* ***Doctor of Philosophy****, under the Faculty of Engineering, is a bonafide research carried out by* ***Smt. Sreedevi B G****, under my supervision and guidance in the School of Engineering, Cochin University of Science and Technology. No part of this thesis has been presented for any other degree from any other institution.*

Kochi
August 2014

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DECLARATION

*I hereby declare that the work presented in this thesis entitled '**A STUDY ON THE PERFORMANCE OF FLEXIBLE PAVEMENTS ON MATURE SOIL SUBGRADES'** being submitted to Cochin University of Science and Technology for the award of the degree of* **Doctor of Philosophy***, under the Faculty of Engineering, is based on the original work done by me under the supervision and guidance of* ***Dr. Benny Mathews Abraham****, Professor of Civil Engineering, School of Engineering, Cochin University of Science and Technology, Kochi. No part of this thesis has been presented for any other degree from any other institution.*

Kochi ***SREEDEVI B.G***
August 2014

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Kochi **SREEDEVI B.G**

August 2014

Dedicated to my beloved parents….

ABSTRACT

The country has witnessed tremendous increase in the vehicle population and increased axle loading pattern during the last decade, leaving its road network overstressed and leading to premature failure. The type of deterioration present in the pavement should be considered for determining whether it has a functional or structural deficiency, so that appropriate overlay type and design can be developed. Structural failure arises from the conditions that adversely affect the load carrying capability of the pavement structure. Inadequate thickness, cracking, distortion and disintegration cause structural deficiency.

Functional deficiency arises when the pavement does not provide a smooth riding surface and comfort to the user. This can be due to poor surface friction and texture, hydro planning and splash from wheel path, rutting and excess surface distortion such as potholes, corrugation, faulting, blow up, settlement, heaves etc. Functional condition determines the level of service provided by the facility to its users at a particular time and also the Vehicle Operating Costs (VOC), thus influencing the national economy.

Prediction of the pavement deterioration is helpful to assess the remaining effective service life (RSL) of the pavement structure on the basis of reduction in performance levels, and apply various alternative designs and rehabilitation strategies with a long range funding requirement for pavement preservation. In addition, they can predict the impact of treatment on the condition of the sections. The infrastructure prediction models can thus be classified into four groups, namely primary response models, structural performance models, functional performance models and damage models.

The factors affecting the deterioration of the roads are very complex in nature and vary from place to place. Hence there is need to have a thorough study of the deterioration mechanism under varied climatic zones and soil conditions before arriving at a definite strategy of road improvement. Realizing the need for a detailed study involving all types of roads in the state with varying traffic and soil conditions, the present study has been attempted.

This study attempts to identify the parameters that affect the performance of roads and to develop performance models suitable to Kerala conditions. A critical review of the various factors that contribute to the pavement performance has been presented based on

the data collected from selected road stretches and also from five corporations of Kerala. These roads represent the urban conditions as well as National Highways, State Highways and Major District Roads in the sub urban and rural conditions.

This research work is a pursuit towards a study of the road condition of Kerala with respect to varying soil, traffic and climatic conditions, periodic performance evaluation of selected roads of representative types and development of distress prediction models for roads of Kerala. In order to achieve this aim, the study is focused into 2 parts. The first part deals with the study of the pavement condition and subgrade soil properties of urban roads distributed in 5 Corporations of Kerala; namely Thiruvananthapuram, Kollam, Kochi, Thrissur and Kozhikode. From selected 44 roads, 68 homogeneous sections were studied. The data collected on the functional and structural condition of the surface include pavement distress in terms of cracks, potholes, rutting, raveling and pothole patching. The structural strength of the pavement was measured as rebound deflection using Benkelman Beam deflection studies. In order to collect the details of the pavement layers and find out the subgrade soil properties, trial pits were dug and the in-situ field density was found using the Sand Replacement Method. Laboratory investigations were carried out to find out the subgrade soil properties, soil classification, Atterberg limits, Optimum Moisture Content, Field Moisture Content and 4 days soaked CBR. The relative compaction in the field was also determined. The traffic details were also collected by conducting traffic volume count survey and axle load survey.

From the data thus collected, the strength of the pavement was calculated which is a function of the layer coefficient and thickness and is represented as Structural Number (SN). This was further related to the CBR value of the soil and the Modified Structural Number (MSN) was found out. The condition of the pavement was represented in terms of the Pavement Condition Index (PCI) which is a function of the distress of the surface at the time of the investigation and calculated in the present study using deduct value method developed by U S Army Corps of Engineers. The influence of subgrade soil type and pavement condition on the relationship between MSN and rebound deflection was studied using appropriate plots for predominant types of soil and for classified value of Pavement Condition Index. The relationship will be helpful for practicing engineers to design the overlay thickness required for the pavement, without conducting the BBD test. Regression analysis using SPSS was done with various trials to find out the best fit relationship between the rebound deflection and CBR, and other soil properties for Gravel, Sand, Silt & Clay fractions.

The second part of the study deals with periodic performance evaluation of selected road stretches representing National Highway (NH), State Highway (SH) and Major District Road (MDR), located in different geographical conditions and with varying traffic. 8 road sections divided into 15 homogeneous sections were selected for the study and 6 sets of continuous periodic data were collected. The periodic data collected include the functional and structural condition in terms of distress (pothole, pothole patch, cracks, rutting and raveling), skid resistance using a portable skid resistance pendulum, surface unevenness using Bump Integrator, texture depth using sand patch method and rebound deflection using Benkelman Beam. Baseline data of the study stretches were collected as one time data. Pavement history was obtained as secondary data. Pavement drainage characteristics were collected in terms of camber or cross slope using camber board (slope meter) for the carriage way and shoulders, availability of longitudinal side drain, presence of valley, terrain condition, soil moisture content, water table data, High Flood Level, rainfall data, land use and cross slope of the adjoining land. These data were used for finding out the drainage condition of the study stretches.

Traffic studies were conducted, including classified volume count and axle load studies. From the field data thus collected, the progression of each parameter was plotted for all the study roads; and validated for their accuracy. Structural Number (SN) and Modified Structural Number (MSN) were calculated for the study stretches. Progression of the deflection, distress, unevenness, skid resistance and macro texture of the study roads were evaluated. Since the deterioration of the pavement is a complex phenomena contributed by all the above factors, pavement deterioration models were developed as non linear regression models, using SPSS with the periodic data collected for all the above road stretches. General models were developed for cracking progression, raveling progression, pothole progression and roughness progression using SPSS. A model for construction quality was also developed.

Calibration of HDM–4 pavement deterioration models for local conditions was done using the data for Cracking, Raveling, Pothole and Roughness. Validation was done using the data collected in 2013. The application of HDM-4 to compare different

maintenance and rehabilitation options were studied considering the deterioration parameters like cracking, pothole and raveling. The alternatives considered for analysis were base alternative with crack sealing and patching, overlay with 40 mm BC using ordinary bitumen, overlay with 40 mm BC using Natural Rubber Modified Bitumen and an overlay of Ultra Thin White Topping. Economic analysis of these options was done considering the Life Cycle Cost (LCC). The average speed that can be obtained by applying these options were also compared. The results were in favour of Ultra Thin White Topping over flexible pavements. Hence, Design Charts were also plotted for estimation of maximum wheel load stresses for different slab thickness under different soil conditions. The design charts showed the maximum stress for a particular slab thickness and different soil conditions incorporating different k values. These charts can be handy for a design engineer.

Fuzzy rule based models developed for site specific conditions were compared with regression models developed using SPSS. The Riding Comfort Index (RCI) was calculated and correlated with unevenness to develop a relationship. Relationships were developed between Skid Number and Macro Texture of the pavement.

The effort made through this research work will be helpful to highway engineers in understanding the behaviour of flexible pavements in Kerala conditions and for arriving at suitable maintenance and rehabilitation strategies.

Key Words: Flexible Pavements – Performance Evaluation – Urban Roads – NH – SH and other roads – Performance Models – Deflection – Riding Comfort Index – Skid Resistance – Texture Depth – Unevenness – Ultra Thin White Topping

CONTENTS

LIST OF FIGURES

LIST OF TABLES

Chapter 1

INTRODUCTION

1.1 GENERAL

The increasing traffic intensity, high tire pressure, increasing axle loads etc are causing early signs of distress to bituminous pavements throughout the world. The deterioration of the paved roads in tropical and subtropical countries differ from those in the more temperate regions of the world. This can be due to the harsh climatic conditions and sometimes due to the lack of good pavement materials and construction practices.

Pavement performance can be defined as the ability of the road to meet the demands of traffic and environment during its design life. The reduction in the performance level of the pavement with time is termed as deterioration. Flexible pavements deteriorate due to many factors, predominantly traffic, climate, material, construction quality and time. These multiple parameters make the process very complex. The condition of the road at any time can be predicted approximately using performance models.

For managing the transport infrastructure system, prediction and modelling of their performance are the main inputs as well as major challenges. The predicted deterioration play major roles at both network level and project level. The overall facilities can be planned for justifying the budget and resources with help of deterioration models. The planning and scheduling of the maintenance work for individual project is dependent on the time at which the section becomes deficient in service. This can be predicted through accurate deterioration models. Development of appropriate transportation policy and evaluation of the economic impacts also depend on the performance and interplay between the infrastructure facility and its user (traffic). One such example is the imposition of axle load limits, which is responsible for the damage of the pavement at exponential rates.

Lack of necessary maintenance results in deterioration of the pavement, which in turn cause damage to the vehicles and higher fuel consumption, thereby increasing the vehicle operating (VOC) and user costs. To ensure an acceptable level of service, comfort and safety on these roads, road maintenance activities are very essential. Also, for increasing the life of the pavement, timely and appropriate maintenance is very essential. In this context, by understanding the performance of the pavement accurately has great significance. Performance prediction of flexible pavements is an essential activity in the design of flexible pavement overlays, can be used to develop appropriate strategies, and improved design methodologies. Also a developing country like India is now facing the challenge of preserving and enhancing its transportation system infrastructure within limited budget allocation. So, prioritization of roads is required for planning optimum Maintenance and Rehabilitation (M&R) strategies.

Efficient management of the road infrastructure can be achieved by predicting their performance accurately. Since the deterioration and performance of the pavements depend on multiple factors like traffic, climate, environment, construction quality, age, etc., the process is very complex. Many researchers all over the world have developed performance prediction models applicable for particular set of conditions. But these models require more generalisation to have accurate and comprehensive predictive ability.

The main input data considered for performance prediction is the structural strength of the pavement. The structural strength of a pavement is determined by measuring the deflection of pavement under the traffic loads. In the case of pavement, which has been well compacted conditioned by the continuously moving traffic, there will be elastic deformation under each wheel load application. When the wheel load is released, the pavement surface will perform an elastic recovery, also termed as rebound deflection. Decisions on strengthening or reconstruction of pavement are made from analysis of structural strength data. This data gives insight to the right cause of deterioration. Pavement engineers should understand the factors, which affect the long-term performance of overlays so as to design and provide long lasting overlays.

Prioritization or ranking or optimization models are used for determining the best maintenance and rehabilitation package for a given road network. The present quality of the pavement is represented some indices with the data on the condition using the prioritization models. The parameters which are commonly used for ranking are traffic volume, category of road, quality index etc. The prioritisation and ranking are used to determine the allocation of the M & R resources to the road sections. The basic criteria for the analysis of the optimization models can be the minimization of improvement cost

& vehicle operating cost and residual value of the pavements during the period selected as the life span, or maximization of the quality and performance of the entire network within the available annual budgets and minimum quality levels. Budgeting and resource allocation at the network level and activity planning and project prioritisation at programming level can be done using the performance prediction models. At project level, an accurate model is helpful for determining and designing the corrective measures including the M & R strategies.

1.2 PRESENT SCENARIO

India's total road network has increased to 4.69 million km now from 2 million in the 1990s (Indian Infrastructure, 2013) and is the world's second longest road network spread over the entire 29 states of the country. Among this, Kerala has 0.16 million km of roads, which consists of 3.9% of India's total roadways. The large variation in terrain, climatic, environmental, soil and traffic characteristics of Kerala roads, differentiate it from other roads in the country. In the case of Kerala roads, flexible pavements contribute the major portion and are subjected to intense rain and heavy traffic.

In India, it is estimated that annually 7,200-30,000 billion rupees is lost to the economy due to constraints of road network capacity and quality. The annual road safety toll also is alarming and exceeds 1.5 lakh deaths. Studies conducted by the World Bank have reported that an amount of Rs 15,000 crore per annum is estimated as the economic loss due to poor riding quality, inadequate thickness of pavement and insufficient capacity. In India, the Road User Cost Study (RUCS) has brought out that due to bad condition of roads and low design speeds, the average utilization per day of commercial vehicle is only 200 to 250 km, incurring high Vehicle Operating Cost as compared to 500 to 600 km in developed countries. These trends together with the constraints on resources highlight the need for a better system management for road infrastructure.

1.3 NEED FOR THE STUDY

The roads of the country, especially those in the southern region, are showing signs of distress after each monsoon leading to heavy loss on their maintenance along with loss resulting in poor riding quality, accidents and reduction in speed. The factors affecting the deterioration of the roads are very complex in nature and vary from place to place.

Hence, there is a need to have a thorough study of the deterioration mechanism under varied climatic zones and soil conditions before arriving at a definite strategy of road improvement. Realizing the need for a detailed study involving all types of roads in the state with varying traffic and soil conditions, the present study has been attempted. It is hoped that this study would help to provide the data gap for sustainable road development in the State.

1.4 OBJECTIVES OF THE STUDY

The broad aim of the study is to evaluate the performance of in-service roads in terms of reliability, surface condition, structural failure and safety with respect to the strength of the pavement layers and subgrade soil properties and develop relationships and models for prediction. Towards achieving this aim, the objectives of this research work have been formulated as given below:

i. Evaluate the structural and functional condition of the study stretches.

ii. Study the condition of pavements under different soil conditions in Kerala.

iii. Develop correlation between the strength of the pavement with subgrade soil properties and pavement condition.

iv. Study the periodic performance of the study roads.

v. Review the models available for predicting the pavement condition.

vi. Develop models to predict the performance of flexible pavements and compare the same with models developed using other techniques.

vii. Calibrate HDM-4 to Kerala conditions.

viii. Compare the different overlay strategies and select the most feasible option.

ix. Develop models to predict the Riding Comfort and Skid Resistance of the pavement to support timely intervention.

1.5 SCOPE OF THE STUDY

To fulfil the above objectives, the scope of the study was set to two parts. In the first part, study of the strength parameters of urban roads with respect to different subgrade soil properties were attempted using one time data collected from 44 road stretches divided into 68 homogeneous sections distributed in five Municipal Corporations in Kerala. The field data include structural and functional condition of the pavement, inventory, traffic and drainage characteristics. In the second part of the study, performance evaluation of 8 road stretches representing NH, SH and MDR located in different geographical settings was attempted using time series data collected from the field to develop deterioration models.

1.6 WORK PROGRAM

In order to achieve the above objectives, attempts were made to identify the parameters that affect the performance of roads and to develop performance models suitable to Kerala conditions. A critical review of the various factors that contribute to the pavement performance was presented based on the data collected from five Corporations and also from selected road stretches of Kerala as indicated in the scope. These roads represent the urban condition as well as National Highways, State Highways and Major District Roads in the sub urban and rural conditions. Relationships were established between the strength and ride quality of the pavement with technical parameters such as Structural Number, subgrade soil classification, gradation, relative compaction, CBR value of subgrade, pavement history, IRI value, status of road and category. The universally accepted World Bank Software ‘HDM-4, Highway Development and Management’ was also used for analysis. Application of SPSS was used to develop deterioration models and those were compared with HDM-4 and Fuzzy Rule based models.

1.7 ORGANISATION OF THE REPORT

This report is organized into **7** chapters.

Chapter 1 gives an introduction on the status of flexible pavements, importance of pavement evaluation and deterioration modeling, broad objectives of the study, scope of the study, work program and the chapter scheme.

Chapter 2 contains an outline of the flexible pavement deterioration mechanism, pavement evaluation methods and techniques, types of models, global scenario on pavement performance modelling, previous studies reported and different softwares so far applied for performance modelling. Reported models are reviewed. The salient features of Ultra Thin White Topping as an overlay option to flexible pavements is also included.

The basic methodology, instrumentation and procedure adopted for the study are described in **Chapter 3.** The surveys carried out for data collection and tools adopted for model development are illustrated. Brief description of modeling with HDM 4, Fuzzy logic and non- linear regression using SPSS are also presented in this chapter.

A detailed description of the study stretches is given in **Chapter 4**. The roads identified for pavement condition analysis consist of 68 homogeneous sections on 44 roads in 5 Corporations of Kerala namely Thiruvananthapuram, Kollam, Kochi, Thrissur and Kozhikode. Deflection studies, Condition survey and soil investigations were done on these study roads. Eight roads representing NH, SH and MDR were selected for pavement evaluation. All the study sections are representative of varying terrain, traffic, soil parameters, land use etc. The results of the data analysis and values of various parameters and interventions based on adopted standards are also included in this chapter.

In the **Fifth Chapter,** data collected from the urban roads are analyzed and presented to develop relationships of the influencing parameters on deflection. The effect of pavement condition and soil type on the strength of the pavement was examined using appropriate plots. Relationships were developed for different classifications of soil predominant in the study stretches for classified values of pavement condition indexes. These relationships were validated for their accuracy and adoption to specific site conditions. Non-linear relationships were developed in SPSS considering Field Dry Density, Maximum Dry Density, Optimum Moisture Content, Field Moisture Content, Liquid Limit, Plastic Limit, Plasticity Index, Gravel, Sand, Silt & Clay fractions and CBR as variables. Different combinations were tried and the best-fit equations are reported.

Chapter 6 contains the results of the data analysis of the pavement evaluation study of eight road stretches. Progression of unevenness, deflection, pavement distress, skid resistance and texture depth are presented. Models that are fitting to the study roads were developed in SPSS and compared with the reported models. HDM-4 models were also tried and compared with Fuzzy rule based and other available models. The application of HDM-4 to compare different maintenance and rehabilitation options was studied considering the deterioration parameters like cracking, pothole and ravelling. The alternatives considered for analysis were base alternative with crack sealing and patching, overlay with 40 mm BC using ordinary bitumen, overlay with 40 mm BC using Natural Rubber Modified Bitumen and overlay of Ultra Thin White Topping. Economic analysis of these options was done considering the Life Cycle Cost (LCC). The average speed that could be obtained by applying these options were also compared. The correlation between skid resistance and macro texture of the pavements was formulated using appropriate plots. The Riding Comfort Index (RCI) was calculated and correlated with unevenness index to develop a relationship.

The report is concluded with **Chapter 7** and specific recommendations are evolved. The references and papers published are also listed in the end.

1.8 SUMMARY

This chapter contains a brief description of the performance of pavements, importance of performance modelling and prediction, input data considered, present scenario of road availability in the country and the loss due to constraints of road network capacity and quality. The felt need for the study is highlighted along with the objectives and scope within which the study is confined. The work program and chapter scheme are also outlined.

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Chapter 2

LITERATURE REVIEW

2.1 GENERAL

Performance of pavement can be generally defined as to the change in their condition or function with respect to age. It can also be indicative of the ability of a pavement to carry the intended traffic and satisfy the environment during the design life, both functionally and structurally. With the increased economic and development activities in India, the traffic has increased multi fold during the last 3 decades resulting in the overstressing of road network. The development of higher stresses leads to performance failure of the pavements. If the pavements fail to carry the design loads satisfactorily, then the failure is of structural type. It is of functional type, if the pavement does not provide a smooth riding surface. The uneven surface not only causes discomfort, but also increases the Vehicle Operating Cost (VOC), thus influencing the overall transportation cost. This chapter gives a broad outline of the importance of pavement performance evaluation, type of models, applications of performance models in other countries for their Pavement Management System and the research studies carried out so far.

2.2 FLEXIBLE PAVEMENT DETERIORATION MECHANISM

The structural and functional conditions of flexible pavements changes with time due to continued effects of its structural adequacy, volume, composition and loading characteristics of traffic, environment, surrounding conditions and the maintenance inputs provided. The failure of the pavement occurs due to internal damage caused by traffic loads within an operational environment, over a period of time; and is not an abrupt phenomenon. Deterioration can also be defined as the process of accumulation of damage and the failure of the pavement is said to have reached at the limiting stage of serviceability level. Studies conducted all over the world have established that even though design and construction techniques vary from country to country, the deterioration pattern of pavements shows the same trend.

The various factors which cause deterioration of flexible pavements can be represented as shown in **Fig. 2.1**

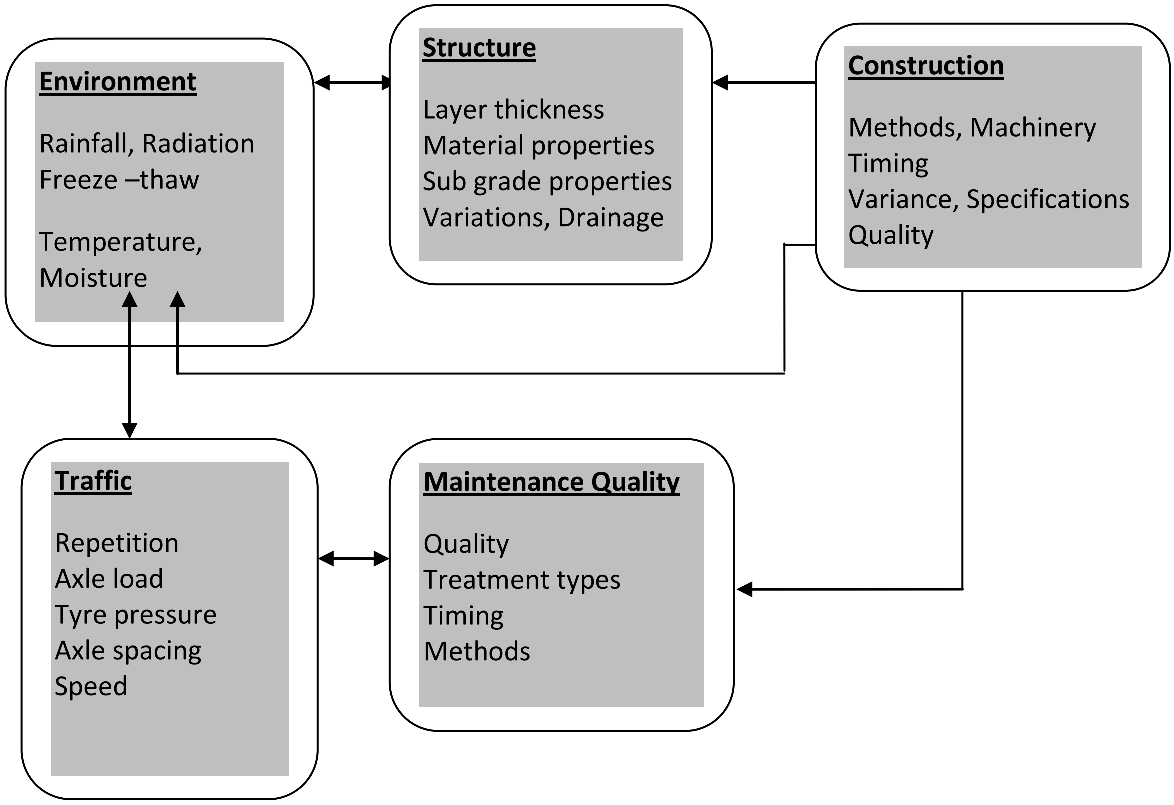

Fig. 2.1 Factors influencing pavement performance (Gedafa D S, 2007))

The main deteriorations include cracking, potholes, rutting along wheel path and roughness of road surface. The physical manifestation of the internal damage (cracking, rutting, potholes etc.) is known as distress. Percentage of distress gives an indication of the pavement condition. Different modes of distress occur either independently or simultaneously with mutual interaction. For planning purpose, the distress can be based on distress type and the most important are those, which trigger decisions. The distresses can be broadly classified as:

i. Fatigue Cracking

- Load – Associated cracking
- Thermal cracking (due to freeze and thaw)
- Longitudinal cracking at edges due to moisture movement through shoulder
- Deflection cracking

ii. Load Associated pavement distortion

- Transverse distortion or rutting
- Longitudinal distortion or roughness

iii. Non – load associated pavement distortions due to foundation movements

iv. Disintegration (raveling, stripping, potholing etc.)

2.3 PERFORMANCE EVALUATION OF PAVEMENTS

In order to build more durable roads for tomorrow, it is imperative to find out how pavements and materials will perform under repeated heavy loads. The deterioration of the pavements show slow progress during the initial years after construction, but very fast progress during later years. Performance evaluation involves a thorough study of various factors such as subgrade support, pavement composition & its thickness, traffic loading and environmental conditions. The evaluation is broadly classified into (i) Structural evaluation and (ii) Functional evaluation. Pavement evaluation process is normally represented using four criteria, namely, Pavement Roughness (Reliability), Pavement distress (Surface condition), Pavement deflection (Structural failure) and Skid resistance (Safety). Certain terms are defined by researchers and are mentioned here before looking upon the models developed. They are:

- Present Serviceability. This is the term used to represent the ability of a specific section of pavement to serve high-speed, high-volume, mixed traffic within the existing conditions.
- Individual Present Serviceability Rating (PSR). This denotes an independent rating by an individual on the present serviceability of a specific section of roadway. The ratings usually range from 0 to 5. The individuals may also be asked to indicate whether or not the pavement is acceptable as a primary highway.
- Present Serviceability Index (PSI). PSI is a mathematical combination of values obtained from certain physical measurements formulated to predict the PSR for those pavements within prescribed limits.
- Performance Index (PI). This denotes the summary of PSI over a period of time, which can be represented by the area under the PSI versus time curve. There are

many possible ways in which the summary value can be computed. The simplest summary consists of the mean ordinate of the curve of PSI against time.

Aggarwal et al. (2005) has given an overall picture of the problems of road networks in developing countries, which are rapid traffic growth, inadequate funding for maintenance and upkeep, lack of skilled man power, attitude towards maintenance etc. Thube et al. (2005) critically reviewed the maintenance management strategy for low volume roads in India and stressed the need for development of pavement distress data base, deterioration models, optimal investment and maintenance strategy and highlighted the need for a suitable National level policy regarding paving of unpaved low volume roads in India.

2.4 PERFORMANCE PREDICTION MODELS (PPM)

According to the World Road Association (Ferreira *et al.* 1999, 2004), a PPM is a mathematical representation that can be used to predict the future state of pavements, based on current state, deterioration factors (traffic and climate) and effects resulting from maintenance and rehabilitation actions (or simply M&R actions).

The projection of the present condition of the performance of the pavement is done within user-defined scenarios of future loading and maintenance using accurate and realistic models. But the outcome of the applied models will be acceptable only if they conform to a reasonable set of generic criteria. Road deterioration modeling is considered as an important parameter in Road Infrastructure Management and Road Maintenance Management Systems. (RMMS). Good RD models should be able to suggest efficient and economically viable treatment options, which could be converted into realistic work programs and strategies for planning. They assume importance in a PMS due to many reasons as given below:

- The type of the deterioration indicated by the model, its severity, intensity and extent with time of occurrences are used to decide upon the nature of the proposed treatment.
- The severity of deterioration of pavement has influence on surface unevenness, which will be used to determine the RUE costs modeling. If the RD modeling is not perfect and realistic, RUE modeling will be influenced, even if RUE models are correctly formulated.

- Modeled maintenance effects can be realistic only if the RD models produce accurate predictions.

- The Work Effect (WE) modeling and parameter resets are used for determining the treatment. Then the RD models perform the modeling of the performance of the pavement until the next set of WE is applied. The RD models are most important for predicting the performance of pavements, and they are to be very accurate to get, both technically and financially realistic WE applications.

2.4.1 Parameters Predicted

Using the models, the condition of the pavement can be predicted in terms of one or more of different measures. These measures can be grouped into four groups namely (i) Primary response, (ii) Structural performance, (iii) Functional performance and (iv) Damage performance models. These models can also predict the impact of treatment on the condition of sections.

i. **Primary response models:** The primary mechanistic response of sections to external loads are predicted by these models. The most basic parameters are deflection, stress and strain. These models are generally used at project level. These are also called mechanistic models. The data input consist of information gathered from construction records, laboratory testing or in-situ non-destructive testing.

ii. **Structural performance models:** Individual types of damages are predicted by these models. This damages include fatigue cracking in flexible pavements and corner breaks in rigid pavements. Condition measures also can be predicted in terms of condition indices based on more than one damage types and severities. Structural performance models are normally Empirical and Mechanistic–Empirical models. In order to use this type of models, structural or other related informations are required.

iii. **Functional condition models:** Measure of the condition of the pavement with regard to its basic function is predicted by these models. Safety related prediction models are also available and they are used to predict characteristics such as the surface friction of the pavement based on skid numbers obtained through skid resistance testing. Though these models are empirical in nature, they may use material properties also.

iv. **Damage models:** and are a Normalized measure of distress, unevenness, surface friction etc. or other parameters of condition are derived from either structural or functional models. The integration of different measures of condition with different maximum and minimum levels into a single function can be achieved by using damage function.

2.4.2 Classification of pavement prediction models (Ferreira et al., 2002, 2004, Stephenson et al., 2004). The pavement performance prediction models are classified into different types as given below:

i. Deterministic and Probabilistic models:

Deterministic models can be sub divided into Mechanistic, Empirical and Mechanistic – empirical models. Mechanistic models are based on physical models. In empirical models, regression analysis is used to relate the estimated variables to the deterioration. The variables can be deflection, traffic, age etc.

In the case of mechanistic – empirical models, the calculated responses are used with other variables like traffic to relate to loss of serviceability or deterioration. The responses can be subgrade strain, stresses or strains of pavement layer etc.

Probabilistic models are represented by transition probability matrices, ie, with probabilities of transition between quality states of the pavement with or without application of rehabilitation and maintenance. They are purely empirical in nature.

ii. Network or project level models:

Pavement performance models are used to predict the future condition of pavements at network level. These are used to evaluate alternate pavement design strategies at project level to find the most cost effective solution for individual sections of the road network.

iii. Relative or Absolute models:

Future condition of the pavement based on measured condition data like bearing capacity, defects, roughness, skid resistance, cracking and rut depth etc. can be predicted by relative models. Relative models have usually only one independent variable, like age or traffic.

Absolute models include independent variables explaining the pavement deterioration process, like layer thickness, resilient modulus, asphalt characteristics, pavement response etc.

2.4.3 Techniques of Pavement Performance Models (PPM)

PPM's can be developed using different techniques such as Regression analysis, Bayesian methodology, Homogeneous Markov Process, Non Homogeneous Markov Process, Semi- Markov process etc. Bayesian methodology can be used in both relative and absolute models. ANN, Fuzzy systems and hybrid computing systems also can be used.

KENLAYER is a Computer Program that can be used to find out the damage ratio using distress models. Cracking and rutting are the distress models in KENLAYER and they are considered as the most critical elements for bituminous pavements. The horizontal tensile strain (E_t) at the bottom of the bituminous layer which causes fatigue cracking and the vertical compressive strain (E_c) on the surface of the subgrade which causes permanent deformation or rutting are the critical elements. The performance of the pavement can be predicted using pavement deterioration models in HDM -4. (Gedafa, 2006).

2.5 GLOBAL SCENARIO

Pavement prediction models have been used worldwide for Pavement Management System (PMS). In United States of America, Washington State Dept. of Highways developed the first PMS model in 1970's. Ohio Department of Transportation (ODOT) uses a deterministic prediction model for forecasting network condition. The state of Iowa has an Iowa pavement management program. This is characterized by the integration of a pavement performance modelling tool with a new pavement network optimization model which can be used for identifying and selecting cost effective projects for rehabilitation and maintenance.

Juang and Amirkhanian, (1992) documented the findings of a study carried out on the use of Pavement Management System (PMS) in the United States. A model using fuzzy logic for a PMS based on priority ranking was developed. An index called Unified Pavement Distress Index (UPDI) was also developed and this was used to measure the distress condition of the pavement. Guidelines for rating six types of distresses, weights among the different types of distresses, fuzzy set representations, fuzzy mathematics and the definition of UPDI and its use in pavement database were given by this approach.

Collop and Cebon, (1995) reported a whole-life performance model (WLPPM). This model is capable of making deterministic pavement damage predictions resulting from realistic traffic and environmental loading. Realistic predictions of pavement degradation with traffic has been obtained by taking into account most of the primary factors of vehicle/pavement interaction. Simulation by WLPPM shows that short- wave length surface – roughness components can be smoothed out, and traffic loading increases the amplitude of long wave length components.

The PMS of the Nevada Department of Transportation developed a total of 16 PPMs to cover the most frequent rehabilitation and maintenance actions used in all Nevada districts. The aims of these models are to predict the performance of flexible pavement sections under the combined influence of traffic and environment. These models use traffic, environment, materials and hot-mix asphalt data along with actual performance data, which is measured by the PSI, for predicting the long-term performance of pavement sections with application of rehabilitation and maintenance programs. In the Nevada PMS, a typical performance model for bituminous concrete overlays was formulated (Sebaaly *et al*. 1996, 1999).

The effects of pavement characteristics on condition of the street network in Irbid City in Northern Jordan was evaluated using the concept of Pavement Condition Index (PCI) and it showed that Alligator cracking, rutting, depression and swell distresses were the most frequent distress types that caused the pavement deterioration. Pavement age, traffic level and pavement thickness were found to be highly significant and affect the pavement condition to a great extent. Some of the asphalt mix properties such as air voids, bulk specific gravity and bitumen contents were found to have small effect on pavement condition. Pavement section of low air voids in the asphalt mix suffered from distortion and cracking due to the small resistance to compaction under traffic (Turki et al., 1996).

Morosick et al., 2000 has reported calibration of HDM-4 relationships for cracking, rutting and roughness against observed rates of deterioration of inter-urban roads in West Java. The detailed models were found to be successful at using the extent of defects on under designed or poorly constructed roads. The calibration factor in the detailed model was close to unity, with a value of 1.3 for roads with heavy traffic and 1.0 for roads with light to medium traffic. The value of K for the aggregate model had to be increased approximately five fold to compensate for lack of distress terms *ie,* 5.3 for roads with heavy traffic and 5.5 for the roads with lighter traffic. The paper highlights that if the issues related to institutional data, engineering and systems are resolved and major reinvention is applied, it can be turned into opportunities which will substantially strengthen the pavement.

For use in mild (Mediterranean) climates and operating conditions, innovative performance models were developed, for asphaltic pavements based on HDM-4 (Andreas et al., 2002). Official HDM-4 models for cracking were compared with models in terms of a study for crack modeling using Greek SHRP LTPP data. Overall structural adequacy within the model was considered essential to simulate the degradation effects on pavement strength and capacity of condition, climate and traffic. Structural adequacy relationship between SNP and residual capacity (8.2 MSA) was formulated. The Portuguese Road Administration, called Junta Autonoma de Estradas initiated the implementation of a PMS in 1990, considering a probabilistic PPM (Golabi and Pereira 2003).

An appraisal of the pavement management system - its definition, need frame work, what a PMS can do, categories and application around the world was outlined with global PMS scenario in 26 countries like USA, Canada, Australia, UK, France, Germany, Denmark, New Zealand, Sweden, Austria, Norway, Italy, Belgium etc. Efforts needed to enhance the pavement management process were also indicated (Agarwal et al., 2002, MORTH, 2004). A preliminary calibration of HDM-4 was done for the North South Expressway in Malaysia (Chai, 2004). The factors considered in this were roughness, age and environmental data.

In Canadian PMS, integration of the pavement database with other road system was facilitated using GIS, and the structural evaluation is deflection based. The Australian

Road Research Board uses a PMS using GPS and GIS. The United Kingdom PMS uses data on pavement condition using a common reference system. The French Directorate of Road also assesses the condition of the pavement for its PMS. Germany developed methods for forecasting the behavior of flexible pavements considering the condition and climatic influences. Denmark uses Dynatest Pavement Maintenance and Rehabilitation Management System, which is capable of predicting future pavement condition at project level and network level.

New Zealand National PMS uses a hybrid set of predictive models from HDM III & 4. Sweden PMS produces objective information as decision support system. Austrian PMS uses pavement condition module, which predicts surface defects, cracking, rutting, roughness and skid resistance. The Norwegian Public Road Administration PMS uses prediction models that are simple and based on historically measured data for each road section. PMS for Italy uses high performance equipment to measure the physical parameters.

The Belgium PMS uses longitudinal unevenness, rut depth, skid resistance and bearing capacity with different prediction models and statistical approach. South Africa also uses PMS, which adopts most cost – effective rehabilitation strategy incorporating performance prediction models. Brazil uses HDM- III model to simulate life cycle performance.

Chile has implemented PMS, which gives pavement condition, prediction of future behavior and economical evaluation as output. Other countries which have successfully implemented PMS based on pavement performance evaluation are China, Japan (GIS based), Korea, Thailand, Indonesia, Malaysia, Pakistan and Kuwait.

The Portuguese Road Administration decided to implement a new PMS considering deterministic PPMs in 2003. (Picado-Santos and Ferreira 2007, Estradas de and Portugal, 2008). The methodology adopted was to analyze a large set of PPMs used in PMS of road administrations around world, select the most accurate ones for further evaluation and finally to select one for implementation. The important models analyzed were the models used in the Highway Development and Management System (HDM-4), the 1993 AASHTO pavement design method, the PMS of the Nevada Department of Transportation, the Collop - Cebon whole-life PPM (WLPPM) used in New Zealand and the Swedish and Spanish Road Administrations PMS (Ferreira *et al.* 2008).

Nasir et.al., (2010) compared six pavement condition indices from five DOT's in the United States, using the distress and ride quality data obtained from the Pavement Management Information System of the Texas Department of Transportation. The computed scores were compared visually using scatter plots and statistically using paired t –test. The results showed significant differences among seemingly similar pavement condition indexes.

Ferreira et al. (2011) compared the reported pavement performance models (PPM'S) to recommend the ones for use in the Portuguese Pavement Management Systems (PMS). The models analyzed were the HDM, AASHTO, the Nevada PMS, the Collop- Cebon whole-life pavement performance, the Swedish PMS and the Spanish PMS. The study recommended AASHTO model for an initial phase of implementation of the Portuguese PMS. The same model was used in an LCCA model in the USA (Chen and Flintsch, 2007).

A summary of the work conducted in the Nord FoU Projects (Rabbera, 2011), reported that the project was divided into two parts; project level models and network level models. Evaluation, choice and calibration of relevant performance prediction models to Nordic conditions were done. The models were calibrated using data from test sections of the Swedish long-term pavement performance program (LTPP). The work conducted in both the project level and the network level parts of the project are reported. HDM-4 models were selected and calibrated to Nordic conditions at the network level. Calculations of pavement condition using HDM-4 models were done using MATLAB application. Separate material models were developed for bound (bituminous) and unbound (granular layers) at project level.

Gary et al., (2011) have made a study of the road network condition and pavement management system of the city of Abbotsford and found that the condition of the major road network in 2004, in terms of cracking as a percentage of surface area was on an average of 7.7%. It is also reported that the average International Roughness Index (IRI) was 2.1 mm/m. The GHG reduction due to the application of PMS was significant by adopting $ 3.8 million funding scenario and has now reached 1,300 tons and could total to more than 17,000 tonnes over 20 years.

Owolabi et al. (2012) conducted comprehensive investigations and developed performance models for a typical flexible road pavement in Nigeria. Data on traffic characteristics, pavement condition ratings, distress types, pavement thickness, roughness index, rainfall and temperature were collected to develop models to pavement condition score (PCS) and international Roughness index (IRI). The models were validated which showed that they can predict the deterioration of pavements with reasonable accuracy and can be used to update pavement condition data prior to each maintenance program.

2.6 PAVEMENT PERFORMANCE MODEL BY AASHTO

The AASHTO pavement design method (AASHTO 1993) for flexible pavements is the most commonly used method, not only in North America but also in the world (C-SHRP, 2002). In this method, several factors are applied in the model. The change in present serviceability index (PSI) over the design period is one such factor. In AASHTO pavement design method, the material properties, predicted equivalent single axle load (80 kN), drainage and performance reliability are used to indicate the strength of the flexible pavements in terms of an index called Structural Number (SN). The value of SN is then modified incorporating the structural coefficients of the layers, which represent the relative strength of the layer materials and pavement layer thicknesses. The basic AASHTO model for flexible pavement is given by (AASHTO 1993):

$$\log_{10}(W_{18}) = Z_R S_O + 9.361\log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32\log_{10}(M_R) - 8.07 \qquad \text{---------------} \quad (2.1)$$

and $$SN = \sum C^e{}_n C^d{}_n H_n, \qquad \text{---------------} \quad (2.2)$$

where, W_{18} is the number of 18-kip (80 kN) ESAL applications estimated for a selected design period and design lane,

S_0 denotes the combined standard error of the traffic prediction and performance prediction,

Z_R is the standard normal deviate, ΔPSI is the difference between the initial or PSI_0 and the terminal serviceability index (PSI_t,),

SN is the structural number which represents the total pavement thickness required as given by Equation (2.2),

M_R is the sub-grade resilient modulus (pounds per square inch),

C^d_n is the drainage coefficient of layer n

H_n is the thickness of layer n (inch) and

C^e_n is the layer (structural) coefficient of layer n,

Equation (2.1) can be converted into Equation (2.3) for using the same to predict the PSI value for every year of the planning period.

$$PSI_t = PSI_o - (4.2 - 1.5) \times 10^{[\log_{10}(W_{18}) - Z_R S_o - 9.361 \log_{10}(SN+1) + 0.2 - 2.32 \log_{10}(M_R) + 8.07][0.4 + \frac{1094}{(SN+1)^{5.19}}]} \quad \text{---------------} \quad (2.3)$$

Since the AASHTO model was based on PSI and a quality index was already in use in new Portugese PMS for finding the pavement quality, this PPM was used for testing also in Portugese PMS. In the case of a change in load applications represented as $((\Delta W_{80})_{t}$-$1,_{t})$ corresponding change in the PSI is $((\Delta PSI_t$-$1,_t)$ for a change in service time $(\Delta T_{t-1,t})$. The PSI in year t represented as (PSI_t,) can be denoted as the difference between the serviceability index in year t - 1 (PSI_{t-1}) and the incremental change in the PSI ($\Delta PSI_{t-1,t}$). PSI can be defined as the difference between the initial serviceability index (PSI_0) and the total incremental change in the PSI (ΔPSI_{0t}). The range of PSI_t, lies between its initial value of about 4.5 (value for a new pavement) and the AASHTO lowest allowed PSI value which depends on the road class.

Modified Structural Number (SNC) or Adjusted Structural Number (SNP) was used to define the structural capacities of various pavements. Modified Structural Number is the linear combination of the strength co-efficients of layers a_i and thickness H_i of the individual layers above the subgrade with a factor from the subgrade soil and is denoted as SNSG for the HDM model (Paterson, 1987 and Watantada, 1987). The modified structural number over predicted the capacity of pavements with thickness over 700 mm and hence adjusted structural number was used in HDM-4. SNP applies a weighting factor, which reduces with increasing depth, to sub base and sub grade contributions so that the strength for deep pavements are not over predicted. (Graham Salt et al., 2012).

Paterson (1987), found that the BBD readings gave an approximate relationship between central deflection and SNC of the form:

$SNC = 3.2\ D_B^{-0.63}$, if the base is unbound, and --------------- (2.4)

$SNC = 2.2\ D_B^{-063}$, if the base is bound, --------------- (2.5)

where, D_B is the BBD reading in mm.

Paterson's data was limited to deflections generally less than 2 mm. Based on further study, a relatively poor correlation was developed for SNP of the form;

$SNP = 3.2\ D_B^{-0.5}$, $(R^2 = 0.56)$ --------------- (2.6)

Ralph et al. (2007) presented two cases of implementing an M-E design procedure. In the first case, a simple, equivalent two layer elastic model was used. In this, the calibration was done in two stages. The first was dependent on extensive field data on performance, and the second was based on the design estimates of an expert panel, in a matrix of factor combinations. In the second case, AASHTO new Mechanistic-Empirical Pavement Design Guide (MEPDG) was used where the hierarchical levels of design inputs were first described. A summary description of a recent sensitivity analysis of the input factors in the rutting and fatigue models were then described. The paper suggested some opportunities and challenges, which can be a comparative sensitivity or interactions of factors for analysis for MEPDG. Calibration of the models and validation of results and dissemination are other challenges.

2.7 HIGHWAY DEVELOPMENT AND MANAGEMENT MODELS:

The Highway Development and Management model (HDM-4) is the successor to the World Bank Highway Design and Maintenance Standards model (HDM-III). This is being used by many road agencies since last 20 years. The preliminary observation in using HDM-4 is that the PPMs should be calibrated to correspond to the observed rates of deterioration of the road sections where the models are applied. Several applications were reported in the use of HDM.

The calibration and adaptation aspects of HDM Road Deterioration and Maintenance Effect (RDME) relationship for Indian conditions were done in 1995. (Chakrabarti et al., 1995). The deterioration factors were derived for the pavement types and traffic loading levels appropriate for the country. The study reported that DDM RDME is robust, yet flexible enough to predict the deterioration for roads in the country with the pavement deterioration factors very close to default values. Pavement management system for urban roads by using the data on road inventory, functional evaluation, structural evaluation and traffic from 12 roads in Delhi was developed using the HDM –III, duly adopted to Indian conditions and was used for Life Cycle costing (Jain et al., 1998). From this study, it was found that the pavement management system provides a rational basis and unbiased prioritization of roads for maintenance at appropriate time within the available funds. Also a new ‘Condition Responsive-cum-Scheduled Maintenance methodology was proposed. Besides prioritization, the PMS provides the availability of timely information with respect to pavement condition, maintenance, rehabilitation, reconstruction actions and cost for each section. (Mrawira. *et al.*,1999, Odoki and Akena, 2008).

It is reported that HDM-4 was developed as a support system to help in decision making for administrators and highway engineers for predicting the economic, social and environmental effects that might occur when implementing road improvement projects (PIARC, 2000, Jain *et al.*, 2004, 2005, Parida *et al.*, 2005).

Roy et al., (2003) conducted a study to calibrate HDM-4 Road deterioration and Work Effect models to Indian conditions, by comparing the HDM–4 models with the Pavement Performance Study (PPS) models, developed for Indian conditions. The calibration factors were developed for two types of pavement surfacing, viz, Bituminous Concrete (BC) and Premix Carpet (PC) for varying Cumulative Standard Axle loads (CSA) and Modified Structural Numbers (MSN). Regression equations were developed, relating the calibration factors and the CSA and MSN values.

Studies were conducted by Jain et al., (2004, 2005) and developed a Pavement Management System (PMS) for NH 58, 72, 72A, 73 and 74 located in Uttaranchal state for a sub network of 65 km long. The pavement deterioration models available in HDM-4 were calibrated for local conditions to find out the calibration factors. The selected segments were confined within Uttaranchal State and U.P. Models were developed for cracking, ravelling and roughness progressions. The road segments were chosen from different types of pavement sections, with varying terrain and climatic conditions, pavement compositions and traffic condition. Pavement deterioration models were developed and two alternative Maintenance Rehabilitation strategies were defined for each pavement section. HDM-4 model was used for predicting the life cycle costing & optimization. The network level pavement management analysis was done using the 'Program Analysis' application module of HDM-4. Attempt were also made to calibrate the HDM–4 pavement deterioration models for the National Highway network located in UP and Uttaranchal states of India. The study observed a variation of 10.8 to 28.2% for cracking area, 15.4 to 39.8% for raveling area, and 0 to 66% for pothole area, but the variability obtained for roughness was from 2.1 to 15.1%. The pavement deterioration model relationships were derived from 145 pavement sections located along National Highways (NH) and State Highways (SH) in the Indian States of Gujarat (44 sections), Haryana (17 Sections), Rajasthan (47 Sections) and Uttar Pradesh (37 Sections). Results showed that crack initiation begin 2.3 times earlier, raveling initiation 2.7 times earlier and pothole formation 2.2 times earlier in Indian conditions. Rate of progression of raveling was almost half (K_{vp} = 0.52), rate of progression of potholing (K_{pp} = 0.95) was slow by 5%, that of roughness (K_{gp} = 0.85) was slow by 15 percent, and that of cracking (Kcpa = 1.25) was 25% faster.

Singh et al. (2005) did the program analysis using HDM–4 and carried out the life cycle analysis for a period of 15 years. The budget requirement for unconstrained program was obtained. The optimized program was developed for three scenarios namely 75, 50 and 35 percent of the required budget, respectively. HDM-4 contains PPM's for major pavement distresses like cracking, potholes, rutting and roughness. The initiation phase is separated from the progression phase, and developed the cracking initiation and progression models. The pavement condition and change in condition were predicted every year for each mode of distress in the order given below (Attoh-Okine and Paris, 2005)**:**

a) The age for initiation of all cracking and increment in area of all cracking.

b) Initiation and increment in the area of all potholes,

c) Increment in rut depth (mean and standard deviation), and

d) Increment in roughness,

These PPMs were developed from the results of extensive field experiments conducted in different conditions.

HDM–4 computer software was used by Daba (2006), for predicting the performance by dividing into two parts; project level models and network level models. The work involved evaluation, choice and calibration of relevant performance prediction models to pavement deterioration models. Out of the eight deterioration models, cracking model was found to be governing. Studies reported that life of pavement predicted by HDM-4 was less than the life predicted by the software KENLAYER.

HDM-4 analysis showed that among the various alternatives recommended in Government of India specifications, the maintenance treatment with 25 mm SDBC was optimum for urban roads and average periodicity interval required for renewing the road surface was 2 years (Sudhakar et al., 2009).

2.8 PAVEMENT PERFORMANCE MODELS IN THE INDIAN PERSPECTIVE

The most commonly used models are HDM- 4 and AASHTO performance models**.** The performance models that are developed for Indian perspective are briefly reviewed here to compare the same with the models developed in the present study. These are categorized under three groups, considering the attributes that are related. These are:

i. **Distress Characteristics Based Models:** These models predict the information on roughness, rut depth, raveling, potholes etc. being developed as a result of traffic factor and age.

ii. **Pavement Performance Rating Models:** These models define the performance of the pavement using certain arbitrary or weighted values. These values varies within a certain range. Different researchers have proposed various indices. These include PSI (Present Serviceability Index), PCI (Pavement Condition Index), PCR (Pavement Condition Rating), etc.

iii. Models Based on Environmental Factors: These models consider the effect of various environmental factors like temperature of soil, pavement layers and surroundings, freeze and thaw cycles, humidity and precipitation, movement of ground water, capillary water or surface water etc. on the performance of the pavements.

In addition to the above classifications, some researchers worked on the application of computing techniques and methods for predicting the performance of pavements. These are also discussed here.

2.8.1 Distress characteristics based models

CRRI (1994) reported deterioration models for cracking, raveling and potholing as given in **Table 2.1**

Table 2.1 Pavement Deterioration Models developed by CRRI (1994)

Sl. No	Model Description	Pavement Deterioration Models for PC Surfacing
1	Cracking Initiation	$\mathrm{AGECRIN} = 2.74 * \mathrm{EXP}\left[-2.57 * \frac{\mathrm{CSALYR}}{\mathrm{MSN}^2}\right]$
2	Cracking Progression	$\frac{\Delta CR_t}{t_i} = 5.41 * \left[\frac{CSALYR}{MSN}\right]^{0.54} SCR_i^{0.28}$
3	Ravelling Initiation	$\mathrm{AGERVIN} = 3.18 * \mathrm{AXLEYR}^{-0.138} * (\mathrm{CQ}+1)^{-0.38}$
4	Ravelling Progression	$\frac{\Delta RV_t}{t_i} = 3.94 * \mathrm{AXLEYR}^{0.32} * SRV_i^{0.46}$
5	Potholing Initiation	$\mathrm{AGEPHIN} = 0.21 * \mathrm{THBM}^{0.23} * \mathrm{EXP}\,[-0.18\ \mathrm{AXLEYR}]$
6	Potholing Progression	$\frac{\Delta \mathrm{PH_t}}{\mathrm{t_i}} = 1.49 \frac{\mathrm{CR_i} * \mathrm{AXLEYR}\,(1+\mathrm{CQ})}{\mathrm{THMB} * \mathrm{MSN}} + 3.60 * \mathrm{PH_i} * \mathrm{AXLEYR}\,(1+\mathrm{CQ}) + 3.47 \frac{\mathrm{RV_i} * \mathrm{AXLEYR}\,(1+\mathrm{CQ})}{\mathrm{THMB} * \mathrm{MSN}}$
7	Roughness Progression	$\Delta RG_t = [58121\,(\Delta CSAL/SNCK^5)] * EXP\,(mPAGE) + [4.13\,\Delta\, CR_t] + [184.48\,\Delta\, PH_t] + [33.46\,\Delta PT_t] + [m\,RG_i * t_i] + [9.39\,\Delta RV_t]$

where,

SCRi	= Min. {Cri, (100-Cri)};
SCVi	= Min. {RVi, (100-Cri)};
AGECRIN	= Age of pavement at the time of crackin g initiation in years
AGERVIN	= Age of pavement at the time of raveling initiation in years
AGEPHIN	= Age of pavement at the time of pothole initiation in years
AXLEYR	= Number of vehicle axle per year in millions
E_Z	= Vertical strain on the subgrade
ΔCR_t	= Percent change in cracked area over time 't' in years
$\Delta CSAL$	= Change in cumulative standard axles (msa) over time 't' in years
ΔPH_t	= Percent change in pothole area over time 't' in years.
ΔPT_t	= Percent change in patched area over time 't' in years
ΔRG_t	= Change in roughness over time 't' in years (mm/km)
ΔRV_t	= Percent change in raveled area over time 't' in years
CSALYR	= Cumulative standard axles per year (msa)
PH_i	= Initial Pothole area (%)
PH_t	= Pothole area (%) at time t

Das and Pandey (1999) reported to have developed a mechanistic design method by correlating the performance data of bituminous pavements from different parts of India to the critical stress-strain parameters of the pavement composition leading to the failure of pavements and developed relationships with axle loading as given below:

For AC, $N_f = 1.001*10^{-1}\,[1/\varepsilon_t]^{3.565}[1/M_R]^{1.4747}$ --------------- (2.7)

For BM, $N_f = 2.26*10^{-2}\,[1/\varepsilon_t]^{3.565}[1/M_R]^{1.4747}$ -------------- (2.8)

where,

N_f = Number of cumulative standard axle repetitions to produce 25 percent surface crack due to flexural fatigue on in-service pavement

MR or M_R = Resilient modulus

Reddy et al. (1999) developed Deflection progression models and cracking models for Indian conditions and are given in **Tables 2.2 and 2. 3**

Table 2.2 Deflection Growth Model

iDEF range (mm)	Model form
0.44 <iDEF< 0.61	D_t = iDEF + 0.07884 [(N_t*Age)^{iDEF}]
0.66 <iDEF< 0.8	D_t = iDEF + 0.0027 exp [(iDEF*N_t)^{iDEF}] + 0.0859 (Age)
0.84 <iDEF<1.05	D_t = iDEF + 0.04513(expN_t)$^{0.45}$+ 0.0924 (exp Age)^{logiDEF}
1.10 <iDEF< 1.25	D_t = iDEF + 0.03658 [exp (iDEF*N_t)]0.5 + 0.19864 (Age)$^{0.26}$

Table 2.3 Cracking Models

Model Type	Model Form
CRACK AREA	(BM+PMC) surfaced pavement C_t = 1.8 [log N_t + 0.115 (iDEF* N_t)$^{1.48}$]
	(BM+AC) surfaced pavement C_t = 3.49 [(iDEF* N_t)$^{0.34}$ + 3.24* 10 -5* exp (N_t)]
CRACK AREA PROGRESSION	(BM+PMC) surfaced pavement CA_t = iCA [1 + 0.744 (iDEF*N_t)*0.32 + 0.0054* exp (N_t)]
	(BM+AC) surfaced pavement CA_t = iCA [1 + 1.49 (iDEF*N_t)*0.15 + 0.00547* exp (N_t)]

where,

N_t = Cumulative standard axle in millions at time 't',

DEF = Deflection

Reddy and Veeraragavan (1999) studied the effect of overloading and introduction of tandem axle trucks on pavement life. The deflection growth model was developed using the historical data from six overlaid flexible pavement sub stretches with different initial deflection values. The model is of the following form:

$$DEF_t = 0.2386 \exp [CSA^{0.1713}] + 1.2445 (DEF_0) - 0.6071 \qquad \text{----------} \quad (2.9)$$

Roy et al. (2003) suggested regression equations as given in **Table 2.4** for cracking initiation and progressions and roughness progressions for premix carpet surfaced pavements with a relationship between calibration factors, CSA, MSN and AXELYR.

Table 2.4 Cracking and Roughness Progression models for P C

Sl. No	Name	Model
1	Cracking Initiation	a_0 = 0.04124*CSA – 0.03059* MSN + 0.461
2	Cracking Progression	a_1 = 0.229*CSA – 0.07129* MSN + 0.537
3	Cracking Retardation Age	a_2 = 0.02825*CSA + 0.04123* MSN + 2.722
4	Roughness Environmental Coefficient	a_3 = -0.0176*AXELYR + 0.0460* MSN + 2.722
5	Roughness Progression	a_4 = -0.106*AXELYR + 0.481* MSN + 2.674

where,

CSA = Cumulative standard axle in millions at time 't'

Jain et al. (1992) analyzed the data of nine test sections of overlaid flexible pavements located in the States of Uttar Pradesh and Himachal Pradesh. The performance and life of the overlays was assessed on the basis of acceptable limits for deflection, rut depth, cracks and cracking pattern and maintenance cost. Models were also incorporated for the choice of type and thickness of materials for overlays on different sub grade soils economically without sacrificing the safety of road structure.

Models developed in these studies are capable of predicting the life of an overlay for given values of pavement thickness. The general model included wide variation of climate, terrain, rainfall and temperature. The performance of twelve test sections of overlaid flexible pavement located in the states of Himachal Pradesh and Uttar Pradesh was studied by Jain et al. (1996). The influencing parameters considered were deflection, roughness, rutting, cracking and potholes. The availability of resources for the choice of the type and thickness of materials for overlay and the parameters were recorded for the year 1994 and 2003 whose comparison showed the deterioration of flexible pavements with passage of time.

Sood and Sharma (1996) reported a pavement Performance Study conducted with a view to develop data for total transportation cost model for Indian conditions, to be achieved through development of pavement performance data and attempted development of layer equivalence and strength coefficients. Data was collected on the construction and maintenance inputs of different pavements based on studies carried out on nine pavement sections for a period of about 10 years. Models were developed for cracking, cracking progression, raveling, potholes and roughness progression. Validations of models were done based on limited fieldwork.

Sharma and Pandey (1997) made a study on the existing pavements completed in recent years and developed total transportation cost model for Indian conditions. Indian research results were used to develop this model and its predictions and results were considered truly reflective of Indian conditions. The model helps to apply a rational approach in road maintenance decisions for obtaining best results from available funds including benefits of periodic maintenance, cost effectiveness of maintenance strategies etc.

Reddy et al. (l999) collected extensive data on performance of in service flexible highway pavements on NH & SH over a period of 10 years for development of indigenous deterioration models. The deterioration prediction models were validated and used to predict the performance of different highway pavements during their design life. Computer programs were developed which can be used by the practicing engineers to select the best overlay strategy, among different combinations of overlay materials and thickness, duly considering life cycle cost and design life. The models were applied to predict the remaining service life of existing highway pavements based on different pavement performance indicators. The program to evaluate the best overlay strategy based on the developed deterioration models were applied to a project level management of NH section in Tamil Nadu.

Labi et al. (2009) investigated the economic viability of different levels of lifecycle cost with preventive maintenance for three pavements constructed with asphaltic concrete for cost effectiveness. Statistical analysis was done to develop models indicating the relationship between life cycle preventive maintenance and its applicability in extending the pavement life. This study showed cost effectiveness increases with more investment in preventive maintenance. The paper also gives a general approach to estimate the anticipated changes in service life pavements for various preventive maintenance activities.

A decision support system for multi – year maintenance program for NH section was developed by Muralikrishna and Veeraragavan (2011). The study evolved roughness and deflection progression equations and estimated the remaining service life of the pavement by considering different trigger levels as performance indicators. Routine maintenance, preventive maintenance and corrective maintenance treatments were considered as maintenance alternatives. Roughness and deflection progression equations were developed using SPSS. Road user cost models were used to compute the optimal timing of alternative maintenance strategies. They have carried out a Life Cycle Cost analysis to compute the optimal maintenance option.

Kumar (2012) developed a methodology for assessment of pavement performance. Seven pavement distress models for urban mains were reported for Block cracking, longitudinal and Transverse cracking, Patching, Pothole, Depression, Weathering and Raveling. The pavement prediction model developed could predict the deterioration of pavements, priorities of repair, prediction of time of maintenance or rehabilitation and also the estimate of the finds for repair. In order to validate the model, an application on a secondary road in India was done. The results of the simulation showed that work productivity is extremely important to the optimal level of investment.

2.8.2 Pavement Performance Rating Models

Different pavement performance rating models were reported by researchers over times, which are represented in terms of Pavement Serviceability Index (PSI), Present Serviceability Rating (PSR) and Performance Index (PI). The relationship between PSI and typical distresses observed on flexible pavements during the AASHO Road Test is given in the equation given below:

$$PSI = 5.03 - 1.91 * \log(1+SV) - 1.38\, RD^2 - 0.01\sqrt{C + P} \qquad \text{------------ (2.10)}$$

Garcia et al. (2000), summarized the performance of pavements in terms of three indices viz (i) Present Serviceability index, (ii) Distress area index and (iii) Distress severity index. The model predicts more realistic long-term behaviour, which is an improvement over the original AASHO Road Test performance equation. This could be achieved by the use of a Sigmoidal or S - Shaped curve which can recognize the capacity of the pavement to reduce the rate of deterioration when the traffic level approaches the end of the pavement service life.

Krishna Murthy (1991) proposed a model, which represents Unevenness Index (UI) (in cm/km) in terms of Pavement Serviceability Index (PSI) values.

$$PSI = 315\,(UI)^{-0.822} \qquad \text{--------- (2.11)}$$

For calculating UI, the relationship used was: $I = B/W*R*2.54$ -------- (2.12)

Ramesh et al. (1999) collected extensive field data to develop the performance models and in the determinations of life cycle cost. A computer program was developed to calculate the initial cost, first stage strengthening cost, user delay cost and salvage value. The program has the capability to compute the life cycle cost for any design period and for any number of sections by varying the threshold Present Serviceability Index (PSI) value which is on a scale of 1 to 10. The budget scenario can also be varied and the effect of budget level on the performance of the pavement can be studied. A relationship was developed between the present serviceability index values on a 10-point scale, cracking in percentage, rut depth in mm and unevenness index in cm/km. The developed equation is given by:

$$PSI = 14.79 - 0.029\,(C_t) - 0.0086\,(RD)^{1.85} - 0.642\,(UI)^{0.4} \qquad \text{----------- (2.13)}$$

The ranges of values considered were, Ride Rating- 4 to 9.5 (10 point scale), Unevenness Index – 150 to 860 cm/km, Rut Depth – 2 to 15 mm, Cracking – 0 to 25 percent.

An unevenness progression model was developed which considered the initial unevenness value and a structural condition parameter viz., deflection. The unevenness progression model is given as follows:

$$UI_t = iUI\,[1+0.3012\,(CSA^*\,Def)^{0.08\,age}] \qquad \text{-----------} \quad (2.14)$$

Reddy et al. (1999) presented models for functional condition deterioration as Serviceability Rating (PSR) model and structural condition as Unevenness progression model in terms of rebound deflection. The models are as given in Table **2.5.**

Table 2.5 Functional condition Deterioration Models

Model Type	Model Form
PSR Model	$PSR_1 = 14.3765 - 1.932 \log (UI_1)$
UI Growth Model	$UI_t = iUI[1+0.065187(N_t)^{1.22} + (DEF_0)^{0.61} *^{AGE}]$

Veeraragavan et al. (2003), developed Unevenness progression model of the form,

$$UI_t = UI_0\,[1+ 0.065187^*\,(CSA_t)^{1.22} \;\; +0.184261^*Def^*Age)^{0.61}] \qquad \text{------------} \quad (2.15)$$

PCI based pavement condition prediction models for a family of rural roads were developed by Verma (2006) as given below in **Table 2.6.**

Table 2.6 PCI Based Pavement Condition Prediction Models

Sl. No	Terrain Type	Suggested Model	Model Equation
1	Plain, 0-15 CVPD	Linear	PCI = -0.4039* Age+103.46
2	Plain, 15-45 CVPD	Linear	PCI = -0.2875* Age+100.19
3	Plain, >45 CVPD	Linear	PCI= -0.2974 * Age+99.994
4	Rolling, 0-15 CVPD	Linear	PCI= -0.3051* Age+96.952

5	Rolling, 15-45 CVPD	Linear	PCI= -0.3107* Age+99.481
6	Rolling, >45 CVPD	Polynomial 2	PCI= 0.0007* Age^2-0.49* Age +104.29
7	Hilly, 0-15 CVPD	Linear	PCI= -0.3794* Age + 100.73
8	Hilly, 15-45 CVPD	Linear	PCI= -0.4277* Age + 101.83
9	Hilly, >45 CVPD	Linear	PCI= -0.341* Age +99.42

PSI progression models for a family of rural roads were developed by Verma (2006) as given below in **Table 2.7**

Table 2.7 PSI Based Pavement Serviceability Prediction Models

Sl. No	Terrain Type	Suggested Model	Model Equation
1	Plain, 0-15 CVPD	Polynomial 2 Degree	PSI= -0.0002*Age^2 +0.0111*Age+ 3.9448
2	Plain, 15-45	Linear	PSI= -0.0066* Age + 4.1207
3	Plain, > 45 CVPD	Power	PSI= 5.8672 * Age $^{-0.1699}$
4	Rolling, 0-15 CVPD	Linear	PSI= -0.005 * Age + 4.1655
5	Rolling, 15-45 CVPD	Linear	PSI= -0.0042 * Age + 3.895
6	Rolling,> 45 CVPD	Polynomial 2 Degree	PSI= -6E-5* Age^2 + 0.0024* Age+ 3.5808
7	Hilly, 0-15 CVPD	Polynomial 2 Degree	PSI= -0.0002 * Age^2+ 0.0058*Age +4.0289
8	Hilly, 15-45 CVPD	Linear	PSI= -0.005 * Age+ 3.9338
9	Hilly, > 45 CVPD	Linear	PSI= -0.004 * Age+ 3.7011

Sandra et al. (2008) developed relationship between PSI and noticeable distress parameters commonly observed on Indian roads as given below:

$$PSI_R = 0.00886\ RL + 0.00938\ RM + 0.01237\ RH + 0.00976\ PAL + 0.01063\ PAM + 0.01426\ PAH + 0.0239\ PL + 0.02979\ PM + 0.0325\ PH + 0.00686\ CL + 0.00786\ CM + 0.00986\ CH + 0.000974\ RUL + 0.001015\ RUM + 0.001815\ RUH \quad \text{------------} \quad (2.16)$$

where,

PSI_R = reduced PSI due to pavement distress parameters;

RL, RM and RH = low, medium and high severity raveling in % of area;

PAL, PAM and PAH = low, medium and high severity patching in % of area;

PL, PM and PH = low, medium and high severity potholes in % of area;

CL, CM and CH = low, medium and high severity cracking in % of area;

RUL, RUM and RUH = low, medium and high severity rutting in meters.

To find the PSI of pavement stretch, the reduced PSI (PSI_R) was deducted from the maximum possible PSI (PSIM) obtained from different functional classes of road.

$$PSI = PSI_M - PSI_R \quad \text{-----------------} \quad (2.17)$$

where,

PSI = Present Serviceability Index on a pavement at any point of time;

PSI_M = Maximum possible present serviceability index after construction.

(3.3 for National Highway, 3.0 for State Highway and 2.9 for Major District Road);

PSI_R = Reduced PSI due to pavement distress parameter

Reddy and Veeraragavan (1997) developed deterioration models for in-service flexible pavements in India and a deterministic model form is arrived as given under:

Future condition = f {Present condition, pavement strength, incremental traffic, age characteristics and climate}.

The performance of pavement was represented in terms of characteristic rebound deflection (average plus standard deviation) corrected for pavement temperature and subgrade soil moisture, Unevenness Index (UI) in mm/km and Riding Comfort Index (RCI) which is an average value of ride rating from the panel of raters, crack area (%) and rut depth (mm). Functional Condition models developed were Riding Comfort Index (RCI) Model and Unevenness Progression Model. Structural condition models developed were for growth of deflection, rut depth progression models and crack area progression models.

Reddy B and Veeraragavan (1998) reported a study conducted to develop data collection methods for pavement performance. The study established a sample size for the collection of (i) Benkelman Beam rebound deflection, (ii) Rut depth, (iii) Uneven index (UI) and (iv) Riding Comfort Index (RCI), so that the sample size could adequately predict the structural and functional condition of the pavement. Based on the study, it was concluded that total of 12 deflection readings and 10 rut depth readings for a 2-lane pavement stretch of 1 km length is minimum for structural ranking of pavements. The results of the study are applicable to estimate the sample size for the evaluation for homogeneous flexible pavements in a highway network.

Arya et al. (1999) developed economically judicious guidelines for maintenance operations of roads under different traffic, terrain and environmental conditions based on extensive field data collected on some experimental test sections and based on maintenance norms. Performance rating on test sections was done based on visual observation, unevenness index values and characteristic deflection. It was concluded that variables such as terrain, subgrade type and environments interacted and suppressed the eventual influence of structural deficiency of the pavement test section.

Hund and Bunker (2002) developed a method for calculating the roughness progression and displaying the same effectively with the effects of pavement maintenance. The paper concluded historical roughness progression of a pavement segment can be defined by linear regression and prediction of pavement roughness (LRPR) based on an extrapolation of the pavements'. LRPR could be a useful method of predicting roughness over a 5 year time frame.

Jain et al. (2000) conducted studies on pavement management system for rural roads. The study developed 'A Rational Approach for Low Cost Pavement Management System for Rural Roads' under which five roads were studied in the districts of Muzaffarnagar and Haridwar in Uttar Pradesh State. The functional evaluations of identified road sections were done by calculating present serviceability index (PSI) and the extent of cracking, rutting and patching. The study found that the poor condition of shoulders and side drains are primarily responsible for premature failure of pavements. It is hoped that the approach for low cost PMS for rural roads would be useful taking rational decisions for planning and safe design of maintenance activities to avoid premature failure and to preserve huge investment made on the construction of existing rural roads.

Narasimha et al. (2003) conducted various studies and field surveys on various categories of roads and data regarding the present serviceability conditions were collected for a chosen study area. The collected data was then analyzed using a subjective rating technique. The rating index thus obtained for each category of road in the chosen network was then used to find out the priority of maintenance. Depending upon the proximity and availability of data and ease with which data can be collected, a segment of road network located in Cuddalore sub-division, Tamilnadu was chosen as study area. From the studies carried out on the applicability of the subjective rating technique, it was observed that the above method is more versatile and adoptable for the field conditions and is a systematic rating technique for arriving at the pavement condition, which is useful for documenting the pavement characteristics and in preparing the budgetary estimates. It was concluded that the subjective rating technique is a rational tool in the hands of field engineers, for evolving a maintenance management for pavements, at network level.

Naidu et al. (2004) reported a maintenance management plan developed for the Inner Ring road of Delhi. The prevailing maintenance norms and deficiencies were discussed. Serviceability indicators for different levels of highways were presented. The performance and economic viability of the different maintenance alternatives at different intervention levels were evaluated at project level.

Reddy et al. (2005) developed a preservation frame work for flexible pavements in order to achieve an integrated management of the asset. Preservation needs were determined by establishing the Riding Comfort Index (RCI). The program integrates pavement condition data, pavement performance and its standards for generating a pavement preservation schedule.

Teiborlang (2011) reported distress studies that were carried out on the four rural roads of Assam, which were opened to traffic between January 2009 and January 2010 revealed that the PCI's of rating of these pavements at the time of studies as from Fair to Satisfactory. PCI of the section was calculated using the following formulae and the obtained value of 'n' is rounded to the next highest whole number.

$$n = Ns^2/\left((e^2/4)(N-l) + s^2\right) \qquad \text{------------------- (2.18)}$$

where,

e = acceptable error in estimating the section PCI

s = standard deviation of the PCI from one sample unit to another within the section (15 for bituminous pavement)

N = Total number of sample units in the section

The total severity and percentage density of the particular stress were calculated. From a plot between the distress and percentage density at any severity level, a deduct value was determined from the graph. The Highest Deduct Value (HDV) was identified and used to calculate the allowable number of deducts, m from the equation;

$$m = 1 + (9/98) \times (100-HDV) < 10 \qquad \text{------------------ (2.19)}$$

Corrected Deduct Value (CDV) was determined from a plot of DV & CDV. Maximum CDV was used to determine PCI.

2.8.3 Models based on Environmental factors

Several studies are reported about the influence of environment on different properties of pavements, but still no major work has been reported in India on development of performance models based on these factors. In the AASHTO method, modifications were done considering the environmental factors also.

Three major environmental factors that are of importance in the case of flexible pavements are given under:

- Temperature variations for bituminous concrete. Since the dynamic modulus of bituminous concrete mixture is very sensitive to temperature, temperature distributions in bituminous concrete layers are to be predicted and used to define the stiffness of the mixture in the sub layers. Thermal cracking prediction models also need temperature distributions as inputs.
- Moisture variation for sub grade. Optimum density and moisture content affect the resilient modulus of unbound materials. Based on the predicted moisture content, a correction factor is defined to modify the resilient modulus.

Sinha et al. (2007) studied the effect of high pavement temperature on the stability of mix in conjunction with lower softening point of bitumen in top pavement layers. The effect of high temperature of pavement layers on the behaviour of compacted bituminous mixes was the aim of the study. Rutting has been one of the commonly seen distress that is of permanent nature and was the parameter of this case study. The study was conducted on a typical rutted stretch, which form part of a four-lane road, which was widened and strengthened with thick bituminous layers.

Roy et al. (2009) correlated the CBR value of soil with compaction characteristics of the soils of CH, CI and CL group. A correlation is developed for predicting the soaked CBR values from compaction characteristics of optimum Moisture content and Maximum dry Density (Υ_{max}) involving two newly introduced parameters α & β. This can predict the value of soaked CBR values from OMC and MDD.

Syed et al. (2009) has presented a methodology for determining the relative influence of site factors and design on roughness development on in-service flexible pavements. Analysis was done to find out the effect of surface layer thickness of bituminous concrete, type and thickness of base and drainage on roughness growth of flexible pavements constructed in different site conditions and climate. It is reported that among the design factors, type of base has the most significant effect on roughness progression.

Grover et al. (2010) quantified the benefits of providing good drainage throughout the service life of the pavement. The effect of benefit – cut off values, vehicle damage factors and traffic growth rate were also analyzed for the maintenance treatment. It is reported that for the pavement section with poor drainage, the rate of deterioration of the functional parameters viz. roughness, cracking and raveling are faster by 41%, 20% and 25% respectively.

Gupta et al. (2011, 2012) carried out a review study of the pavement performance models developed for Highways, Rural Roads and low volume roads in India. It was found that most of these models are limited to particular conditions and environments. With the help of this review, one can evaluate the usefulness of various models in some particular condition having similar characteristics, pavement composition, soil type, climatic condition and terrain type. A brief discussion on the gaps and limitations of the different performance models are also given. The study showed that age is by far the most significant predictor of serviceability. The traffic volume and weight expressed in terms of equivalent single-axle loads (ESALS) and the structural make up of the pavement described by the composite structural number has got only a secondary role in the performance of pavements.

Nagakumar et al. (2000) critically reviewed the various factors that contribute to the stresses due to combined action of traffic factors and climatic factors. The use of heat transfer equations in the prediction of pavement temperature was also examined. The deflection spreadability decreases with temperature and the thermal gradient have a significant influence on deflection bowl parameters. A highly significant relation was found to exist between resilient modulus and moisture content and degree of saturation of subgrade. The relationship between number of repetitions of traffic wheel loads and strains both in the laboratory and in service pavements and certain aspects of prediction of pavement life were reviewed.

2.9 APPLICATION OF OTHER COMPUTING TECHNIQUES

Reddy and Veeraragavan (2001) developed a model for the network level management of flexible pavements. A priority-ranking module was developed which can provide a systematic procedure to prioritize road pavement sections for improvement and select suitable maintenance strategies depending upon the budget. This was based on priority index concept. Opinion survey and experience of experts were used to find out pavement distresses and weightages.

Satyakumar and Kumar (2004) has conducted a study for certain major roads in Thiruvananthapuram to develop a methodology for priority ranking using composite criteria. The road stretches were selected on the basis of user response. Functional evaluations of the stretches were done to determine the crack area, percentage of potholes, present serviceability rating and unevenness index. Based on the result of the functional evaluation, expert opinion was again collected to rank the pavements for priority maintenance. The mean rating of the expert was found out and compared with user opinion. From the study it was concluded that user opinion varies but not to large extent with the expert opinion. Considering the vast number of roads in our country user opinion can be used for initial ranking and thus narrowed down to the one, which require immediate attention, and experts can concentrate on the selected roads in detail and develop better strategy.

Reddy et al. (2004) highlighted in his paper that the IRC guidelines for strengthening of road pavements using Benkelman Beam technique are based on limited experience in India gathered during the sixties and seventies and has little relevance to the present day traffic and axle load. Though overlay design methods using falling Weight Deflectometer and Deflectograph are accurate and scientific, they are costly and require skilled manpower.

Freeman et al. (2004) proposed a method for predicting stresses in pavements under vehicular loadings. Each pavement layer was characterized by a co-efficient of lateral stress, which is similar to the commonly used co-efficient of lateral earth pressure (k). Several instrumented test sections were established and studied to determine coefficients of lateral stresses for common paving materials. A closed form solution based on the central limit theorem of probability is presented which can be used for predicting stresses within pavement structures. It is also applicable to multilayered structures.

A three-stage model was used to describe the primary, secondary and tertiary stages to establish the relationship between the number of load repetitions and permanent deformation. The algorithm can also be used for identifying the transition point between stages like flow number, and determine the model parameters from typical laboratory data (Fujie et al., 2004).

Uzhan (2004) reported a mechanical empirical framework for determining the permanent deformation in flexible pavements. The procedure uses rational material properties. It can be used as an analysis tool, for design also. This method is a compromise between simple and advanced approaches, between linear elasticity and finite element approaches. The proposed procedure uses the actual temperature distribution in the asphalt layer for every hour in the whole design period. It was found that increasing the thickness of the AC layer (with a stable material) for stiffening the pavement structure will lead to reduction in permanent deformation of the pavement.

Joseph et al. (2004) attempted to incorporate reliability aspect into pavement design procedure, and established different reliability levels for different levels of traffic intensities for a particular road.

Khaled et al. (2005) developed flexible pavement overlay design models by considering the performance reduction that has developed over a specified service period instead of performance curve parameters. The model generated can be an alternative method to the overlay design models that are in use and it can be applied up to a certain age of pavement, which is to be determined based on assessment of in situ pavement condition.

Bassam et al. (2005) examined the effect of quality of subgrade and base and base thickness on the mechanical response of conventional flexible pavement foundation to dynamic traffic loading by using a three dimensional, implicit dynamic finite element method, ADINA. From this study, it was found that quality of subgrade and base and thickness of base have remarkable impact on rutting strains but in the case of fatigue strains, subgrade quality has a little impact.

Bassam et al. (2005(2)) examined the dynamic response of conventional flexible pavement system to single wheel loads. This was done in terms of the pavement design criteria, ie, the fatigue strain at the bottom of the bituminous concrete layer and rutting strain at the top of the subgrade layer. Model set up with geometry, boundary conditions and characterization of load wave were presented. The effect of elasto-plasticity of the base material and elasto plasticity with strain hardening of the subgrade material on the dynamic response of the pavement system was studied. A parametric study was conducted to find out the effect of strength and thickness of the base and quality of the subgrade on the fatigue and rutting strains on the vertical deflection at the surface.

Bose et al. (2005) reported the studies conducted on premature distress and failure of bituminous pavements. Five case histories of failure of bituminous pavements due to moisture induced damage was reported as due to improper sub-surface drainage and stripping type of aggregate. Use of coarse graded and permeable granular sub base with fines less than 5% and addition of lime as filler in bituminous mixes reduce the premature failure due to moisture induced damage.

Mariaa et al. (2005) summarized a study done theoretically to have a better understanding of the effect of poor bond on the performance flexible pavements. Two approaches were used for modelling. In the first approach, analysis of the pavement structure was done using a layered linear elastic program which has considered the different degrees of interface bond between different layers of the pavement. A horizontal static load was applied in addition to the standard vertical dual load. From the analysis, it was concluded that poor bond between the base and binder course can reduce the life of the pavement structure up to 80%. Similarly, when the bond between the surface course and binder course is poor, the life of the pavement was found to be sensitive to any type of horizontal loading applied by the traffic. These results were confirmed by static linear and nonlinear 2D Finite Element modelling.

Mammen et al. (2005) determined the Pavement Classification Number (PCN) using CAN – PCN method. Statistical analysis of the PCN value was also done. It was concluded that evaluation by nondestructive testing measures moduli in each layer, which in turn leads to calculation of critical stresses or strains in each material and thus helps the designer to determine the effects of different rehabilitation measures.

Amara et al. (2006) reported the comparison of vertical compressive stresses and transverse horizontal strains, which were measured under the Hot Mix Asphalt (HMA) layer with stresses calculated theoretically, and strains obtained using soft ware based on layered elastic theory like Kenpave, Bisar 3.0 Elsym 5 and Everstress 5.0. From the study, it was found that the layered elastic theory overestimates the pavement responses at low and intermediate temperatures. But, at high temperature, the pavement responses to vehicular loading is underestimated significantly.

Masad et al. (2006**)** reported the study done to compare the measurement of deflection at the surface of the pavement under wheel loads to results obtained from finite element prediction based on models that incorporate isotropic and anisotropic properties for the unbound base and sub-base layers. It is concluded that the anisotropic behaviour of pavement layers cause part of the shift and calibration factors are to be used to relate laboratory measurements to field performance. But, when the pavement layers are modelled as isotropic, the tensile stresses that develop at the lower regions of the bituminous layer are substantially higher than those calculated.

Salama et al. (2006) carried out the analysis of the relative damaging effect of different configuration of truck traffic on flexible pavements in the development of cracking, rutting and roughness. Simple, multiple and stepwise regression were used. The analysis results showed that trucks with multiple axles produce more rutting damage than single and tandem axle and trucks with single and tandem axles causes more cracking than multiple axle trucks.

Sreedevi (2006) suggested a method for prioritization of 80 road links connecting to tourist destinations in Kerala by assigning weights to the pavement condition, tourism potential and importance of the connecting primary road. A priority index was reported.

Mathew et al. (2008) developed pavement deterioration model for PMGSY roads using Artificial Neural Network (ANN) and SPSS for eight roads. Artificial Neural Network models were developed for construction quality and drainage and compared with regression models developed using the λ^2 test. Models are developed for raveling initiation and progression, pothole progression, roughness progression and edge failure. The data collected included pavement age, pavement thickness, subgrade strength and severity of different stretches. ANN models were found to be more suitable for rural roads.

Ali et al. (2009) presented an analysis of rutting of urban pavements using finite element modeling, by considering the non-linear behavior of pavement materials and the complex traffic condition. Visco-plastic constitutive relation was used for the description of the behavior of the bituminous concrete layer, and elasto- plastic constitutive relation, based on the Mohr- Coulomb criterion was used for the other layers. It was reported in the study that urban traffic conditions like low speed and reduced wheel wander are detrimental for urban pavements with increase rutting and the use of high resistant bituminous concrete constitutes an efficient alternative for rehabilitation of urban pavements.

Augus et al. (2010) reported the pavement distress loss cost resulted from overloading. The loss cost of road pavement distress due to overloading is calculated based on damage factor (DF) and deficit design life (DDL). The loss cost the overload can user shall bear is 60% of total DFC (damage factor cost) and DDLC (deficit design life cost), considering that not all pavement structural distresses are absolutely caused by overloading freight transport; there are other factors such as low quality construction, low quality material, substandard maintenance and minimum design standard.

Gedafa et al. (2010) presented a methodology for the estimation of flexible pavements remaining service life by using the surface deflection data. Models were developed using nonlinear regression procedure in the Statistical Analysis Software and Solver in Microsoft Excel. This study reported a sigmoidal relationship between remaining service life and central deflection.

Kumar et al. (2010) developed deterioration prediction models for deflection and roughness of 17 road sections in Uttarakhand, using Artificial Neural Network (ANN) and linear regression. Pavement Serviceability Rating (PSR) and Riding Comfort Index (RCI) were worked out based on visual inspections of the test sections.

Kumar et al. (2010 (2)) carried out a study to assess the performance of rural roads constructed under PMGSY and develop pavement deterioration model by using artificial neural network and multiple linear regression analysis to forecast the pavement deterioration. Also compared the ability of this models for the prediction after a given time period. The regression analysis indicates that the factors like age of pavement, CBR, thickness of pavement are not significant as individual, in relationship with roughness.

Ranadive et al. (2010) studied the effect of variation in thickness of different component layers on performance of flexible pavement under applied load by axis- symmetric analysis using ANSYS. It was found that increase in thickness of base course and subbase course layer does not help to reduce stress and deflection, where as in the case of asphalt concrete layer there is substantial reduction in stress due to increase in thickness. The interface between AC layer and base course layer affects the distributions of stress in the lower layers of road structure. As stiffer materials are employed in upper layers, noticeable reduction in subgrade stress and deflection is observed.

Deepthi et al. (2013) reported the theory of perpetual pavements, which is gaining acceptance with increasing traffic demand. The authors suggested the use of Mechanistic – Empirical Design (MED) philosophy wherein limiting pavement responses are used to evaluate a proposed design. KENPAVE software is used.

Sha et al. (2013) presented a methodology for developing an Urban Pavement Maintenance Management System (UPMMS). The UPMMS procedure begins with network identification, and then carryout pavement evaluation followed by data analysis (deterioration modeling, life cycle cost analysis). Maintenance decisions and maintenance priorities are fixed. Supervision and follow up also form part of the system. For the development of UPMMS, HDM-4, Multi criteria Decision Making Techniques (MCDM), Artificial Neural Network (ANN), Analytical Hierarchy Process (AHP) and Analytical Network Process (ANP) are applied. Integration of the developed system with Geographical Information System (GIS) was done which ends with tailoring and decision support. This UPMMS will provide with a computer based tool to help the managers and pavement engineers of Municipal Corporations to manage their urban roads efficiently and effectively.

Jaritngam et al. (2001) compared the performance of flexible pavement using Finite Element Method (FEM) and KENLAYER (Huang, 1993). The study investigated the sensitivity of the variables in reducing the vertical deflections at the surface. The study can help the engineer to select the appropriate pavement structure. Comparisons made between FEM and KENLAYER indicate that the pavement deflection predicted by KENLAYER is less than that of FEM. It is also concluded that the subgrade modulus is the primary element, which affects the vertical surface deflections in flexible pavements.

2.10 PAVEMENT REHABILITATION TECHNIQUES

Pavements, both flexible and rigid should withstand harsh conditions and external influences. Wheel loads from moving vehicles cause strains, stresses and deflections both at the paved surface and at underlying layers. Since these loads are repetitive in nature, their application over an extended period of time will cause fatigue damage to the pavement. The pavement must withstand environmental effects also. The changes in climatic factors cause temperature variations throughout the depth of the pavement, thereby inducing internal stresses. Binder properties, especially the stiffness changes with temperature.

Variations in moisture of the pavement layers will cause changes in foundation support. Because of the induced damages due to traffic and environment, a pavement exhibits distress and requires rehabilitation during its service time. In order to prolong the service life of the pavement, different strengthening and rehabilitation techniques have been developed. If applied adequately at the appropriate time, these measures significantly extend the total life of the pavements.

2.10.1 Overlays

The technique of overlay is the strengthening of pavements done by providing additional thickness of the pavement of in one or more layers over the existing pavement. This technique is well established for over five decades both for functional and structural improvement. Sometimes composite pavement (concrete overlay on bituminous pavement or vice versa) is also adopted since it combines the advantages of both flexible and rigid pavements. Generally, a concrete overlay over a deteriorated bituminous pavement has a thickness of 100 mm and so.

The overlay thickness required for a flexible pavement can determined either by the conventional pavement design method or by conducting a non-destructive testing method like the Benkelman Beam deflection method. The basic principle of deflection method of pavement evaluation or overlay design is that a well compacted pavement section or one which has been well conditioned by traffic deforms elastically under each wheel load application such that when the load moves away, there is an elastic recovery or rebound deflection of the deformed pavement surface. The maximum deflection under a design wheel load depends on several factors such as subgrade soil properties, moisture in the subgrade, pavement thickness and its composition, temperature of the pavement, loading particulars etc. Therefore, the amount of pavement deflection under a design wheel load or its rebound deflection on removal of this load is a measure of the structural stability of the pavement system under the prevailing conditions of the test. Larger rebound deflection indicates weaker pavement structure, which may require earlier strengthening or higher overlay thickness. It is desirable to carry deflection measurement soon after the monsoons when the pavement system may be at the weakest condition due to maximum subgrade moisture content.

2.10.2 Types of overlays

2.10.2.1 Hot-Mix Overlays

Treatments using thin hot-mix overlays can be adopted to correct surface irregularities which cannot be addressed with the other maintenance treatments. They are similar to conventional overlays, but the thickness is less than 37.5 mm. They include dense, open and gap-graded mixes and most often contain modified binders such as polymers or rubber. Dense and gap graded mixes seal the pavement surface, improve the ride quality and skid resistance. Open-graded mixes improve the ride quality, surface friction and facilitates the quick draining of water from pavement. The service life of thin dense graded overlays ranges from 2 to 10 years.

2.10.2.2 Thin concrete overlays (Ultra Thin White Topping) on bituminous layer

One of the techniques for rehabilitating distressed bituminous pavements is the use of concrete overlay. In North America, this technique is often referred as 'white topping'. White topping has been routinely used in the United States since the mid-1970s. The first documented use of a concrete overlay over an existing asphalt roadway in the U.S can be traced to 1918, when South 7th Street in Terre Haute, Indiana was white topped. It is a well accepted method of asphalt pavement rehabilitation. White topping, that is the placement of a concrete overlay on top of deteriorated asphalt pavement has been an acceptable pavement restoration practice since 1977. Much of white topping technology was developed in IOWA, where more than 300 miles of asphalt pavement has been successfully rehabilitated since 1960 (Kumar, 2004).

Another new method of rehabilitating asphalt pavements was developed in the early 1990s. This technique is called ultra-thin white topping (UTW) (Rao and Thombare, 2013). In this method, bonding of a relatively thin layer of concrete to the underlying asphalt is done to create a composite pavement section. In this process, 2 to 4 inches of high strength fibre reinforced concrete is placed over a specially prepared surface of distressed asphalt. The resulting composite pavement will give long life with better performance characteristics of concrete pavement at a comparative cost with ordinary bituminous overlays (Bordelon, 2012).

Ultra Thin White Topping (UTW) is a concrete product reinforced with synthetic fibers, which is an option to rehabilitate a deteriorated bituminous pavement that has adequate structural strength, but a poor or rutted surface. The asphalt pavement that has to be milled, broomed and cleaned for placing the UTW layer. The bituminous layer should have sufficient structure to permit bonding. UTW can be placed using conventional paving equipment, even inexpensive vibrating screeds or hand leveling equipment. An advantage is that by using Fast Track paving materials and techniques, UTW projects can handle traffic in less than 24 hours after construction, developing compressive strengths over 3000 psi. The first ever UTW project was undertaken in Loiuisville, Kentucky in 1991. The experiment included construction of two thicknesses of concrete overlay (90 mm and 50 mm) on the road leading to a landfill site. The road served 400 to 500 trucks daily throughout the working days in the week. This project was successful and a number of different projects in other states like Georgia, Tennessee, Virginia, Florida and Iowa were taken up subsequently. Sinha et al. (2007), also attempted to research on the concept of White topping. They have demonstrated the actual savings in per kilometer cost in the case of Ultra-thin & Thin White topping. It was concluded based on analysis that White topping with thickness of 100 mm to 250 mm is suitable for rehabilitating existing bituminous roads having low to moderate traffic.

Sinha et al. (2007) also reported the concept of white topping which can be used to rehabilitate deteriorated bituminous pavements. The paper highlights that white topping is practiced in developed countries due to its cost effectiveness. White topping has proved to give an additional life of 20 to 30 years on an average and at the same time, it is cost effective. When distresses like rutting or cracking occur on pavements, there will be differences in the condition of pavement layers causing variations in pavement deflections and shape of deflection basin along the road. But, the available procedures for the analysis of deflection data cannot identify the presence, location, and extent of distress within the pavement structure. Therefore, a technique to assess the pavement layer condition based on surface deflection data and CBR value of subgrade soil is to be developed and so that reduce the need for destructive testing will be minimized.

2.10.2.3 Advantages of Ultra Thin White Topping

It is reported that life of Ultra-Thin White Topping will last 2-3 times more than bituminous overlays. UTW gives better appearance and cool light reflectance like concrete surface over the life of the surface. Rutting which appears within a year on a bituminous surface will be negligible on UTW surface.

This is fast and easy to lay also. The existing pavement is to be milled to a uniform depth and then cleaned. The UTW concrete can be directly placed on the milled surface using a conventional vibratory screed and consolidated to final grade. The finishing can be done with tire broomed or burlap dragged finish. The joints are made using entry or green cut saw. Joint spacing is usually laid out in three foot square.

UTW overlay is competitively priced with bituminous overlays. When Fast Track construction techniques are adopted, curing time becomes less than 24 hours and common paving jobs like intersection repair etc. can be completed in a day or two.

2.10.2.4 Fibre Reinforced Concrete for Ultra Thin White Topping

With a lot of new highway projects coming up, highway sector in India is poised to undergo a sea change in the quality and quantity of road construction activities. Newer materials have been tried in the field with the aim of optimizing life cycle costs and enhancing the life of the pavements by reduction in pavement thickness with the use of materials, which are of high strength and durability. The concept of fiber reinforcement is as old as 1920s. Substantial development in the production of fiber reinforced concrete has been undertaken in the last two decades. This has been used in UTW also.

2.10.2.5 Factors Affecting UTW Performance

The performance of a UTW system is determined by the following factors (Bordelong 2012).

a) Bond between the UTW overlay and the Hot Mix Asphalt (HMA) base.

   The tensile load stresses in the concrete are reduced as solid bond at the concrete asphalt interface creates a composite action and pushes the neutral axis away from the concrete layer **(Fig. 2.2**). As the neutral axis moves further down, the tensile stresses in the asphalt also decrease, as the HMA layer is closer to the neutral axis.

b) Shorter joint spacing or small panels.

   Smaller joint spacing helps to reduce the stresses generated by bending as well as curling and warping effects on the pavement due to temperature and moisture gradients. In UTW, joint spacing used are short in order to ensure that energy is absorbed by deflection and not by bending. **(Fig. 2.3).** Typically the joint spacing for a UTW system are somewhere between 0.6m and 1.5m.

c) Sufficient thickness of the remaining asphalt.

   It is to be ensured that after the surface preparation, (ie. the milling of the asphalt surface), thickness of remaining asphalt should be sufficient to form a composite section to carry the load.

d) The UTW projects constructed in the last few years have provided promising results for further implementation and analysis of the white – topping as a rehabilitation measure for rutted and distressed asphalt pavements.

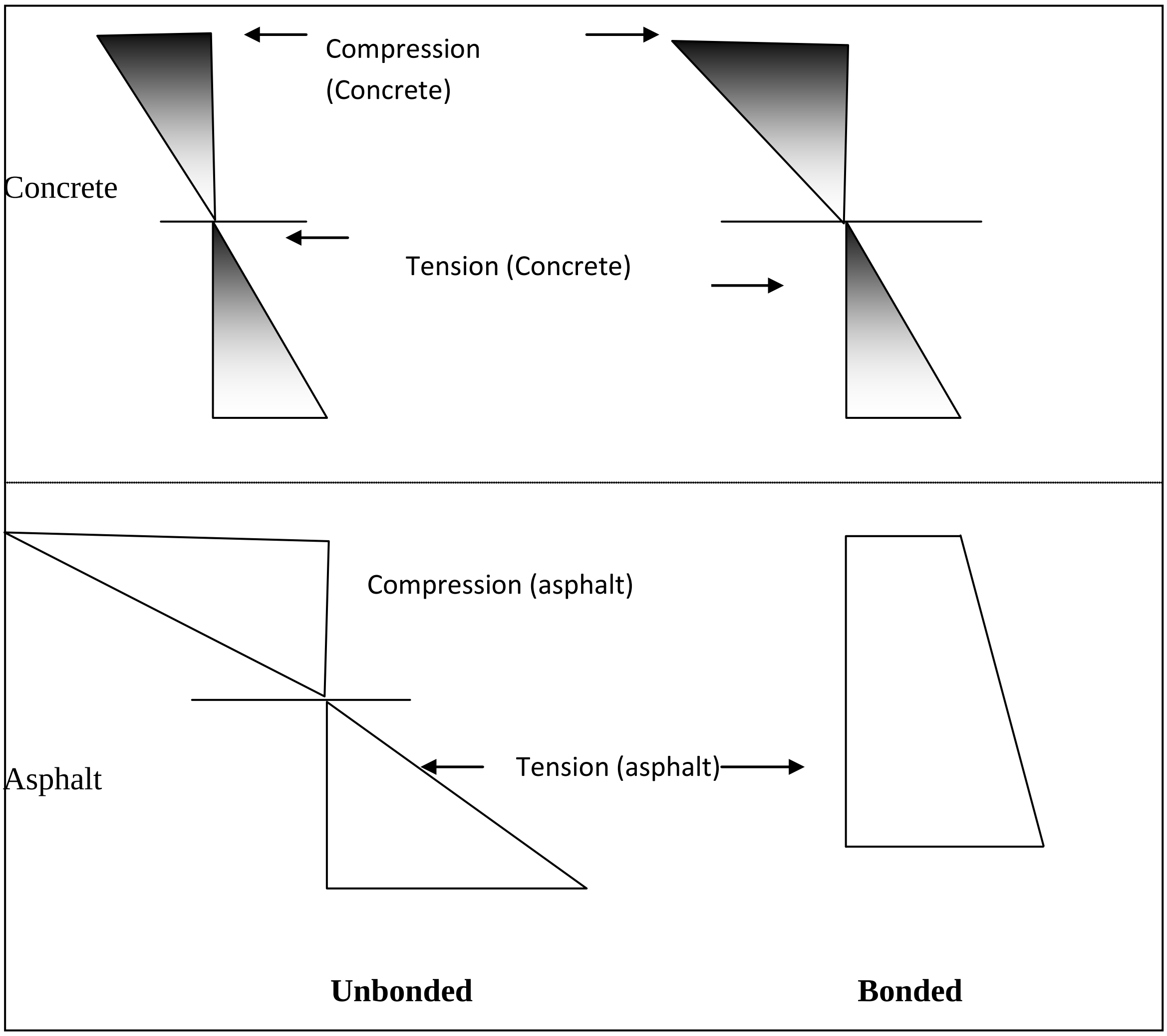

Fig. 2.2 Stress distribution in unbonded and bonded overlays

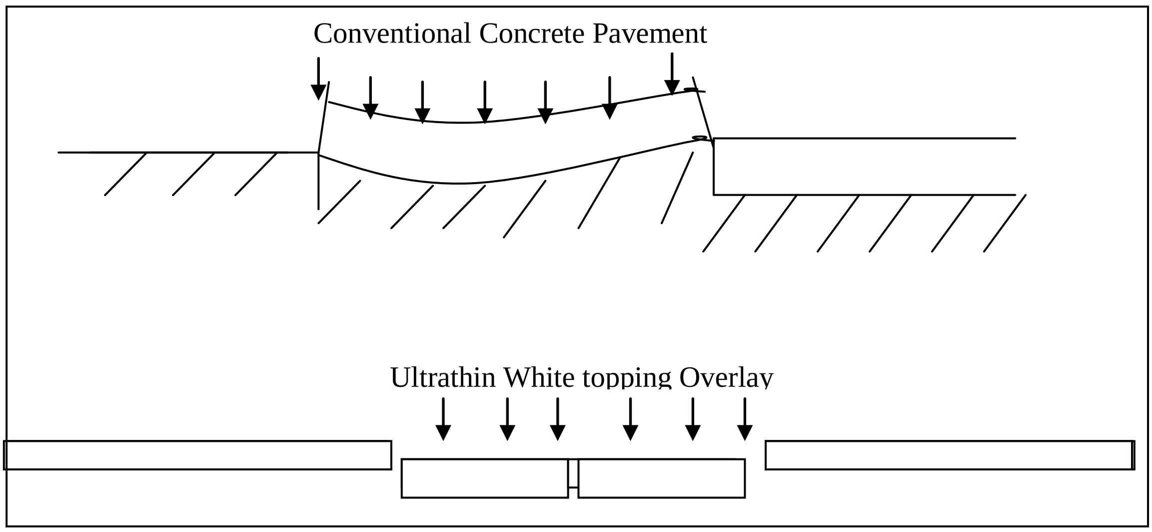

Fig. 2.3 Stress in C C pavement and UTW

2.11 FUZZY LOGIC

Soft computing is an emerging approach to computing which parallels the remarkable ability of the human mind to reason and learn in an environment of uncertainty and imprecision for modeling and computing. It consists of many complementary tools such as Artificial Neural Network (ANN), Fuzzy Logic (FL), and Adaptive Neuro-Fuzzy Inference System (ANFIS).

2.11.1 Fuzzy sets

The concept of Fuzzy Logic (FL) was introduced (Krishnan et al., 2010) as a way of processing data by allowing partial set membership rather than crisp set membership or non-membership. The membership function $\boldsymbol{\mu_A}$(x) is associated with a fuzzy set A such that the function maps every element of the universe of discourse X (or the reference set) to the interval [0,1]. Formally, the mapping is written as:

$$\mu_A(x) : X \rightarrow [0, 1]$$

A fuzzy set is defined as follows: If X is a universe of discourse and x is a particular element of X, then a fuzzy set A defined on X may be written as a collection of ordered *pairs*

$$A=\{(x,\mu_A(x), x \in X\} \qquad \text{--------------- (2.20)}$$

Where each pair (x, μ_A(x)) is called a singleton. In the crisp sets μ_A(x) is dropped. An alternative definition which indicates a fuzzy set as a union of all μ_A(x)/x singeltons is given by

$$A=\textstyle\sum_{x \in X} \mu_A(xi)/(xi), \text{ in the discrete case} \qquad \text{------------- (2.21)}$$

$$A=\int\mu_A(x)/x, \text{in the continuous case} \qquad \text{------------- (2.22)}$$

Here, the summation and integration signs indicate the union of all μ_A(x)/x singletons.

2.11.2 Membership function

The membership function values need not always be described by discrete values. Quite often, these are described by a continuous function (Jang et al., 2012). The Membership functions are Triangular Membership Functions, Trapezoidal Membership Functions, G-

Membership Functions, S-Membership Functions, Gaussian Membership Functions and Exponential-Like Membership Functions.

2.11.3 Basic Fuzzy set Operations:

Given X to be the universe of discourse and A and B to be fuzzy sets with $\boldsymbol{\mu_A}$(x) and $\boldsymbol{\mu_B}$(x) as their respective membership functions, the basic fuzzy set operations are asgiven below: (Rajasekaran et al., 2003)

Union:

The union of two sets A and B is a new fuzzy set A U B also on X with a membership function defined as

$$\boldsymbol{\mu_{AUB}}(x)=\boldsymbol{\mu_A}(x),\boldsymbol{\mu_B}(x)) \quad \text{------------------} \quad (2.23)$$

Intersection:

The intersection of fuzzy sets A and B is a new fuzzy set A n B with membership function defined as

$$(x)= \min(\boldsymbol{\mu_A}(x),\boldsymbol{\mu_B}(x)) \quad \text{------------------} \quad (2.24)$$

Complement:

The complement of a fuzzy set A is a new fuzzy set A with membership function

$$\mu_A(x) = 1- \mu_A(x) \quad \text{------------------} \quad (2.25)$$

Product of two fuzzy sets:

The product of two fuzzy sets A and B is a new fuzzy set A. B whose membership function is defined as

$$\mu_{A.B}(x) = \mu_A(x)\mu_B(x) \quad \text{------------------} \quad (2.26)$$

Equality:

Two fuzzy sets A and B are said to be equal

$$(A=B) \text{ if } \mu_A(x) = \mu_B(x) \quad \text{------------------} \quad (2.27)$$

Power of a fuzzy set:

The α power of a fuzzy set A is a new fuzzy set, Aˆ α whose membership function is given by

$\mu_{A\hat{}\,\alpha}(x) = (\mu_A(x))\,\hat{}\,\alpha$ ------------------ (2.28)

Raising a fuzzy set to its second power is called Concentration (CON) and taking the square root is called Dilation (DIL).

Difference:

The difference of two fuzzy sets A and B is a new fuzzy set

A –B defined as: A–B = (AnB^c) ------------------ (2.29)

2.11.4 Fuzzy Models

In general, fuzzy models constitute highly abstract and flexible constructs since they operate at a level of information granules fuzzy sets. Given the environment of physical variables describing the surrounding world and an abstract view of the system under modeling, a very general view of the architecture of the fuzzy model can be portrayed.

2.11.5 Main Categories of Fuzzy Models

There are several categories of models where each class of the constructs comes with interesting topologies, function and characteristics, learning capabilities, and the mechanisms of knowledge representation. The landscape of fuzzy models is highly diversified. Some of the architectures that are most visible and commonly envisioned in the area of fuzzy modelling (Pedrycz and Gomide, 2007) are, Tabular Fuzzy Models, Fuzzy Decision Trees and Fuzzy Neural Networks.

2.12 DISCUSSION

Based on the literature survey carried out as listed above, the present status of pavement deterioration modeling is briefly discussed here. The techniques so far used for developing pavement deterioration models are straight line extrapolation, mechanistic-empirical, regression (empirical), polynomial constrained least squares, probability distribution, S- shaped curves and Markovian technique. But, the degree of accuracy needed for model depends on its intended use. For project level analysis, these models shall be more accurate than that for network level analysis.

Straight line extrapolation is the simplest of the techniques available, but can be applied only for individual sections of pavement. Straight line extrapolation will not give a generalised model. This method assumes that traffic loading and other factors prevailing on the pavement section will remain the same in future. This technique requires at least one condition measurement after construction and the condition at the time of construction. A straight line variation in conditions is assumed between the points.

Extrapolation of this line predicts the condition at a future date. This technique account for the factors like traffic, subgrade properties, climate, pavement structure etc. passively. Straight line extrapolation is accurate enough for the short term condition forecasting. The assumption of straight line variation in pavement condition is the biggest drawback of this technique. Hence, this is normally not recommended for longer duration and for new pavement section.

Regression technique is empirical in nature and it tries to establish a relationship between the pavement condition and its causative factors. Pavement age, intensities of different modes of distress, traffic load etc. form the variables for regression. The variables are described in terms of their variance and mean. Regression can be either linear or non-linear. Linear regression has got wide acceptance because of its simplicity and the physical sense it gives. If there are more than one variable deterioration model, non-linear regression or multiple linear regression is resorted to. The reliability of a regression model is to be measured by its goodness of fit, which is defined in terms of co-efficient of determination (R^2 value).

A pure mechanistic modelling approach can be adopted only for finding out the pavement responses like stress, strain and deflection. These responses are developed by climate, forces due to traffic, or a combination of the two. The pure mechanistic model cannot be termed as a prediction model, but the calculated stress and strain or deflection can be used as input to a regression model for predicting pavement performance.

Polynomial constrained least square technique is one of the most powerful techniques for predicting pavement condition. Here pavement condition is modeled as a function of any factor causing deterioration, like traffic or climate. A polynomial of n^{th} degree is established to predict pavement condition applying the method of least squares. S-shaped curve fitting technique is also an appropriate for prediction of pavement condition with age. Non-linear regression analysis of periodic performance data is used to fit the curve.

In probability distribution technique, measure of a pavement condition like Pavement Condition Index (PCI) or International Roughness Index (IRI) is considered as a random variable with probabilities associated with its values. A probability reduces from 1 to 0 with time. The probability plot with time gives the percentage of pavement that remains in service with PCI or IRI greater than a selected value. Such a plot between PCI and time is called ‘survivor curve’. This technique is particularly useful for prediction of distress of individual pavement.

In Markovian technique, a pavement condition measuring scale is divided into discrete intervals, which are called ‘condition states’. Condition prediction is dependent on determination of probabilities associated with the pavement in a given state of condition after one duty cycle. A study cycle is taken as the one-year effect of traffic loading, weather or similar measure.

Different pavement condition rating models were also reviewed. New techniques are currently being incorporated in determining the condition indices. Various techniques reported for developing deterioration models were reviewed. These techniques included straight line extrapolation, regression (empirical), mechanistic- empirical, polynomial constrained least squares, S- shaped curves, probability distribution and Markovian technique. One of the soft computing techniques, fuzzy logic including theory of fuzzy sets, fuzzy membership functions and fuzzy models were reviewed. Analytical Networking Process (ANP) and Analytical Hierarchy Process (AHP) are also discussed.

To summarise, a number of models to predict performance of pavements have been reported by researchers. But, some of these models were developed for a country or a particular region under defined traffic and climatic conditions. Hence they cannot be applied in countries with different climatic conditions and traffic. Much research has been done for modelling the performance of pavements, there is need for the development of a reliable and accurate model which can predict the performance at all traffic and climatic conditions.

2.13 SUMMARY

This chapter discusses the developments that have taken place for predicting the behavior of pavements throughout the world. HDM and AASHTO models have been widely used for the Pavement Management Systems in most of the countries. Use of software like KENLAYER, ANSYS etc. and techniques like Regression, Artificial Neural Network and Fuzzy Logic for modeling have been reported. The theory of perpetual pavements and use of Mechanistic - Empirical design (MED) philosophy also have gained acceptance. Use of Finite Element Method (FEM) for predicting the deflection behavior of flexible pavements has been reported. Ultra Thin White Topping, one of the pavement rehabilitation techniques, which is gaining acceptance, is also mentioned briefly. It is proved that the HDM-4 Software should be calibrated to suit to local conditions to get more reliable and accurate results. A review of the works reported through published papers show that only limited pavement evaluation studies have been done for the roads of Kerala State for research purpose which warrants more systematic data collection and evaluation studies.

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Chapter 3

METHODOLOGY

3.1 GENERAL

The methodology in general comprise of selection of study stretches, field investigations, laboratory investigations, data preparation, analysis, development of relationships and models. Extensive literature survey was done. The works reported in journals at the national as well as international level were reviewed. Research work being carried out in various academic and research institutions were also referred. The methodology adopted is represented in **Figure 3.1** and explained in the succeeding sections.

The steps adopted in the present study are as given below:

i. Review of the previous works in pavement evaluation: The published literatures were reviewed to get an insight on evaluation of flexible pavements, application of different tools, geographical area so far covered in such studies, models reported and their adaptability to Kerala conditions etc.

ii. Selection of study area: Since it was aimed to study the performance of all types of roads, focus was given on urban roads and other roads separately. Based on a preliminary scanning of the list of urban roads in the five corporations of Kerala using maps and secondary data on the traffic and other details, 44 roads were selected for pavement evaluation. In order to conduct periodic performance evaluation and develop deterioration models, eight representative road sections including NH, SH and other roads were selected considering the type of road, traffic, geographical location, climatic condition, construction aspects etc.

iii. Data collection, compilation and analysis: Field investigations were done as per the methodology discussed in the sections given below.

iv. For urban roads, using the one time data collected from field, relationships between pavement strength represented by deflection, condition of the pavement and subgrade soil characteristics were developed.

v. For other roads (NH, SH, etc.), from the data obtained through periodic evaluation, deterioration models were developed using different tools. The predicted values were compared with observed values for validation.

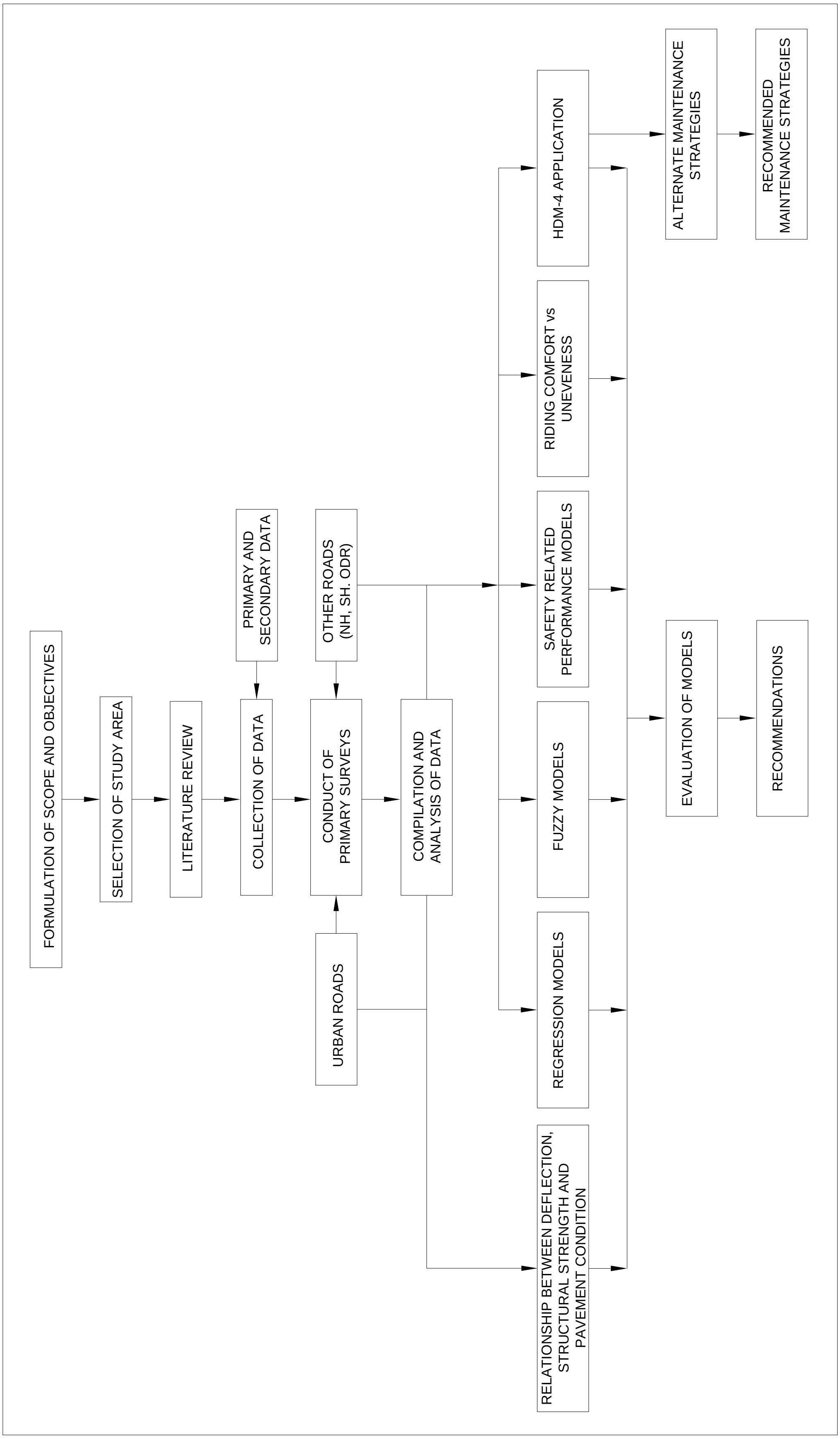

Figure 3.1: Methodology adopted for the study

The procedure adopted for field data collection, analysis and model development are discussed here. The codes and specifications adopted for the study are referred wherever necessary.

3.1.1 Selection of Road Stretches

i. Urban roads

Based on a preliminary assessment with respect to the road conditions and traffic, 44 roads from five corporations, which represent urban conditions, were selected for the study. These road stretches were further divided into 68 homogeneous sections.

ii. Study roads for periodic pavement performance evaluation

The road stretches for conducting the periodic evaluation were identified to represent variation in pavement composition, traffic composition, climatic conditions, terrain and soil characteristics. Eight roads with twelve homogeneous sections were selected as study stretches based on the above factors. Details of the study roads are discussed in Chapter 4.

3.1.2 Field Data Collection and Laboratory Investigations

Data collection ranged from simple ‘windshield surveys’ to the use of testing vehicles that measure deflection, unevenness, skid resistance and cracking on the surface and axle load surveys. The data include:

i. Inventory of the study sections
ii. Pavement drainage characteristics
iii. Pavement surface condition (cracks, raveling, potholes, rutting, edge break etc.).
iv. Unevenness using Bump Integrator (IRC SP 16, 2004).
v. Skid Resistance using portable skid-resistance Pendulum (BS 812-1967).
vi. Rebound deflection using Benkelman Beam (IRC 81, 1997).
vii. Traffic studies including axle load surveys using Portable Weigh Bridge (TRL Overseas Road Note 40).

viii. Pavement composition from trial pits and core cutter method.

ix. Texture depth using sand patch method (BS 598 part 105, 1990).

x. Field Density of the subgrade soil using sand replacement method (IS 2720 Part 28, 1974).

xi. Laboratory investigation of the subgrade soil properties including CBR.

3.1.3 Data Compilation and Analysis

The collected data were compiled to suit to the requirements of the study and analyzed to determine the performance parameters like percentage of distress, rebound deflection in mm, roughness values in cm/km, IRI values and Vehicle Damage Factor (VDF). For analysis in HDM-4 software, the data was prepared under four categories namely pavement condition data, vehicle fleet data, Maintenance and Rehabilitation Works data and cost data.

3.1.4 Development of Pavement Performance Models and Relationships

The influencing parameters were identified from the literature. Non-linear regression models using SPSS were developed. Calibration of HDM-4 deterioration models to suit to Indian conditions was done. Fuzzy rule based models were also developed for deflection and compared with other models. The effect of subgrade soil type on the rebound deflection and Modified Structural Number relationship has been checked using appropriate plots. Performance models were developed for cracking, pothole, raveling, roughness and for safety considerations.

3.2 FIELD AND LABORATORY INVESTIGATIONS - INSTRUMENTATION AND PROCEDURE

The instrumentation and procedure adopted for data collection is discussed in this section.

3.2.1 Inventory

Detailed inventory of all study stretches were taken and the data were collected as per Indian Roads Congress format.

3.2.2 Pavement drainage characteristics

The drainage characteristic of the pavement is controlled by the following factors and hence data on the same were collected from the field and secondary sources.

i. **Camber or cross slope:** The cross slope/ camber of the pavement surface was noted using a camber board (slope meter) in percent slope at 5 to 10 locations in each study stretch of 100 to 200m and the mean camber was recorded **(Fig. 3.2).** Typical sections selected were at level road and with gradient, for rolling terrain and flat terrain. Under each category, at least one typical section was selected with (a) adequate camber (b) inadequate or deficient camber and (c) practically nil or zero slope. The cross slope of shoulders (in percentage or as ratio) were noted at typical locations. The shoulder level in mm, i.e., shoulder drop or raised shoulder with respect to pavement edge was also recorded.

ii. **Presence of valley stretch:** Availability of cross drainages or culverts at valley locations were noted.

iii. **Soil moisture content:** Measurement of moisture content of soil and its variations with depth (0.5, 1.0 and 1.5 m) during 2 or 3 typical seasons.

iv. **Water table data:** This was collected from local enquires**.**

v. **High Flood Level Data:** This was obtained from local enquiry.

vi. **Rainfall data:** Average rainfall data in the locality for the past 3 years of the study period was collected from secondary sources.

vii. **Land use data:** The collected data included land use of the adjoining road formation, type of cultivation with or without irrigation or built up along the road side. The relative level of the road land with respect to the road formation level had been collected from the field as primary data.

viii. **Cross slope:** Cross slope of the adjoining land were also recorded.

ix. **Longitudinal side drain:** Availability of side drains on left and right side of the road with details including the depth with respect to road level were recorded.

Fig. 3.2(a) Measurement of Camber

Fig. 3.2(b) Measurement of Camber

3.2.3 Pavement Surface Condition

(i) Cracking: 3 to 6 study stretches of 15 to 50m length with some cracks already developed on the surface were identified from the selected roads. Location of cracking, cracked area, wheel path (if visible) and distance of the cracked area with respect to the pavement edge or centre were noted. The areas of classified cracks (fine, medium and wide cracks) were recorded in terms of the total area of pavement. The percentage of cracked area with fine cracks (width less than 1.0mm), medium crack (width 1.0 to 3. 0 mm), wide cracks (width more than 3.0 mm) and mixed cracks were noted. **(Fig. 3.3, 3.4 and 3.5)**

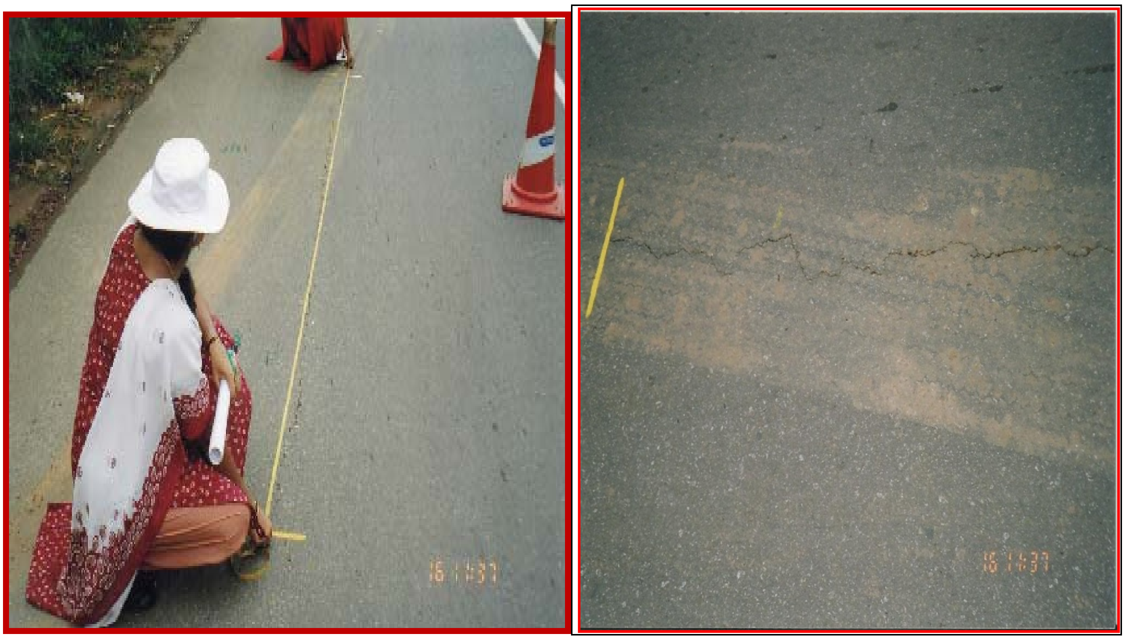

Fig. 3.3: Measurement of crack length and width

(ii) Crack length propagation studies on selected control stretches:

Control sections of dimensions 1m x 1 m were marked on the road and the crack length data were periodically measured. The selected area consisted of the following types

a. Area without any crack

b. Area with only fine cracks of width less than 1.0 mm

c. Area with medium cracks of width 1.0 mm to 3 mm

d. Area with wide cracks of width more than 3.0 mm

e. Area with all the above types of cracks

Fig. 3.4 Cracks with patching

(iii) **Potholes:** Study stretches of length about 200 to 500 m was selected considering identical conditions. The data of potholes on each study stretch were collected and recorded periodically two or more times a year covering different seasons including the periods before and after monsoon season. **(Fig. 3.6)**

Open or unfilled potholes: The average depth, length and width of unfilled potholes were recorded in mm and classified into groups such as small, medium and large size. The numbers of potholes of each group for each study stretch were recorded to know the growth pattern.

Improperly filled potholes: Details of improperly filled potholes (size and number) were recorded with details such as unsuitable materials like soil, brick aggregates, large boulders, stones etc in adequate compaction, not cut to rectangular shape with vertical edges, not properly cleaned or loose materials not removed, a tack coat not properly applied and level difference of the finished surface with the adjoining surface.

Patched potholes: The features of the patched potholes such as slippery with excess binder, absence of seal coat, level of the patch, etc. were noted including the number and area.

Fig. 3.5 Alligator cracks

Fig. 3.6 Pothole and Alligator Cracks

(iv) **Rutting:** The data collected consists of location of the longitudinal rut with respect to pavement edge or centre line or wheel path, mean depth and width of the longitudinal ruts in mm and location of the wheel path with respect to pavement edge, if visible.

(v) **Undulations:** Measurement of undulations or depressions were done using straight edge and wedge scale on stretches of length 30 to 50m, along three longitudinal lines, representing two wheel paths and centre line, and were grouped as per IRC/ MoRTH guidelines and number in each category was recorded. The Bump Integrator was used to measure the undulations as discussed in **Section 3.2.5**. Shear failure denoted by large size depressions with heaving in adjoining portion were measured using a straight edge and vertical scale or steel tape. The mean length, width and depth of such depressions were noted.

(vi) **Raveling:** Raveling is the loss of material from the surface of the pavement. The reason that can be attributed to raveling is mainly thin surfacing, such as surface dressing, seal coat and premix carpet. The affected area is expressed as percentage of the total pavement area.

3.2.4 Traffic Studies:

Traffic studies were conducted on the study road stretches with axle load survey on roads indentified for deterioration modeling. The data collected include:

i. Classified traffic volume count survey was done for 24 hours on a representative day.

ii. Direction wise count of classified Heavy Commercial Vehicle (HCV, 2 – axle rigid, 3 – axle rigid, multi – axle articulated and load tractor – trailer units) for 24 hours was done on a representative day.

iii. Axle load study on 2 axle, 3 axle, and multi axle HCV using probable wheel weigh bridge, on each direction with a minimum of 20 percent sample was carried out. **(Fig. 3.7).** Axle load studies on a small sample of Light Commercial Vehicle (LCV) and Tractor Trailers was also done.

Fig. 3.7 Axle load Survey

The data obtained from the axle load survey was used to calculate the Equivalent Single Axle Load (ESAL) for each class of commercial vehicles. The load equivalency for the front axle, rear axle and intermediate axle were determined separately. The total of the equivalent single axle load of all axles of a vehicle were summed up to get the total equivalency. (**TRL Overseas Road Note 40**)

The fourth power rule for the determination of Equivalent Single Axle Load is

$$ESAL = (Li/La)^4 \qquad \text{------------------------} \qquad (3.1)$$

where,

L_i is the axle load of the ith vehicle and

L_a is the Standard Axle Load

Standard Axle Load taken for an axle with single wheel assembly is 6000kg and dual assembly is 8160 kg. For Tandem axle with dual wheel assemblies in each axle, the standard axle load is taken as 15100 kg. The vehicle damage factor which is an indicator of the damaging power of an axle with respect to that of standard axle is given by

$$\text{Vehicle Damage Factor (VDF)} = \frac{\text{Total ESAL}}{\text{No.of vehicles weighed}} \qquad \text{-----------} \ (3.2)$$

VDF is taken as a parameter for crack and raveling progression models.

3.2.5 Roughness Survey

Roughness or ride quality is a measure of the unevenness of the pavement surface along a linear plane. It represents the ability of the pavement to provide a comfortable ride to the users. Pavement roughness is considered as the most important road feature by the public. Roughness is important on roads with higher speed limits, for those above 70 km per hour. Even though it is important for state highway agencies, it is generally of less importance to cities due to the difference in speed limits. Roughness is normally converted into an index such as the Present Serviceability Index (PSI) or the International Roughness Index (IRI).

The roughness of pavement surface is commonly designated as unevenness index value and is expressed in surface roughness measured by a Bump Integrator. Roughness index is represented as the ratio of the cumulative vertical displacement to the distance travelled and is expressed in mm/km. A Towed Fifth wheel Bump Integrator was used in the present study, which consists of a trailer towed by a vehicle **(Fig. 3.8 and 3.9).**

Fig. 3.8 Fifth Wheel Bump Integrator

A standard pneumatic tire wheel inflated to a tire pressure of 2.1kg/sq.cm was mounted within the trailer chasis, with a single leaf spring on both sides of the wheel supporting chasis. The test was done at a speed of 32$\pm$1 km per hour. The readings of the revolution counter and integrating counters were noted and entered in the data sheet. One measurement in each lane was taken.

Fig. 3.9 Roughness Survey in Progress

Calibration of the Bump Integrator was done using MERLIN – Machine for Evaluating Road Roughness using Low Cost Instrumentation. **(Fig. 3.10)**

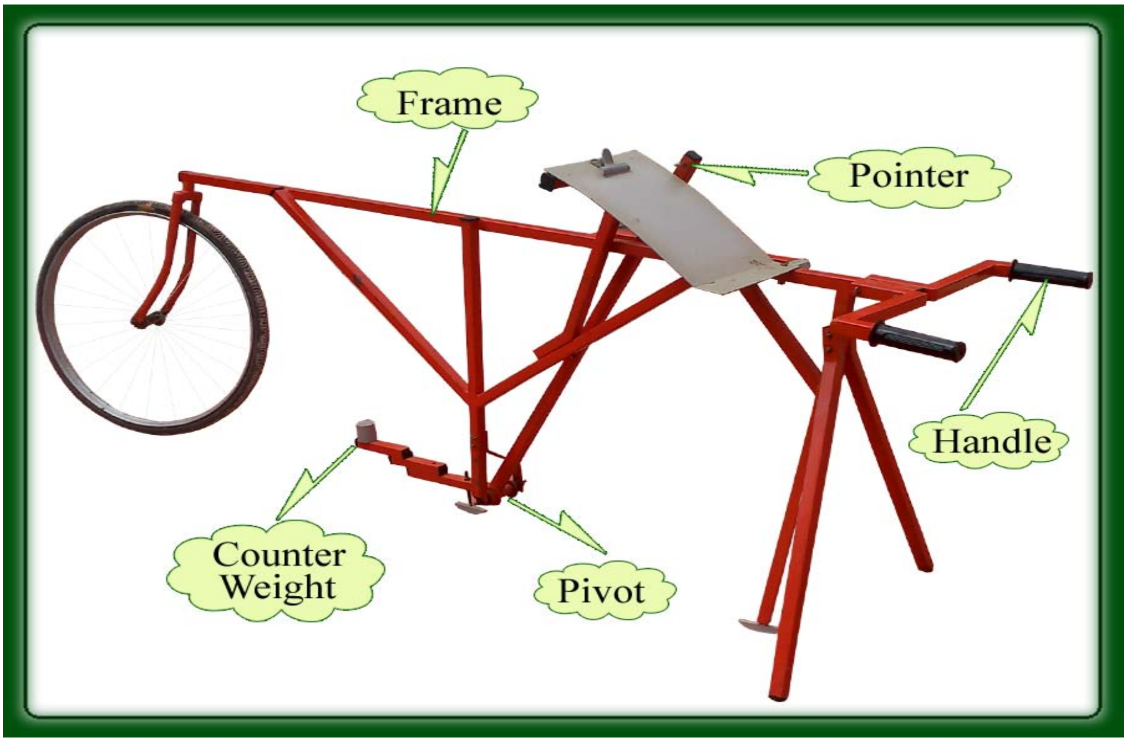

Fig. 3.10 MERLIN

Recommended Standards for Roughness Values: The maximum permissible values of surface roughness measured with Bump Integrator for different surfaces as per IRC SP 16 (2004) are given in **Table 3.1.**

Table 3.1 Maximum permissible surface unevenness for road

Type of surfacing	Maximum permissible surface unevenness	
	Longitudinal profile (mm)	Transeverse profile (mm)
Surface dressing	10	8
Open graded Premix Carpet	8	6
Mix seal surfacing	8	6
Semi dense bituminous Concrete	6	4
Bituminous concrete	5	4
Cement concrete	5	4

(Source: IRC SP 16, 2004)

Newly constructed surfaces are expected to give roughness values corresponding to ‘Good’ category while the values under ‘average’ and ‘poor’ category indicate level of service and intervention level for maintenance as given in the **Table 3.2**. Surfacing with very low roughness values loose skid resistance and are not desirable.

Table 3.2 Maximum permissible values of roughness for road surface (mm/km)

Type of surface	Condition of road surface		
	Good	Average	Poor
Surface Dressing	<3500	3500-4500	>4500
O G P C	<3000	3000-4000	>4000
Mix seal surfacing	<3000	3000-4000	>4000
SDBC	<2500	2500-3500	>3500
Bituminous Concrete	<2000	2000-3000	>3000
Cement Concrete	<2200	2200-3000	>3000

(Source: IRC: SP: 16-2004)

O G P C - Open Graded Premix Carpet,

S D B C - Semi Dense Bituminous Concrete

3.2.6 Skid Resistance Test

The basic function of a pavement is to extend smooth and safe surface for the travelling public. The travelling public is primarily interested in this functional condition, which is primarily measured with roughness and surface friction. The engineers and managers are interested in developing the most cost-effective maintenance and rehabilitation program. They are interested in an engineering analysis of the structural condition, as well as the functional condition. The friction between the road surface and the tire is a detrimental factor for safe stopping speed and distance requirements. Skid resistance is defined as the resistance offered by the surface of the pavement against skidding of vehicle. Portable Skid Resistance Pendulum was used for the study as shown in **Fig. 3.11.** The test was conducted on wet surface. The apparatus was set on the road surface in the line of the wheel track as shown in **Fig. 3.12**. The instrument directly gives the co-efficient of friction on a scale, which is graduated, and it is recorded as 100 times the co-efficient of friction. The value is represented as Skid Number (SN). Readings were taken at a point until same values were obtained for three consecutive trials. The testing was done as per procedure adopted in TRRL 1969 report.

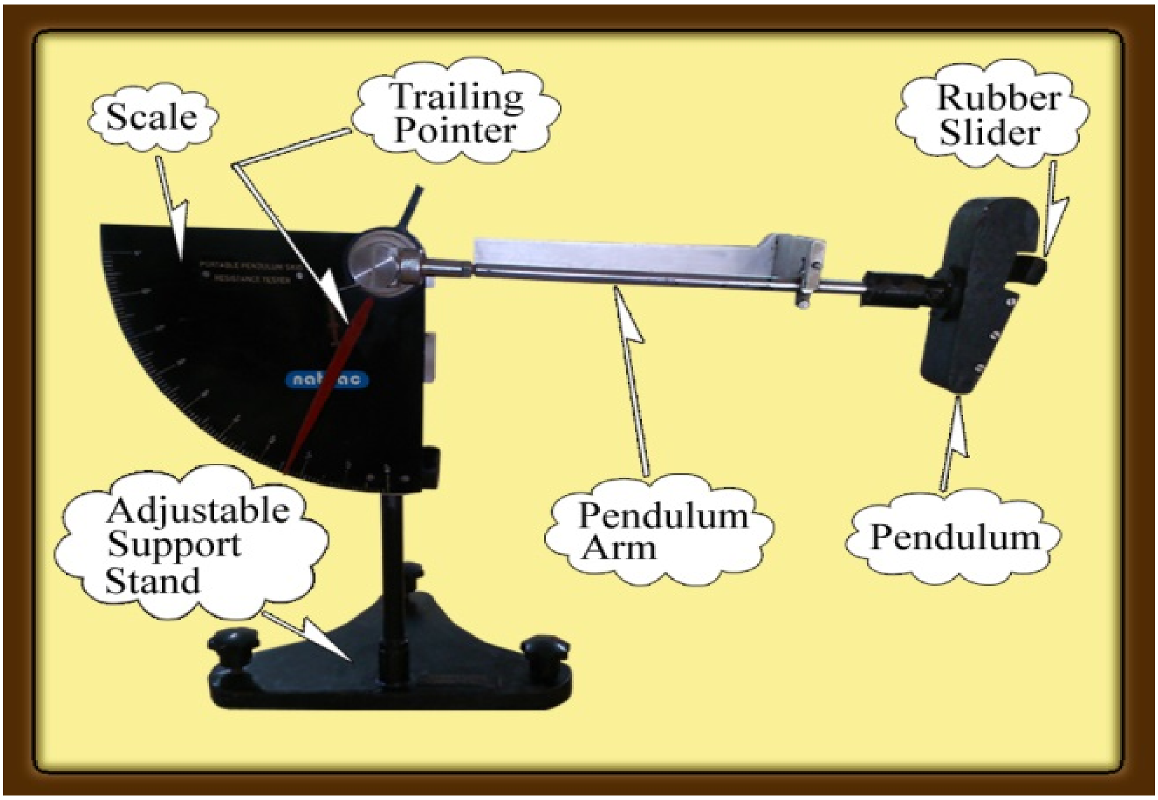

Fig. 3.11 Portable Skid Resistance Pendulum

Fig. 3.12 Skid Resistance Test

3.2.7 Texture Depth Studies (Sand Patch Test)

The texture depth gives the macro texture of the surfacing, which contributes to the net skidding resistance at high speeds. This is achieved by providing drainage routes for water between road surface and tire. The apparatus consist of a measuring cylinder of 50 ml. volume and spreader disk comprising of a flat wooden disc 65 mm in diameter with a hard rubber disc 1.5 mm thick, stuck to one face. The reverse face is provided with a handle as shown in **Fig. 3.13.**

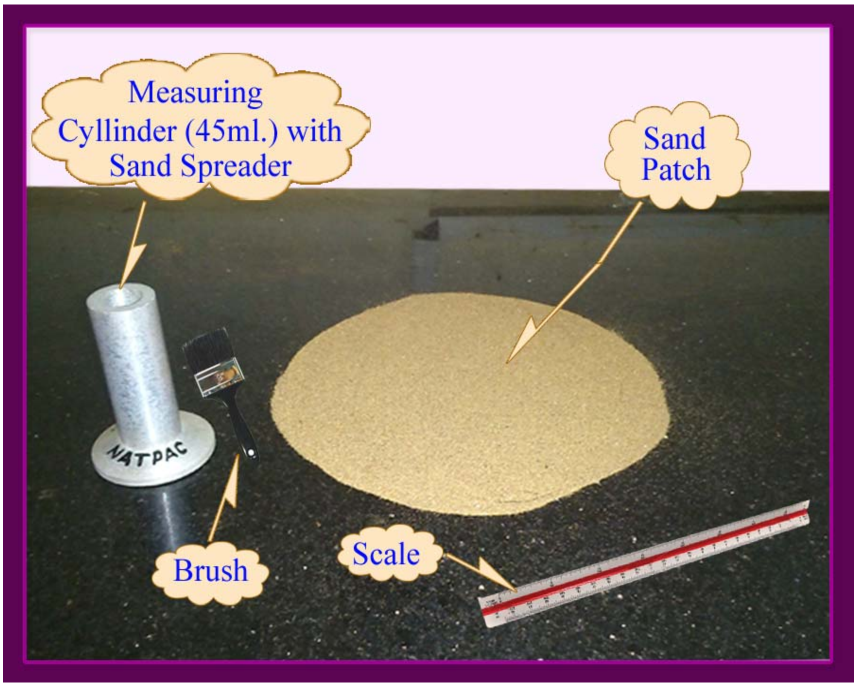

Fig. 3.13 Sand Patch Test Apparatus

The testing was done as per Clause 602.12 of MORTH Specification. The grading of the sand used for the test is given in **Table 3.3**

Table 3.3 Grading of Sand for Sand Patch Test

BS Test Sieve (mm)	% by Mass Passing
0.6	100
0.3	90-100
0.15	0-15

Washed dried sand with round particle passing through 0.6 mm sieve and 15% retained on 0.15 mm sieve was used for the test. The cylinder was filled with sand and poured into a heap on the surface, spread into a circular patch to its largest diameter as shown in **Fig. 3.14.**

Fig. 3.14 Sand Patch Test in progress

The diameter was measured to nearest 1 mm at four points at every 45 degrees. From the measurements, the mean diameter D was calculated from the equation 3.13 and was adjusted to the nearest 0.01 mm.

Texture depth $= 63660 / D^2$ mm -------------- (3.3)

For surface, which give a texture depth of less than 1mm, the volume of sand is reduced to 25 ml or less and the texture depth was again calculated using the equation 3.4.

Texture depth = Volume of sand (ml) * 1000/area of patch (mm^2) in mm ----------- (3.4)

3.2.8 Benkelman Beam Deflection Studies

i. Structural capacity:

Structural capacity analysis is normally conducted at the project-level to determine the load-carrying capacity of the pavement and the capacity needed to accommodate the projected traffic. Structural capacity can be taken as the maximum load and number of repetitions a pavement can carry before reaching some defined condition. Non-destructive deflection testing of the pavement is a simple and reliable method to evaluate the pavement; but destructive testing such as coring and component analysis techniques are used. Structural evaluation of pavement is required for the selection of appropriate treatments at the project-level.

ii. Characteristic deflection

Deflection measurements are tests used to evaluate the response of the pavement structure to a realistic load. The stress, strain and deformation condition in the pavement can be assessed by these tests. It is also possible to assess the deformation characteristics of the individual layers (Molenaar, 1983).

A number of road authorities use the representative maximum deflection for estimating the carrying capacity of road (Kennedy and Lister, 1978, Asphalt Institute, 1983). But the deflection criteria curves recommended in these design procedures (ie, the relationship between deflection and traffic carrying capacity) may not be applicable for the pavements of tropical and sub tropical regions. (TRL Overseas Road Note 18).

The Benkelman Beam instrument can be used to measure the rebound deflection of a pavement due to a dual wheel load assembly or the design wheel load as shown in **Fig. 3.15**. Deflection measurements are made at 20 points in a kilometer. The points are to be staggered at 50 meter interval on both directions. The loading is applied by a truck having rear axle load of 8.17 tonnes and tire pressure of 5.6 kg/sq.cm. The measurements are done as per the CGRA procedure laid down in IRC 81-1997.

iii. Procedure for Deflection measurement using Benkelman Beam

- Based on the pavement condition survey, the road length to be surveyed was divided into homogeneous sections of lengths not less than 500 metre. The loading points on the pavement for deflection measurement were located along the wheel paths on a line 0.9 m from the pavement edge in the case of pavements of total width more than 3.5 m and the distance from the edge is reduced to 0.6 m on narrower pavements. The testing point shall be selected and marked in advance on the pavement. For highway pavements, the distances from the edge of the lane for the testing points shall be as given in **Table 3.4.**

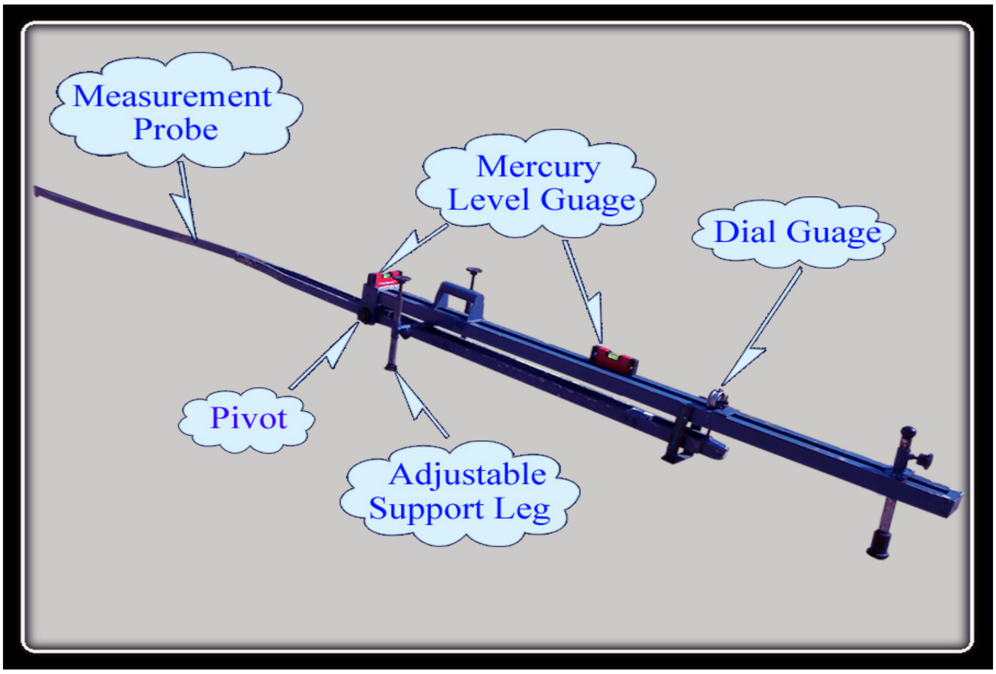

Fig. 3.15 Benkelman Beam

Table 3.4 Distance from the Edge of the Lane for deflection measurement

Lane Width (Metres)	Distance from lane Edge (Metres)
2.8 or less	0.5
3.0	0.6
3.2	0.7
3.4	0.8
3.6 or more	0.9

- A minimum of 10 deflection observation points are taken on each of the selected stretch of pavement. The truck is driven slowly parallel to the edge and stopped such that the left side rear dual wheel is centrally placed over the first point for deflection measurement **(Fig. 3.16).**

- The probe end of the Benkelman beam is placed between the gap of the dual wheel and is placed exactly over the deflection observation point **(Fig. 3.16).**

- When the dial gauge reading is stationary the initial dial gauge reading D_o was noted.

- The truck was moved forward slowly through a distance of 2.7m from the point and stopped. The intermediate dial gauge reading D_i was noted when the rate of recovery of the pavement was less than 0.025 mm per minute.

- The truck was then driven forward through a further distance of 9.0m and the final dial gauge reading D_f was recorded as before. The schematic plan of the survey segment is given in **Fig. 3.17.**

- The three deflection dial readings D_o, D_i, and D_f form a set of readings at one deflection point under consideration. Similarly, the truck is moved to the next deflection point and the procedure is repeated.

Fig. 3.16 BBD Survey in progress - Deflection Observation point

Fig. 3.17: BBD Survey In Progress- Location of Probe

- The temperature of the pavement surface is recorded at intervals of one hour during the study. The moisture content of the subgrade soil is also to be determined at suitable intervals

- The rebound deflection value D at any point is given by one of the following two conditions

  (i) If D_i - $D_f \leq$ 2.5 divisions of the dial gauge or 0.025mm, D = 2 (D_o - D_f) divisions of 0.01mm units = 0.02 (D_o - D_f) mm

  (ii) If D_i- D_f> 2.5 divisions of the dial gauge or 0.025mm , this indicates that correction is needed for the vertical movement of the front legs. Therefore D = 2 (D_o - D_f) + 2 K (D_i - D_f) divisions.

The value of K is to be determined for every make of the Benkelman Beam and is given by the relation:

$$K = (3d – 2e)/f \qquad \text{------------- (3.5)}$$

where,

d = distance between the bearing of the beam and the rear adjusting leg

e = distance between the dial gauge and rear adjusting leg

f = distance between the front and rear legs.

The value of K of Benkelman Beam generally available in India is found to be 2.91. Therefore, the deflection value in case (ii) with leg correction is given by:

$$D = 0.02\ (D_o - D_f) + 0.0582\ (D_i - D_f)\ \text{mm} \qquad \text{---------------}\ (3.6)$$

Correction for Pavement Temperature: Correction for temperature variation for pavement temperature other than 35°C shall be applied on the deflection values measured. For each degree centigrade change from the standard temperature of 35°C, a value of 0.01mm will have to be applied. The correction will be positive for pavement temperature lower than 35°C and negative for pavement temperature higher than 35°C.

Correction for Seasonal Variations: Correction for seasonal variation shall depend on type of sub grade soil, field moisture content at the time of survey and average annual rainfall in the area. The moisture correction factors (or seasonal correction factors) were obtained from the charts given in IRC: 81 – 1997 for a given field moisture content and type of subgrade soil. Limits of Benkelman beam deflection values are shown in **Table 3.5**

Table 3.5: Limits of Benkelman Beam Deflection Values (IRC 81, 1997)

Rebound Deflection (mm)	Strength of pavement
0.5-1	Reasonably Strong
1-2	Moderate
2.3	Weak
>3	Very Weak (Permanent Deformation)

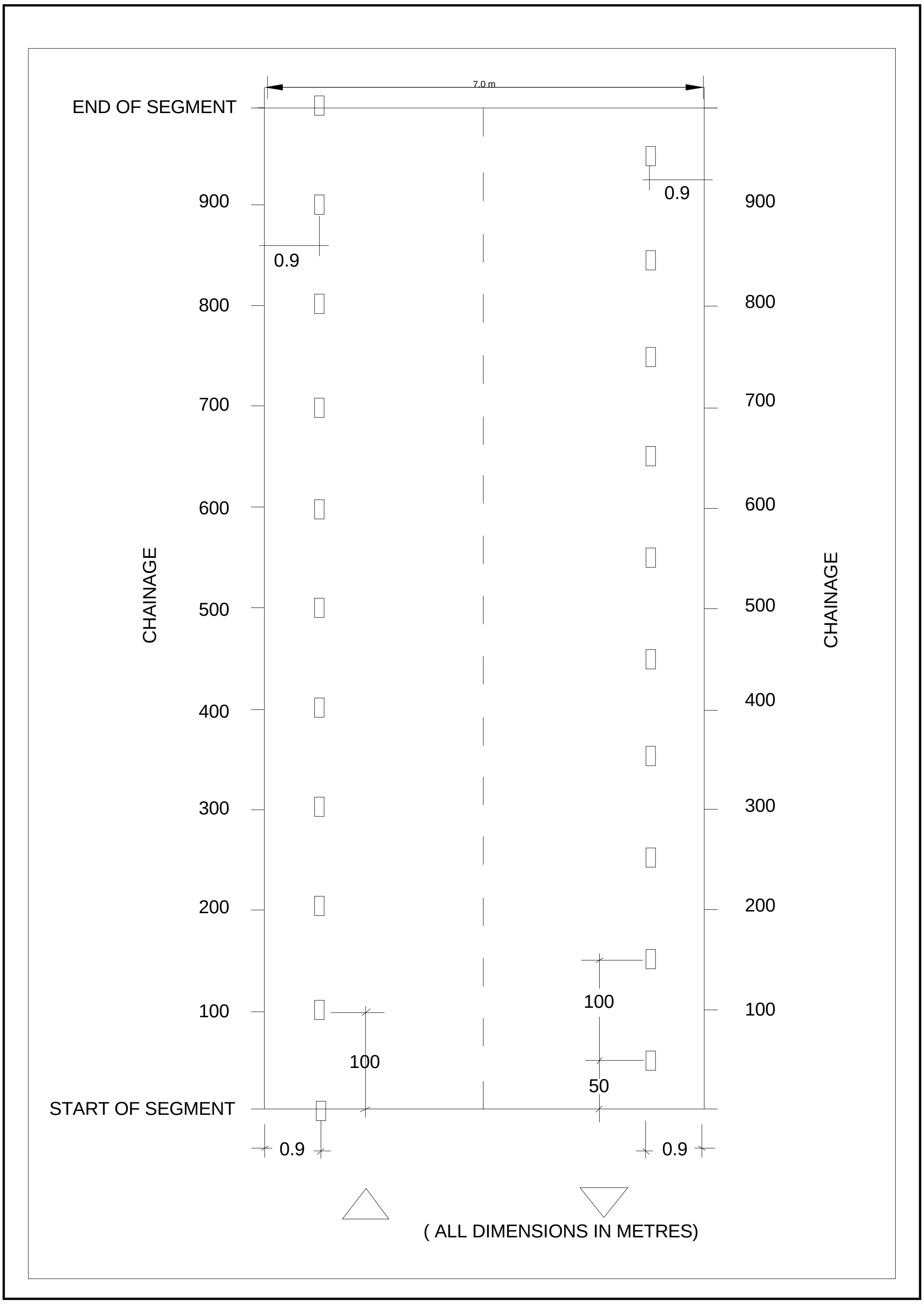

Fig. 3.18: Location diagram of BBD Test

3.2.9 Extraction of core

Cores were extracted from the study roads wherever needed using Core Cutter to identify and record the pavement layer details as shown in **Fig. 3.19.**

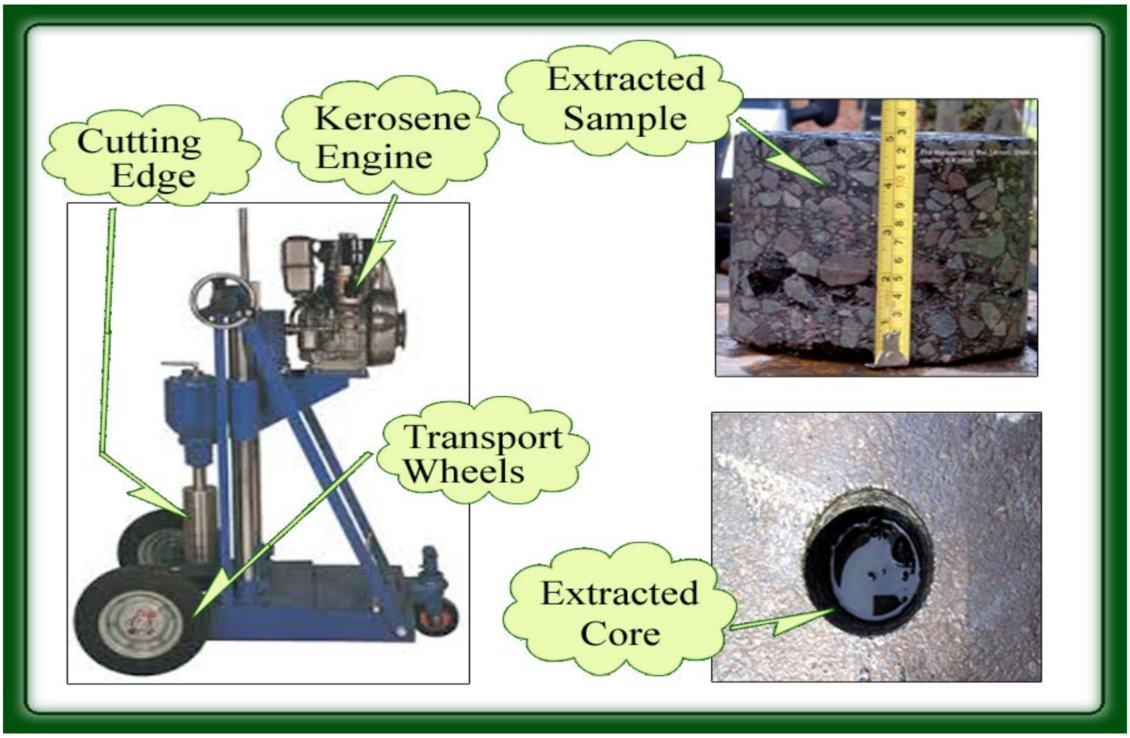

Fig. 3.19 Core Cutter and sample extracted

3.2.10 Trial Pits and Sand Replacement Test

For all the urban roads under study, test pit of size 0.5m*0.5m was taken by the edge of the carriageway to find out the layer details. The field density of the subgrade soil was determined using the sand replacement method in the field as shown in **Fig. 3.20** and **3.21.**

Laboratory tests were done to find the CBR value of the subgrade soil, Optimum Moisture Content (OMC), Maximum Dry Density (MDD), Atterberg Limits, PI and the Soil Classification.

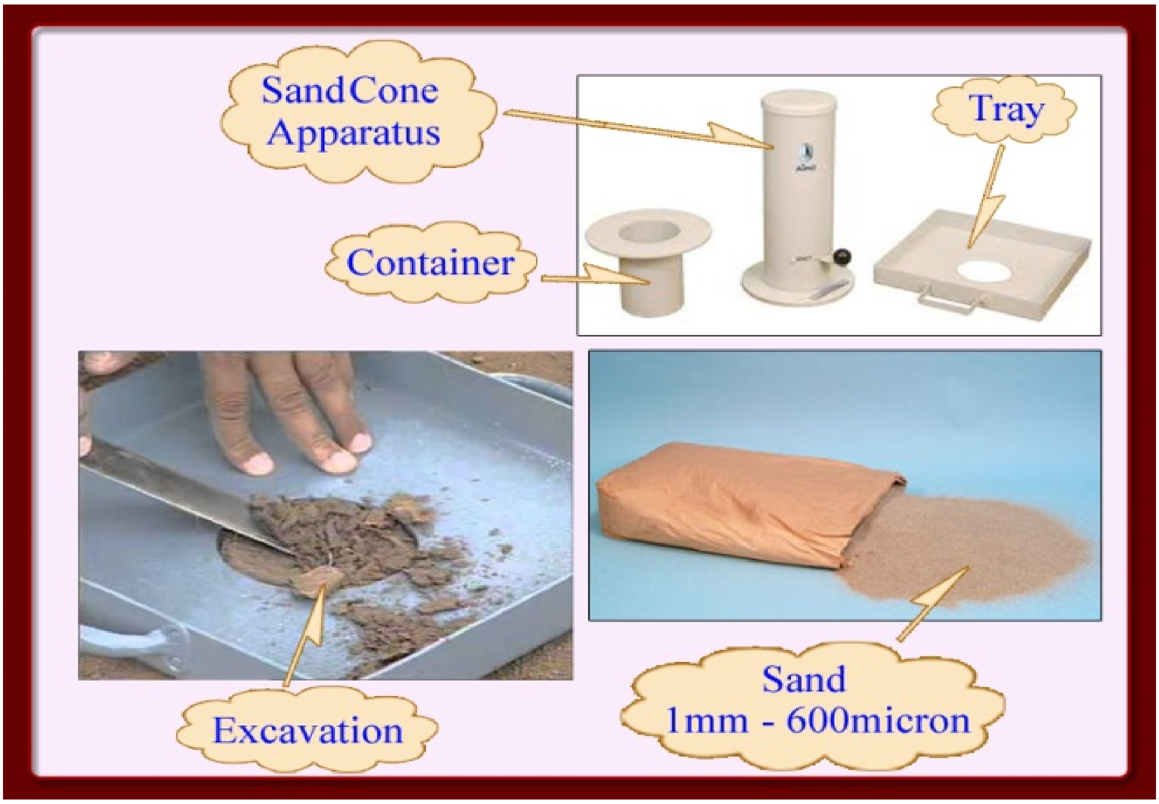

Fig. 3.20 Sand Cone Apparatus

Fig. 3.21: Trial Pit and Sand Replacement Test in progress

3.3 PAVEMENT DETERIORATION CONDITION

3.3.1 Riding Comfort Index (RCI)

The riding quality of pavement is represented by an index called Riding Comfort index (RCI) (**Fwa, 2005**). RCI is a function of pavement unevenness and is measured on a scale of 0 to 5 as given in **Table 3.6**. RCI value of zero represents a well constructed new pavement and five indicates an extremely rough pavement. As the first step, the measured unevenness values were classified in to different ranges. Range selection was based on the data obtained. The RCI value can be obtained directly from the table below, if the unevenness index value of a particular pavement stretch is known.

Table 3.6: Riding Comfort Index values

Unevenness Index (mm/km)	Riding Comfort Index (RCI)
<2500	0
2500-3500	1
3500-5000	2
5000-7000	3
7000-10000	4
>10000	5

3.3.2 Distress surveys and Pavement Condition Index (PCI)

The Pavement Condition Index (PCI) is used to indicate the present condition of the pavement numerically. It is directly related to the operational condition and structural integrity of the pavement surface like ability to resist fracture, distortion and disintegration. Surface distress is the term, which indicates the damage observed on the pavement surface. The PCI is represented as a function of the type, severity and density of surface distress **(Fwa, 2005).**

Distress surveys were performed to find out the type, severity and quantity of surface distress. This information was used to determine the pavement condition index (PCI), which is a measure of the rate of deterioration. Distress is the measure mostly used by maintenance personnel to decide the type of maintenance treatment required, when it is needed and the funding needs in the PMS process. PCI method is based on a visual survey of the number and types of distresses in a pavement. PCI provides numerical rating between 0 and 100, with 0 representing the worst possible condition and 100 representing the best possible condition. To determine the PCI of a pavement section, PAVER procedure and deduct value curves developed for asphalt pavements are used as given below:

i. The pavement section is divided into sample units.

ii. The sampled units are inspected to determine the distress types and severity levels. This is recorded to compute density.

iii. The deduct value for each distress type and severity is determined; for example a (for alligator cracking) and b (for longitudinal and transverse cracking). Distress density is computed first to use the deduct curves. Density is the amount of distress (extend) divided by the sample unit area for bituminous pavement. A deduct value is a number from 0 to 100 with a zero indicating that the distress has no impact on pavement condition and 100 indicating an extremely serious distress which will cause the pavement to fail.

iv. The individual deduct values are summed up to compute the Total Deduct Value.

v. Correction curves are used to determine the Corrected Deduct Value (CDV) from the TDV. The correction curves used are for different number of entries with deduct value over five points.

vi. The deduct value is adjusted by using correction chart. The result is CDV. This is done to ensure that the total deduct value do not exceed 100, if the pavement is badly deteriorated. PCI is computed using the following equation for each sample unit inspected PCI= 100 – CDV

vii. PCI of entire section is computed by taking average of all PCI values.

viii. Condition rating of the pavement section is determined using PCI and rating scale.

3.3.3 Structural Number and Modified Structural Number (MSN)

The Structural Number (SN) concept was first developed from the AASHO Road test. It denotes a measure of total thickness of the pavement weighted according to the strength of each layer and is calculated using the equation given below:

$SN = \sum a_i$ ------------------- (3.7)

where, I = summation over layers, a_i= strength coefficient for each layer **(Table 3.7).**

d_i = thickness of each layer measured in inches

Table 3.7: Strength coefficients for pavement layers

Layer/Specification	Strength coefficients
Bituminous Concrete (BC) 40mm	0.3
Bituminous Concrete (BC) 25mm	0.28
Semi Dense Bituminous Concrete (SDBC) 25mm	0.25
Dense Bituminous Macadam (DBM)	0.28
Premix Carpet (PC) 20 mm(in the case of overlaidpavements with PMC as original surfacing)	0.18
Bituminous Macadam (BM)	0.18
Water Bound Macadam (WBM Gr I,II,or III) Wet mix macadam / (Lime cement) stabilized	0.14
Granular Sub base (GSB)/Quarry Rubish /Moorum	0.11

(Source: Guidelines for maintenance of primary, secondary and urban roads, Indian Roads Congress, 2004)

The AASHO Road Test was conducted on a single uniform subgrade and hence the effect of different types of subgrades could not be not included in the equation. Pavements with a particular structural number, but built on different subgrades may not carry the same traffic to a given terminal condition. But, the structural number did not include the contribution of the subgrade. In order to extend the structural number concept to all types of subgrade soils, a contribution in terms of CBR value of the subgrade was derived and a modified structural number was developed by the equation given below:

$$MSN = SN + 3.51 \log_{10}(CBR_s) - 0.85 (\log_{10} CBR_s)^2 - 1.43 \qquad \text{-----------} \quad (3.8)$$

where, CBR_s = California Bearing Ratio of the subgrade. **(Guidelines for maintenance of primary, secondary and urban roads, Indian Roads Congress, 2004)**

Many PPM's which were developed later used this equation for defining the pavement strength and is extensively being used.

3.4 HIGHWAY DEVELOPMENT AND MANAGEMENT SOFTWARE

Highway Development and Management Software (HDM-4) is an effective tool, which can be used for the analysis of different alternatives for management of highways (**Morosiuk et al.**, 2002). This facilitates make comparison of cost estimates and economic evaluations of different construction and maintenance options. Different time-staging alternatives also can be evaluated, for a given road project on a specific alignment or for groups of links on an entire network.

3.4.1 Applications: The applications of HDM-4 are Strategy Analysis, Programme Analysis and Project Analysis.

a) **Strategy Analysis:** Strategy analysis deals with entire networks or sub-networks managed by one road organisation. In this module, a chosen network can be analyzed as a whole to prepare medium or long term planning of investments. Different scenarios of road development can be analyzed.

   Typical applications of strategy analysis are**:**

   (a) Forecasting of funding requirements on medium and long term basis for pre defined road maintenance standards and targets.

   (b) Forecasting pavement performance under varying levels of funding on a long term basis.

(c) Optimum allocation of funds according to defined budget heads such as routine maintenance, periodic maintenance and budgets.

(d) Optimum allocations of funds to sub-networks. This can be allocated by functional road class (main, feeder and urban roads etc.), administrative region etc.

(e) Policy studies also can be done. This includes impact of changes to the axle load limit, energy balance analysis, provision of NMT facilities, sustainable road network size, pavement maintenance standards, evaluation of pavement design standards, etc.

HDM-4 applies the concept of a road network matrix to predict the medium to long term requirements of an entire road network or sub-network. This consists of a road network matrix with categories of the roads defined according to the key attributes that has influence on the performance of pavements and road user costs. The users can define the road network matrix to represent the most important factors affecting transportation costs in the area. The road network matrix can be categorized according to Pavement types, Pavement condition, Traffic volume or loading and Environment or climatic zones.

b) Programme analysis: The prioritization of a defined list of road projects into a one-year or multi-year work programme with defined budget allocations can be done in the programme analysis module. The major difference between strategy analysis and programme analysis is in the physical identification of road links and sections. Programme analysis deals with individual links and sections, which are unique physical units identifiable from the road network throughout the analysis. This can also be used to prepare a multi-year rolling programme, within the resource constraints. Incremental NPV/cost ratio is used as the ranking index, which provides an efficient and robust index for prioritization purposes. Also, it satisfies the objective of maximizing economic benefits for each additional unit of expenditure proposed.

c) **Project analysis:** One or more road projects or investment options are analyzed in Project Analysis. A road link or section with user-selected treatments, with associated costs and benefits, projected annually over the analysis period can be analyzed. For different investment options, economic indicators are determined. Project analysis allows the users to assess the physical, functional and economic feasibility of specified project alternatives by comparison against a base case (do nothing). The key parameters are structural performance of Pavement, prediction of Life cycle costs, deterioration, Maintenance effects & costs, Road user costs & benefits and Economic evaluation of project alternatives.

3.4.2 Analysis Methods: The two methods of analyzing investment options provided in HDM-4 Project Analysis are; Analysis by Section and Analysis by Project.

a) **Analysis by Section:** In the method of Analysis by Section, each of the road sections selected for the project are analyzed separately. Several alternatives like maintenance and/or improvement standards can be defined for any of the section, with one alternative designated by the user as the base alternative and all other alternatives will be compared with the base alternative. Economic indicators (e.g. NPV, IRR and NPV/C) are calculated for each section alternative.

b) **Analysis by Project:** In the method of Analysis by Project, a project is defined as the set of road works to be carried out on one or more road sections that can be grouped together conveniently to be undertaken as one contract or work instruction. Several project alternatives can be analysed to determine the most cost-effective option. Analyses involving new sections and diverted traffic can be performed only using this method.

3.4.3 HDM-4 Modules

The three analysis tools (Strategy, Programme and Project) work on the data defined by one of the four data managers namely Road Network, Vehicle Fleet, Road Works and HDM Configuration.

The Road Network module contains the physical characteristics of road sections in a network or sub-network, which will be analysed. It provides the basic facilities for storing data of one or more road sections. The users can define the networks and sub-networks, and road sections, and it is the fundamental unit of analysis. Sections, Links and Nodes are the data entities provided within the road network.

The Vehicle Fleet module contains facilities for the storage and retrieval of vehicle characteristics data necessary for calculating speeds, operating costs, travel time costs and other effects. Multiple fleet data sets can be provided for doing different analyses, and a notable range of default data also is provided.

The Road Works module defines maintenance and improvement standards, along with their unit costs. This will be applied to different road sections to be analysed. Also, the standards defined in the Road Works Standards folder can be used in any of the three analysis tools namely Project analysis, Programme analysis or Strategy analysis**.**

The HDM Configuration module defines the default data used in the applications. When HDM-4 is first installed, a set of default data is provided, but the users can modify these to suit to local conditions.

3.4.4 Procedure for Project Analysis:

1. The road project to be analyzed is created by giving it a title. Specify the road network to be analyzed.
2. General information about the project is specified and the project is defined. Define the method of analysis and Road sections to be analyzed.
3. The maintenance and improvement standards proposed for each selected road section is provided. Then go to Set-up and run the analysis.
4. The reports are generated and if necessary, print the required outputs.

3.4.5 Maintenance & Rehabilitation Activities:

As per the 'Report of the Committee on Norms for Maintenance of Roads in India' [MORT&H 2001], Maintenance & Rehabilitation (M&R) treatments are actions taken on a given section to either reduce the pavement deterioration rate (prevention) or to repair the effects of deterioration. These have been categorized as Ordinary Repairs (Routine Maintenance) and Periodic Renewals (Periodic Maintenance). In HDM-4, maintenance standards are defined to set the targets or levels of condition and responses that are to be achieved. Routine and periodic maintenance are the two kinds of maintenance treatment available in HDM-4. All maintenances can be carried out based on scheduled and condition-responsive out puts. But overlays are always defined in terms of condition responsive works.

The routine maintenance works on bituminous roads are patching, crack sealing, edge-repair, and drainage works. The effects of these on the performance of pavements are modeled. The periodic maintenance works on bituminous roads are preventive treatment, resealing, overlay, mill & replace, inlays, and reconstruction.

3.4.6 Alternative Maintenance Strategies: The most recent definition of preventive maintenance by AASHTO Standing Committee on Highway states that preventive maintenance is “A planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional conditions of the system without increasing structural capacity”. For adopting different Maintenance options, various Maintenance and Rehabilitation Works Data is to be collected.

3.4.6.1 Serviceability Levels for Maintenance: [MORT&H, 2004] Serviceability levels for maintenance is a qualitative rating of the effectiveness of a highway in terms of operating conditions such as traffic volume, speed, comfort and safety. The maintenance program can be divided into three levels, level 1, 2 and 3 for maintenance purposes. Level 1 is the desired level that provides for highest level of comfort, convenience and safety. Level 2 is the level to which the road deteriorates from Level 1 after two-three years of use before fresh maintenance is implemented. Level 3 represents the minimum level necessary to protect the investment and provide reasonable levels of safety.

The suggested serviceability levels and the limiting levels of surface defects based on measurement of roughness, cracking, rutting etc., are given in **Table 3.8.**

3.4.7 Economic Analysis: The 'Committee for Maintenance Norms for Roads in India', (2001), has recommended the total costs for carrying out various types of maintenance and rehabilitation (M&R) works on bituminous pavements situated in various price zones of the country. The economic indicators such as Net Present Value (NPV) and Internal Rate of Return (IRR) are also provided for comparison purposes.

Table 3.8 Serviceability Levels for Maintenance

Sl. No	Serviceability Indicator	Serviceability Levels		
		Level 1	Level 2	Level 3
1	Roughness by Bump Integrator (max. permissible) Equivalent IRI	2000mm/km 2.8m/km	3000mm/km 4.0m/km	4000mm/km 5.2 m/km
2	Potholes per km(max. number)	Nil	2-3	4-8
3	Cracking and patching area (max permissible)	5%	10%	10-15%
4	Rutting – 20 mm (maximum permissible)	1%	1.5%	2.5%
5	Skid number (minimum desirable)	50SN	40SN	35SN

Source: (MORT&H, Guidelines for maintenance of primary, secondary and urban roads, Indian Roads Congress, 2004)

(i) Net Present Value (NPV) criterion: Under this method the cost and benefits due to the project over an analysis period of 10 years is estimated and discounted at a predetermined discount rate of 10%. Benefits are treated as positive and cost as negative values and the summation gives the NPV. The NPV is expressed as follows:

$NPV= (B_O-C_O) + (B_1-C_1)/ (1+ r/100) +(B_2-C_2)/(1+r/100)^2(B_{10}-C_{10})/(1+r/100)^{10}$ ... (3.9)

where, NPV = Net Present Value

B_1 = Benefits in year 1; C1 = Cost in year 1; R = Rate of discount chosen

(ii) Internal Rate of Return (IRR): The IRR is the discount rate at which the discounted cost stream and discounted benefit stream are equal. It is expressed as follows:

$NPV = (B_1-C_1)/(1+ r/100)+(B_2-C_2)/(1+r/100)^2……(B_n-C_n)/(1+r/100)^n$

$$\text{NPV} = \sum_{t=1}^{n}(Bt - Ct)/\left(1+\frac{r}{100}\right)^t \qquad \text{--------------- (3.10)}$$

3.5 APPLICATION OF FUZZY LOGIC

In the present study, a fuzzy rule based system was developed to represent the expert knowledge using Fuzzy Rules, 'If- Then'. Linguistic Fuzzy model was used.

3.6 DATA ANALYSIS AND MODEL DEVELOPMENT

In the present study, pavement condition analysis for urban roads was done. Relationship between modified structural number and deflection were found out for predominant types of subgrade soil. Both linear and non linear relationships were tried. Influence of pavement condition on MSN - Deflection relationships were studied using appropriate plots. For roads other than urban roads, periodic evaluation was done and the data was used to develop pavement deterioration models. The regression models developed using SPSS tool was compared with models developed using HDM 4 and Fuzzy Logic. HDM 4 was used to find out the best maintenance option with selected alternatives. The relationship between Skid resistance and Texture Depth was tried to get a best fit model. The RCI was related with unevenness using appropriate plots.

3.7 SUMMARY

The methodology used for the study is discussed in this chapter. The instrumentation and techniques used for field and laboratory investigations were inventory survey, drainage study, condition survey, unevenness survey, skid resistance study, deflection study, texture depth study, pavement composition and field density measurement. Laboratory investigations of the subgrade soil are also discussed. Applications of SPSS, HDM-4 and Fuzzy logic adopted in the study for developing deterioration models are briefly outlined. The computation of Riding Comfort Index (RCI), Unevenness Index (UI), Pavement Condition Index (PCI), Structural Number (SN) and Modified Structural Number (MSN) are also discussed. Models developed in the study are also mentioned in this chapter.

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Chapter 4

STUDY AREA, FIELD INVESTIGATIONS AND RESULTS

4.1 STUDY AREA

The present research study is divided into two parts. 44 in-service urban roads distributed in the five Corporations of the State were selected for the first part of the study. The data on the sub grade soil properties and deflection characteristics of 68 stretches in these urban settings were collected for analysis and development of relationships with pavement condition, soil properties and deflection. The findings based on the analysis of the results and plots are discussed in Chapter 5.

The second part of the study focuses on to develop pavement deterioration models applicable to Kerala conditions using time series data. Eight road sections representing NH, SH and MDR with variation in traffic composition, soil properties, climate, drainage characteristics and land use were selected. Periodic data were collected from the field. The plots and developed models are described in Chapter 6.

4.1.1 Urban Roads selected for the study

From a preliminary study of the traffic and condition of the roads, 44 road sections were selected from five corporations namely Thiruvananthapuram, Kollam, Kochi, Thrissur and Kozhikkode. The roads were selected based on a preliminary assessment of the road condition, traffic, importance of the road, variations in soil type, climatic conditions and terrain. These roads were again divided into 68 homogeneous sections for further studies. The list of roads is given in **Table 4.1.**

Table 4.1 List of Urban Roads selected for study

Sl No.	Road Name	Length km	No. of HS	Section ID
I	**THIRUVANANTHAPURAM CORPORATION**			
1	NH Bypass - Veli	2.00	2	TR 01 & TR 02
2	PMG – Law College	0.50	1	TR 03
3	Peroorkada - Pipinmoodu	2.00	2	TR 04 & TR 05
4	Pipinmoodu - Sasthamangalam	0.80	1	TR 06
5	Sasthamangalam - Edapazhanji	1.00	1	TR 07
6	Edapazhanji - Jagathy	1.40	1	TR 08
7	Jagathy - Killipalam	1.90	1	TR 09
8	Uloor - Kesavadasapuram	1.20	1	TR 10
9	Poojappura - Thirumala	2.50	3	TR 10, TR 11 & TR 12
10	Valiyavila - Peyad	2.50	3	TR 13, TR 14 & TR 15
11	Attakulangara - Manacaud	4.40	1	TR 16
12	Manacaud – NH byepass		2	TR 18 & TR 19
II	**KOLLAM CORPORATION**			
13	Kappalandi mukku- Jawahar Jn	0.28	1	KL 20
14	Jawahar Jn – Kadappakkada Jn	1.16	1	KL 21

15	Kadappakkada – Ashramam Jn	1.5	1	KL 22
16	Anandavalleeswaram temple - Wadi	1.20	1	KL 23
17	Kochupilamoodu Jn – AR Camp Jn	0.70	1	KL 24
18	Mundalummoodu - Thirumullavaram	1.60	2	KL 25 & KL 26
19	Lekshmi Nada - Valaittambalam	3.00	2	KL 27 & KL 28
20	AR Camp Jn- Kappalandimukku jn	1.00	1	KL 29
III	**KOCHI CORPORATION**			
21	Karshaka Road	0.40	1	KO 30
22	NH Byepass – Palarivattom	0.70	1	KO 31
23	Palarivattom – Vytilla Road	4.00	2	KO 32 & KO 33
24	MG Road	4.30	2	KO 34 & KO 35
25	High Court Jn – Edappally	6.60	3	KO 36 , KO 37 & KO 38
26	SA Road	3.25	2	KO 39 & KO 40
27	Chittoor Road	2.60	1	KO 41
28	Thoppumpady – beach – Fort Kochi	6.10	3	KO 42, KO 43 & KO 44
29	TD Road	1.80	1	KO 45

30	Church Landing Road	0.50	1	KO 46
31	DH Road	0.60	1	KO 47
32	Hospital Road	0.60	1	KO 48
IV	**THRISSUR CORPORATION**			
33	Kizhakkumpattukara – NH byepass	3.40	3	TH 49, TH 50 & TH 51
34	KSRTC Jn – Koorkanchery	2.40	2	TH 52, & TH 53
35	West Fort - Ollari	2.60	3	TH 54, TH 55 & TH 56
36	Chiyyaram - Koorkanchery	2.40	2	TH 57 & TH 58
V	**KOZHIKKODE CORPORATION**			
37	Arayidathupalam - Eranjipalam	2.90	3	KZ 59, KZ 60 & KZ 61
38	Jail Road	0.30	1	KZ 62
39	Oyitti Road	0.70	1	KZ 63
40	Francis Road	1.20	1	KZ 64
41	Red cross Road	0.43	1	KZ 65
42	Gandhi Road	1.00	1	KZ 66
43	PT Usha Road & Bazar Road	1.20	1	K Z 67
44	PT Usha Road	1.10	1	K Z 68
	TOTAL HOMOGENOUS SECTIONS & TRIAL PITS		**68**	

The study area locations are shown in **Fig. 4.1 to 4.5.**

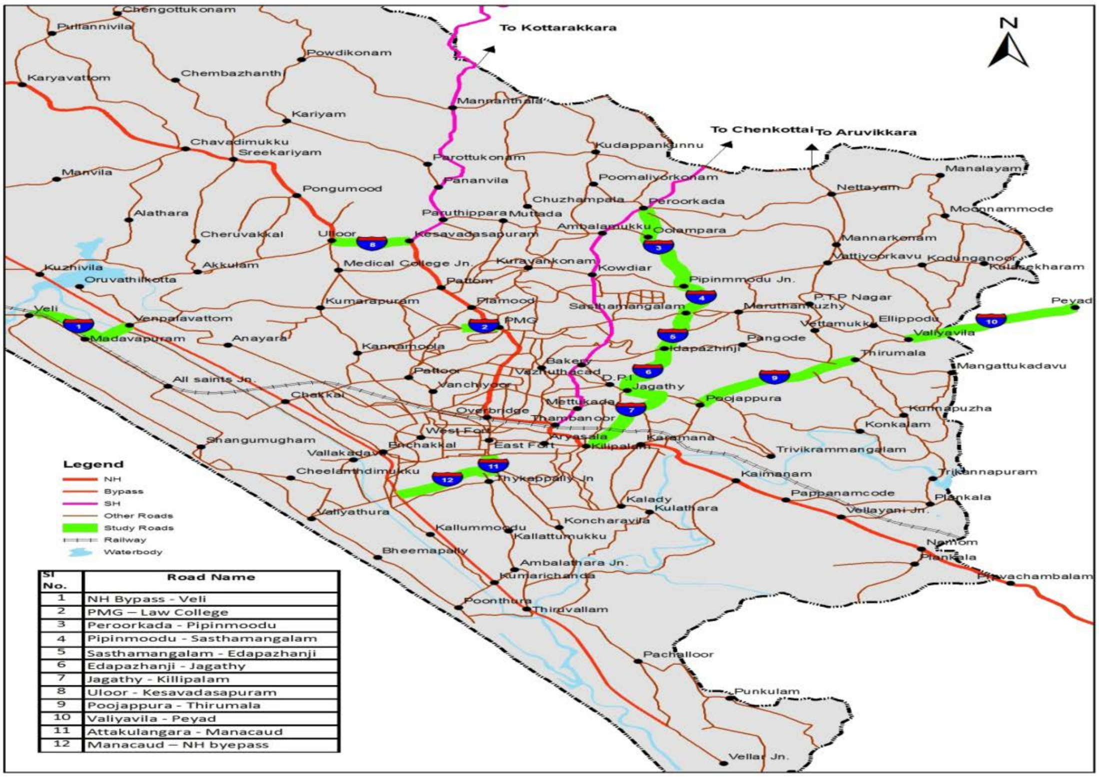

Fig. 4.1: Study Roads in Thiruvananthapuram Corporation

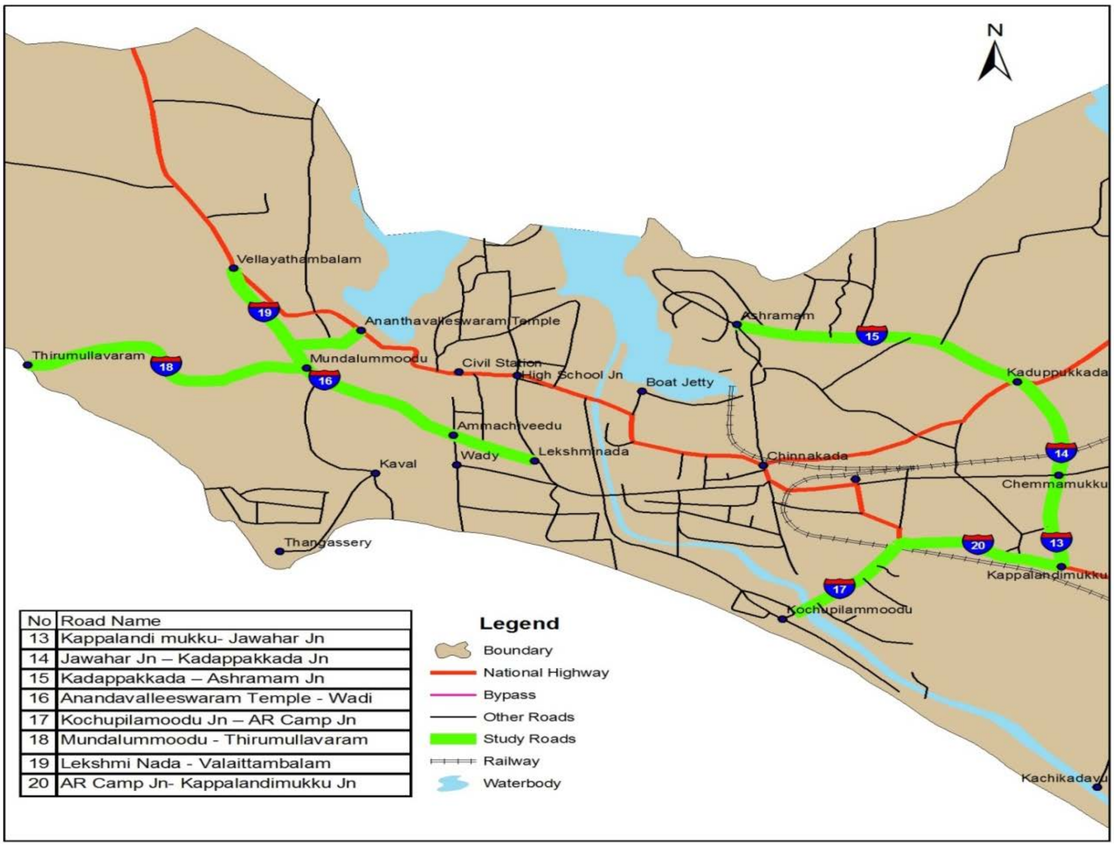

Fig. 4.2: Study Roads in Kollam Corporation

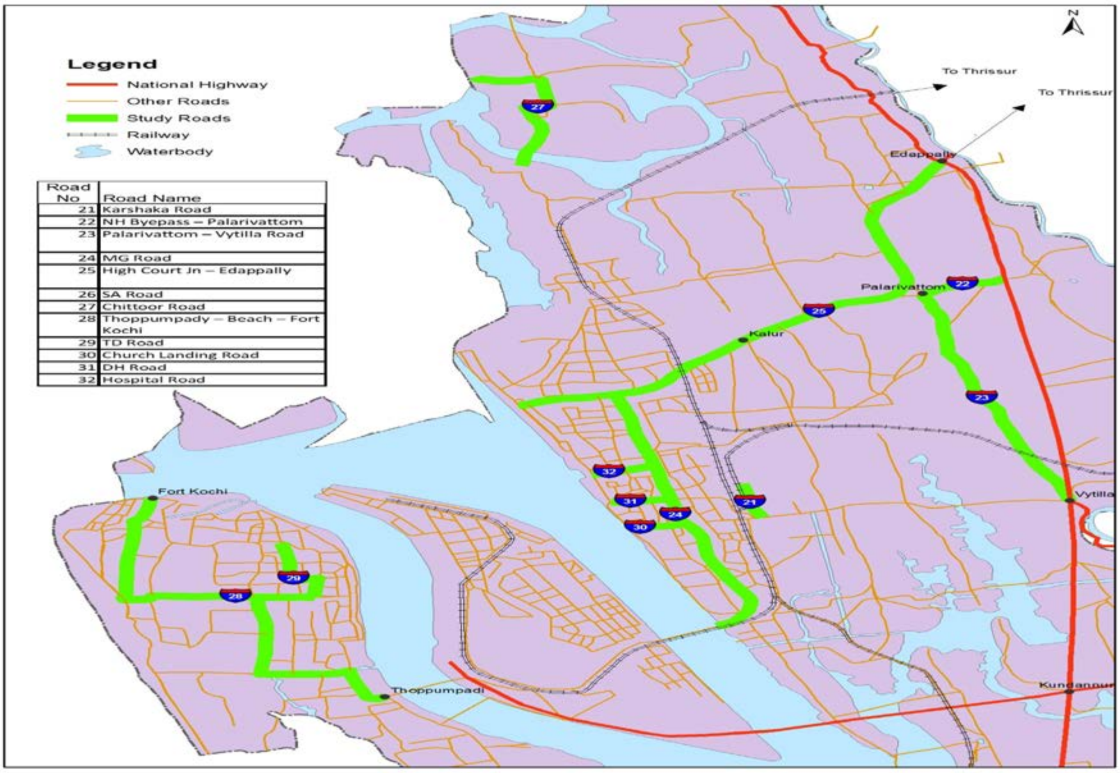

Fig. 4.3: Study Roads in Kochi Corporation

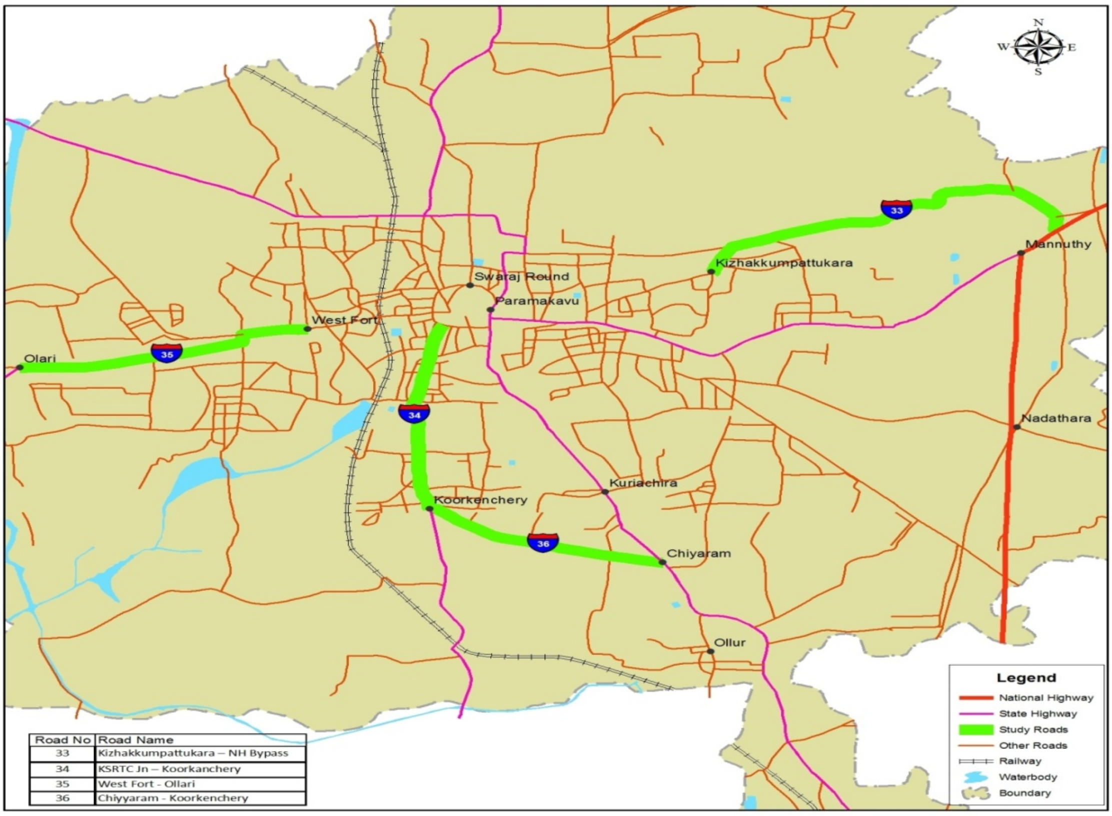

Fig. 4.4: Study Roads in Thrissur Corporation

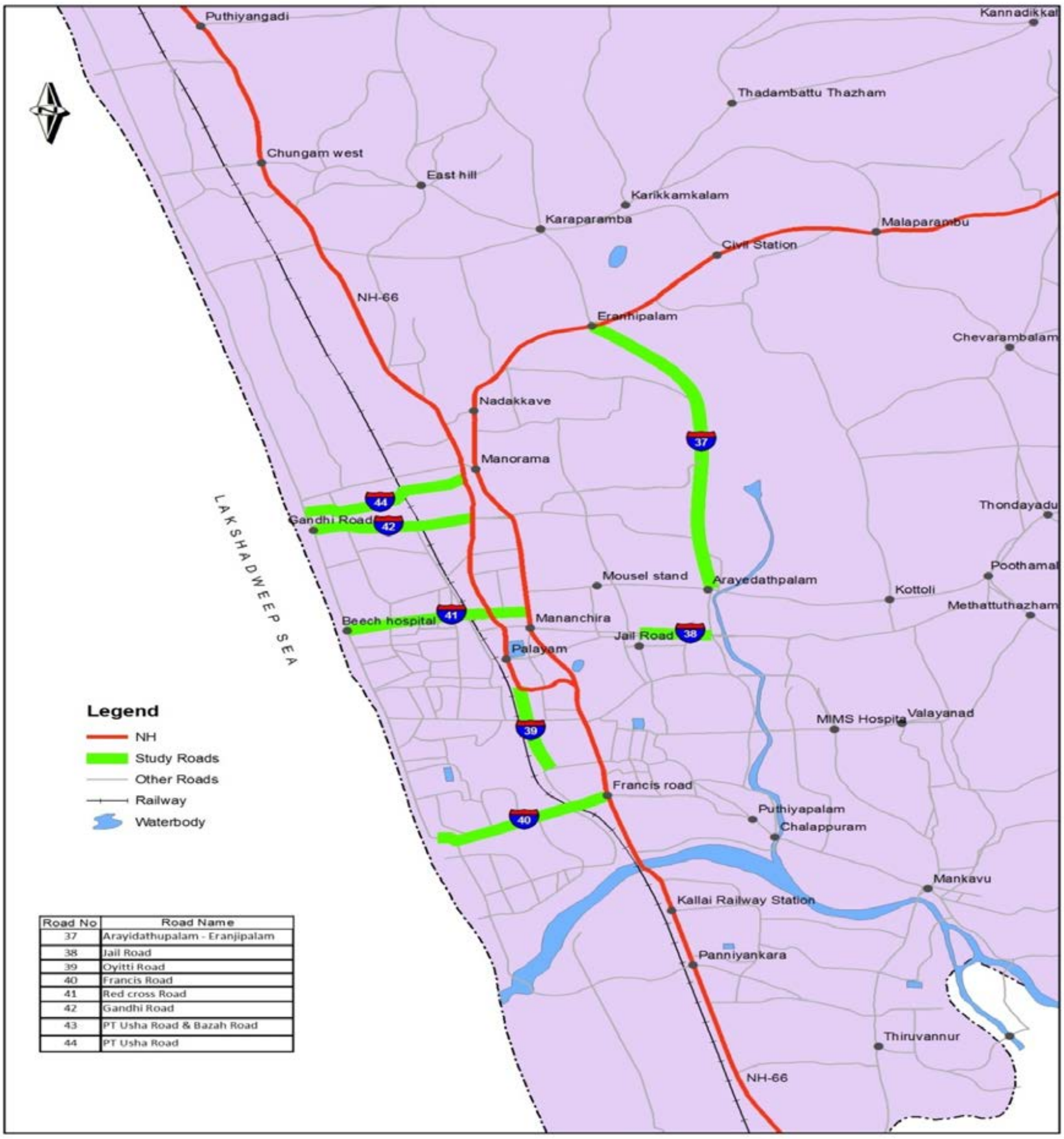

Fig. 4.5: Study Roads in Kozhikode Corporation

4.1.2 Road Stretches selected for periodic Pavement Evaluation

Study stretches were selected based on the category of road, terrain, traffic characteristics, geographical location, land use etc. 15 Homogeneous sections were selected for detailed study and periodic data collection. Road stretches selected for the present study are listed below in **Table 4.2.**

Table 4.2: Road Stretches Studied for pavement evaluation

Sl. No	Section	Stretch ID	Category	Pavement History	Terrain	Length (km)
1.	Kesavadasapuram - Plammoodu	KP	NH 66 (Old NH 47)	4 lane, urban section, Widened in 2003	Plain	2
2.	Attingal-Kallambalam	AK	NH 66 (Old NH 47)	2 lane, rural road. Overlay 50 mm BC in 2008	Plain	8
3.	Kazhakuttam-Kovalam	KO	NH 66 Bypass (Old NH 47)	2 lane, new surface & old surface	Plain	22
4.	Chavadimukku-Pallippuram	CP	NH 66 (Old NH 47)	2 lane, sub-urban, Surfacing in 2003	Plain	10
5.	Varkala - Kallambalam	VK	ODR	2 lane, Last resurfacing BM - 60/70,6 cm thick MSS- 20 mm in 2006	Plain	11.3
6.	Kottayam - Kumily	KK	NH 183 (Old NH 220) HS-1 HS-2 HS-3	2 lane, Overlay during the study period 50 mm BM, 20 mm BC in 2003, PMC, 2003 75 mm BUSG in 2005	HS I - Plain HS II - Plain HS III – Hilly	108
7.	Mannanthala - Venjaramood	MV	SH 1	2 lane, sub- urban, Strengthened in 2004 (KSTP)	Plain	17
8.	Seaport- Airport	SA	NH Standard	2 Lane (2004)	Plain	13

4.1.2.1 Kesavadasapuram - Plammoodu (NH 47)

Kesavadasapuram - Plamoodu road, 2km long stretch, is a part of National Highway 66 (formerly known as National Highway 47) connecting Panvel to Kanyakumari. This study section lies within the Thiruvananthapuram city and was upgraded to 4 lane divided carriageway in the year 2000. Main Central Road (SH-1), which connects Angamaly and Thiruvananthapuram, originates from Kesavadasapuram on National Highway 66 (NH-66). There are three major intersections on the study road stretch namely Plamoodu, Pattom and Kesavadasapuram. The study road has an average carriageway width of 14 m with central median of 0.5m. The road is characterized by mixed land use with number of commercial establishments on either side. **Fig. 4.6** shows the map of the study road stretch on National Highway 66.

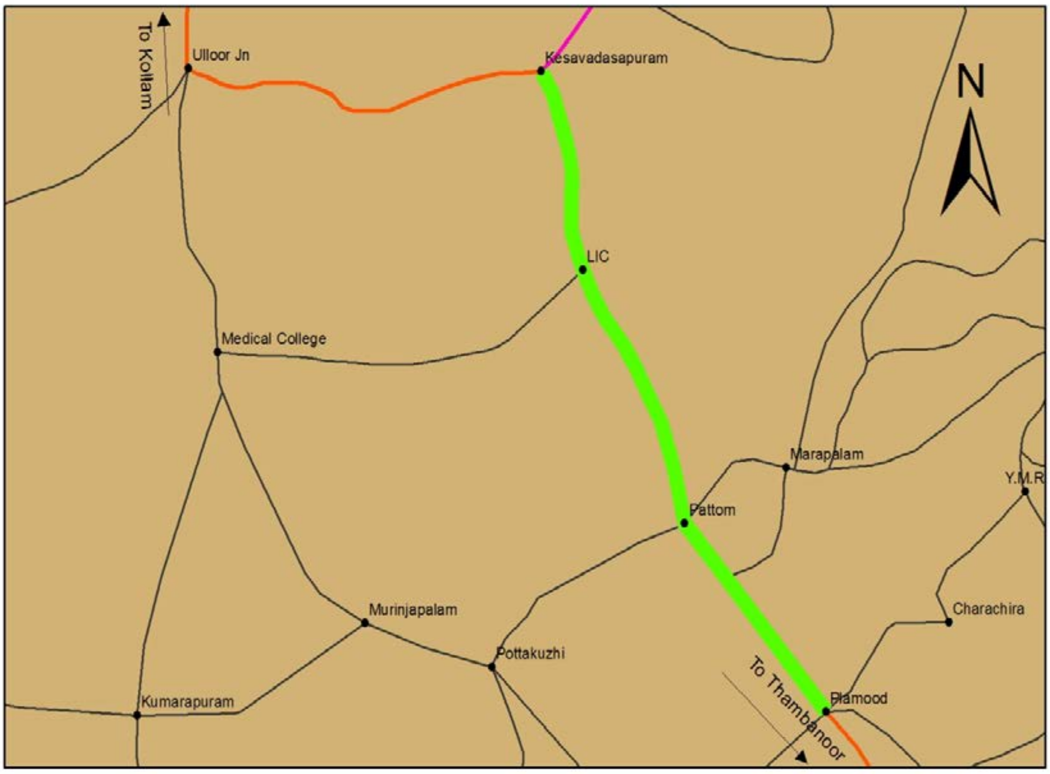

Fig. 4.6: Map showing Kesavadasapuram - Plamoodu study stretch on NH 66

4.1.2.2. Attingal – Kallambalam

Attingal – Kallambalam road, of length 8km, is a two-lane road on plain terrain and form part of National Highway 66 connecting Panvel to Kanyakumari. Attingal is the important town connecting Trivandrum and Kollam cities. The study road stretch passes through suburban area and has four major intersections namely Attingal KSRTC junction, Kacheri Nada, Alamcode and Kallambalam. State Highway (SH-46) connecting Alamcode on NH 66 and Kilimanoor on SH 1 lies in this road stretch. The road is characterized by mixed land use with number of commercial establishments at the junctions and open land use along the stretch on both sides. Government Homeo Hospital, Attingal civil station, four educational institutions, and other important government institutions of the taluk are situated in this road stretch. **Fig. 4.7** shows the map of the selected study road stretch on National Highway 66.

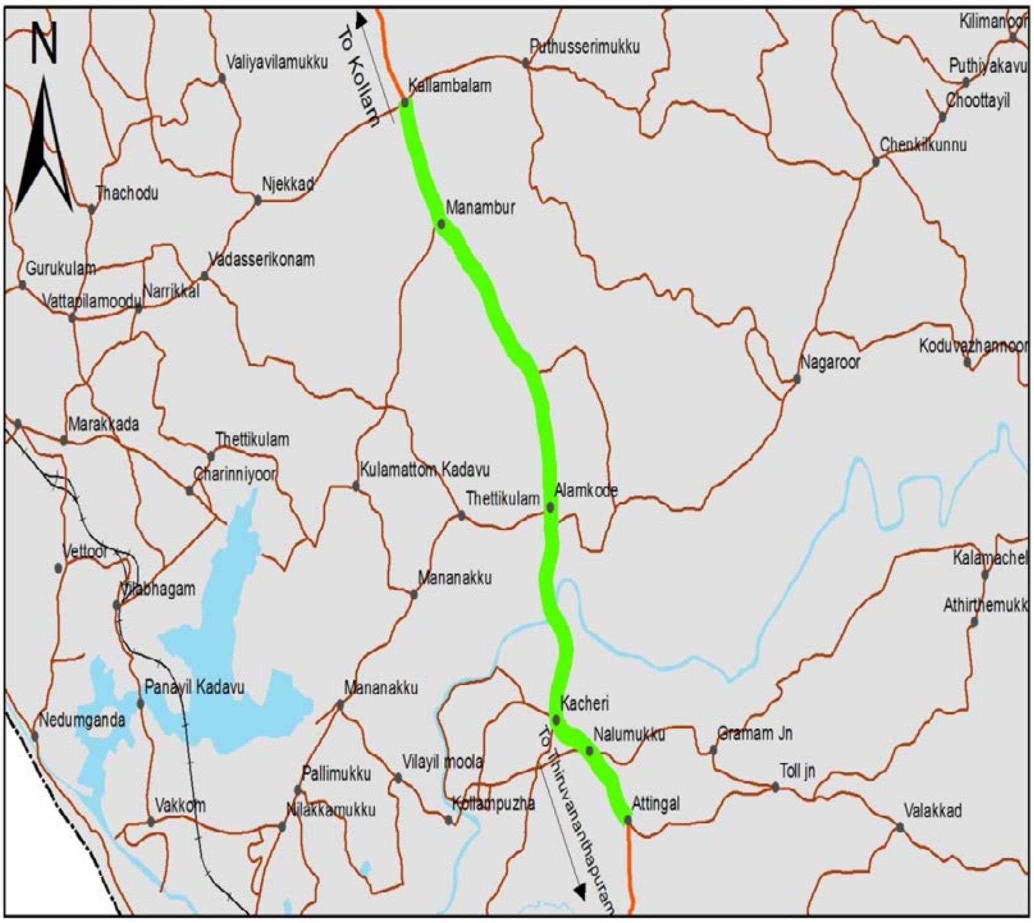

Fig. 4.7: Map showing Attingal - Kallambalam study stretch on NH-66

4.1.2.3. Kazhakkuttam - Kovalam (NH 66 bypass)

Kazhakkootam - Kovalam road stretch, of length 22.4km, is two lane roads on plain terrain and forms a part of National Highway (NH 66) bypass connecting Kazhakkoottam in Thiruvananthapuram to Tamil Nadu. Widening of NH bypass from

Kazhakkoottam to Mukkola Jn, to four lane divided carriageway is in various stages of construction. The four laning of NH bypass will help to decongest the traffic in Thiruvananthapuram city. This road provides connectivity to Kovalam and Vizhinjam which are the hotspots of international tourism and trade, Air Port, Bus Terminus, Technopark etc. The study road stretch is characterized by mixed land use near to Kazhakkootam and open land use after Thiruvallam. **Fig. 4.8** shows the map of the selected study road stretch on NH 66 Bypass road.

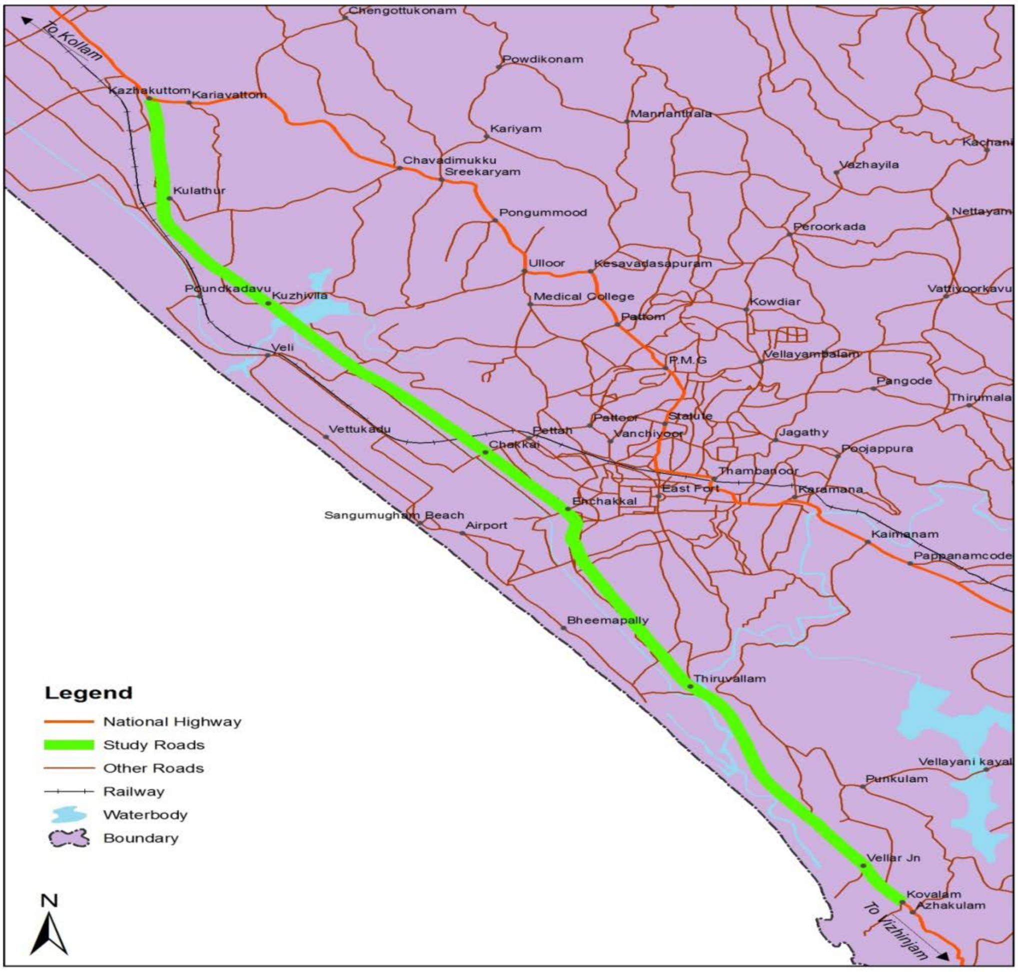

Fig. 4.8: Map showing Kazhakkootam - Kovalam study stretch on NH-66 Bypass

4.1.2.4. Chavadimukku - Pallippuram (NH 66)

Chavadimukku - Pallippuram road section, of length 10.4 km, is a two lane road on plain terrain and forms a part of National Highway 66 connecting Panvel to Kanyakumari. Two major roads join National Highway 66 in this road stretch-namely Vettu Road and National Highway Bypass Road. Vettu Road developed as a part of Kerala State

Transport Project (KSTP) connects Main central road (SH-1) with National Highway-66. National Highway Bypass road starts from Kazhakootam in Thiruvananthapuram and terminates at Kovalam. Major intersections in this road stretch are Chavadimukku, Kariyavattom, Kazhakkootam, Vettu Road Junction, and Pallippuram. The proposed alignment for monorail for Thiruvananthapuram city passes through this study road stretch and terminates at Pallippuram. In addition to Kariavattom University campus and Technopark, the International Multi-Purpose Greenfield Stadium at Kariyavattom and Technocity at Pallippuram are the two major projects under construction in this road stretch. The land use is characterized by mixed land use on either sides of the road throughout the study stretch. **Fig. 4.9** shows the map of the selected study road stretch on National Highway 66.

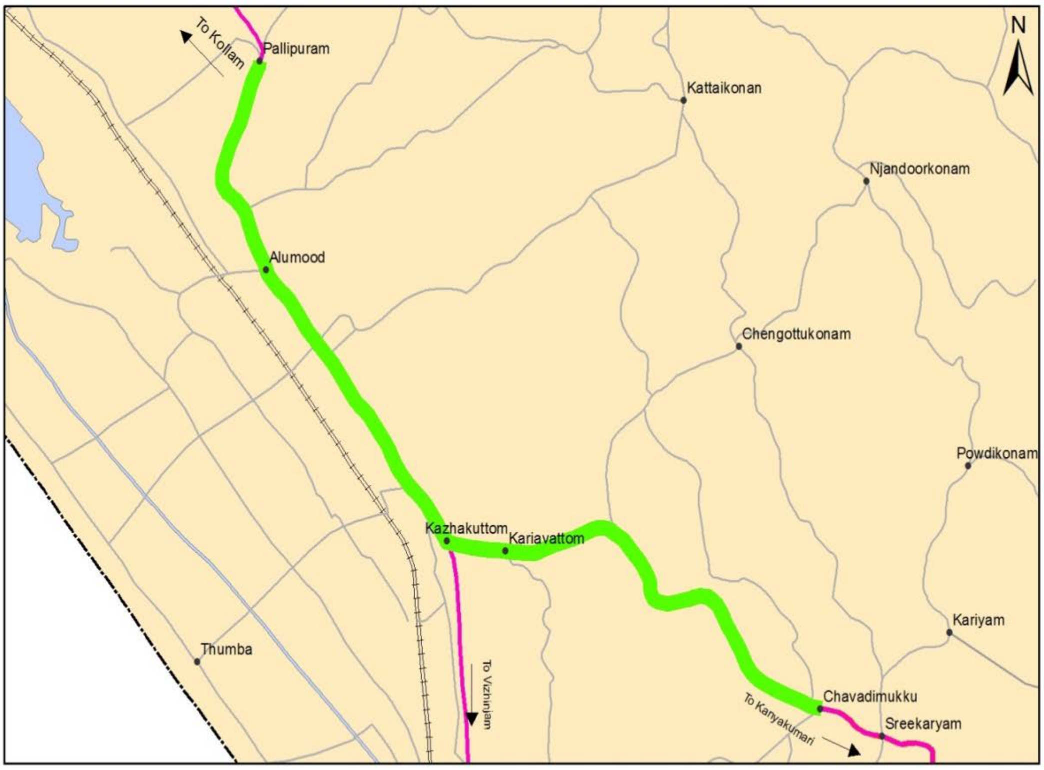

Fig. 4.9: Map showing Chavadimukku - Pallippuram study stretch on NH-66

4.1.2.5. Kallambalam - Varkala

Kallambalam - Varkala road, of length 11.3 km, is a two lane road comes under the category of Other District Road (ODR). This road stretch provides connectivity to popular tourist spots, Varkala beach and Sivagiri Maddam with National Highway 66. Varkala - Madathara road (SH-64) crosses National Highway 66 at Parippally and terminates in the study road stretch. Varkala railway station is located near to Kallambalam-Varkala road. Major intersections in this road stretch are Kallambalam, Njekkad, Narikkal, Naalumukku and Railway station road junction. Map of the selected study road stretch is shown in **Fig. 4.10.**

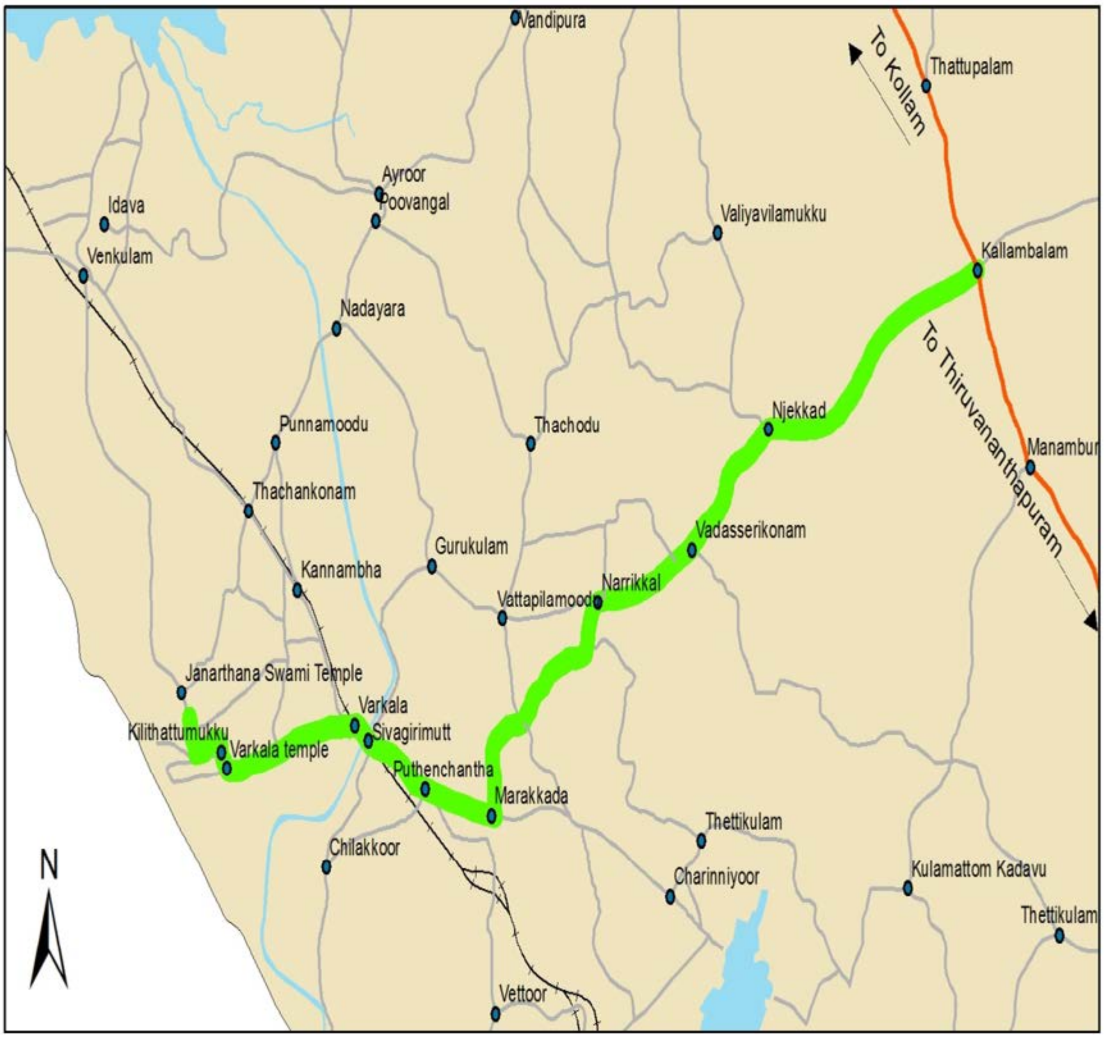

Fig. 4.10: Map showing Kallambalam - Varkala study stretch

4.1.2.6. Kottayam-Kumily Road (NH 183)

Kottayam-Kumily road (108km) is a two lane road and forms a part of National Highway 183 (formerly known as NH 220), which connects Kollam in Kerala with Dindigul in Tamil Nadu. The study road stretch starts from Kottayam and runs along the northern border of Periyar Wildlife Sanctuary connecting Kanjirappally, Peerumade, Vandiperiyar and terminates at Kumily. Kottayam-Kozhencherry road (SH-9), Main Eastern Highway (Punalur Muvattupuzha Road – SH8), Sabarimala – Kodaikkanal Road

(SH-44), Hill Highway (SH-59) and Ernakulam – Thekkady Road (SH-41) are the major roads intersecting with the study road stretch. Kottayam-Kumily road has a varying terrain conditions from plain terrain to hilly terrain throughout its entire length. **Fig. 4.11** shows the map of the selected study road stretch on National Highway 183. Road Composition and pavement history data as obtained from PWD (NH Sub Division) are as follows:-

HS-1 section 50 BM + 25mm BC was laid in April 2003. HS-2 is an unimproved section with only chipping carpet layer laid in 2003. HS-3 is in a hilly terrain, the widening of which was done using 75mm Built up Spray Grout (BUSG). 20mm Mixed Seal surfacing was done for the entire section in 2005.

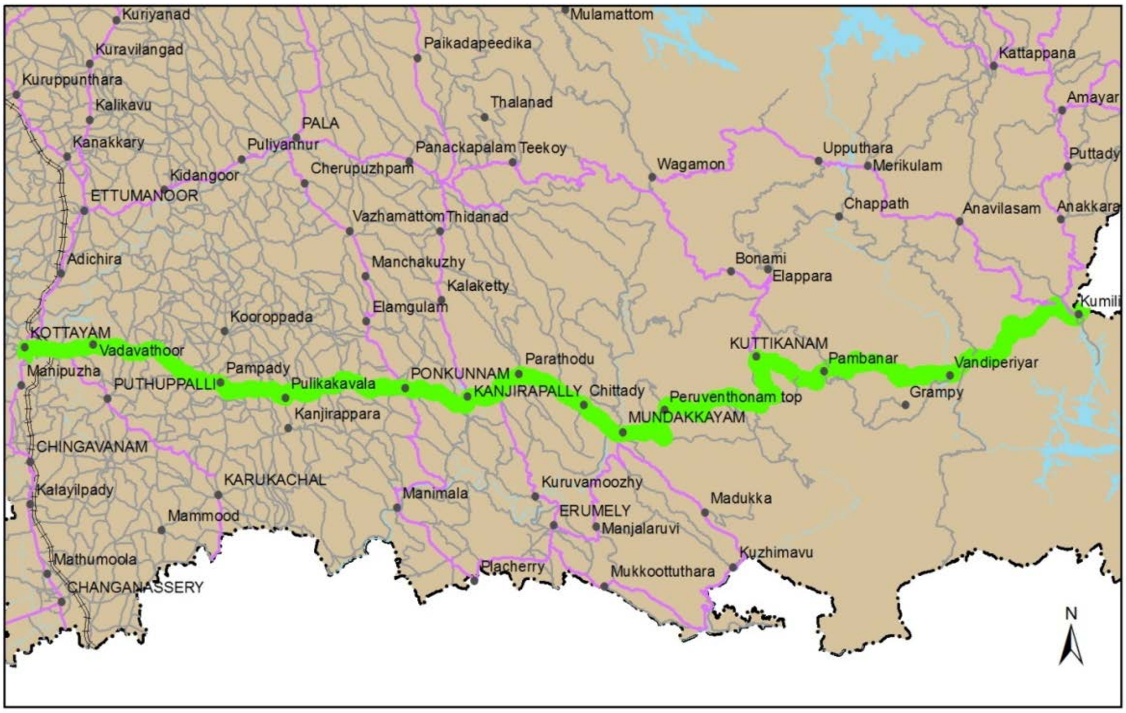

Fig. 4.11: Map showing Kottayam - Kumily study stretch on NH-183

4.1.2.7. Mannanthala - Venjaramoodu (SH-1)

Mannanthala – Venjaramoodu road stretch, of road length 17km, forms a part of Main Central Road (SH-1), which connects Angamaly in Ernakulam and Kesavadasapuram in Thiruvananthapuram. MC Road connects many important towns of Central and South Kerala. The study road stretch is a two lane sub urban section of SH-1, which was improved under the Kerala State Transport Project (KSTP) under World Bank funding.

Attingal - Nedumangad Road (SH-47) and Vettu Road are the two major roads intersecting with the study road stretch. Major intersections in the study road stretch are Mannanthala, Vattapara, Vembayam and Venjaramoodu. KSRTC Bus Stand at Venjaramoodu is located in this road stretch. **Fig. 4.12** shows the map of the selected study road stretch on Main Central Road (SH-1).

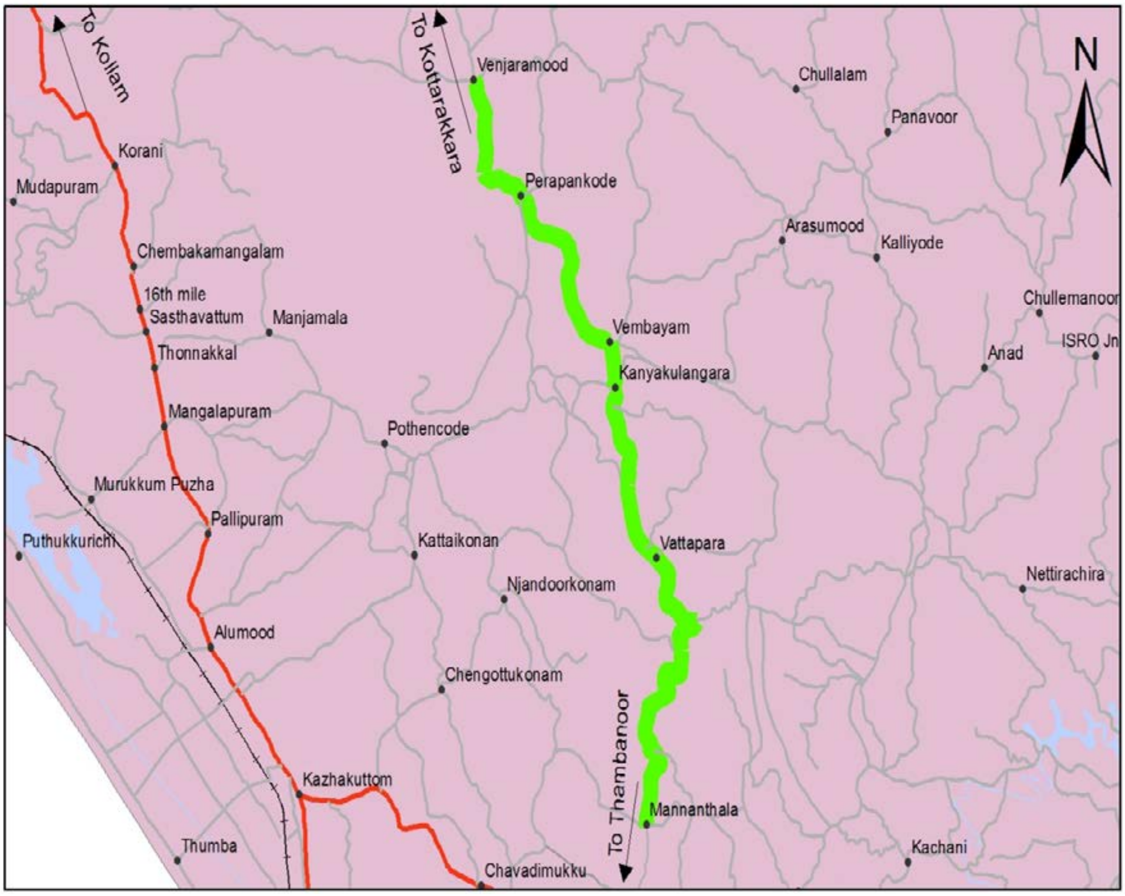

Fig. 4.12: Map showing Mannanthala - Venjaramoodu study stretch on SH-1

4.1.2.8. Seaport – Airport Road (HMT-Karingachira)

The proposed Seaport-Airport Road (30km) connects Cochin seaport to the Cochin International Airport aimed at improving the transport infrastructure in the Kochi city. First phase development of Seaport-Airport road of length 13 km, between HMT in Kalamassery and Karingachira in Tripunithura, was constructed by Roads and Bridges Development Corporation Kerala Ltd (RBDCK) and conforms to NH standards. The study road stretch has a road length of 13km starts from HMT in Kalamassery and terminates at Karingachira in Tripunithura. It is a two lane road on plain terrain characterized with goods movement. The study road acts as a by-pass to the NH 66 within the Kochi city limits. Seaport-Airport Road passes through the Cochin Special Economic Zone and connects major industrial units such as HMT, FACT, Kochi Refineries and various oil terminals at Irumpanam. **Fig. 4.13** shows the map of the selected study road stretch on Seaport-Airport Road.

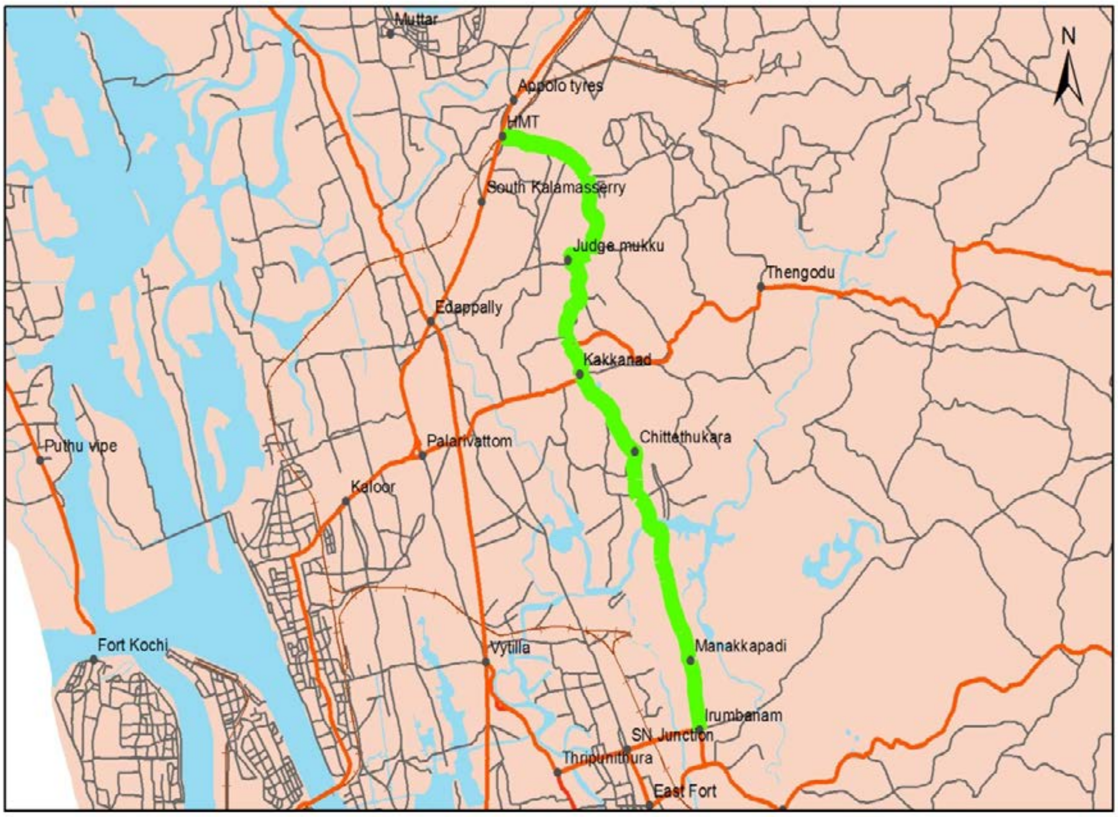

Fig. 4.13: Map showing the Seaport- Airport study Road

4.2 FIELD INVESTIGATIONS AND LABORATORY TESTING

The pavement condition and strength evaluation data were collected from different sections of roads at Trivandrum, Kollam, Ernakulam, Thrissur and Calicut Corporation limits. The following procedure was adopted:

- Pits were dug over the shoulder area just near to the carriageway edge and the material from each layer was taken.
- The pavement layer details about type, thickness etc. for base course, sub base, if provided, and surface course were obtained from the design details or by the field investigation of the materials in the pavement layer.

- The soil material from the sub grade layer was subjected to sieve analysis and tests for determining Atterberg limits to determine the plasticity indices. The soil types were assigned depending upon the percentage of particle size in the material and the liquid limits. The material was then subjected to compaction test for determining the Optimum Moisture Content (OMC) and maximum dry density.

- The field dry density and field moisture content of the soil material were determined by Sand Replacement Method and calcium carbide method.

- CBR values were also determined in the laboratory.

- The deflection of the pavements was measured using Benkelman Beam Deflection (BBD) method.

- Condition Survey of the pavements were done to collect the details of cracks, potholes, patching and other defects and represented as per IRC guidelines.

4.3 RESULTS

4.3.1 Subgrade soil properties for urban roads

The results of the sub grade soil analysis are given in **Table 4.3.** The values indicated that the soil predominantly belong to SC and SM class.

Table 4.3 Subgrade Soil Properties for urban roads

Sl. No	Section ID	LL (%)	PL (%)	PI	Grain size Distribution			Soil Type
					Gravel (%)	Sand (%)	Silt & Clay (%)	
	THIRUVANANTHAPURAM CORPORATION							
1	TR 01	-	-	-	11	70	19	SM
2	TR 02	36	20	16	20	36.	44	SC
3	TR 03	33	18	15	31	40	29	SC

4	TR 04	16	9	7	16	64	20	SC
5	TR 05	47	32	15	17	48	35	SC
6	TR 06	52	35	17	40	33	27	SC
7	TR 07	15	-	-	27	49	24	SM
8	TR 08	-	-	-	6	66	28	SM
9	TR 09	30	21	9	24	44	32	SC
10	TR 10	36	18	18	24	46	30	SC
11	TR 11	24	14	10	15	57	28	SC
12	TR 12	23	17	6	37	41	22	SC
13	TR 13	18	10	8	23	57	20	SC
14	TR 14	21	12	9	5	56	39	SC
15	TR 15	15	-	-	12	58	30	SM
16	TR 16	25	15	10	15	55	30	SC
17	TR 17	18	-	-	20	53	27	SM
18	TR 18	-	-	-	10	78	12	SM
19	TR 19	-	-	-	3	87	10	SM
KOLLAM CORPORATION								
20	KL 20	36	23	13	45	32	23	SC
21	KL 21	-	-	-	8	81	11	SM
22	KL 22	-	-	-	4	84	12	SM
23	KL 23	18	-	-	14	64	22	SM
24	KL 24	-	-	-	13	69	18	SM
25	KL 25	-	-	-	7	76	17	SM
26	KL 26	-	-	-	19	52	29	SM

27	KL 27	-	-	-	2	85	13	SM
28	KL 28	-	-	-	21	56	23	SM
29	KL 29	-	-	-	15	67	18	SM
KOCHI CORPORATION								
30	KO 30	24	14	10	18	45	37	SC
31	KO 31	23	16	7	21	49	30	SC
32	KO 32	-	-	-	6	84	10	SM
33	KO 33	-	-	-	23	65	12	SM
34	KO 34	21	12	9	29	38	33	SC
35	KO 35	-	-	-	23	63	14	SM
36	KO 36	-	-	-	23	66	11	SM
37	KO 37	-	-	-	28	56	16	SM
38	KO 38	-	-	-	11	64	25	SM
39	KO 39	34	22	12	20	50	30	SC
40	KO 40	32	19	13	30	48	22	SC
41	KO 41	-	-	-	20	71	9	SM
42	KO 42	-	-	-	5	77	18	SM
43	KO 43	26	19	7	32	54	14	SC
44	KO 44	-	-	-	13	64	23	SM
45	KO 45	-	-	-	23	70	7	SM
46	KO 46	-	-	-	23	65	12	SM
47	KO 47	-	-	-	15	72	13	SM
48	KO 48	-	-	-	6	77	17	SM

THRISSUR CORPORATION								
49	TH 49	-	-	-	10	61.19	29	SM
50	TH 50	19	12	7	16	56	28	SC
51	TH 51	21	16	5	5	56	39	SC
52	TH 52	17	11	6	20	57	23	SC
53	TH 53	-	-	-	18	57	25	SM
54	TH 54	-	-	-	10	62	28	SM
55	TH 55	-	-	-	12	61	27	SM
56	TH 56	17	13	4	14	58	28	SC
57	TH 57	32	21	11	28	47	25	SC
58	TH 58	-	-	-	11	70	19	SM
KOZHIKODE CORPORATION								
59	KZ 59	18	-	-	36	47	17	SM
60	KZ 60	29	-	-	6	65	29	SM
61	KZ 61	-	-	-	19	69	12	SM
62	KZ 62	-	-	-	23	56	21	SM
63	KZ 63	-	-	-	9	78	13	SM
64	KZ 64	-	-	-	13	69	18	SM
65	KZ 65	-	-	-	14	77	9	SM
66	KZ 66	-	-	-	8	83	9	SM
67	KZ 67	-	-	-	16	70	14	SM
68	KZ 68	-	-	-	40	45	15	SM

4.3.2 Compaction characteristics and CBR values for urban roads

The compaction characteristics and CBR values are tabulated in **Table 4.4.**

Table 4.4 Compaction and CBR for urban roads

Sl. No	Section ID	Field Dry Density (g/cc)	Field Moisture Content (%)	Max Dry Density (g/cc)	Optimum Moisture Content (%)	CBR (%)	Relative compaction (%)
THIRUVANANTHAPURAM CORPORATION							
1	TR 01	1.24	13.32	2.10	8.5	24	59
2	TR 02	1.47	11.2	1.85	16.6	19	79
3	TR 03	1.61	15.7	1.97	11.3	39	82
4	TR 04	1.66	5.6	2.28	6.7	40	73
5	TR 05	1.14	21.2	1.67	21.4	32	68
6	TR 06	1.34	22.5	1.74	19.0	26	77
7	TR 07	2.50	11.3	2.28	7.65	24	110
8	TR 08	1.64	7.0	2.20	6.7	26	74
9	TR 09	1.36	18.1	1.83	16.0	30	74
10	TR 10	1.48	14.5	1.97	11.2	10	75
11	TR 11	1.83	6.5	2.17	8.2	33	84
12	TR 12	1.88	10.7	2.19	6.6	54	86
13	TR 13	1.54	7.7	2.18	7.0	24	71
14	TR 14	1.52	10.6	2.15	9.4	25	71
15	TR 15	2.34	4.0	2.30	6.0	64	102
16	TR 16	1.76	10.0	2.15	8.1	26	82
17	TR 17	1.46	11.5	2.1	9.5	9	70
18	TR 18	1.45	8.0	2.07	5.0	43	70
19	TR 19	1.32	2.4	1.99	8.3	41	66
KOLLAM CORPORATION							
20	KL 20	1.66	15.7	2.05	13.5	11	81

21	KL 21	1.49	8.9	2.02	8.2	34	74
22	KL 22	1.52	6.1	2.06	8.2	34	74
23	KL 23	1.99	9.0	2.29	7.8	37	87
24	KL 24	2.01	6.8	2.23	8. 0	36	90
25	KL 25	1.77	5.3	1.95	7.7	39	91
26	KL 26	1.81	7.0	2.19	7.0	48	83
27	KL 27	1.49	8.1	2.16	7.5	45	69
28	KL 28	1.58	11.2	2.21	8.3	64	71
29	KL 29	1.60	4.5	2.13	7.4	53	75
KOCHI CORPORATION							
30	KO 30	1.28	24.0	1.91	13.1	31	67
31	KO 31	1.19	21.0	1.93	12.5	31	62
32	KO 32	1.72	7.9	1.99	7.6	22	87
33	KO 33	1.60	4.0	2.12	8.4	27	75
34	KO 34	1.35	12.9	1.99	12.2	05	68
35	KO 35	1.57	9.3	2.01	8.4	31	78
36	KO 36	1.70	10.8	2.13	7.5	59	80
37	KO 37	1.66	12.3	2.11	9.8	56	79
38	KO 38	1.43	6.3	2.06	9.2	41	69
39	KO 39	1.20	19.5	1.85	14.0	64	65
40	KO 40	1.45	18.0	1.92	12.9	78	76
41	KO 41	1.43	13.1	2.00	9.3	30	70
42	KO 42	1.40	14.1	1.92	11.9	27	73
43	KO 43	1.35	16.7	1.93	11.9	54	70
44	KO 44	1.12	5.1	2.08	8.4	48	54
45	KO 45	1.48	6.7	2.01	7.7	15	74

46	KO 46	1.60	6.4	2.13	7.6	56	75
47	KO 47	1.37	9.3	2.13	6.6	30	64
48	KO 48	1.32	8.2	2.11	7.5	56	63
THRISSUR CORPORATION							
49	TH 49	1.58	10.7	2.19	7.4	15	72
50	TH 50	1.56	12.1	2.16	7.9	13	72
51	TH 51	1.51	16.3	2.00	10.0	31	76
52	TH 52	1.79	8.9	2.25	7.0	58	80
53	TH 53	1.80	11.8	2.12	9.1	10	85
54	TH 54	1.76	9.5	2.13	8.3	33	83
55	TH 55	1.88	10.8	2.21	8.2	16	85
56	TH 56	1.85	11.0	2.17	8.5	41	85
57	TH 57	1.35	13.1	2.12	10.0	74	64
58	TH 58	1.81	9.3	2.13	7.5	37	85
KOZHIKODE CORPORATION							
59	KZ 59	1.66	13.0	2.28	8.4	53	73
60	KZ 60	1.24	18.3	1.96	10.1	09	63
61	KZ 61	1.79	6.3	2.08	8.5	26	86
62	KZ 62	1.59	7.4	2.23	7.4	59	71
63	KZ 63	1.87	7.8	2.05	7.5	36	91
64	KZ 64	1.82	9.0	2.10	7.5	54	87
65	KZ 65	1.51	6.8	2.13	7.3	33	71
66	KZ 66	1.55	7.2	1.99	9.6	51	78
67	KZ 67	1.43	10.6	2.16	8.0	52	66
68	KZ 68	1.72	10.2	2.25	7.6	52	76

4.3.3 Structural Number (SN) and Modified Structural Number (MSN)

The structural number SN and Modified Structural Number MSN were calculated using the equations given in Chapter 3 (Section 3.3.3). The pavement composition and Structural Number SN for the study stretches are given in **Table 4.5.**

Table 4.5 Pavement Composition and Structural Number for Urban Roads

Sl No.	ID	Layer details			Structural Number: SN= ∑aidi
		Surface (mm)	Base (mm)	Sub base (mm)	
THIRUVANANTHAPURAM CORPORATION					
1	TR 01	PMC 20 mm	WBM – 150 mm	Soling 450mm	2.78
2	TR 02	PMC 20 mm BM 90 mm	WBM 40 mm	Soling 450 mm	4.79
3	TR 03	SDBC 25 mm BM 70 mm	WBM 150 m	Soling 450 mm	1.43
4	TR 04	SDBC 25 mm BM 80 mm	WBM 280 mm	-	2.29
5	TR 05	SDBC 25 mm BM 80 mm	WBM 200 mm		1.91
6	TR 06	BC 40 mm BM 80 mm	WBM 180 mm		2.03
7	TR 07	BC 40 mm BM 90 mm	WBM 280 mm		2.65
8	TR 08	PMC 20 mm BM 100 mm	WBM 200m		1.81
9	TR 09	PMC 20 mm	PMC (old) 20 mm WBM 70 mm	Soling 140 mm	1.13

10	TR 10	PMC 20 mm BM 50 mm	WBM 250 mm		1.73
11	TR 11	SDBC 25 mm BM 50 mm	WBM 150 mm		1.43
12	TR 12	SDBC 25 mm BM 90 mm	WBM 410 mm		3.14
13	TR 13	SDBC 25 mm BM 50 mm	WBM 400 mm		2.81
14	TR 14	PMC 20 mm BM 110 mm	WBM 180 mm		1.77
15	TR 15	PMC 20 mm BM 110 mm	WBM 180mm		1.77
16	TR 16	PMC 20 mm BM 110 mm	WBM 180 mm		1.77
17	TR 17	BC 40 mm BM 50 mm	WBM 170 mm		1.76
18	TR 18	SDBC 25 mm BM 110 mm	WBM 180 mm		2.02
19	TR 19	SDBC 25 mm BM 90 mm	WBM 150 mm		1.71
KOLLAM CORPORATION					
20	KL 20	PC 20 mm BM 90 mm	WBM 100 mm	Soling 150 mm	1.98
21	KL 21	PC 20 mm BM 70 mm	WBM 220mm		1.85
22	KL 22	PC 20 mm BM 70 mm	WBM 110 mm		1.24

23	KL 23	PC 20 mm BM 60 mm	WBM 130 mm		1.28
24	KL 24	PC 20 mm BM 60 mm	WBM 130 mm		1.28
25	KL 25	PC 20 mm BM 100 mm	WBM 80 mm		1.29
26	KL 26	PC 20 mm BM 100 mm	WBM 70 mm		1.24
27	KL 27	SDBC 70 mm SDBC 60 mm	WBM 200 mm		3.63
28	KL 28	SDBC 70 mm BM 100 mm	WBM 180 mm		2.39
29	KL 29	SDBC 120 mm BM 200 mm	WBM 250 mm		3.98
KOCHI COPORATION					
30	KO 30	PC 20 mm BM 70 mm	WBM 150 mm		1.47
31	KO 31	PC 20 mm BM 40 mm	WBM 180 mm		1.42
32	KO 32	PC 20 mm BM 70 mm	WBM 50 mm		0. 91
33	KO 33	PC 20 mm BM 40 mm	WBM 20 mm		1.53
34	KO 34	SDBC 25 mm BM 40 mm BM 160 mm	WBM 150 mm		2.49

35	KO 35	SDBC 25 mm BM 40 mm BM 160 mm	WBM 100 mm		2.22
36	KO 36	PC 20 mm BM 50 mm BM 50 mm	WBM 80 mm		1.29
37	KO 37	PC 20 mm BM 50 mm BM 50 mm	WBM 230 mm		2.12
38	KO 38	PC 20 mm BM 50 mm BM 50 mm	WBM 80 mm		1.29
39	KO 39	PC 20 mm BM 80 mm BM 80 mm	WBM 180 mm		2.27
40	KO 40	PC 20 mm BM 80 mm BM 80 mm	WBM 400 mm		3.48
41	KO 41	PC 20 mm BM 60 mm	WBM 200 mm		1.67
42	KO 42	PC 20 mm BM 50 mm BM 120 mm	WBM 110 mm		1.95
43	KO 43	PC 20 mm BM 150 mm	WBM 150 mm		1.32

44	KO 44	BC 40 mm BM 60 mm BM 80 mm	WBM 100 mm		2.01
45	KO 45	PC 20 mm BM 150 mm	WBM 80 mm		0.94
46	KO 46	PC 20 mm BM 100 mm	WBM 180 mm		1.84
47	KO 47	PC 20 mm BM 60 mm	WBM 130 mm		1.28
48	KO 48	PC 20 mm BM 50 mm BM 50 mm	WBM 200 mm		1.51
THRISSUR CORPORATION					
49	TH 49	PC 20 mm BM 110 mm	WBM 200 mm		1.32
50	TH 50	PC 20 mm BM 60 mm	WBM 300 mm		2.22
51	TH 51	PC 20 mm BM 60 mm	WBM 250 mm	Soling 180 mm	3.01
52	TH 52	PC 20 mm BM 50 mm	WBM 150 mm		1.32
53	TH 53	PC 20 mm BM 50 mm	WBM 150 mm		1.32
54	TH 54	PC 20 mm BM 40 mm	WBM 180 mm		1.42

55	TH 55	PC 20 mm BM 40 mm	WBM 180 mm		1.42
56	TH 56	PC 20 mm BM 50 mm	WBM 180 mm		1.49
57	TH 57	PC 20 mm BM 40 mm	WBM 400 mm	Soling 200 mm	3.50
58	TH 58	PC 20 mm BM 50 mm	WBM 150 mm		1.32
59	TH 59	PC 20 mm BM 40 mm	WBM 60 mm	Soling 150 mm	1.41
KOZHIKODE CORPORATION					
60	KZ 60	PC 20 mm BM 60 mm	WBM 120 mm	Soling 150 mm	1.88
61	KZ 61	PC 20 mm BM 80 mm	WBM 100 mm	Soling 100 mm	1.69
62	K 62	PC 20 mm BM 50 mm BT 50 mm	WBM 170 mm	-	1.79
63	KZ 63	PC 20 mm BM 50 mm	WBM 80 mm BM20 mm WBM 250 mm	-	2.81
64	KZ 64	PC 20 mm BM 50 mm BM 50 mm	WBM 80	Soling 150 mm	1.94
65	KZ 65	PC 20 mm BM 80 mm	WBM 100 mm	-	1.26

66	KZ 66	PC 20 mm BM 40 mm	WBM 10 mm	Soling 150 mm	1.63
67	KZ 67	PC 20 mm BM 50 mm BM 50 mm	WBM 80 mm	Soling 150 mm	2.16
68	KZ 68	PC 20 mm, BM 80 mm	WBM 70 mm	-	1.10

4.3.4 Characteristic Deflection: The characteristic deflection values were calculated from the data collected through Benkelman Beam Deflection studies.

4.3.5 Pavement Condition Index (PCI):

Pavement Condition Index (PCI) is a numerical indicator of present pavement condition that is directly related to the pavement surface operational condition. The PCI value ranges from 0 to 100 with a score of 100 representing a pavement in perfect condition. Deduct value method was used to calculate the pavement strength. The classes of PCI and the maintenance intervention to be adopted are presented in **Table 4.6.**

Table 4.6 Maintenance Intervention based on PCI

PCI	Rating	Type of Maintenance
80-100	Very Good	Preventive
60-80	Good	Resurfacing
40-60	Fair	Overlay
20-40	Poor	Strengthening
< 20	Very Poor	Rehabilitation

The Modified Structural Number obtained from layer composition and CBR of the subgrade soil, Pavement Condition Index obtained through Deduct Value Method, PCI Rating and type of improvement required for the study roads in urban roads are presented in **Table 4.7.**

Table 4.7 Deflection, PCI, Rating and Type of Improvement for Urban Roads

Sl No.	ID	SN	MSN	DEF mm	PCI	Rating	Improvement
THIRUVANANTHAPURAM CORPORATION							
1	TR 01	2.78	4.57	0.51	54	Fair	Over lay
2	TR 02	4.49	6.46	0.87	100	Very good	Preventive
3	TR 03	1.43	3.43	1.17	54	Fair	Over lay
4	TR 04	2.29	4.30	0.58	67	Good	Resurfacing
5	TR 05	1.92	3.84	0.50	54	Fair	Overlay
6	TR 06	2.03	3.87	1.39	54	Fair	Overlay
7	TR 07	2.65	4.45	0.5	54	Fair	Overlay
8	TR 08	1.81	3.65	0.75	54	Fair	Overlay
9	TR 09	1.13	3.03	0.64	54	Fair	Overlay
10	TR 10	1.73	2.96	0.75	54	Fair	Overlay
11	TR 11	1.43	3.37	0.64	54	Fair	Overlay
12	TR 12	3.14	5.24	0.55	54	Fair	Overlay
13	TR 13	2.81	4.60	0.55	54	Fair	Overlay
14	TR 14	1.77	3.59	0.64	54	Fair	Overlay
15	TR 15	1.77	3.91	0.64	54	Fair	Overlay
16	TR 16	1.77	3.61	0.64	54	Fair	Overlay
17	TR 17	1.76	2.91	0.57	54	Fair	Overlay
18	TR 18	2.06	4.05	0.34	54	Fair	Overlay
19	TR 19	1.71	3.73	0.34	54	Fair	Overlay
KOLLAM CORPORATION							
20	KL 20	1.98	3.28	0.29	54	Fair	Overlay
21	KL 21	1.85	3.80	0.64	54	Fair	Overlay

22	KL 22	1.24	2.55	0.46	54	Fair	Overlay
23	KL 23	1.28	3.27	0.5	54	Fair	Overlay
24	KL 24	1.28	3.26	0.5	67	Good	Resurfacing
25	KL 25	1.29	3.29	0.61	54	Fair	Overlay
26	KL 26	1.24	3.30	0.61	54	Fair	Overlay
27	KL 27	3.63	5.68	0.16	81	Very Good	Preventive
KOCHI COPORATION							
28	KL 28	2.39	4.53	0.16	81	Very Good	Preventive
29	KL 29	3.98	6.07	0.2	81	Very Good	Preventive
30	KO 30	1.47	3.38	1.58	67	Good	Resurfacing
31	KO 31	1.42	3.33	0.86	67	Good	Resurfacing
32	KO 32	0.91	2.66	0.67	54	Fair	Overlay
33	KO 33	1.53	3.38	0.69	54	Fair	Overlay
34	KO 34	2.5	3.10	0.89	54	Fair	Over lay
35	KO 35	2.22	4.13	0.89	54	Fair	Over lay
36	KO 36	1.29	3.41	0.8	54	Fair	Overlay
37	KO 37	2.12	4.23	0.8	54	Fair	Overlay
38	KO 38	1.29	3.31	0.8	54	Fair	Overlay
39	KO 39	2.27	4.40	1.08	54	Fair	Overlay
40	KO 40	3.48	5.65	1.08	54	Fair	Overlay
41	KO 41	1.67	3.57	0.62	30	Poor	Strengthening
42	KO 42	1.95	3.81	0.76	30	Poor	Strengthening
43	KO 43	1.32	3.42	0.54	81	Very good	Preventive
44	KO 44	2.02	4.08	0.17	67	Good	Resurfacing
45	KO 45	0.94	2.46	0.74	67	Good	Resurfacing
46	KO 46	1.84	3.95	0.54	67	Good	Resurfacing

47	KO 47	1.28	3.18	0.18	67	Good	Resurfacing
48	KO 48	1.51	3.62	0.62	54	Fair	Over lay
THRISSUR CORPORATION							
49	TH 49	1.32	2.84	0.64	54	Fair	Over lay
50	TH 50	2.22	3.65	0.62	54	Fair	Over lay
51	TH 51	3.01	4.92	0.7	54	Fair	Overlay
52	TH 52	1.32	3.44	CC	54	Fair	Overlay
53	TH 53	1.32	2.55	CC	54	Fair	Overlay
54	TH 54	1.42	3.36	0.56	54	Fair	Overlay
55	TH 55	1.42	2.98	0.56	54	Fair	Overlay
56	TH 56	1.49	3.51	0.56	54	Fair	Overlay
57	TH 57	3.50	5.66	0.71	67	Good	Resurfacing
58	TH 58	1.32	3.31	0.64	67	Good	Resurfacing
59	TH 59	1.41	3.5	1.67	54	Fair	Overlay
KOZHIKKODE CORPORATION							
60	KZ 60	1.88	3.02	2.54	54	Fair	Overlay
61	KZ 61	1.63	3.53	2.54	81	Very Good	Preventive
62	KZ 62	1.79	3.91	0.68	67	Good	Resurfacing
63	KZ 63	2.81	4.78	0.48	81	Very Good	Preventive
64	KZ 64	1.94	4.04	0.38	81	Very Good	Preventive
65	KZ 65	1.26	3.20	0.47	81	Very Good	Preventive
66	KZ 66	1.63	3.71	0.49	81	Very Good	Preventive
67	KZ 67	2.16	4.25	0.57	81	Very Good	Preventive
68	KZ 68	1.10	3.18	0.38	81	Very Good	Preventive

4.4 PERIODIC PAVEMENT EVALUATION STUDIES

Results obtained from the field investigations and laboratory tests for the eight study roads identified for periodic pavement evaluation are discussed here. One time data on subsoil parameters, traffic and drainage characteristics were collected. The results were used as variables in development of models.

4.4.1 Subgrade Soil Investigation results

The results of the various tests conducted on the sub grade soil collected from the study roads are given in **Table 4. 8**

4.4.2 Traffic Volume Count Survey

The Traffic Census Data collected on all the study roads were analysed. The Passenger Car Unit values for each count station were calculated. PCU values were determined for the slow moving vehicles, fast moving vehicles & for the commercial vehicles are given in **Table 4.9**.

4.4.3 Axle load survey and Vehicle Damage Factor

Axle load survey was conducted for five study roads. Vehicle Damage Factor is an input variable for crack and raveling progression models. The VDF values for each Type of Vehicles in both directions were calculated and total ESAL were obtained for each category. The summary of the VDF values on the study roads are given in **Table 4.10.**

4.4.4 Pavement Construction Quality

The construction quality of the pavement arrived at on a five point rating scale, drainage properties, camber values of the shoulder and relative compaction data are presented in **Table 4.11.**

4.4.5 Pavement Strength and History

The pavement thickness, strength represented as Modified Structural Number, Relative compaction of the subgrade soil and history are given in **Table 4.12.**

Table 4.8 Subgrade Soil Parameters for roads selected for periodic evaluation

Sl. No.	Study Stretch	Soil Tests Results						
		OMC (%)	MDD (g/cc)	LL (%)	PL (%)	PI (%)	Soil Type (IS)	CBR (%)
1	Kesavadasapuram -Plamoodu	9.2	2.1	25	16	9	SC	8
2	Attingal - Kallambalam							
	a. HS-1 (1km)	9.8	2.1	20	13	7	SC	19
	b. HS-2 (1km)	9.7	2.1				SC	12
3	Kazhakkuttam-Kovalam							
	a. HS-1 (1km)	15.6	1.8	19	11	8	SC	30
	b. HS-2 (1km)	13.1	1.9				SC	36
4	Chavadimukku-Pallippuram							
	a. HS-1 (1km)	9.2	2.1	25	16	9	SC	6
	b. HS-2 (1km)	9.8	2.1	NIL	NIL		SM	7
5	Varkala -Kallambalam							
	a. HS-1 (1km)	9.8	2.1	24	19	5	SM-SC	38
	b. HS-2 (1km)	9.5	2.1	-	-	Non plastic	SM	4
	c. HS-3 (1km)	11.5	2.0	22	13	9	SC	12
6	Kottayam Kumili							
	a. HS-1 (1km)	10.2	2.1	-	-	Non plastic	SM	42
	b. HS-2 (1km)	9.4	2.1	21	16	5	SM-SC	35
	C. HS-3 (1km)	8.4	2.1	-	-	Non plastic	SM	40
7	Mannanthala-Venjarammoodu							
	a. HS-1 (1km)	8.2	2.2	26	17	9	SC	35
	b. HS-2 (1km)	17.2	1.8	44	31	13	SC	41
8	Seaport Airport							
	a. HS-1 (1km)	9.8	2.1	-	-	Non plastic	SM	33
	b. HS-2 (500m)	9.4	2.1	-	-	Non plastic	SM	35

Table 4.9: Summary of PCU Values on Study Roads selected for periodic evaluation

Sl. No.	Road Category	Name of Road	Count Station	PCU			PCU of CVs	Total CV/day
				Fast moving vehicle	Slow moving vehicle	Total		
1	NH 47	Kesavadasapuram - Plamoodu	Pattom	42262	106	42368	11549	3994
2	NH 47	Attingal Kallambalam	Kallambalam	26838	95	26933	8851	2992
3	NH 47 Bypass	Kazhakkuttam-Kovalam	Thiruvallam	39105	577	39682	18680	6468
4	NH 47	Chavadimukku - Pallippuram	Chavadimukku	24757	84	24841	9573	3249
5	MDR	Varkala - Kallambalam	Varkala	11003	225	11228	4361	1524
6	NH 220	Kottayam - Kumili - HS I	Pampady	16284	109	16393	6223	2222
		HS II	Ponkunnom	11228	55	11283	5333	1854
		HS III	Kuttikkanam	5644	4	5648	3594	1269
7	SH-1	Mannanthala - Venjaramoodu	Vayyet	13256	59	13314	5554	1923
8	NH Standard	Seaport - Airport	Chithirapuzha	19027	80	19106	10126	3259

Table 4.10 Vehicle Damage Factor for roads selected for periodic evaluation

Sl. No	Road Section/ Vehicle Category	Bus	Lcv	Mcv	2 Axle Truck	3 Axle Truck	Trailer-Tandem
1	Mannanthala-Venjaramoodu	0.3	0.1	0.3	3.0	0.2	NIL
2	Venjaramoodu-Mannanthala	0.3	0.5	0.7	2.8	NIL	NIL
3	Kallambalam-Varkala	0.2	0.1	0.3	2.4	0.1	NIL
4	Varkala-Kallambalam	0.2	0.4	0.6	2.6	NIL	NIL
5	Seaport- Airport	2.1	0.6	0.8	5.7	14.2	1.8
6	Airport - Seaport	0.6	0.2	0.3	2.5	5.7	2.5
7	Chavadimukku-Pallipuram	1.0	0.1	0.5	3.7	3.6	NIL
8	Pallipuram - Chavadimukku	1.0	0.2	0.5	2.5	3.3	NIL
9	Kottayam- Kumili	0.5	0.1	0.2	0.50	1.4	0.6
10	Kumili- Kottayam	0.7	1.8	1.7	7.1	18.4	NIL

Table 4.11: Pavement Construction Quality Data for roads selected for periodic evaluation

Sl No	Stretch ID	Construction Quality CQ	Drainage Rating DR	Shoulder Camber SHCAM (%)	Carriageway Camber CWCAM (%)	Relative Compaction%
1	KP	3	4	2.5	2	78
2	KK-1	4	3	2.5	2	81.9
3	KK-2	4	3	3	2.5	80.5
4	KK-3	4	4	3	2.5	82.9
5	CP-1	5	4	2	1.7	84.5
6	CP-2	5	3	3	2.5	84.1
7	VK-1	4	4	2.5	2	88.3
8	VK-2	4	2	2.5	2	88.5
9	VK-3	1	1	3.3	3	73.2
10	SA-1	6	3	2.5	2	92.6
11	SA-2	4	2	2.5	2	87.9
12	MV-1	4	3	2.5	2	91.5
13	MV-2	3	4	2.5	2	88.4
14	AK-1	4	4	2.5	2	79.2
15	AK-2	4	4	2.5	2	82.5

Table 4.12: Pavement Strength and History for roads selected for periodic evaluation

Sl. No.	Stretch ID	Total pavement thickness mm	Structural Number SN	CBR (%)	Modified Structural Number MSN	Year of last renewal
1.	K-P	320	1.73	10	2.96	2007
2.	KK-1	400	2.66	42.0	4.68	2006
3.	KK-2	290	1.93	35.0	3.89	2006
4.	KK-3	240	1.65	40.0	3.67	2006
5.	CP-1	480	3.22	5.9	3.99	2004
6.	CP-2	480	3.22	6.9	4.14	2004
7.	VK-1	320	2.07	38.4	4.07	2002
8.	VK-2	320	2.07	4.2	2.50	2002
9.	VK-3	300	1.54	11.9	2.91	2002
10.	SA-1	640	4.80	32.5	6.74	2003
11.	SA-2	640	4.80	35.0	6.77	2003
12.	MV-1	390	3.50	34.6	5.46	2002
13.	MV-2	390	3.50	40.5	5.52	2002
14.	AK-1	340	1.76	18.8	2.91	2003, 2008
15.	AK-2	340	1.76	12.4	2.91	2003, 2008

The Vehicle Damage Factor for the two axle trucks were found to be higher than the VDF of other class of vehicles. Also on examination of the overloading pattern, it could be observed that overloading is maximum in case of two axle trucks. These overloaded trucks can be substituted by trailers or three axle trucks or the excess load can be carried in another vehicle.

4.4.6 Periodic evaluation of the pavement

The data on the condition of the pavement, deflection, unevenness and texture depth were collected periodically, with two data sets in an year. Hence in most of the cases, six consecutive data sets were obtained. In the case of Skid resistance studies, three sets of consecutive data were collected from the four study stretches and only one set of data could be collected from two stretches. Defects in Bituminous Surfacing: - Surface defects (Bleeding Streaking etc), Deformations (Rutting, Shoving), Disintegration (Stripping, Raveling, Edge breaks), Cracks (Hairline, Alligator, Longitudinal etc.) were recorded and their progression with time was also plotted. The data were further analyzed and those, which showed experimental errors, were omitted from the analysis. The progression of these parameters are represented as graphs and charts in Chapter 6 and used for model development.

4.4.7 General observations on pavement condition

The studies conducted on the study roads led to the following general observations regarding the performance of the roads.

4.4.7.1 Kesavadasapuram - Plamoodu Road

This is the section of NH 66 within the capital city with heavy traffic. The surface condition of the stretch was bad. Different types of distresses such as cracks, potholes, raveling etc. were observed in the stretch during the first four sets of data collection. Roughness of the stretch in terms of IRI was 4.4, which is high and the speed of the vehicle is affected by this. Characteristic deflection of the stretch was 1.13 mm. A minimum overlay was done before the fifth set of data was taken.

4.4.7.2 Attingal - Kallambalam Road

Attingal–Kallambalam study stretch is the sub urban section of NH 66 and 8 km in length. Two homogenous sections HS I and HS II were considered for detailed study. The surface condition of the stretch was good. Cracks were observed throughout the section uniformly, which is the major distress on surface. The extent of other defects such as raveling, potholes etc. were found to be less. The average IRI value of the section conforms to standards.

4.4.7.3 Kazhakkuttam - Kovalam Road

Kazhakkuttam - Kovalam stretch is 22.4 kms in length and was divided into two homogeneous sections for the study based on the surface condition. Distress such as cracks, raveling, potholes etc. was present on some sections, where as major part of the new surfacing was free from all these distresses. The IRI value of new surface was within the limits and that of old surface was high.

4.4.7.4 Kottayam - Kumili Road:

The road composition of Kottayam-Kumili Road consists of:

HS 1: 50 BM + 25mm BC laid in April 2003.

HS 2: Unimproved section with only a chipping carpet laid in March 2003.

HS 3: Hilly terrain. The widening was done using 75mm Built up Spray Grout (BUSG). 20mm Mixed Seal surfacing was done for the entire section in 2005.

HS 1: Characteristic Deflection Value obtained using Benkelman Beam Deflection Technique for HS-1 was 1.56 mm in the first dataset. Overlay was done before the second data collection and hence lower deflection values were obtained during the successive data collection stages. Structural stability of the road stretch could be rated as good based on the deflection values obtained. After the overlay work, the unevenness value was found to be decreased in second dataset, rating the surface condition as good. Thereafter, increase in unevenness values were found in successive datasets.

HS 2: Structural stability of the road stretch could be rated as moderate based on the deflection values obtained in successive datasets. After the overlay work, the deflection value got decreased and thereafter slight increase in the deflection values was observed. Higher value of Unevenness/Roughness (7875 mm/km) was obtained before the overlay of the pavement and as a result of the overlay there was a steep reduction in unevenness values thereby bringing the pavement in the category of average surface condition.

HS 3: Comparatively higher values of Characteristic Deflection were obtained compared to the other sections. Higher values of unevenness index were obtained for this section in successive datasets. This contrasting difference in surface condition and structural stability could be due to the steep terrain and topography of this section. Unevenness/Roughness value was above the serviceability range as per MORTH Guidelines for Maintenance Management. As per IRC: SP 16-2004, the road stretch was in poor riding condition.

4.4.7.5 Chavadimukku - Pallippuram Road

The last resurfacing of the road stretch was done in 2003.

HS 1: The characteristic deflection value obtained in six sets of data collection indicated a marginal increase in course of time. As per the guidelines the pavement can be rated as reasonably good from structural stability point of view. The unevenness index values obtained for the stretch also indicated a slight increase in course of time. As per IRC: SP 16-2004, this road comes under average serviceability range. Less distress was observed on the stretch.

HS 2: The characteristic deflections values of HS II were comparatively lower than HS 1. As indicated by the characteristic deflection values, this road stretch was reasonably good from structural stability point of view. As per the roughness data, the road stretch can be categorized under average serviceability range. No noticeable surface defects were found. The road stretch has desirable skid resistance values. Low values of surface texture depth indicate the pavement surface to be smooth.

4.4.7.6 Kallambalam - Varkala Road

The last resurfacing of the road stretch was done in 2006 with 60 mm BM and 20 mm MSS.

HS 1: The characteristic deflection values was found to be increasing slightly in course of time and as per the guidelines the pavement can be rated as reasonably strong from structural stability point of view. The unevenness index values of the stretch depicted that it was having average surface condition as per IRC: SP: 16-2004. There was an increasing trend for the unevenness index values despite the decreased value in the third phase due to the patch works done on the surface. No noticeable surface defects were found. The surface texture depth along the stretch indicated that the road surface was smooth. The road stretch showed desirable skid resistance also.

HS 2: As per the guidelines, the pavement could be rated as reasonably strong from structural stability point of view as indicated by the characteristic deflection values. There was an increasing trend for the roughness values and deflection values despite the decreased value in the third phase due to the patch works done on the surface. This road stretch was having average surface condition as per IRC: SP: 16-2004. No noticeable surface defects were found. The surface texture depth along the stretch indicated that the road surface was smooth.

HS 3: Compared to HS 1 and HS 2, this section showed higher values of characteristic deflection and unevenness index. The roughness values indicated that this stretch was having poor surface condition as per IRC: SP: 16-2004. Major surface distresses were observed in this stretch and the deterioration rate was more compared to HS 1 and HS 2. The stretch indicated higher values of texture depth with coarser surface appearance.

4.4.7.7 Mannanthala - Venjarammoodu Road

HS 1: This road stretch could be rated as reasonably strong from structural stability point of view as was indicated by characteristic deflection values. As per the unevenness index values, the road stretch came under average serviceability range. The Skid value for the road stretch was satisfactory. Some distresses were observed on the road stretch. The stretch was showing smooth surface texture.

HS 2: This road stretch could be rated as reasonably strong from structural stability point of view. As per the unevenness index of the stretch, it came under average serviceability range. The Skid value for the road stretch was satisfactory. The stretch showed smooth surface texture.

4.4.7.8 Seaport - Airport Road

The Sea Port Airport road constructed in 2003, running between HMT Kalamassery and Kochi Refineries Ltd is a two lane road of NH standards having a total length 13 km. The last resurfacing of the road stretch was done in 2003.

HS 1: The pavement could be rated as strong from structural stability point of view due to the lower characteristic deflection values. As per the roughness values, this road stretch came under average serviceability range. There was slight increase in roughness values and characteristic deflection values in course of time. No noticeable surface defects were found. The road stretch showed sufficient skid resistance also.

HS 2: Compared to HS I, this section gave higher values of characteristic deflection and unevenness. Compared to HS I, more surface defects were observed on this stretch in the initial phases of data collection, but resurfacing was done on the stretch before the sixth set of data collection. Hence, lower roughness values were obtained in the sixth data. This section showed sufficient skid resistance.

4.5 SUMMARY

This chapter highlights a brief description of the study stretches highlighting the condition of the roads with respect to the various investigations carried out. One time data was collected from all sixty eight urban road sections which include pavement layer composition, pavement condition, traffic data, characteristic deflection and field density. From the laboratory analysis of the subgrade soil collected from the trial pits, soil parameters and CBR values were determined. The Structural Number, Modified Structural Number, Pavement Condition Index, Relative Compaction, Soil type and CBR value of subgrade soil for the urban roads are presented in this chapter.

For eight road sections comprising of NH, NH bye pass, urban stretch of NH and other roads, which were divided into fifteen homogeneous sections, periodic pavement evaluation was done. One time data for Traffic including axle load data and subgrade soil properties collected for these roads are presented. VDF values, Modified Structural Number, Construction Quality represented on a five point scale and Relative Compaction calculated are also presented in this chapter for these eight roads for developing the deterioration models.

The general observations about the condition of the roads are given in this chapter based on the field data collected. On further analysis of the periodic data, charts and graphs are prepared and presented in Chapter 6. The information thus derived are used to develop pavement deterioration models which are also discussed in Chapter 6.

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Chapter 5

PAVEMENT CONDITION ANALYSIS FOR URBAN ROADS

5.1 INTRODUCTION

Mechanistic analysis demands accurate characterisation of materials in pavement structural layers. Pavement materials in the base, sub base and subgrade layers do not behave linearly elastic under repeated wheel loads since unbound granular materials are used. With varying traffic and environmental conditions in a pavement structure, the most significant influence on pavement performance is often characterized in the subgrade soil. This influence is mainly pronounced at low subgrade support values, for weak soils. The accurate prediction of pavement performance is significant for efficient management of roads.

Factors that have a significant effect on the soil behaviour are the loading condition, stress state, soil type, compaction and soil physical states. The physical state of the soil is mainly represented by moisture content and dry density for compaction characteristics, Liquid Limit (LL), Plastic Limit (PL), Plasticity Index (PI) and saturation levels. Soil suction is controlled by grain size distribution, internal soil structure and the closeness of the ground water table and has a major influence on subgrade moisture content. The mechanical behaviour of unbound materials is affected by stress history, density, void ratio, water content etc. Mathematical models that correlate all these factors have been attempted in this chapter.

5.1.1 Regression Analysis

Regression models, which are empirical in nature, were developed to describe, predict, and control the dependent variable on the basis of the independent variables. These models were used to establish a relationship between the pavement condition and its causative factors. Pavement age, intensities of different modes of distress, traffic load etc. form the independent variables.

The equations developed, based on regression analysis, were used for predicting the characteristic deflection. Regression models can be either linear or nonlinear. Linear regression has got wide acceptance because of its simplicity and the physical sense it gives. When more than one variable is included in the deterioration model, the equations are developed by multiple linear regression technique. In nonlinear regression, the observed data are modelled by a function which is a nonlinear combination of the model parameters and which depends on one or more independent variables. The reliability of regression model is measured by its goodness of fit, which is represented in terms of co-efficient of correlation (R^2 value).

The parameters of regression models estimated and calibrated are statistically significant as indicated by various statistical parameters like R^2, t-test, F-test. However ability of these models to replicate the observed condition (to match observed and assigned values) with accuracy before being used to predict values need to be checked by validation of models. Regression models developed were validated through internal validation process.

5.2 RELATIONSHIP BETWEEN MODIFIED STRUCTURAL NUMBER AND CHARACTERISTIC DEFLECTION

Mathematical relationship of the structural strength of the pavement combined with the subgrade soil properties to the rebound deflection in terms of both linear and exponential functions were attempted. The predominant soil types observed in the study stretches were Silty Sand (SM) and Clayey Sand (SC). The effect of subgrade soil type and pavement condition on the relationship between modified structural number and characteristic deflection were studied using regression analysis.

5.2.1 Influence of Subgrade Soil Type on MSN - Deflection Relationship

Regression analysis was carried out to determine the correlation between the deflection and MSN for subgrade soil types Silty Sand (SM) and Clayey Sand (SC). It was found that both linear and power function relationships gave good fit for the relationships as given below:

5.2.1.1 Linear relationship between Modified Structural Number (MSN) and Deflection

Analysis of data using regression analysis showed that linear regression model provided best fit for both soil types with less standard error. The linear regression model takes the form,

$$y = ax + b$$

where,

a & b = coefficients determined by method of least squares

y = Characteristic Deflection measured in mm (DEF)

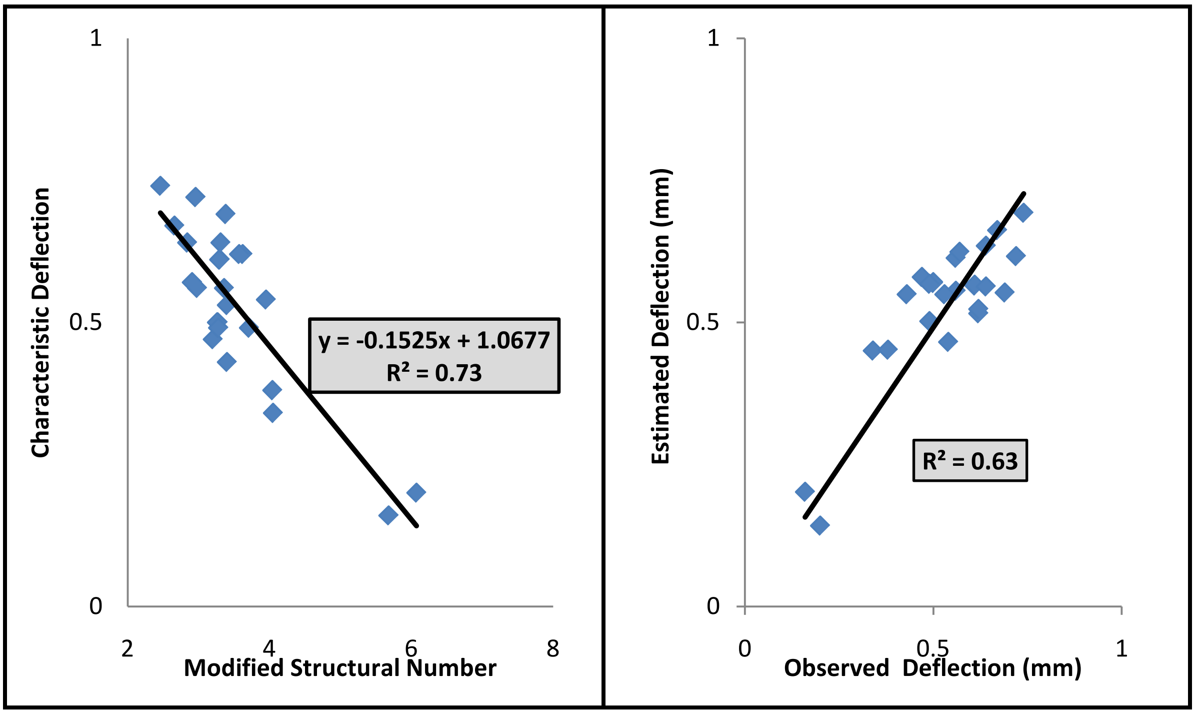
x = Modified Structural Number (MSN)

The linear relationship between Modified Structural Number and Characteristic Deflection for subgrade soil types - Silty Sand and Clayey Sand are tabulated in **Table 5.1**. Arithmetic mean, standard deviation and standard error for characteristic deflection are also calculated.

Table 5.1: Linear Relationship between Modified Structural Number and Deflection

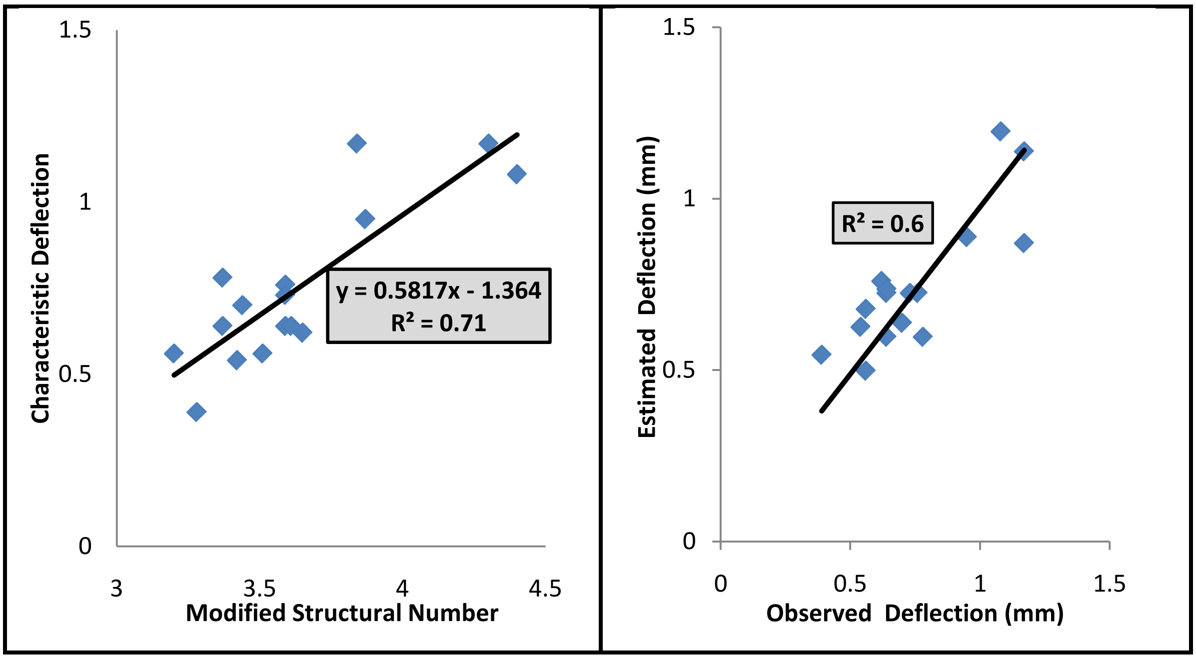
Soil Type	Relationship between Modified Structural Number and Characteristic Deflection	R^2	Characteristic Deflection (mm)		
			Mean	Standard Deviation	Standard Error
SM	DEF = -0.1525 MSN + 1.0677	0.73	0.531	0.145	0.029
SC	DEF = 0.5817 MSN - 1.364	0.71	0.746	0.232	0.058

Plot between modified structural number (MSN) and Characteristic Deflection (DEF) showing linear relationship and the plot between the estimated and observed deflection in a linear scale are shown in **Fig. 5.1 and 5.2.**



Observed vs Predicted in linear scale

Fig. 5.1: Linear Plot between MSN and Deflection for Silty Sand



Observed vs Predicted in linear scale

Fig. 5.2: Linear Plot between MSN and Deflection for Clayey Sand

5.2.1.2 Non linear relationship between MSN and Deflection

From the regression analysis of data, it was found that power function also provided better fit for both subgrade soil types - SM and SC, with less standard error. R^2 value for the power relationship was found to be greater than that of linear relationship between MSN and Deflection.

The power function takes the form,

$$y = ax^b$$

where,

a & b = coefficients determined by least square fitting

y = Characteristic Deflection measured in mm

x = Modified Structural Number (MSN)

Power function relationship between Modified Structural Number and Characteristic Deflection are tabulated in **Table 5.2.** Arithmetic mean, standard deviation and standard error for characteristic deflection are also tabulated.

Table 5.2: Power function relationship between Modified Structural Number and Deflection

Soil Type	Relationship between Modified Structural Number and Characteristic Deflection	R^2	Characteristic Deflection (mm)		
			Mean	Standard Deviation	Standard Error
SM	DEF = 3.692 $(MSN)^{-1.609}$	0.78	0.789	0.364	0.105
SC	DEF = 0.0198 $(MSN)^{2.7904}$	0.67	0.777	0.285	0.095

Plot between Modified Structural Number and Characteristic Deflection showing power function relationship and the plot between the estimated and observed deflection in a linear scale are shown in **Fig. 5.3 and 5.4.**

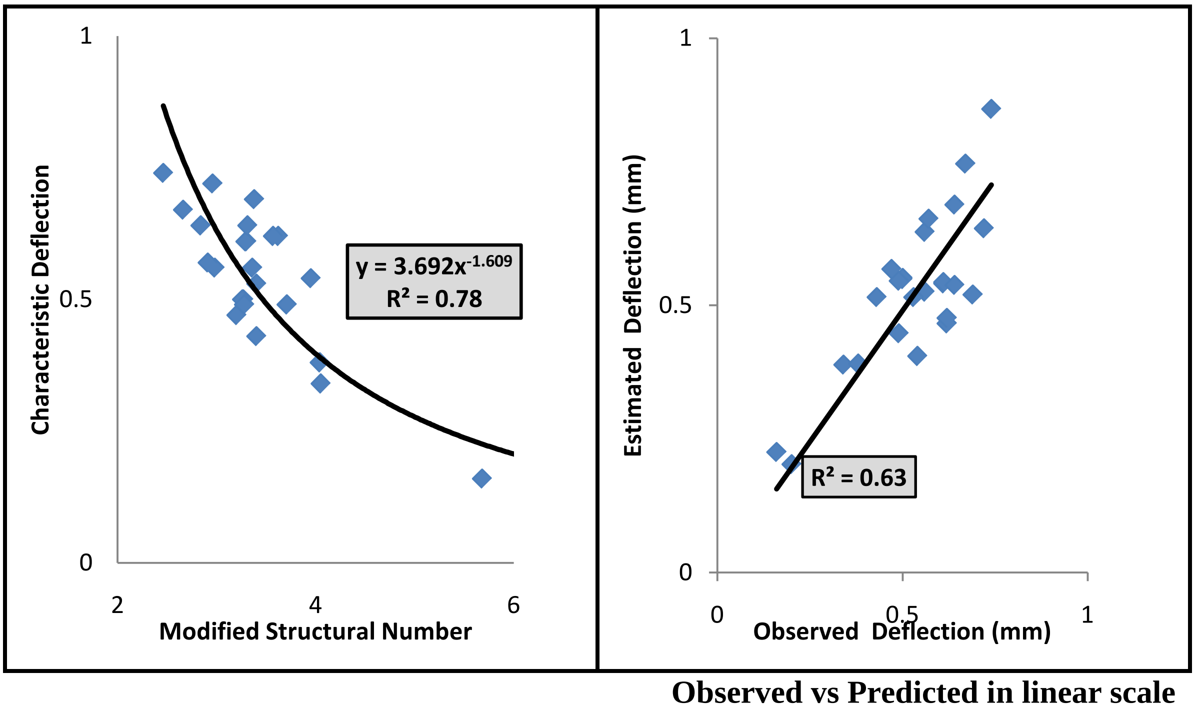


Fig. 5.3: Non- Linear Plot between MSN and Deflection for Silty Sand

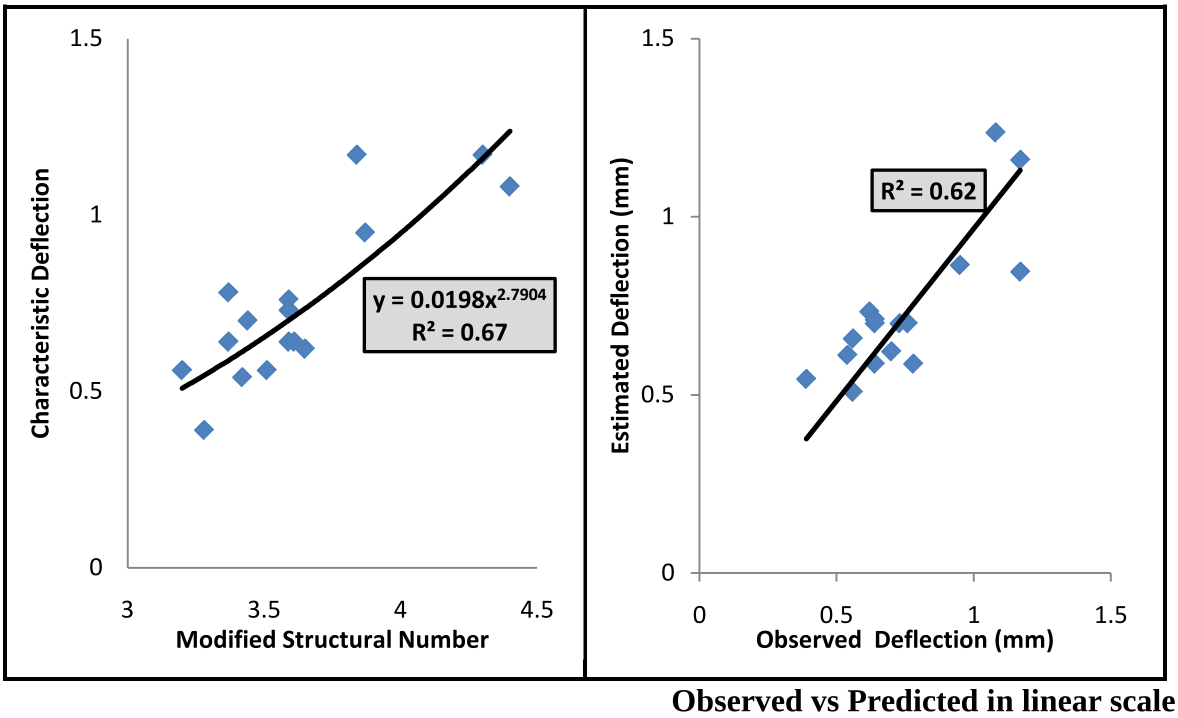


Fig. 5.4: Non- linear Plot between MSN and Deflection for Clayey Sand

The results obtained from the above analysis showed that both linear and power function relationships can be used for estimating the deflection from the strength of the existing pavement. Estimated and observed values of characteristic deflection also showed acceptable correlation. Calibration and validation of the models developed are given below.

5.2.1.3 Calibration and validation of the models

Internal validation method was adopted for validating the developed regression model. A paired t - test for mean was used to evaluate if there exists any significant difference between observed and estimated characteristic deflection values. The calculated t - values in all cases were found to be lower than the critical t - values obtained from statistical tables for 5% level of significance as shown in **Table 5.3**.

Table 5.3 Student's T-test

Soil Type	Type of Relationship	Relationship between Modified Structural Number and Characteristic Deflection	Calculated t test value	Tabular t test value
SM	Linear	DEF= -0.1525 MSN + 1.0677	-0.0003	2.064
	Power	DEF = 3.692 $MSN^{-1.609}$	0.2229	2.064
SC	Linear	DEF = 0.5817 MSN - 1.364	-0.0041	2.132
	Power	DEF = 0.0198 $MSN^{2.7904}$	0.3021	2.132

The parameters of regression models estimated and calibrated are statistically significant as indicated by various statistical parameters like R^2, t-test and F-test. The models developed as part of the regression analysis were used in validating the relationship between the variables, modified structural number and characteristic deflection. The average prediction error (APE) and total prediction error (TPE) for characteristic deflection were calculated and are tabulated in **Table 5.4.**

Table 5.4 Validation of regression models

Soil Type	Type of Relationship	Relationship between Modified Structural Number and Characteristic Deflection	Average Prediction Error (%)	Prediction Error of Totals (%)
SM	Linear	DEF= -0.1525 MSN + 1.0677	2.054	0.001
	Power	DEF = 3.692 MSN $^{-1.609}$	1.471	-0.754
SC	Linear	DEF = 0.5817 MSN - 1.364	2.909	0.017
	Power	DEF = 0.0198 MSN $^{2.7904}$	1.340	-1.305

For Soil type SM, the average prediction error (APE) for characteristic deflection was found to be 2.054 % and 1.471 % for linear and power function models respectively, and the total prediction error (TPE) for characteristic deflection was found to be 0.001 % and -0.754 % for linear and power function models respectively.

The average prediction error (APE) for characteristic deflection calculated for soil type SC was found to be 2.909 % and 1.340 % for linear and power function models respectively, and the total prediction error (TPE) for characteristic deflection was found to be 0.017 % and -1.305 % for linear and power function models respectively.

It may be observed from Table 5.4 that the 'Average Prediction Errors' and 'Prediction errors of Totals' in estimation of variables are much lesser and reasonably accurate. Hence, the regression equations developed are assumed to be acceptable for future predictions.

5.2.2 Influence of Pavement Condition on MSN- Deflection Relationship

Attempts were made to determine the effect of the condition of the pavement on the characteristic deflection. Pavement Condition Index (PCI) values were classified to find the correlation between Modified structural number and characteristic deflection for the different ranges of PCI. The ranges of PCI values adopted are 40-60, 60-80 and 80-100, as per the classification given by Ministry of Road Transport and Highways (MORTH) guidelines.

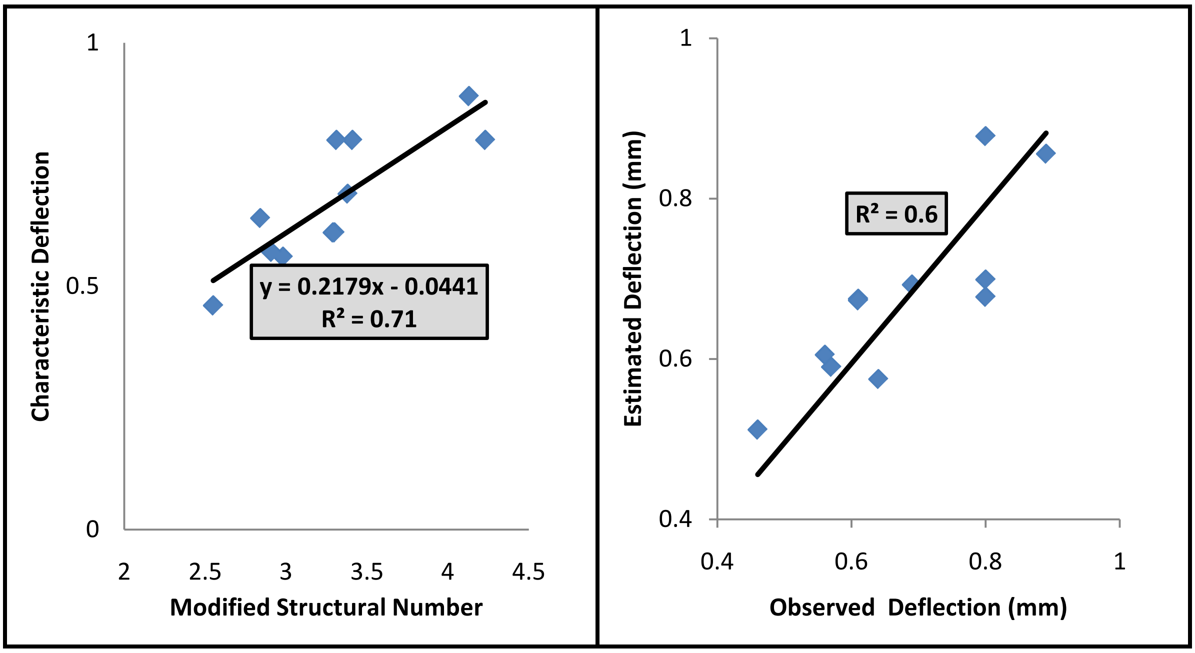
5.2.2.1 Linear Relationship between Modified Structural Number and Deflection for classified values of Pavement Condition Index

Linear regression analysis was carried out to establish relationship between the MSN and deflection variables for each classified values of PCI. Linear relationship between MSN and Characteristic Deflection are tabulated in **Table 5.5.** Arithmetic mean, standard deviation and standard error for characteristic deflection are also given in the table.

Table 5.5 Linear Relationship between MSN and Deflection for classified Pavement Condition Index

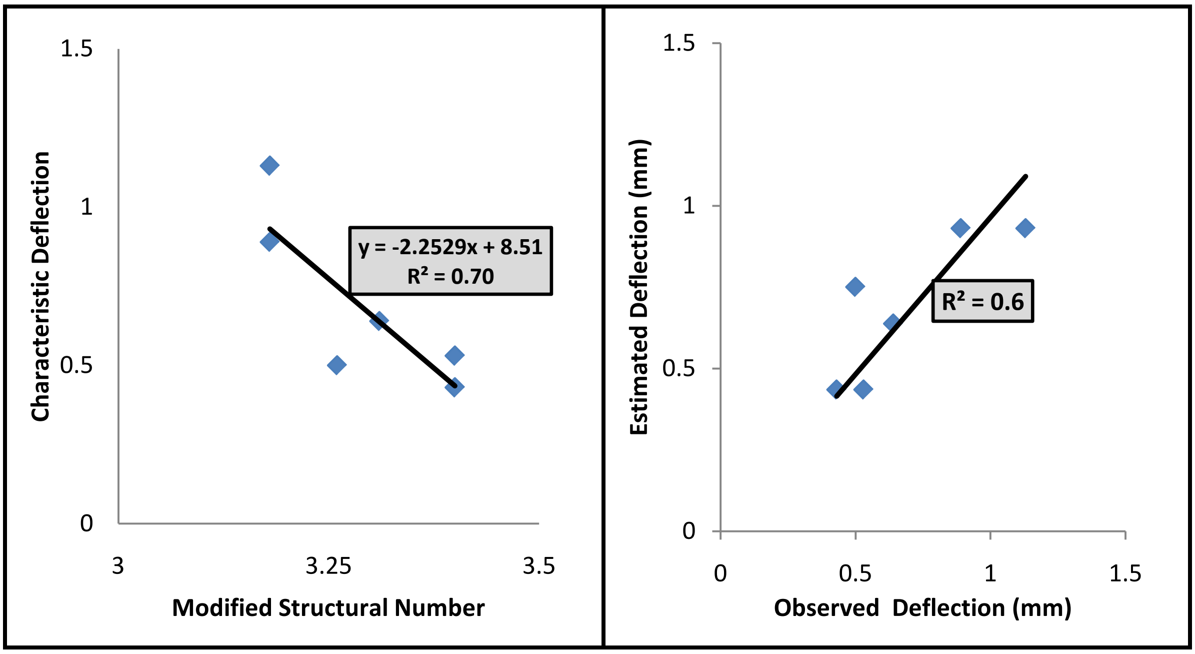
Soil Type	Pavement Condition Index	Relationship between Modified Structural Number and Characteristic Deflection	R^2	Characteristic Deflection (mm)		
				Mean	Standard Deviation	Standard Error
SM	40 - 60	DEF = 0.2179 MSN - 0.0441	0.71	0.675	0.132	0.040
	60 - 80	DEF = -2.2529 MSN + 8.0951	0.70	0.687	0.270	0.110
	80 - 100	DEF = -0.1269 MSN + 0.921	0.74	0.411	0.178	0.063
SC	40 - 60	DEF = 0.5958 MSN - 1.4339	0.75	0.743	0.268	0.081
	60 - 80	DEF = -1.6633 MSN + 7.2018	0.87	1.037	0.506	0.292
	80 - 100	DEF = -0.2444 MSN + 1.6299	0.70	0.798	0.059	0.027

The models developed were used to predict characteristic deflection values and these estimated values were compared with observed values of characteristic deflection. Plots of these models and comparison between observed and estimated values of deflection are shown in **Fig. 5.5, 5.6** and **5. 7** for subgrade soil type SM and in **Fig. 5.8, 5.9** and **5.10** for sub grade soil type SC



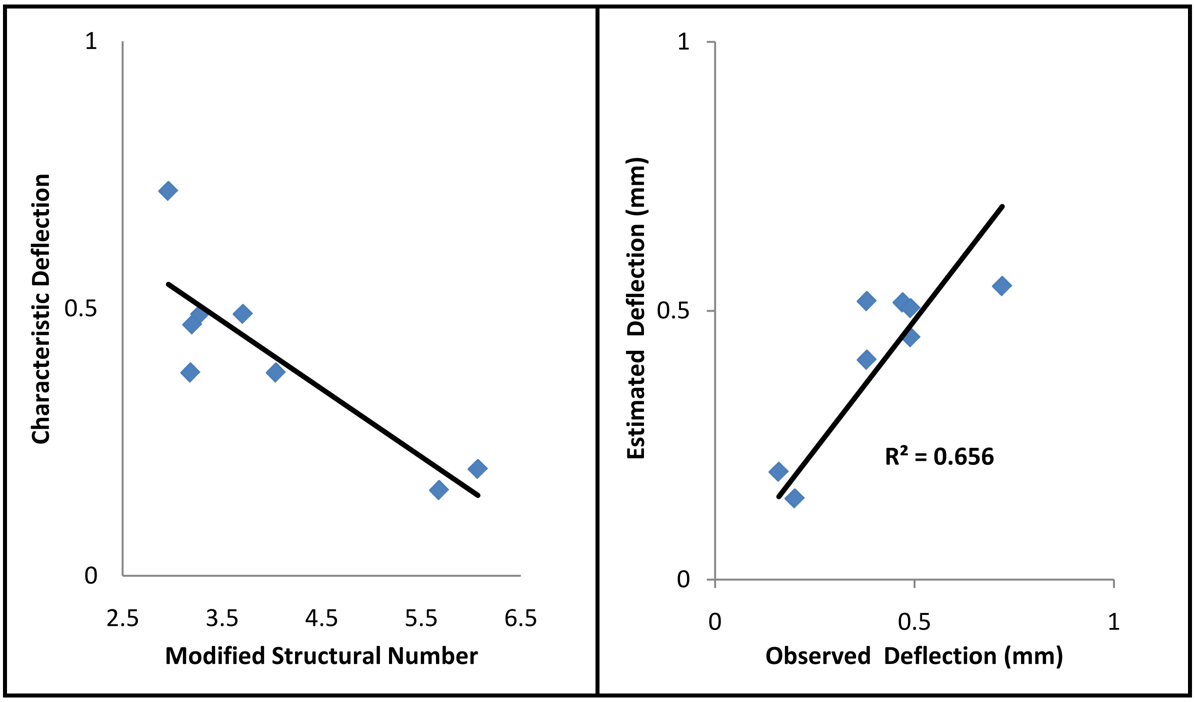
Observed vs Predicted in linear scale

Fig. 5.5: Linear relationship plot between MSN and Deflection for Silty Sand Subgrade soil for PCI range of 40 – 60



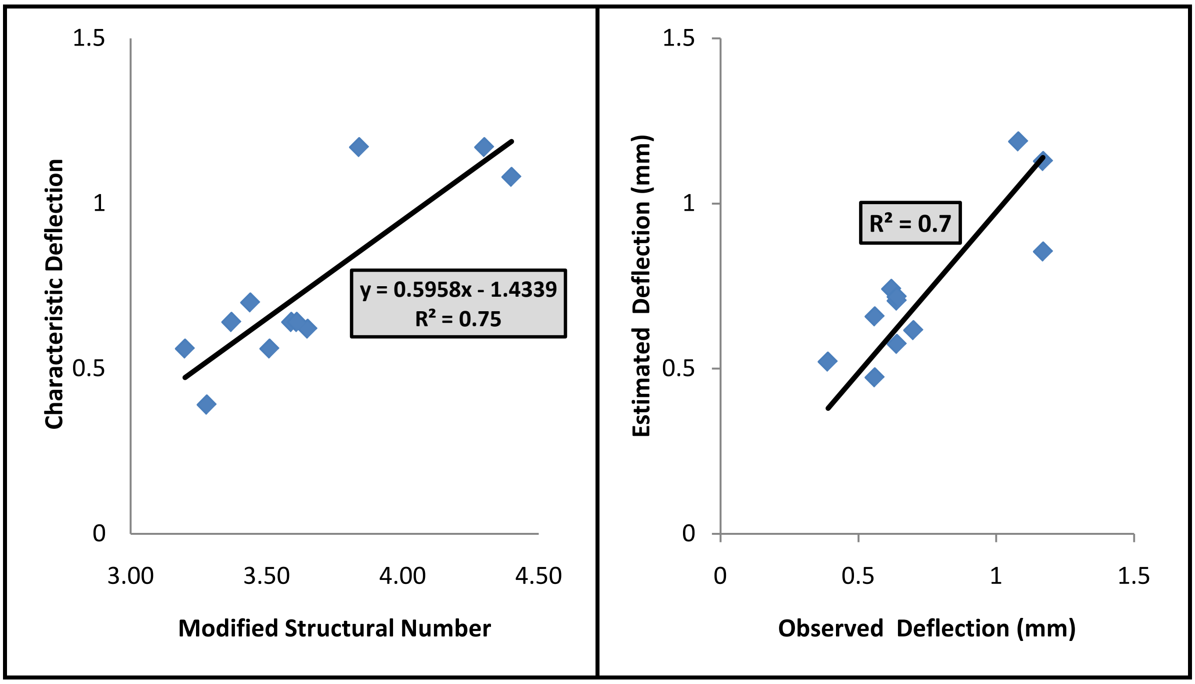
Observed vs Predicted in linear scale

Fig. 5.6: Linear relationship plot between MSN and Deflection for Silty Sand Subgrade soil for PCI range of 60 – 80



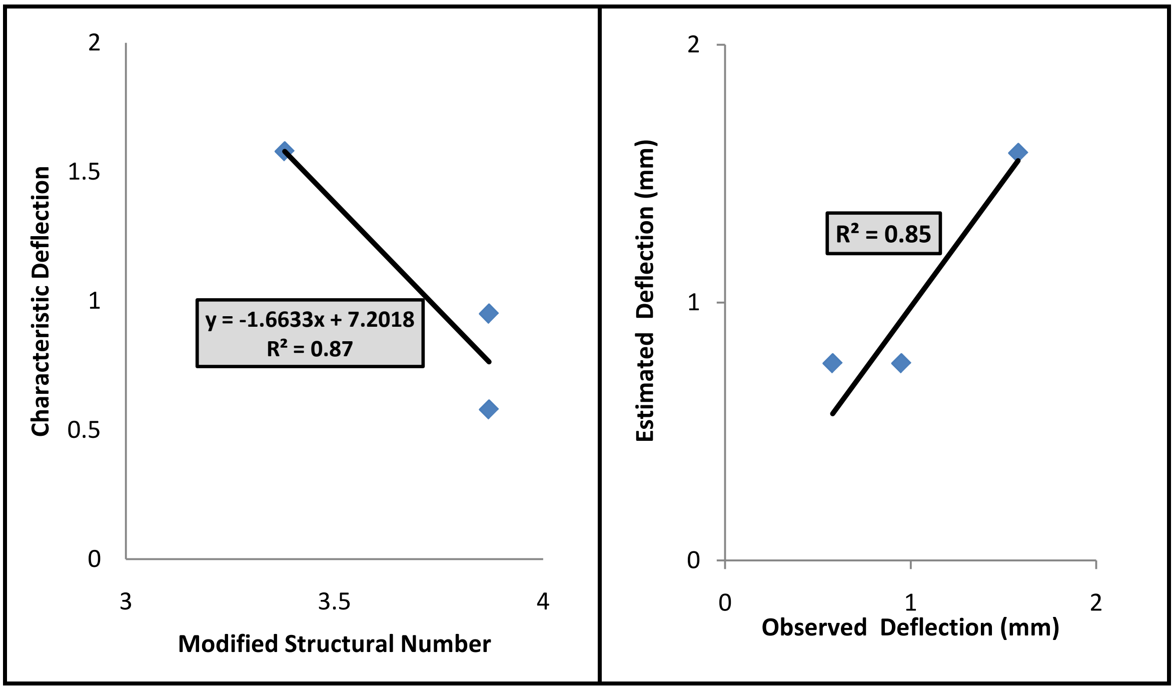
Observed vs Predicted in linear scale

Fig. 5.7: Linear relationship plot between MSN and Deflection for Silty Sand Subgrade soil for PCI range of 80 – 100



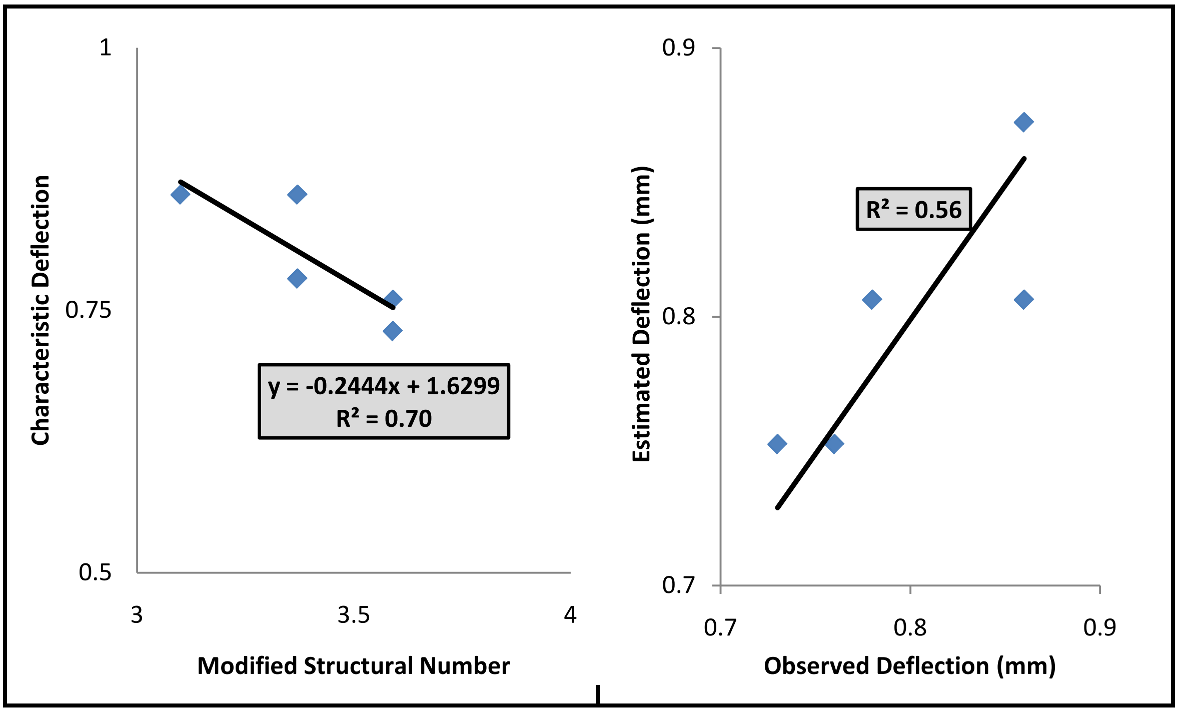
Observed vs Predicted in linear scale

Fig. 5.8: Linear relationship plot between MSN and Deflection for Clayey Sand Subgrade soil for PCI range of 40 – 60



Observed vs Predicted in linear scale

Fig. 5.9: Linear relationship plot between MSN and Deflection for Clayey Sand Subgrade soil for PCI range of 60 – 80



Observed vs Predicted in linear scale

Fig. 5.10: Linear relationship plot between MSN and Deflection for Clayey Sand Subgrade soil for PCI range of 80 – 100

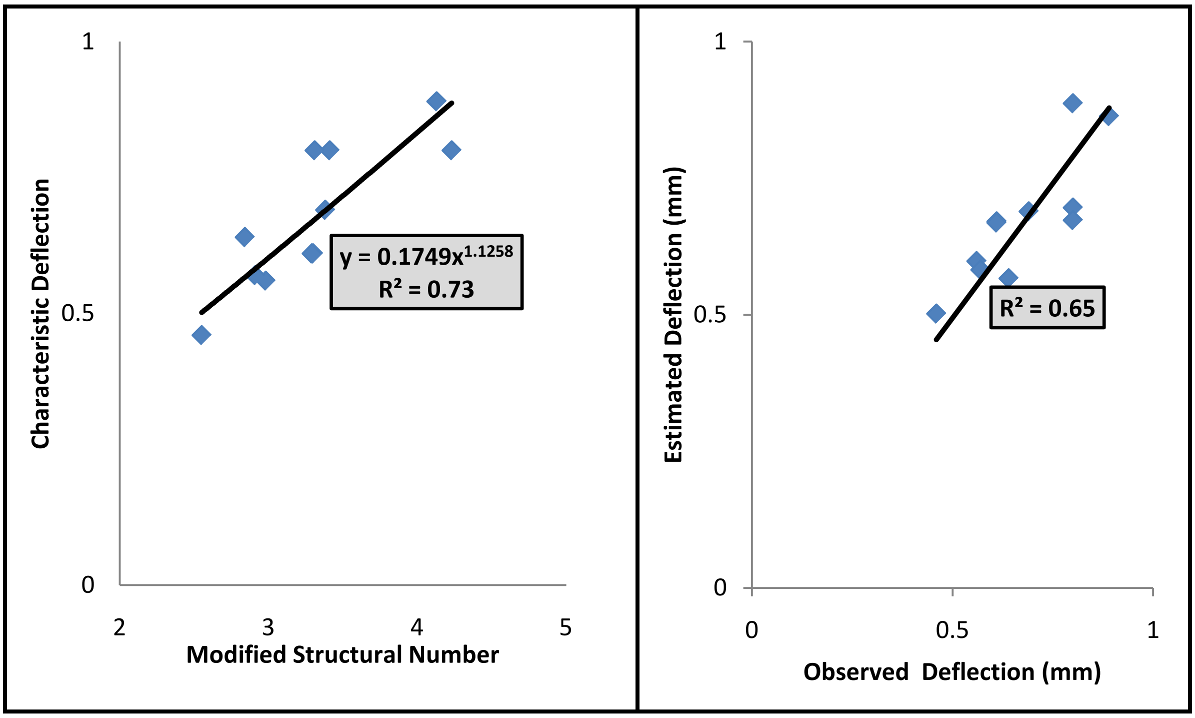
5.2.2.2 Non linear relationship between Modified Structural Number and Deflection for classified values of Pavement Condition Index

Analysis of data using regression analysis indicated that power function also provided better fit for both soil types with less standard error. Modified Structural Number and characteristic deflection values were categorised based on the Pavement Condition Index (PCI) value. Power function regression analysis was then carried out to establish relationship between the independent variable and dependent variable for classified values of PCI. Power function relationship between Modified Structural Number and Characteristic Deflection are tabulated in **Table 5.6.** Arithmetic mean, standard deviation and standard error for characteristic deflection are also given below.

Table 5.6 Power function relationship between Modified Structural Number and Characteristic Deflection

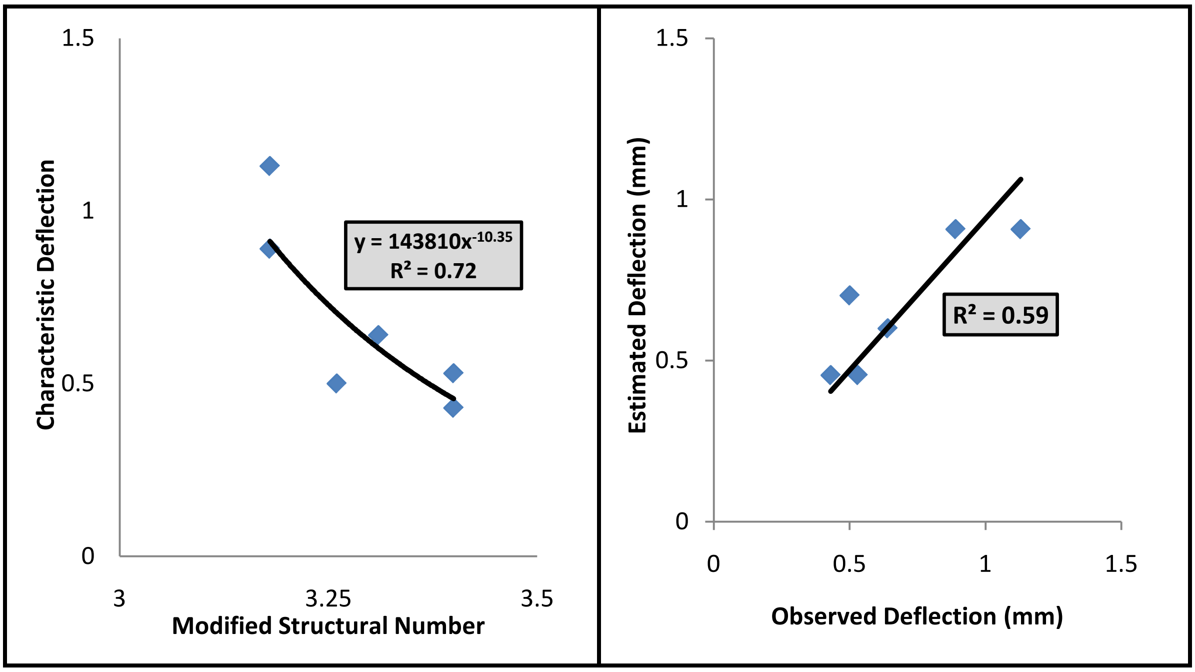
Soil Type	Pavement Condition Index	Relationship between Modified Structural Number and Characteristic Deflection	R^2	Characteristic Deflection (mm)		
				Mean	Standard Deviation	Standard Error
SM	40 - 60	DEF = 0.1749 MSN $^{1.1258}$	0.73	0.675	0.132	0.040
	60 - 80	DEF = 143810 MSN $^{-10.35}$	0.72	0.687	0.270	0.110
	80 - 100	DEF = 3.6267 MSN $^{-1.678}$	0.85	0.411	0.178	0.063
SC	40 - 60	DEF = 0.0171 MSN $^{2.8804}$	0.73	0.743	0.268	0.081
	60 - 80	DEF = 1412.8 MSN $^{-5.58}$	0.76	1.037	0.506	0.292
	80 - 100	DEF = 2.7681 MSN $^{-1.018}$	0.70	0.798	0.059	0.027

The models developed were used to predict characteristic deflection values for each classified value of PCI and the estimated values were compared with observed values of characteristic deflection. Plots of these models and comparison between observed and estimated values of deflection are shown in **Fig. 5.11, 5.12** and **5.13** for subgrade soil type SM and in **Fig. 5.14, 5.15** and **5.16** for subgrade soil type SC.



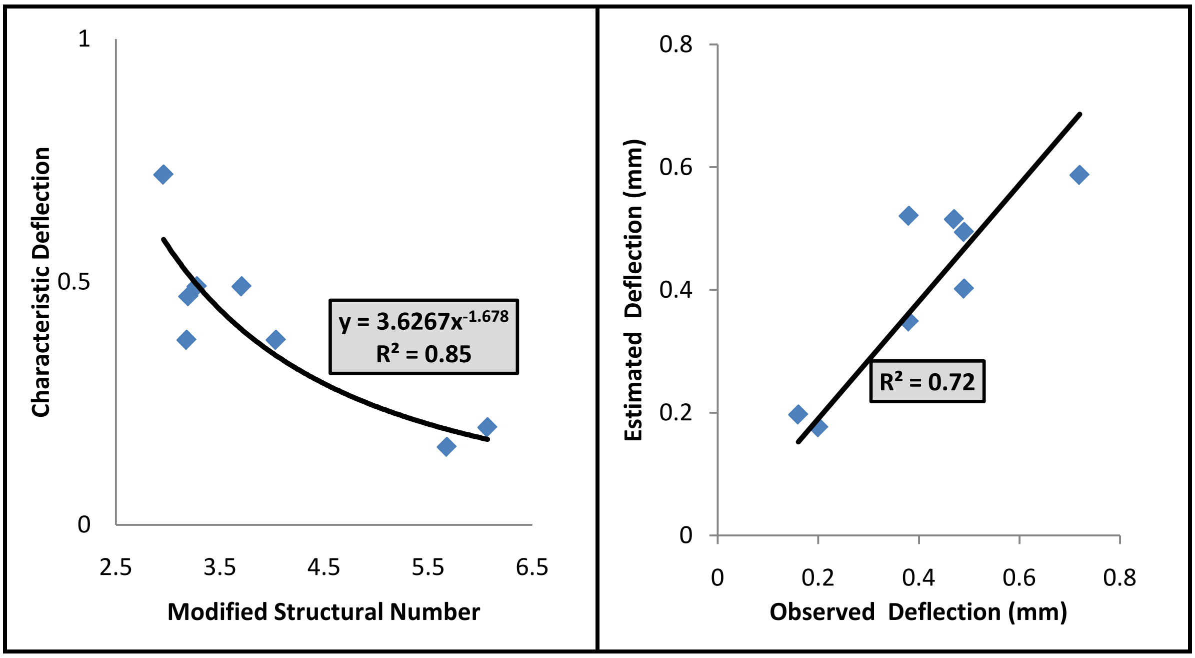
Observed vs Predicted in linear scale

Fig. 5.11: Power function relationship plot between MSN and Deflection for Silty Sand Subgrade soil for PCI range of 40 – 60



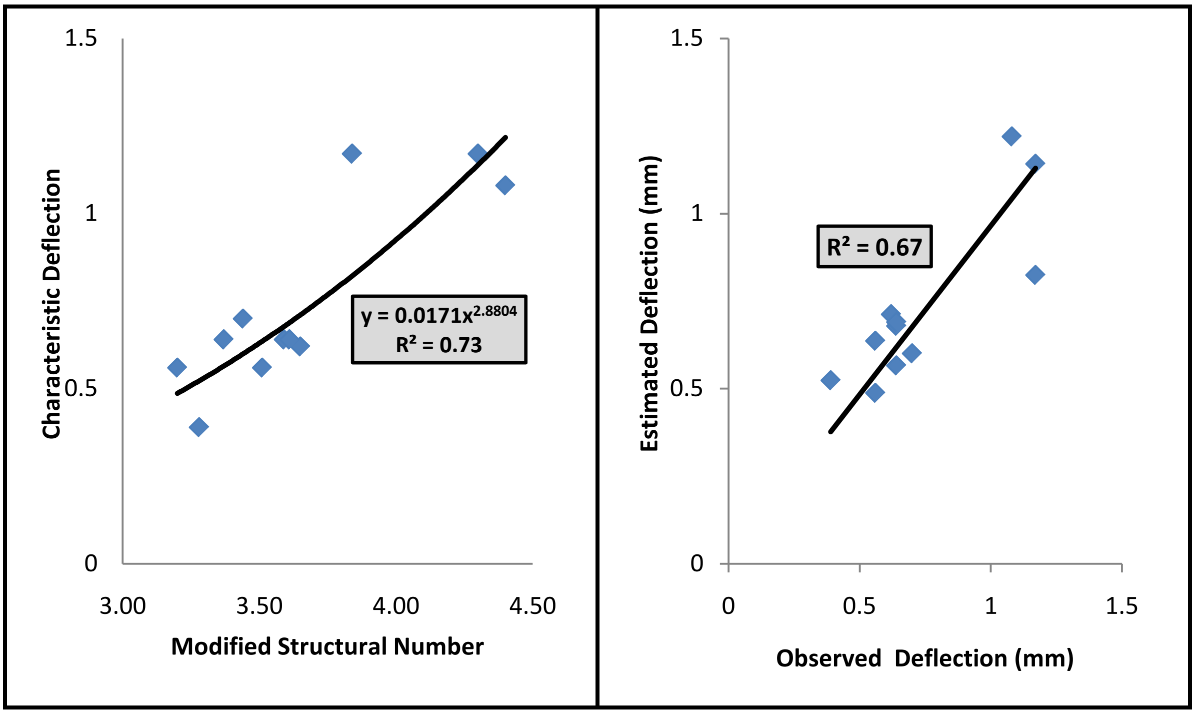
Observed vs Predicted in linear scale

Fig. 5.12: Power function relationship plot between MSN and Deflection for Silty Sand Subgrade soil for PCI range of 60 – 80



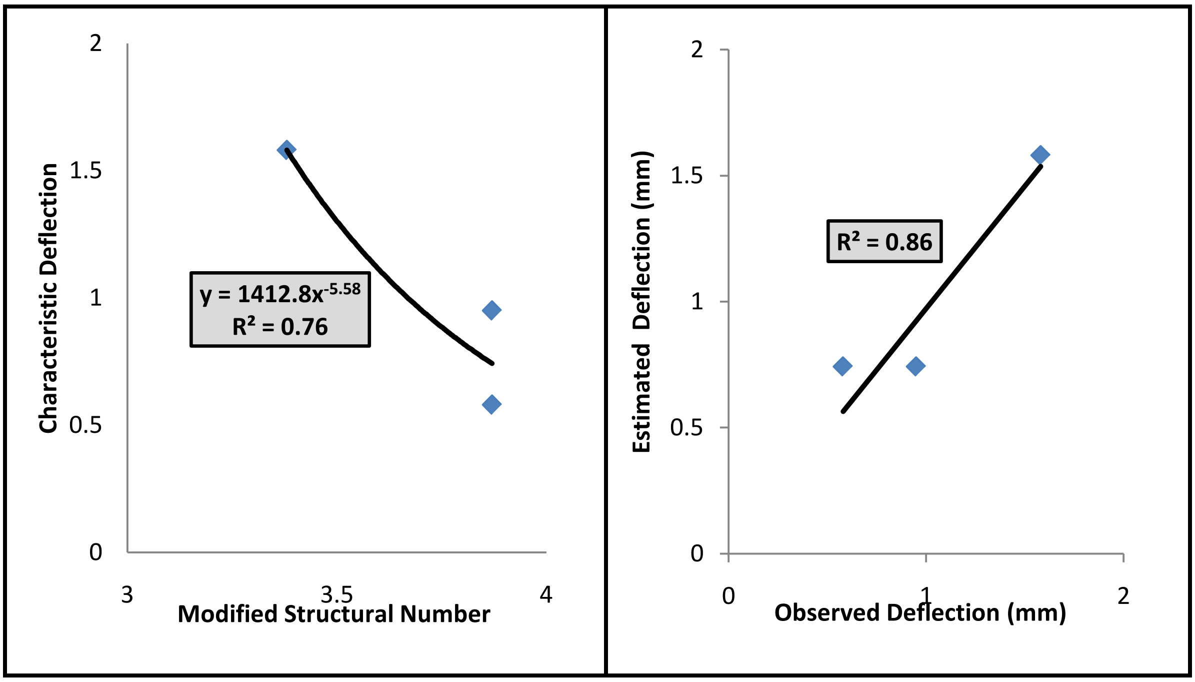
Observed vs Predicted in linear scale

Fig. 5.13: Power function relationship plot between MSN and Deflection for Silty Sand Subgrade soil for PCI range of 80 – 100



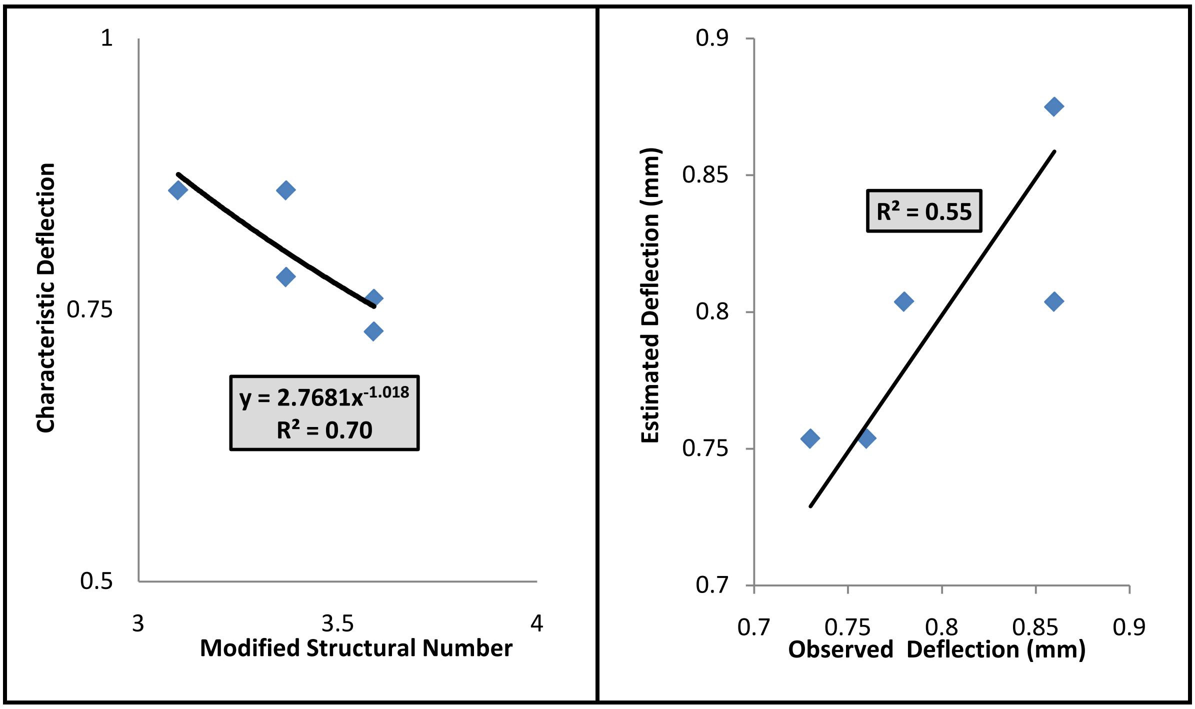
Observed vs Predicted in linear scale

Fig. 5.14: Power function relationship plot between MSN and Deflection for Clayey Sand Subgrade soil for PCI range of 40 – 60



Observed vs Predicted in linear scale

Fig. 5.15: Power function relationship plot between MSN and Deflection for Clayey Sand Subgrade soil for PCI range of 60 – 80



Observed vs Predicted in linear scale

Fig. 5.16: Power function relationship plot between MSN and Deflection for Clayey Sand Subgrade soil for PCI range of 80 – 100

5.2.2.3 Calibration and validation of models

The parameters of regression models estimated and calibrated are found to be statistically significant by various statistical parameters like R^2, t-test, F-test. Internal validation method was adopted for validating the developed regression models. A paired t test for means is used to evaluate if there exists any significant difference between observed and estimated characteristic deflection values. The calculated t-values in all cases were found to be lower than the critical t values obtained from statistical tables for 5% level of significance and are shown in **Table 5.7.**

Table 5.7 Student's T-test Values

Soil Type	Type of Relationship	PCI range	Relationship between Modified Structural Number and Characteristic Deflection	Calculated t-test value	Tabular t-test value
SM	Linear	40 - 60	DEF = 0.2179 MSN - 0.0441	-0.0052	2.2281
		60 - 80	DEF = -2.2529 MSN + 8.0951	-0.0024	2.5706
		80 - 100	DEF = -0.1269 MSN + 0.921	-0.0077	2.3646
	Power	40 - 60	DEF = 0.1749 $MSN^{1.1258}$	0.1446	2.2281
		60 - 80	DEF = 143810 $MSN^{-10.35}$	0.2851	2.5706
		80 - 100	DEF = 3.6267 $MSN^{-1.678}$	0.2110	2.3646
SC	Linear	40 - 60	DEF = 0.5958 MSN - 1.4339	-0.0051	2.2281
		60 - 80	DEF = -1.6633 MSN + 7.2018	0.0016	4.3027
		80 - 100	DEF = -0.2444 MSN + 1.6299	0.0026	2.7765
	Power	40 - 60	DEF = 0.0171 $MSN^{2.8804}$	0.1907	2.2281
		60 - 80	DEF = 1412.8 $MSN^{-5.58}$	0.1398	4.3027
		60 - 100	DEF = 2.7681 $MSN^{-1.018}$	0.0102	2.7765

The models developed as part of the regression analysis were used in validating the relationship between the variables, modified structural number and characteristic deflection. The average prediction error (APE) and total prediction error (TPE) for characteristic deflection were calculated and are tabulated in **Table 5.8.**

For soil type SM, the average prediction error (APE) for characteristic deflection for classified PCI values were found to vary between 1.069 % and 3.355 % for linear models and the total prediction error (TPE) for characteristic deflection was found to vary between 0.016 % and 0.06 %. In case of power function models for soil type SM, the average prediction error (APE) for characteristic deflection for classified PCI values were found to vary between 0.444 % and 1.690 % and the total prediction error (TPE) for characteristic deflection varied between -0.461 % and -2.361 %.

The average prediction error (APE) for characteristic deflection for classified PCI values calculated for soil type SC were found to vary between 0.129 % and 4.122 % and the total prediction error (TPE) for characteristic deflection were found to vary between -0.005 % and 0.028 % for linear regression models respectively. In case of power function models for soil type SC, the average prediction error (APE) for characteristic deflection for classified PCI values were found to vary between 0.116 % and 2.057 % and the total prediction error (TPE) for characteristic deflection varied between -0.019 % and -1.444%.

It may be observed in Table 5.8 that the ‘Average Prediction Errors’ and ‘Prediction errors of Totals’ in estimation of variables are much lesser and reasonably accurate. Hence, the regression equations developed were assumed to be acceptable for future predictions.

Table 5.8 Validation of the Regression Analysis

Soil Type	Type of Relationship	PCI range	Relationship between Modified Structural Number and Characteristic Deflection	Average Prediction Error (%)	Prediction Error of Totals (%)
SM	Linear	40 - 60	DEF = 0.2179 MSN - 0.0441	1.069	0.016
		60 - 80	DEF = -2.2529 MSN + 8.0951	3.355	0.021
		80- 100	DEF = -0.1269 MSN + 0.921	3.039	0.060
	Power	40 - 60	DEF = 0.1749 MSN $^{1.1258}$	0.444	-0.461
		60 - 80	DEF = 143810 MSN $^{-10.35}$	1.220	-2.361
		80- 100	DEF = 3.6267 MSN $^{-1.678}$	1.690	-1.531
SC	Linear	40 - 60	DEF=0.5958MSN - 1.4339	3.078	0.028
		60 - 80	DEF=-1.6633MSN+ 7.2018	4.122	-0.016
		80- 100	DEF = -0.2444 MSN + 1.6299	0.129	-0.005
	Power	40 - 60	DEF = 0.0171 MSN $^{2.8804}$	1.724	-1.084
		60 - 80	DEF = 1412.8 MSN $^{-5.58}$	2.057	-1.444
		60- 100	DEF = 2.7681 MSN $^{-1.018}$	0.116	-0.019

5.2.3 Discussion on the Relationship between MSN and Deflection

5.2.3.1 Influence of subgrade soil type on MSN- Deflection Relationship

The subsoil investigations conducted on the study stretches revealed that Silty Sand (SM) and Clayey Sand (SC) were predominant subgrade soil types in the pavement layers. From the analysis of the observational data, it was found that for silty sand subgrade, the deflection decreased with increase in MSN, where as for soil type SC, the deflection always showed increasing trend. Hence, it can be concluded that subgrade soil type influences the pavement strength.

However, the linear regression relationship between the variables, Modified Structural Number (MSN) and characteristic deflection (DEF) for both the soil types - SM and SC gave good correlation with R^2 value of 0.73 and 0.71 respectively. In the case of power function relationships, the R^2 value for soil types, SM and SC, are 0.78 and 0.67 respectively, which showed a better correlation in the case of SM soil. But, comparison of models using observed and estimated values on a linear scale gave fairly good R^2 values for both soil types, *ie,* 0.63, 0.60, 0.62 and 0.62 indicating that all the relationships are valid.

5.2.3.2 Influence of pavement condition on MSN- Deflection Relationship

In case of classified values of PCI, the linear relationship between MSN and deflection for soil type SC and for PCI ranging between 60 and 80 gave the best fit compared to others, with an R^2 value of 0.87. For all cases, the correlation gives fairly good values ranging between 0.67 and 0.85 and hence can be considered. In the case of power function relation for classified value of PCI, the best fit is obtained for soil type SM for PCI range of 80 - 100 with an R^2 value of 0.85. For all classes of PCI, the R^2 values range between 0.69 and 0.85 and hence can be adopted. Comparison of the function using observed and estimated values on a linear scale gave R^2 greater than 0.5 in all the cases establishing the applicability of the models.

The parameters of both linear and non linear regression models estimated and calibrated are statistically significant as indicated by various statistical parameters like R^2, t-test, F-test. ‘Average Prediction Errors’ and ‘Prediction errors of Totals’ in estimation of variables are much lesser and reasonably accurate.

Power series function relationships can be used for soil type SM, where as in the case of soil type SC, linear regression relationship gave better results. Considering the classified value of PCI also, the same strength is obtained. In the case of pavement in good condition, for PCI value ranging between 60 and 80, linear regression model gave very good results for soil type SC.

5.2.4 Characteristics of the Dataset Considered

Statistical checks were done for the data set considered in the analysis and the results are given below:

Subgrade Soil Type: **Silty Sand (SM)**

Mean = 0.531

Standard Deviation = 0.145

Standard Error = 0.029

Subgrade Soil Type: **Clayey Sand (SC)**

Mean = 0.746

Standard Deviation = 0.232

Standard Error = 0.058

5.3 INFLUENCE OF SOIL PARAMETERS ON DEFLECTION

The deflection of the pavement is influenced by the properties of the subgrade soil and strength of the pavement layers represented as structural number (SN). Attempt was made to establish the correlation of Maximum Dry Density (MDD), Optimum Moisture Content (OMC), Field Dry Density (FDD), Field Moisture Content (FMC), California Bearing Ratio (CBR), Plastic Limit (PL), Liquid Limit (LL), Plasticity Index (PI) and Gravel (G), Sand (S), Silt & Clay (S&C) fractions with characteristic deflection (DEF).

Multiple linear regression attempts to model the relationship between two or more explanatory variables and a response variable by fitting a linear equation to observed data. The multiple linear regression models take the form,

$$y = a_0 + a_1x_1 + a_2x_2 + \ldots + a_nx_n$$

where, $a_1, a_2 \ldots a_n$ = coefficients determined by method of least squares

y = Characteristic Deflection measured in mm (DEF)

$x_1, x_2, \ldots x_n$ = Soil parameters considered

SPSS software was used for multiple linear regression analysis. Different combinations of variables were tried and regression models are reported here. Data given in Tables 4.3 to 4.7 are used for developing the models.

5.3.1 Liquid Limit LL

I. Maximum Dry Density

1) DEF = -0015 LL+0.0103 G-2.422 MDD-0.0388 CBR+0.0025 SN+5.5243

------- (5.1)

(R^2 = 0.50)

2) DEF = -0.014 LL-0.0186 S-2.7855 MDD-0.0012 CBR-0.0714 SN+7.705

------- (5.2)

(R^2 = 0.60)

3) DEF = -0.0018 LL+0.0107 S&C-1.9436 MDD-0.0014 CBR+0.0297 SN+4.4774

------- (5.3)

(R^2 = 0.50)

II. Optimum Moisture Content

4) DEF = -0.0032 LL+0.0061 G+0.1117 OMC-0.0024 CBR-0.0558 SN-0.2697

------- (5.4)

(R^2 = 0.51)

5) DEF = -0.0052 LL -0.0078 S+0.0985 OMC-0.0016 CBR-0.06995 SN+0.4811

------- (5.5)

(R^2 =0.52)

6) DEF = -0.0022 LL+0.0027 S+0.1035 OMC-0.0476 CBR-0.0891 SN+0.5026

------- (5.6)

(R^2 = 0.50)

III. Field Dry Density

7) DEF =0.0226 LL-0.0039 G+0.901 FDD-0.0071 CBR+0.0771 SN-0.5388

------- (5.7)

(R^2 = 0.50)

8) DEF = 0.0143 LL-0.0104 S+0.7922 FDD-0.0052 CBR-0.0047 SN+0.3316

------- (5.8)

(R^2 = 0.52)

9) DEF = 0.0138 LL+0.0249 S&C+1.3355 FDD-0.0041 CBR-0.0089 SN-1.589

------- (5.9)

(R^2 = 0.63)

IV. Field Moisture Content

10) DEF = -0.00296 LL+0.0041 G+0.0441 FMC-0.0044 CBR+0.0743 SN+0.2014

------- (5.10)

(R^2 = 0.60)

11) DEF = -0.0066 LL -0.0081 S-0.0033 CBR+0.0368 SN+0.8643 FMC+0.0417

------- (5.11)

(R^2 = 0.60)

12) DEF = -0.0045 LL+0.0058 S&C+0.0426 FMC-0.0037 CBR+0.0748 SN+0.1744

------- (5.12)

(R^2 = 0.60)

5.3.2 Plastic Limit PL

V. Maximum Dry Density

13) DEF = 0.0085 PL + 0.0132 G-2.854 MDD–0.0014 CBR +0.0052 SN + 6.328

------- (5.13)

(R^2 = 0.50)

14) DEF = -0.0443 PL -0.0282 S - 4.127 MDD 0.004 CBR – 0.1487 SN + 11.1471

------- (5.14)

(R^2 = 0.70)

15) DEF = -0.0016 PL+ 0.0102 S&C-1.8935 MDD-0.0017 CBR+0.02918 SN+4.3878

------- (5.15)

(R^2 = 0.45)

VI. Optimum Moisture Content

16) DEF = 0.013 PL +0.0087 G + 0.1363 OMC–0.0017 CBR – 0.0851 SN-0.4923

------- (5.16)

(R^2 = 0.52)

17) DEF = -0.0141 PL-0.0095 S + 0.1137 OMC - 0.0011 CBR - 0.0948 SN + 0.4861

------- (5.17)

(R^2 = 0.53)

18) DEF = -0.0051 PL+0.0023 S&C + 0.1106 OMC - 0.0022 CBR-0.0365 SN - 0.2179

------- (5.18)

(R^2 = 0.50)

VII. Field Dry Density

19) DEF =0.0367 PL-0.0094 G+0.9424 FDD-0.0064 CBR+0.1088 SN-0.5619

------- (5.19)

(R^2 = 0.50)

20) DEF = 0.02 PL-0.00998 G+0.7065 FDD-0.0047 CBR+0.0113 SN+0.3978

------- (5.20)

(R^2 = 0.50)

21) DEF = 0.0206 PL+0.0257 S&C+1.3145 FDD-0.0035 CBR-0.006 SN-1.5864

------- (5.21)

(R^2 = 0.64)

VIII. Field Moisture Content

22) DEF = 0.0067 PL+0.0052 G+0.0463 FMC-0.0044 CBR+0.0717 SN+0.1678

------- (5.22)

(R^2 = 0.60)

23) DEF = 0.013 PL-0.0094 S+0.0438 FMC-0.0032 CBR+0.02675 SN+0.9479

------- (5.23)

(R^2 = 0.60)

24) DEF = -0.0058 PL+0.005 S&C+0.4246 FMC-0.0039 CBR+0.0731 SN+0.1951

------- (5.24)

(R^2 = 0.60)

5.3.3 Plasticity Index PI

IX. Maximum Dry Density

25) DEF = 0.0278 PI + 0.0052 G - 1.202 MDD-0.0033 CBR-0.0199 SN+3.138

------- (5.25)

(R^2 = 0.52)

26) DEF = 0.0047 PI - 0.0104 S - 1.595 MDD - 0.002 CBR-0.0365 SN + 4.723

------- (5.26)

(R^2 = 0.54)

27) DEF = 0.0289 PI+0.0038 S&C - 0.992 MDD - 0.0031 CBR - 0.0095 SN+2.71

------- (5.27)

(R^2 = 0.51)

X. Optimum Moisture Content

28) DEF = 0.0227 PI + 0.0033 G+0.0612 OMC -0.0035 CBR – 0.0532 SN+0.227

------- (5.28)

(R^2 = 0.53)

29) DEF = 0.0208 PI -0.003 S+0.0562 OMC – 0.0032 CBR-0.0523 SN+0.525

------- (5.29)

(R^2 = 0.53)

30) DEF = 0.0264 PI-0.0007 S&C+0.059 OMC- 0.0036 CBR-0.0421 SN+0.299

------- (5.30)

(R^2 = 0.53)

XI. Field Dry Density

31) DEF =0.0715 PI + 0.007 G+0.9097 FDD-0.0046 CBR-0.028 SN - 0.4196

------- (5.31)

(R^2 = 0.63)

32) DEF = 0.0567 PI - 0.0043 S+0.771 FDD-0.0042 CBR-0.0598 SN - 0.0059

------- (5.32)

(R^2 = 0.62)

33) DEF = 0.0469 PI+0.019 S&C+1.173 FDD-0.0031 CBR-0.0686 SN-1.205

------- (5.33)

(R^2 = 0.70)

XII. Field Moisture Content

34) DEF = 0.0159 PI+0.024 G+0.0302 FMC-0.0046 CBR+0.0303 SN+0.4082

------- (5.34)

(R^2 = 0.60)

35) DEF = 0.0108 PI-0.0036 S+0.0298 FMC-0.0043 SN+0.0237 SN+0.6998

------- (5.35)

(R^2 = 0.60)

36) DEF = 0.0104 PI+0.0023 S&C+0.0323 FMC-0.229 CBR+0.0083 SN+0.611

------- (5.36)

(R^2 = 0.66)

5.3.4 Recommended Models

The following multiple linear regression models are recommended to establish relationship between deflection and soil parameters. It was found in multiple linear regression analysis that Field Dry Density (FDD) had better significance level in the recommended models.

(1) DEF = 0.0206 PL+0.0257 S&C+1.3145 FDD-0.0035 CBR-0.006 SN-1.5864

------- (5.37)

(R^2 = 0.64)

(2) DEF = 0.0715 PI + 0.007 G+0.9097 FDD-0.0046 CBR-0.028 SN-0.4196

------- (5.38)

(R^2 = 0.63)

(3) DEF = 0.0567 PI-0.0043 S+0.771 FDD-0.0042 CBR-0.0598 SN-0.0059

------- (5.39)

(R^2 = 0.62)

(4) DEF = 0.0469 PI+0.019 S&C+1.173 FDD-0.0031 CBR-0.0686 SN-1.205

------- (5.40)

(R^2 = 0.70)

(5) DEF = 0.0138 LL+0.0249 S&C+1.3355 FDD-0.0041 CBR-0.0089 SN-1.589

------- (5.41)

(R^2 = 0.63)

5.3.5 Discussion on Relationship between Soil Parameters and Deflection

All possible combinations of soil parameters were used in multiple linear regression analysis to determine the influence on the deflection. Considering Liquid Limit (LL) and Maximum Dry Density (MDD), only Sand (S) fraction showed acceptable correlation with R^2 value of 0.60. In the case of Liquid Limit (LL) and Optimum Moisture Content (OMC), the correlation is low with R^2 values around 0.5. For Liquid Limit (LL) and Field Dry Density (FDD), Sand (S) and Silt & Clay (S&C) fractions showed good correlation with values of 0.52 and 0.63 respectively. Combination of Liquid Limit (LL) and Field Moisture Content (FMC) with all the three soil fractions independently showed a good relationship with R^2 value of 0.60. Considering Plastic Limit (PL) and Maximum Dry Density (MDD), along with sand fraction (S), it was observed that these parameters influence the deflection with an R^2 value of 0.70. For Gravel (G) and Silt & Clay (S&C) fractions, the correlation is poor with R^2 value less than 0.5. Combinations of Plastic Limit (PL) and Optimum Moisture Content (OMC) with Gravel (G) and Sand (S) fractions gave R^2 value of more than 0.5. In the case of Plastic Limit (PL) and Field Dry Density (FDD), Silt & Clay (S&C) fraction showed better correlation with R^2 value of 0.64. Combinations of Plastic Limit (PL) and Field Moisture Content (FMC) with Gravel (G), Sand (S) and Silt & Clay (S&C) fractions independently gave better correlation with values of 0.60. For the case of Plasticity Index (PI), all the combinations gave R^2 value more than 0.5 showing good relationships. The best fit equation is obtained with Plasticity Index (PI), Field Dry Density (FDD), and Silt & Clay (S&C) combination with an R^2 value of 0.70.

The results show that Field Dry Density and Plasticity Index of the subgrade soil influence the deflection, for all fractions of the soil. It was also found in multiple linear regression analysis that Field Dry Density (FDD) had better significance level than other parameters considered in the models. Maximum Dry Density showed less influence on the deflection in the case of Gravel and Silt & Clay fractions.

For roads of similar traffic and age, and if the subsoil properties and layer details of the pavement are known, the deflection can be calculated using the relationships developed in this study and can be used for overlay design purpose.

5.4 SUMMARY

An attempt has been made in this chapter to establish the linear and non linear relationships between the observed deflection and structural strength of the pavement represented as Modified Structural Number (MSN). Both power function and linear relationship between MSN and deflection gave acceptable values. The effects of subgrade soil type and pavement condition on deflection were studied and explained with regression models. In order to study the influence of the subgrade soil parameters like LL, Pl, OMC, FMC, FDD, MDD and CBR along with the strength of the pavement (SN) on deflection, different combinations of multiple linear regression models were tried using SPSS package. The influence of Gravel, Sand and Silt & Clay fractions on the above relationships were also studied independently. The results obtained from the analysis of the multiple linear models indicated a comparatively low correlation highlighting the complexity of the problem.

But, by using more data, and for different combinations of traffic and pavement condition, the results can be further refined to get generalized relationships and reliably used for field applications by practicing engineers.

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Chapter 6

PAVEMENT PERFORMANCE MODELS

6.1 INTRODUCTION

Pavement is a complex physical structure with non- homogeneous composition of bituminous mixture, aggregate and sub grade soil with variation in traffic, climatic conditions, environment and construction quality. Researchers have developed different models for predicting the performance of flexible pavements using the data generated through extensive studies in different geographical regions.

Pavement deterioration is attributed by various types of distresses, and these have to be modeled separately. Each distress develops and progresses at different rates in different environments, and hence the Road Deterioration relationships should be calibrated to local conditions before using them for road investment analyses. To facilitate this, these relationships manifest a number of user-defined deterioration factors, which affects the magnitude of a particular distress. The model coefficients selected should be able to modify the rates of deterioration for different types of pavement materials. To model the road deterioration properly, homogeneous road sections in terms of physical attributes and condition should be identified which will enable the user to apply a particular set of road deterioration relationships. Therefore, the basic unit is the homogeneous road section and several investment options can be assigned to these sections for analysis.

Various performance rating models for pavements have been reported by researchers. The AASHO Road Test has developed the pavement serviceability performance concept in a better way (Carey and Irick, 1960). Regression models were also developed by correlating the performance data to design inputs.

In AASHO Road Test, five fundamental assumptions were used in the development of the concept of serviceability (HRB, 1962). They are: (1) highways are for the comfort of the travelling user; (2) the user's opinion on the highway performance is subjective; (3) there are characteristics that can be measured and related to the user's perception of performance; (4) performance may be expressed by the average opinion of all users; and (5) performance represents the serviceability with increasing load applications.

Present Serviceability Index (PSI), Present Serviceability Rating (PSR), Pavement Condition Index (PCI) and Unevenness Index (UI) models were developed based on pavement evaluation data, to predict flexible pavement performance. These models depict the functional performance of the pavements and help to develop maintenance strategy for the pavement. In the present study, Pavement Condition Index and Unevenness Index are used for development of models.

6.2 PERFORMANCE EVALUATION

In order to arrive at a realistic evaluation of the study stretches, periodic data were collected at permanent observation points to assess the functional, structural and safety condition of the pavements. The parameters studied were deflection, unevenness, texture depth, condition and skid resistance. Eight road stretches were selected for the study consisting of seventeen sections.

6.3 DEFLECTION STUDIES

The amount of pavement deflection under a wheel load is the measure of the structural stability of the pavement system. For weaker sections, higher value of deflection is shown. From the data obtained through field investigations conducted with Benkelman Beam as per the procedure given in Chapter 3, the progression of the deflection on the study stretches are plotted as given in **Fig. 6.1 to Fig. 6.8.** Slope of the progression line for the homogenous sections, which is a measure of the change in the value of Y corresponding to a unit change in the value of X, were also derived.

On all the study stretches except AK- 2, increasing trend was obtained indicating that the initial stabilization period is over and there is a reduction in the strength of the pavement with the passage of vehicles. For AK-2, due to overlaying, the road is still under stabilization period and hence reversal of trend is seen. Out of the seventeen homogeneous sections studied, very good correlation was obtained for fourteen sections except MV-2, KK-2 and AK-2. The R^2 value obtained is 0.56 or MV-2 and 0.51 for both AK-2 and KK-2, which can be due to the poor condition of the road sections.

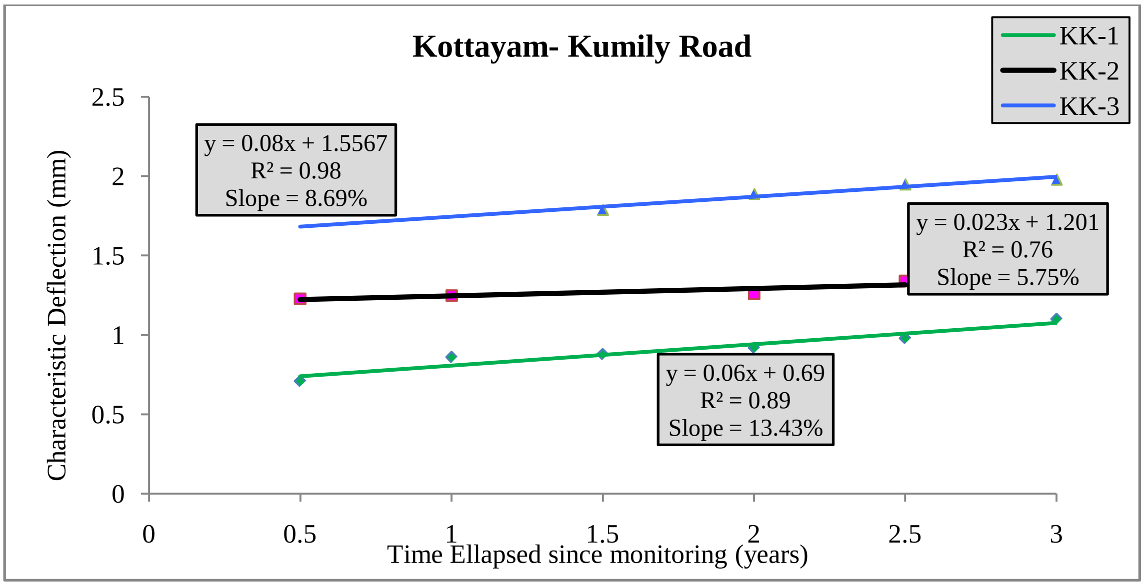

Fig. 6.1 Progression of deflection on Kottayam - Kumily road

The slope of the deflection lines above shows that for the homogeneous section 1, the rate of change of deflection is more than double that of section 2 indicating faster reduction in strength for HS 1.

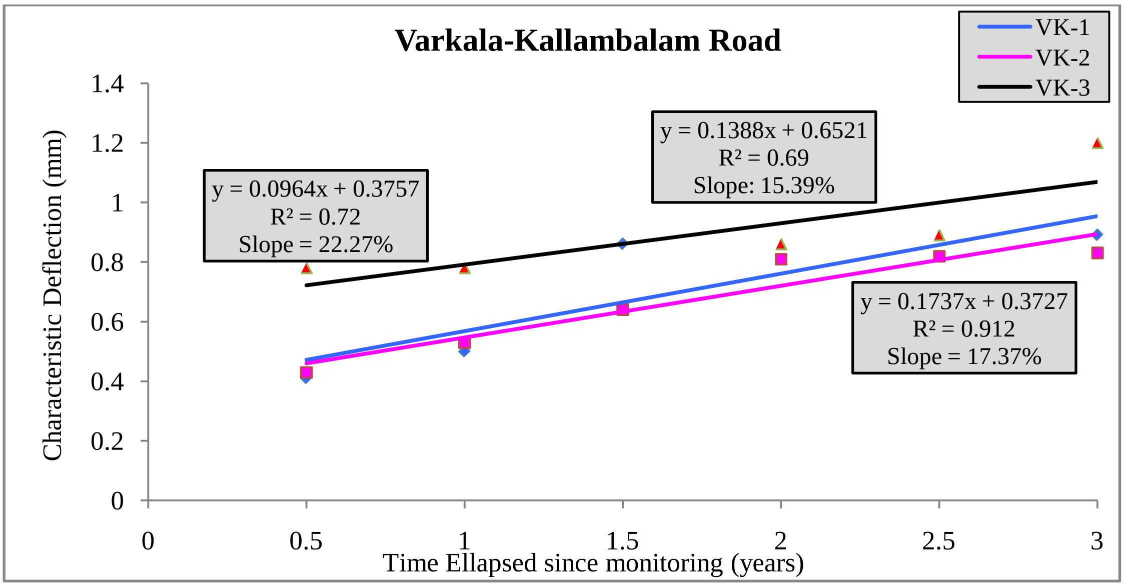

Fig. 6.2: Progression of deflection on Varkala - Kallambalam road

The slope of the deflection lines given above shows that for the homogeneous section 1, the rate of change of deflection is only slightly more than that of section 2 and 3 indicating almost same rate of reduction in strength. For all the sections, faster changes in the deflection values are indicated.

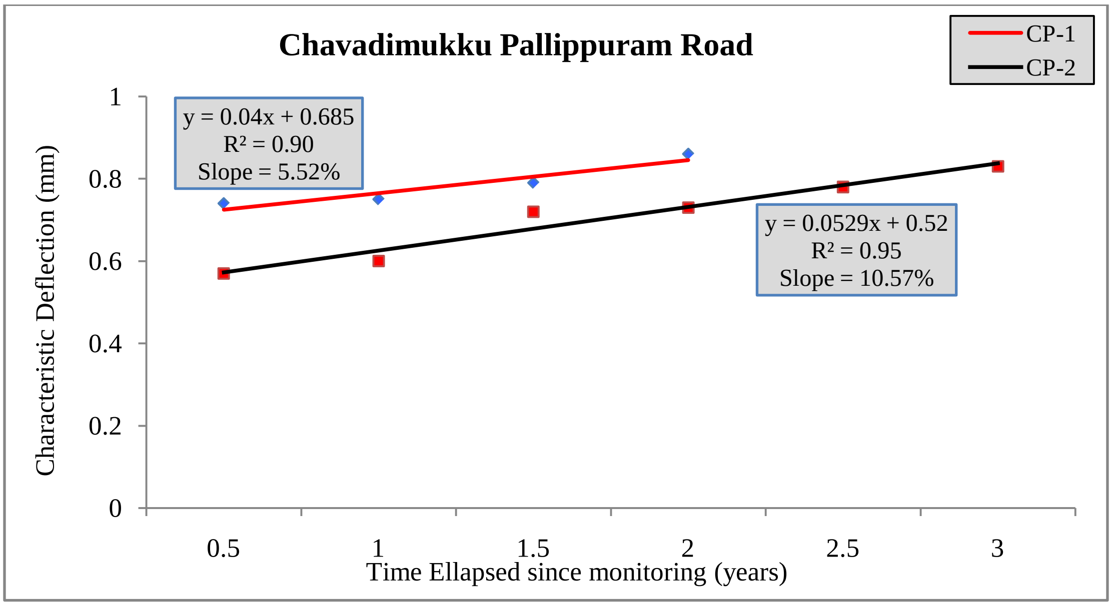

Fig. 6.3: Progression of deflection on Chavadimukku- Pallippuram road

The slope of the deflection lines given above shows that for the homogeneous section 1, the rate of change of deflection is half that of section 2 indicating almost slower rate of reduction in strength for HS 1. Hence section 1 is more stable than 2.

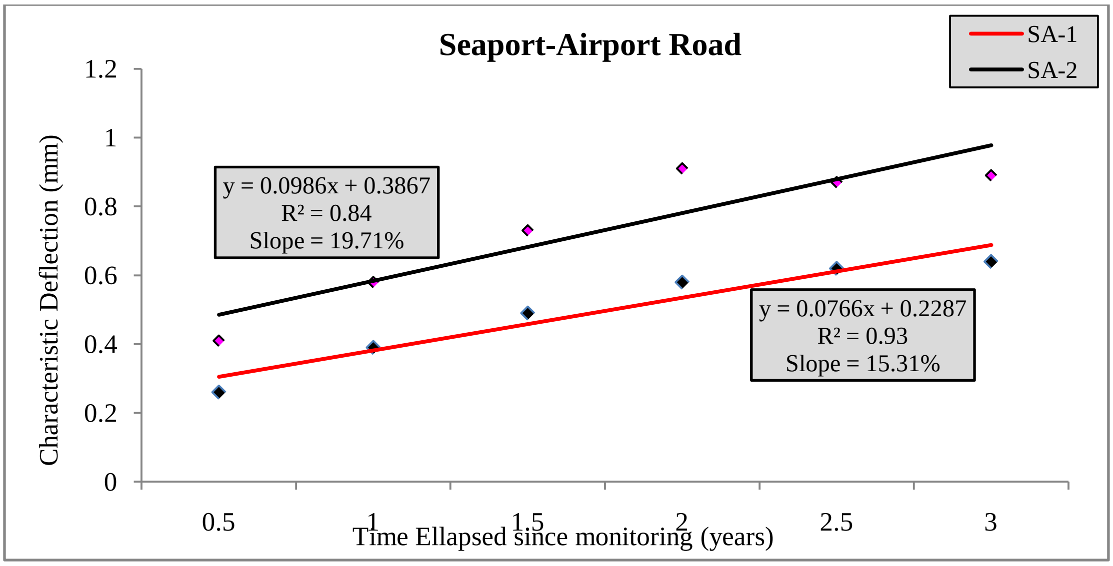

Fig. 6.4: **Progression of deflection on Seaport Airport road**

The slope of the deflection lines given above shows that for both the homogeneous sections, the rate of change of deflection are 15 and 19 percent indicating almost same rate of reduction in strength but the progression is very fast.

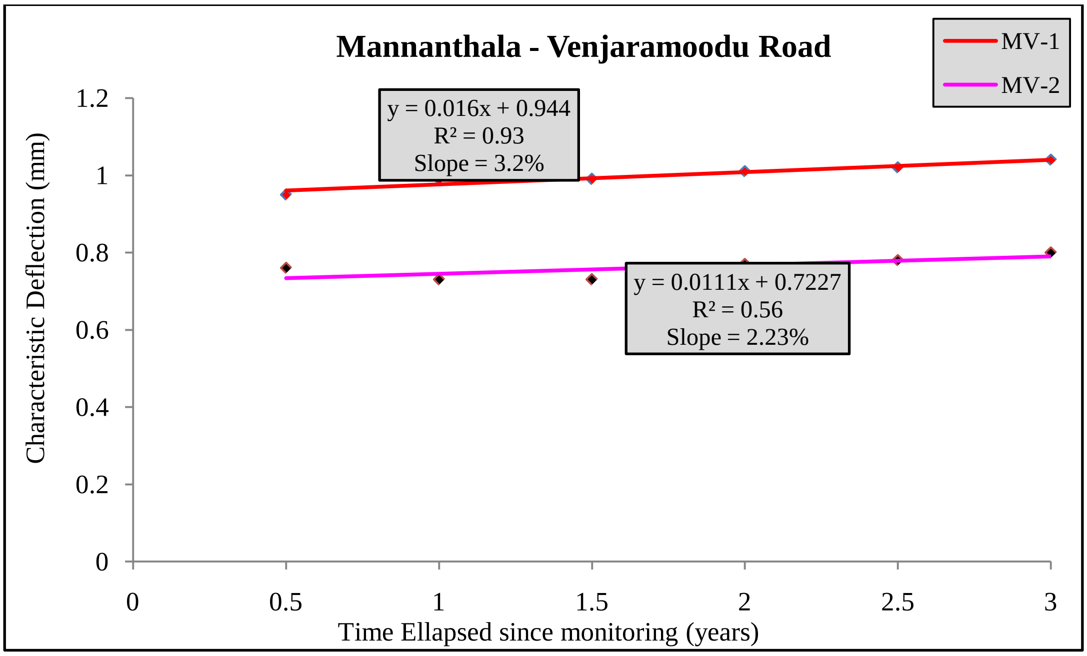

Fig. 6.5: Progression of deflection on Mannanthala Venjaramood road

The slope of the deflection lines are 3.2 and 2.3 percent only which shows that the progression is slow and hence deterioration of the pavement also will be slow.

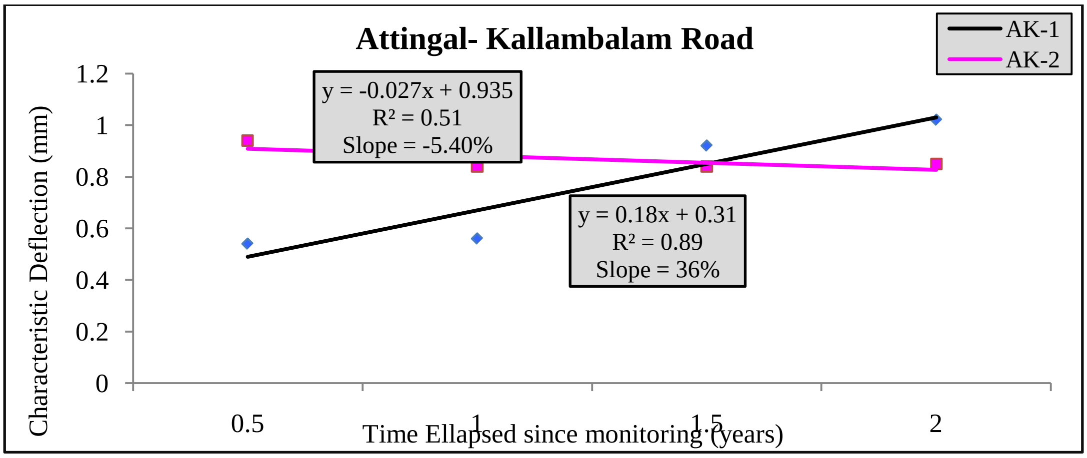

Fig. 6.6: Progression of deflection on Attingal- Kallambalam road

From the trend line, it was observed that the deflection value was on the reducing trend for HS 2. This can be due to the fact that surfacing was done for the stretch after the first set of data was collected and the pavement was in the stabilization period during the subsequent data collection phase. The deterioration of AK-1 is very fast.

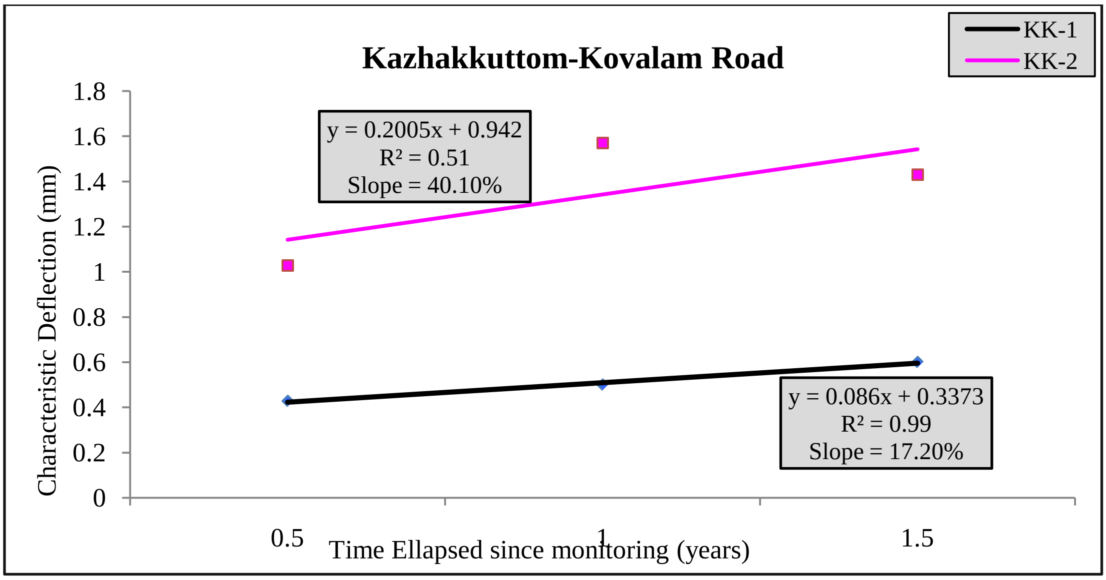

Fig. 6.7: Progression of deflection on Kazhakkuttom-Kovalam road

For the Kazhakkuttam- Kovalam road, HS 1 showed good relation between successive readings with an R^2 value of .99 and the slope of the regression line is less than the other section. The regression line and it's trend for the two sections revealed that the rate of deterioration of HS 2 which was having poor riding quality and condition at the time of the study is very fast and needs immediate strengthening.

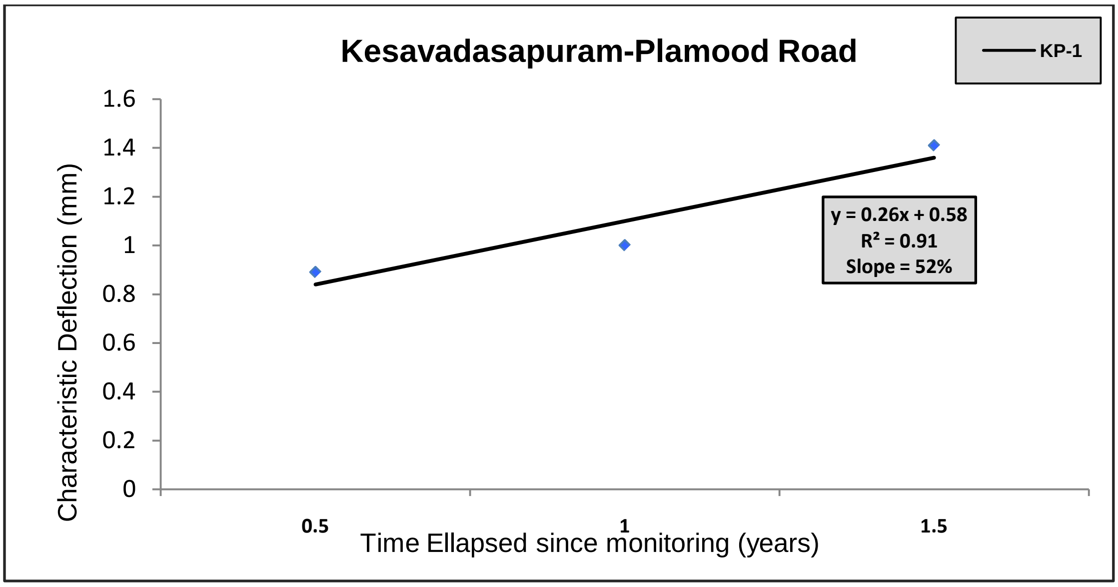

Fig. 6.8: Progression of deflection on Kesavadasapuram- Plamood road

At the time of the study, the road condition was poor and the same trend was observed for the progression of the deflection with steep gradient showing fast deterioration.

6.4 ROUGHNESS STUDIES

The serviceability of a pavement is largely a function of its unevenness, which is represented as IRI values internationally. This is also represented in terms of mm per kilometre and measured using the Bump Integrator. The roughness values on the study stretches were measured from the field studies as per the procedure discussed in Chapter 3. The progression of unevenness on the study roads are given in **Fig. 6.9 to 6.16.** The correlation is very good except for three stretches, with an R^2 value of 0.62 for KP-2, 0.63 for AK-1 and 0.61 for AK-1. For all other study stretches, very good correlation is obtained.

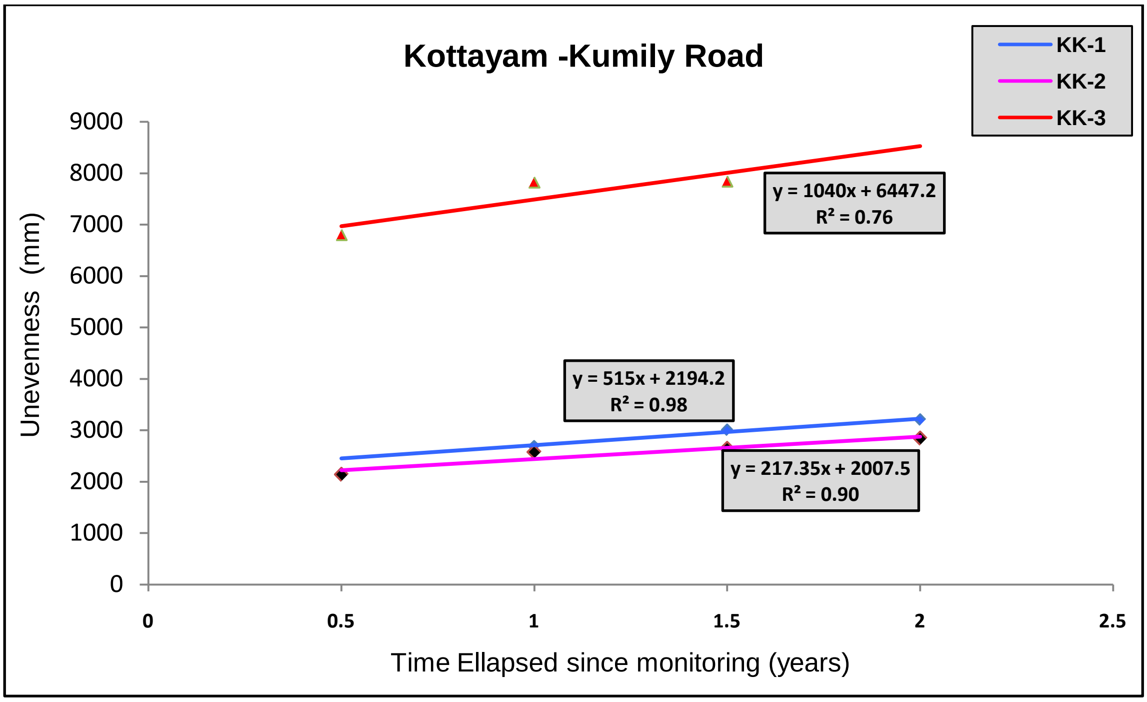

Fig. 6.9: Progression of unevenness on Kottayam-Kumily road

The progression of roughness for KK road shows an annual average incremental increase of 515 mm for KK-1, 217 mm for KK-2 and 1040 mm for KK-2. For KK -3, the roughness at the time of the first measurement also was very high (more than 6000 mm). It can be inferred that KK-2 is more stable and KK-3 is in very poor condition demanding maintenance.

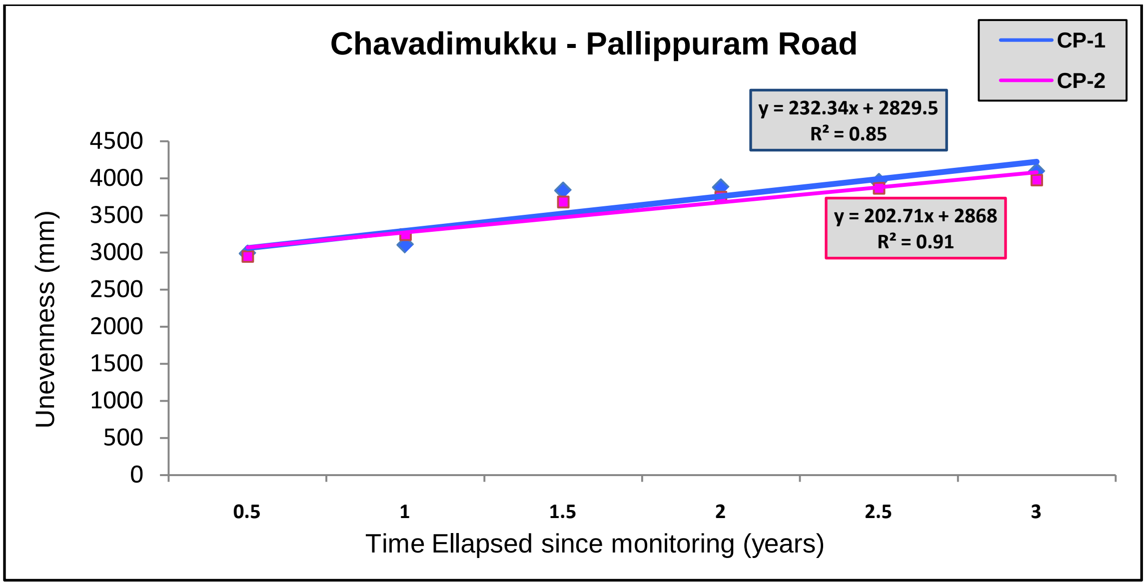

Fig. 6.10: Progression of unevenness on Chavadimukku - Pallippuram road

The progression of roughness for CP road follows the same trend for both homogeneous sections initially, with an annual average increase of 232 mm and 202 mm, but at the fifth measurement (2.5 years) onwards, CP-1 showed faster growth of roughness.

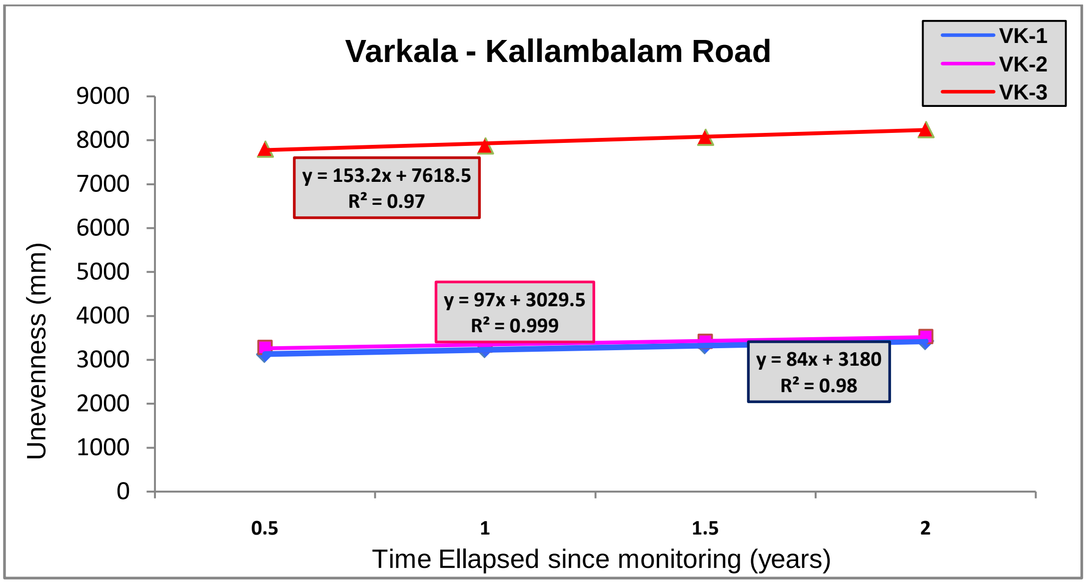

Fig. 6.11: Progression of unevenness on Varkala-Kallambalam road

For all the three homogeneous sections of VK road, the rate of progression is the almost same with an average annual increment of 97 mm, 84 mm and 153 mm respectively for sections 1, 2 and 3. The roughness at the time of first measurement for VK-3 was higher than the other two homogeneous sections demanding maintenance.

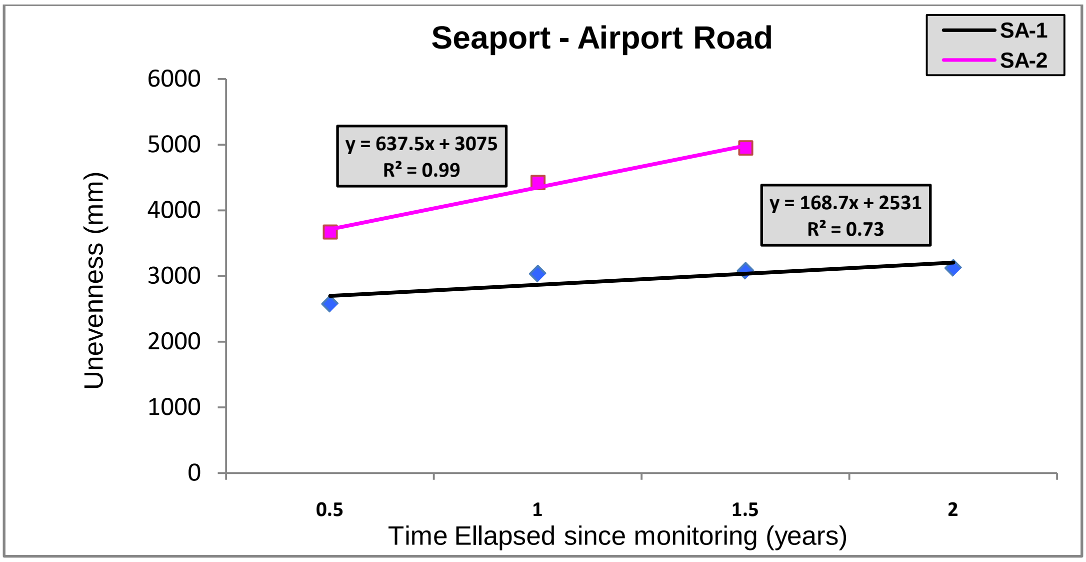

Fig. 6.12: Progression of unevenness on Seaport-Airport road

The unevenness value at the time of initial measurement for SA-2 was higher than that of SA-1 (3075 mm and 2531 mm) and its rate of progression also is faster with an increase of 638 mm per year where as for SA-1, the annual increase in roughness is 169 mm only. This indicates the poor condition of the road in terms of cracks, potholes and raveling and structural inadequacy for SA-2.

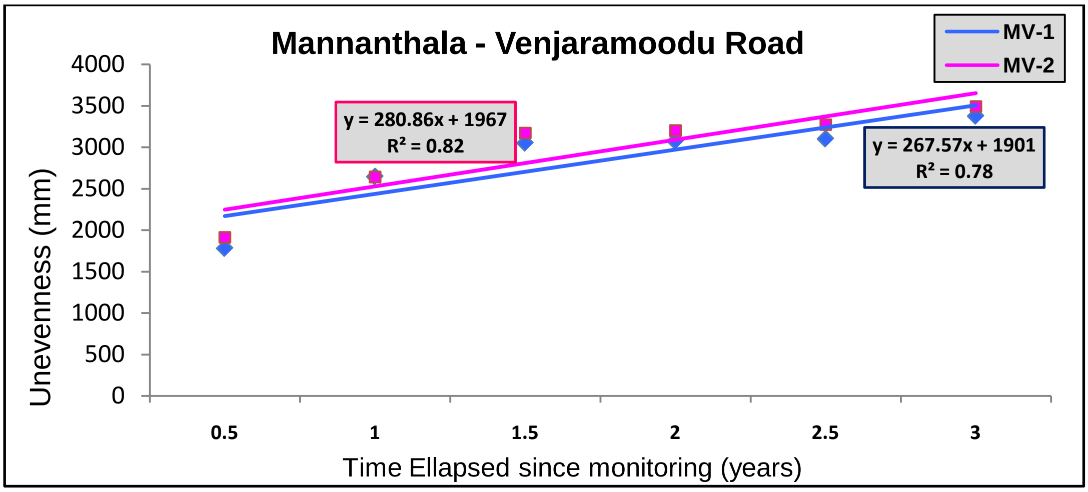

Fig. 6.13: Progression of unevenness on Mannanthala-Venjaramoodu road

For M V road, the value of initial deflection and growth rate are almost same, with slight increase for MV-2. This indicates that both the homogeneous sections are structurally adequate with unevenness increment of 268 mm and 281 mm per year for MV-1 and MV-2 respectively.

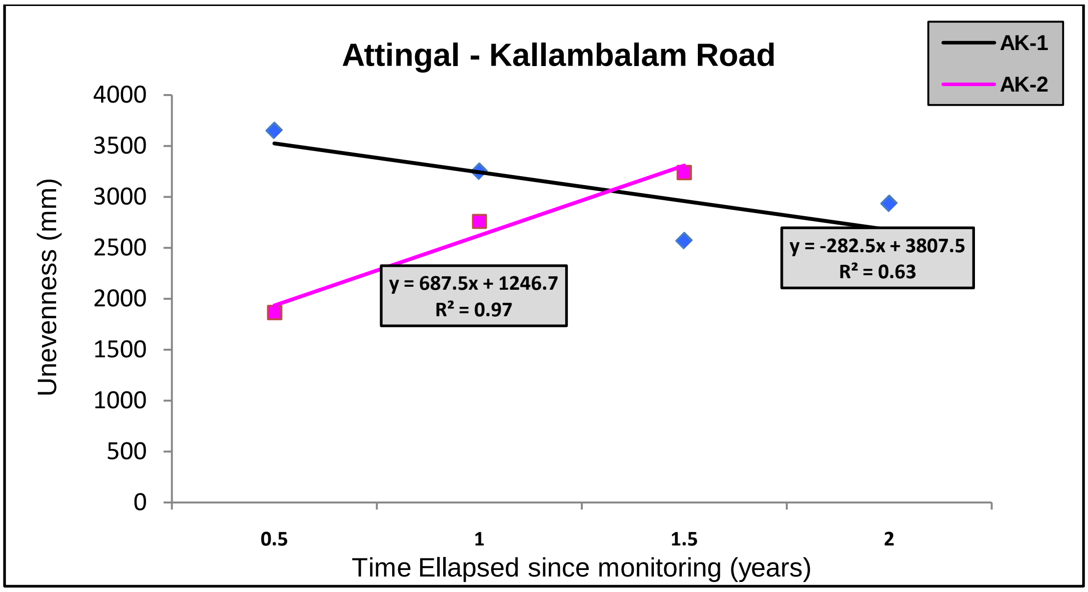

Fig. 6.14: Progression of unevenness on Attingal-Kallambalam road

For AK-2, the roughness shows a decreasing trend due to the fact that after the first measurement, maintenance work was done and this has stabilized only after the third measurement.

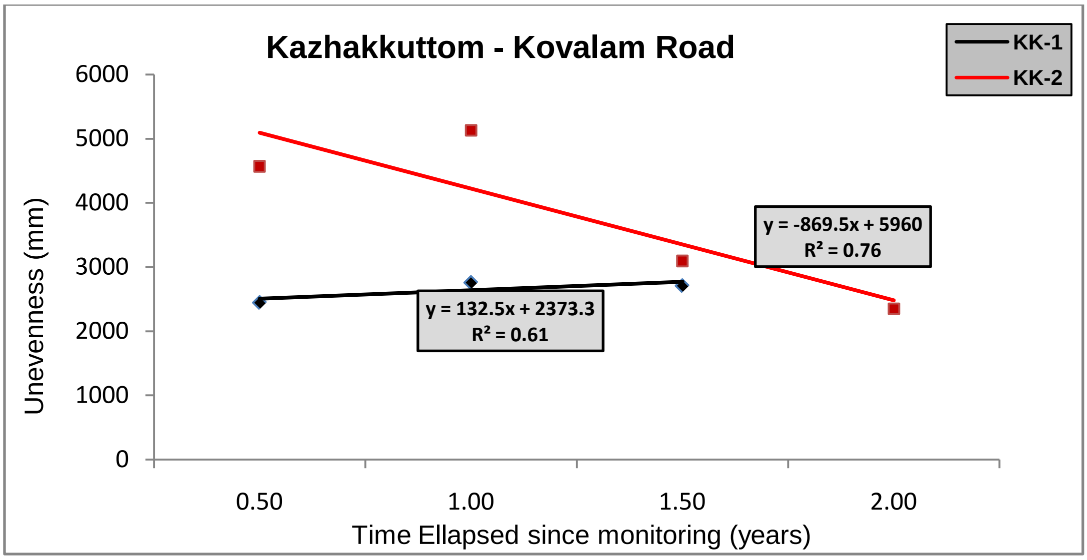

Fig. 6.15: Progression of unevenness on Kazhakkuttam-Kovalam road

For the second stretch, maintenance work was done after the second measurement and the roughness showed decrease for the next two measurements, which can be due to the stabilization of the surface.

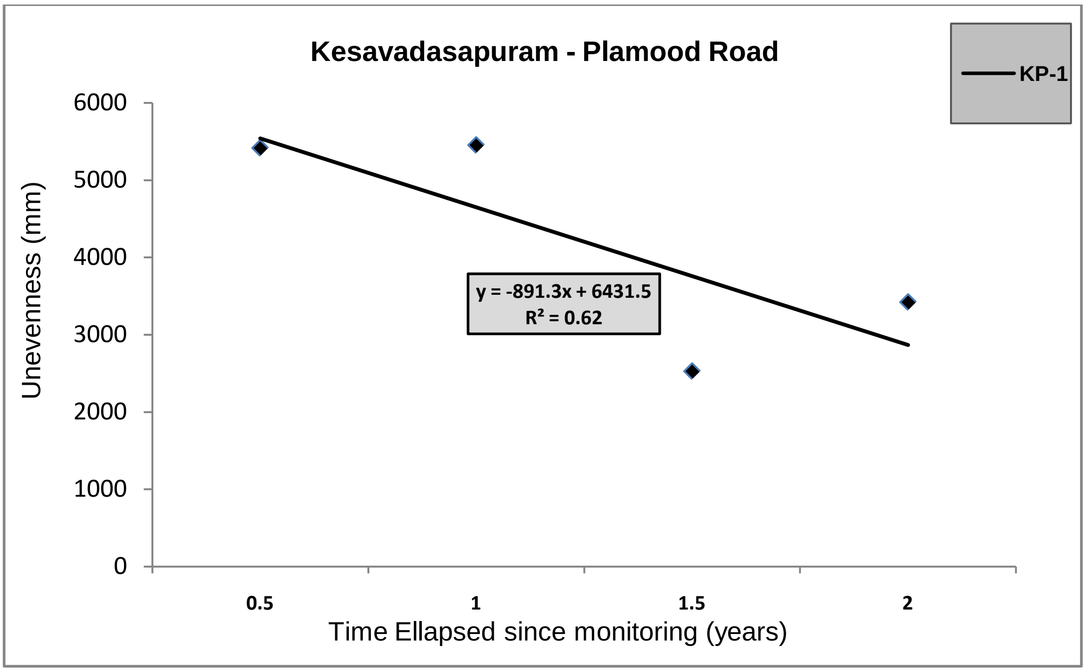

Fig. 6.16: Progression of unevenness on Kesavadasapuram-Plamood road

This being an urban section of NH, is showing signs of deterioration very fast and after the second measurement, patching and pothole filling was done. But, the section showed increasing trend for roughness in the next measurement also showing the instability.

6.5 STUDY OF MACRO TEXTURE

The macro texture of the surface and micro texture of the aggregate have an influence on the surface unevenness and skidding properties of the pavement. Hence the texture depth studies were done on the study stretches using sand patch test. Progression of the texture depth is given in **Fig. 6.17 to 6.23.**

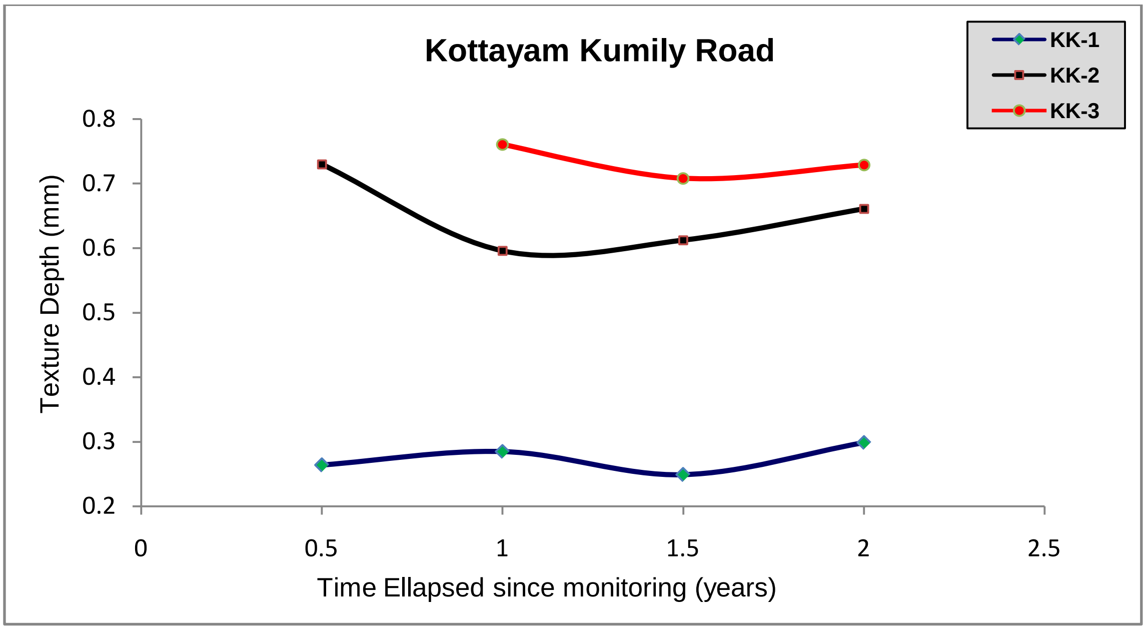

Fig. 6.17: Progression of Texture Depth on Kottayam-Kumily road

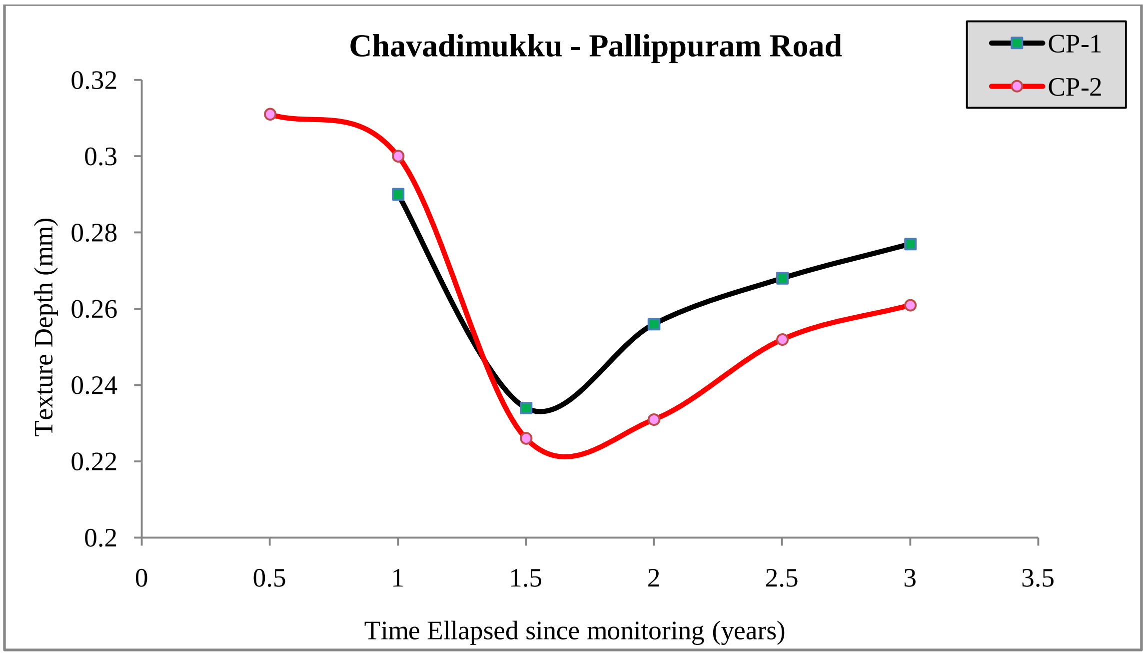

Fig. 6.18: Progression of Texture Depth on Chavadimukku - Pallippuram road

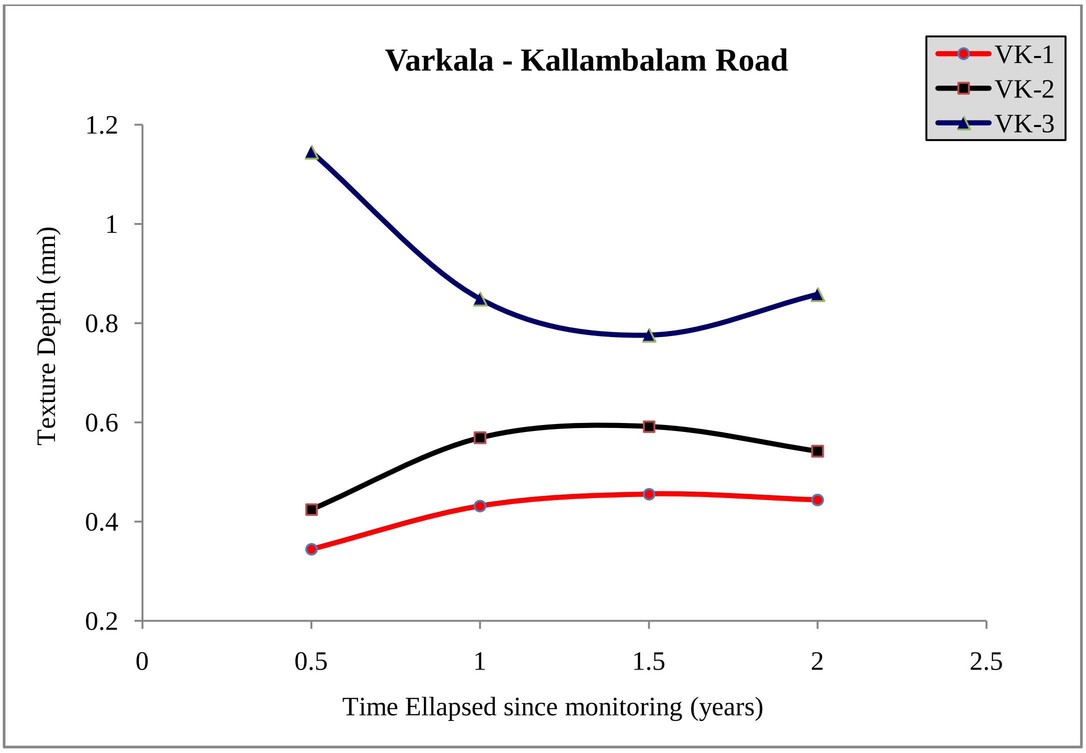

Fig. 6.19: Progression of Texture Depth on Varkala-Kallambalam road

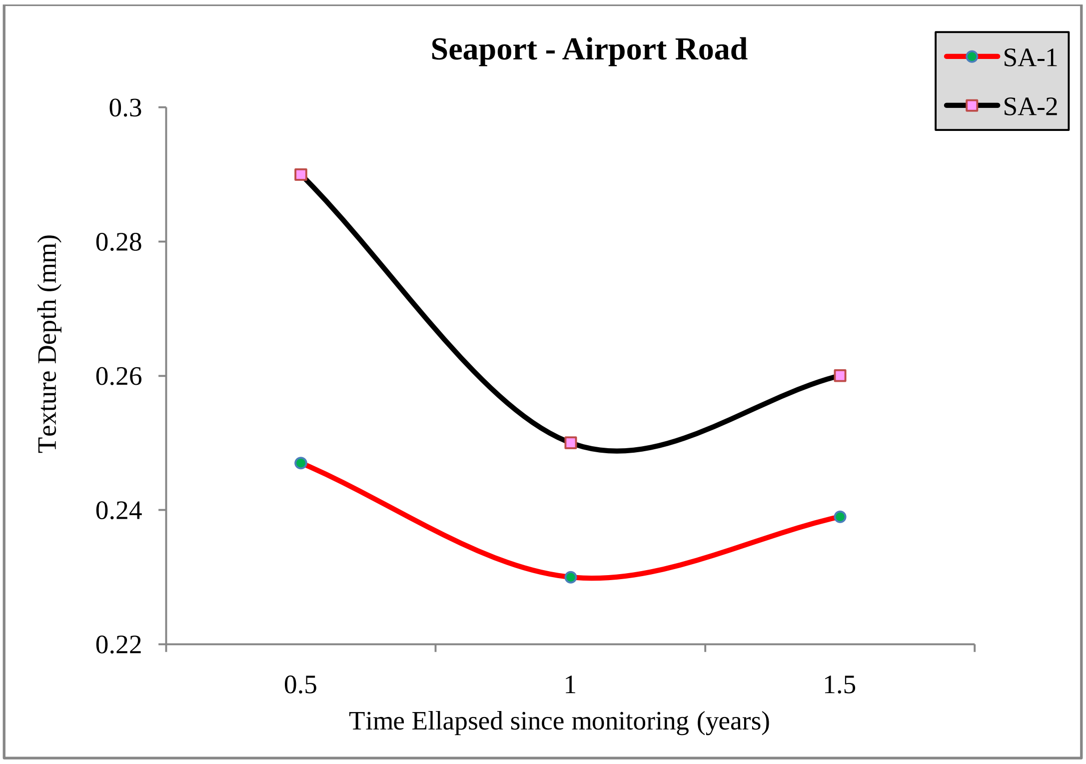

Fig. 6.20: Progression of Texture Depth on Seaport-Airport road

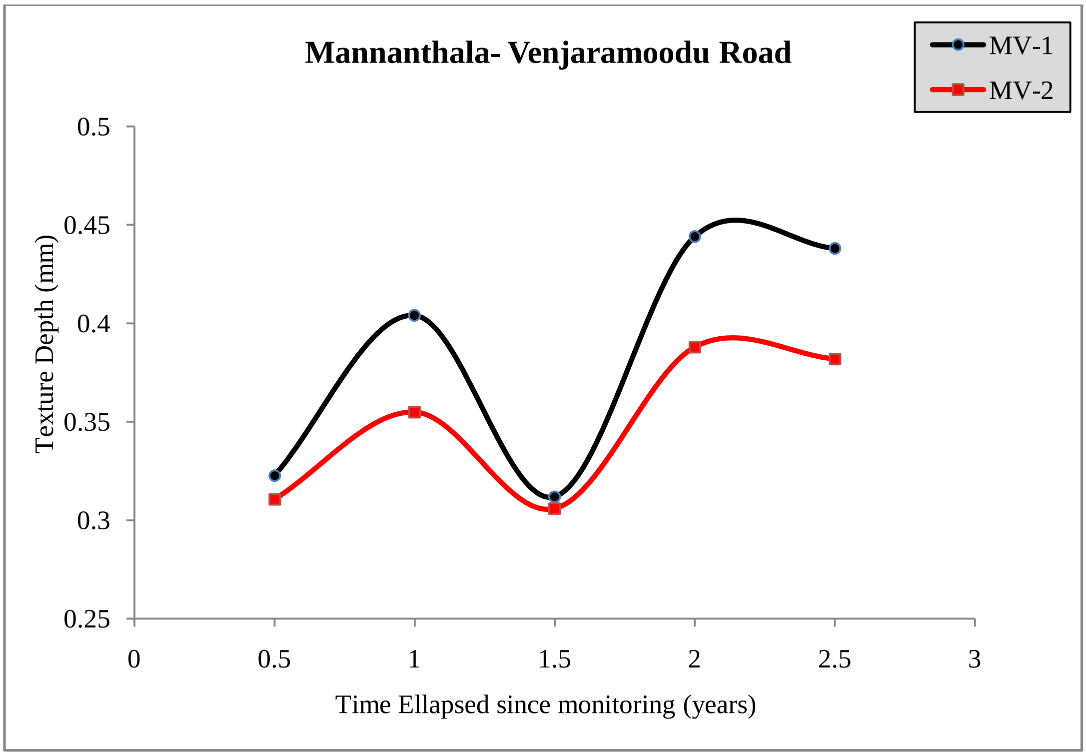

Fig. 6.21: Progression of Texture Depth on Mannanthala-Venjaramoodu road

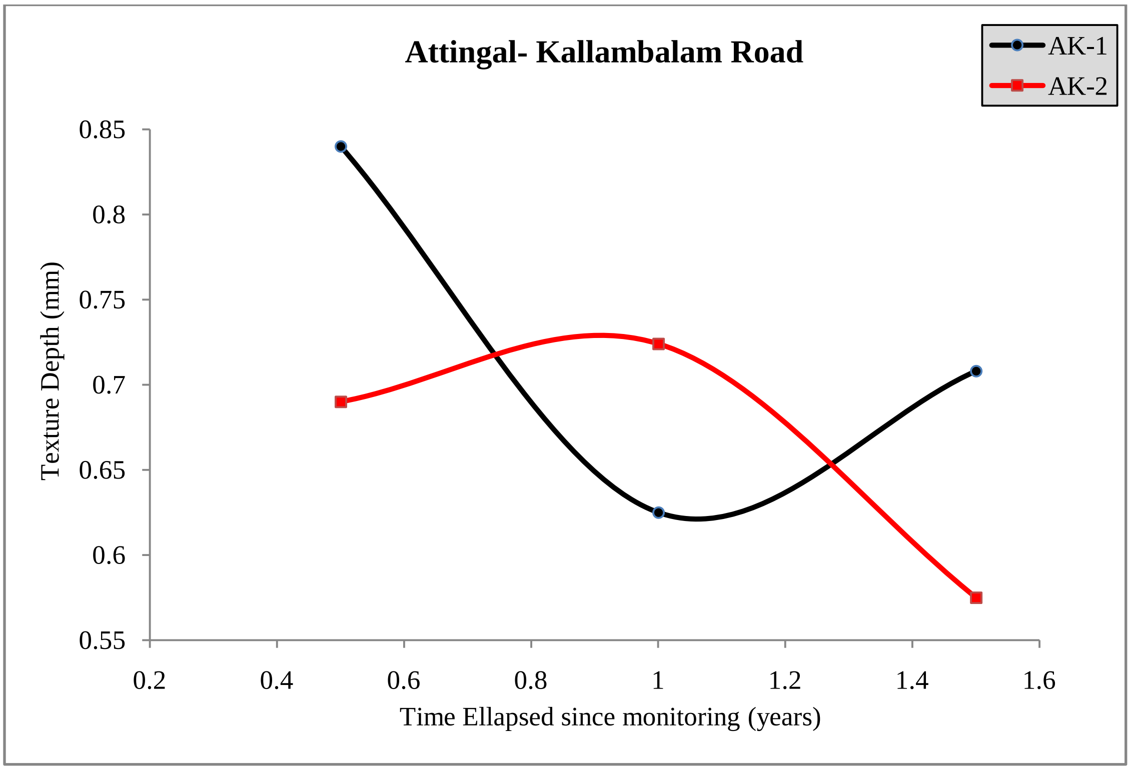

Fig. 6.22: Progression of Texture Depth on Attingal-Kallambalam road

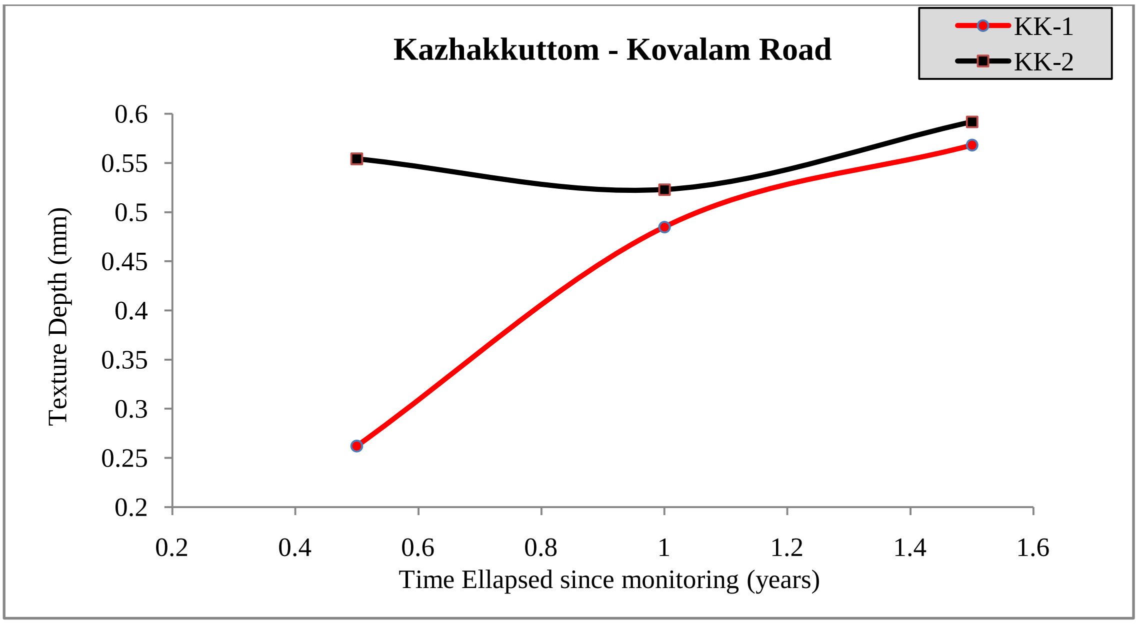

Fig. 6.23: Progression of Texture Depth on Kazhakkuttam-Kovalam road

The maintenance done on section KK-2 after the first measurement was done indicates for the decreased value for the second data.

Texture depth studies were not conducted on Kesavadasapuram Plamood road, since the condition of the road at the time of start of the study was very bad with potholes and cracks and the length of the road was only 2 km.

6.6 CONDITION EVALUATION OF THE PAVEMENTS

From the condition survey charts shown in **Fig. 6.24 to 6.31,** the progression of the major distresses are plotted and given in **Fig. 6.32 to 6.39.** Only representative data are presented here.

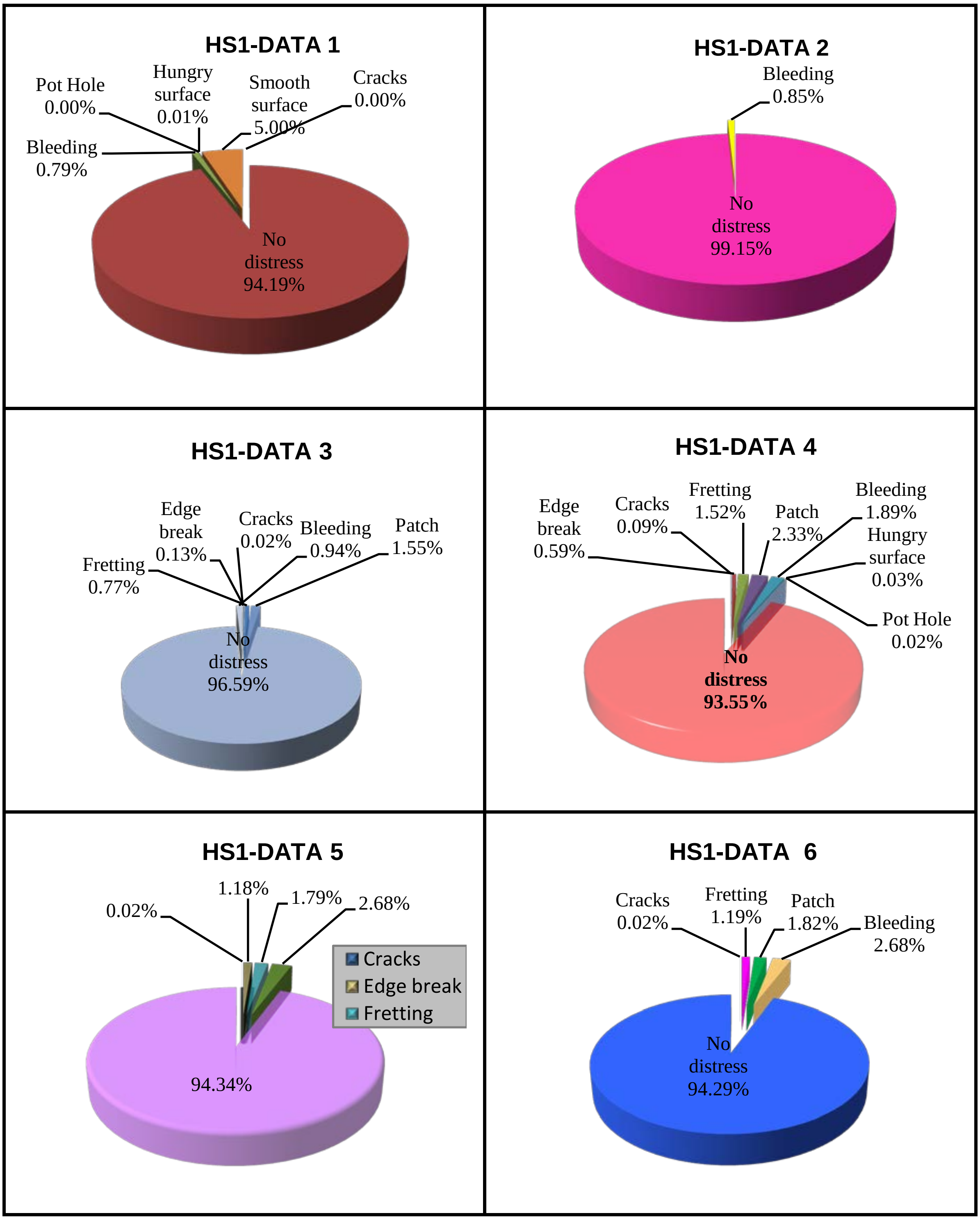

Fig. 6.24: Condition Survey- Chavadimukku-Pallippuram Road HS 1

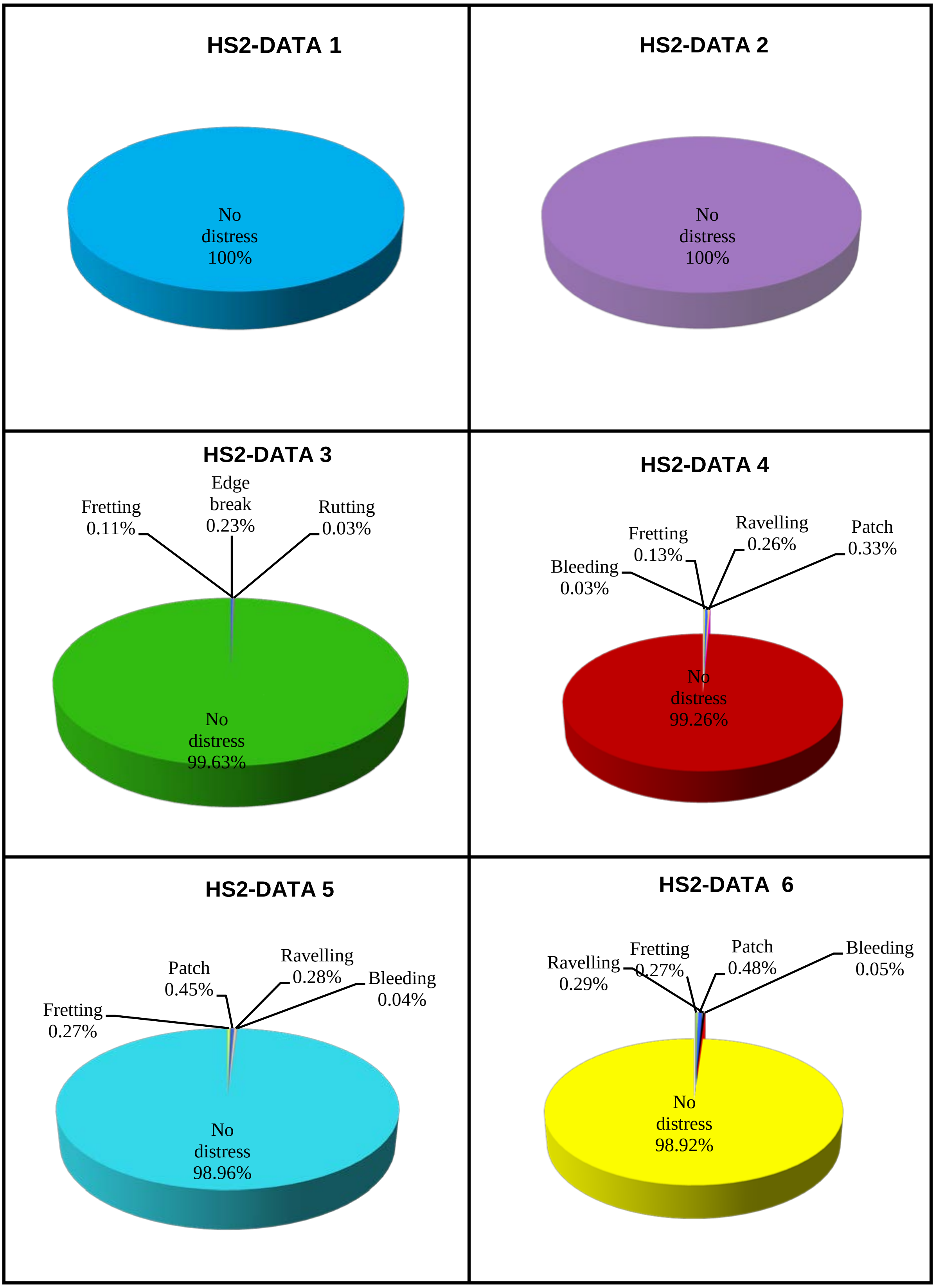

Fig. 6.25: Condition Survey- Chavadimukku-Pallippuram Road HS 2

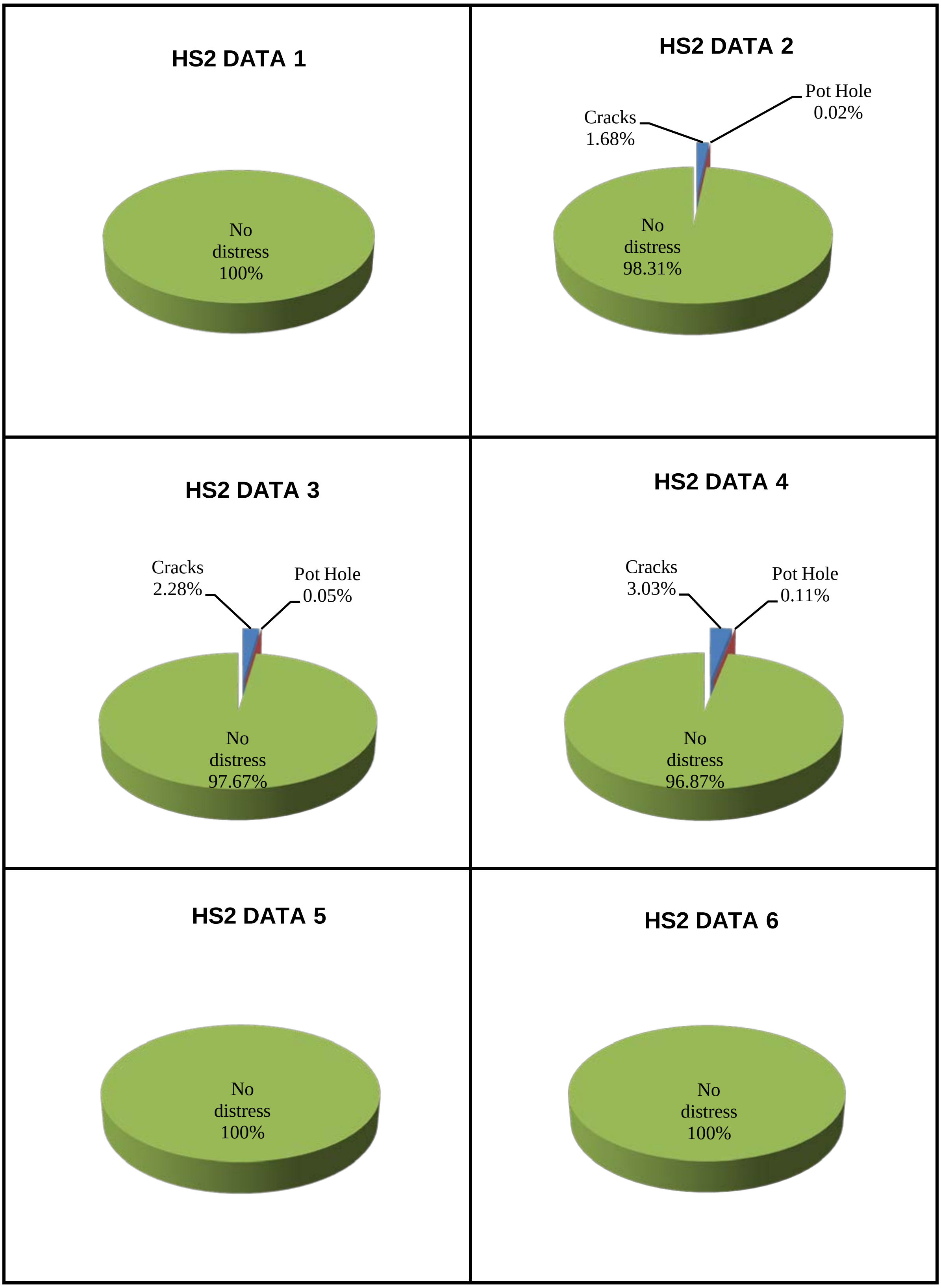

Fig. 6.26: Condition Survey – Seaport Airport Road HS 2

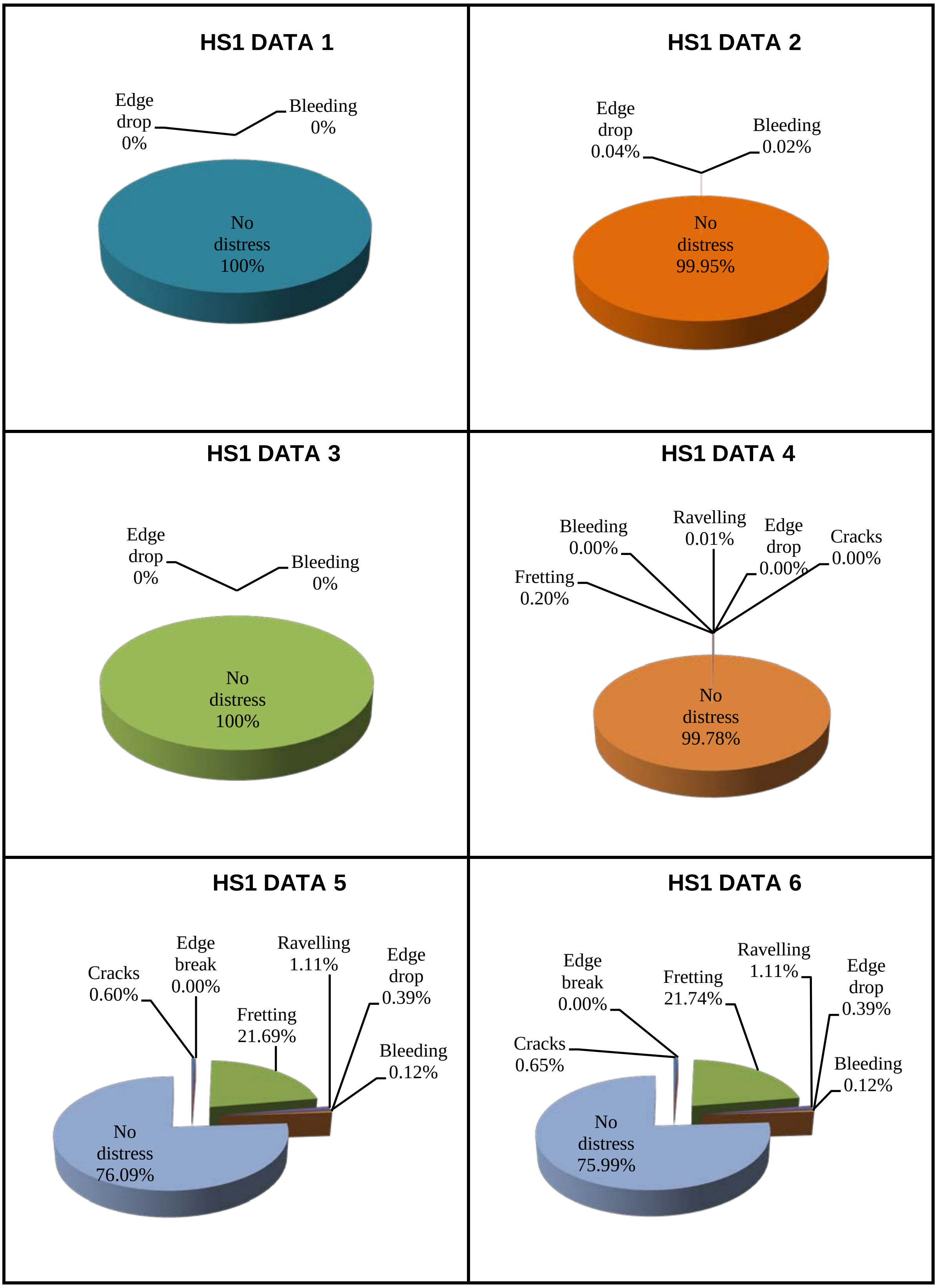

Fig. 6.27: Condition Survey - Varkala Kallambalam Road HS 1

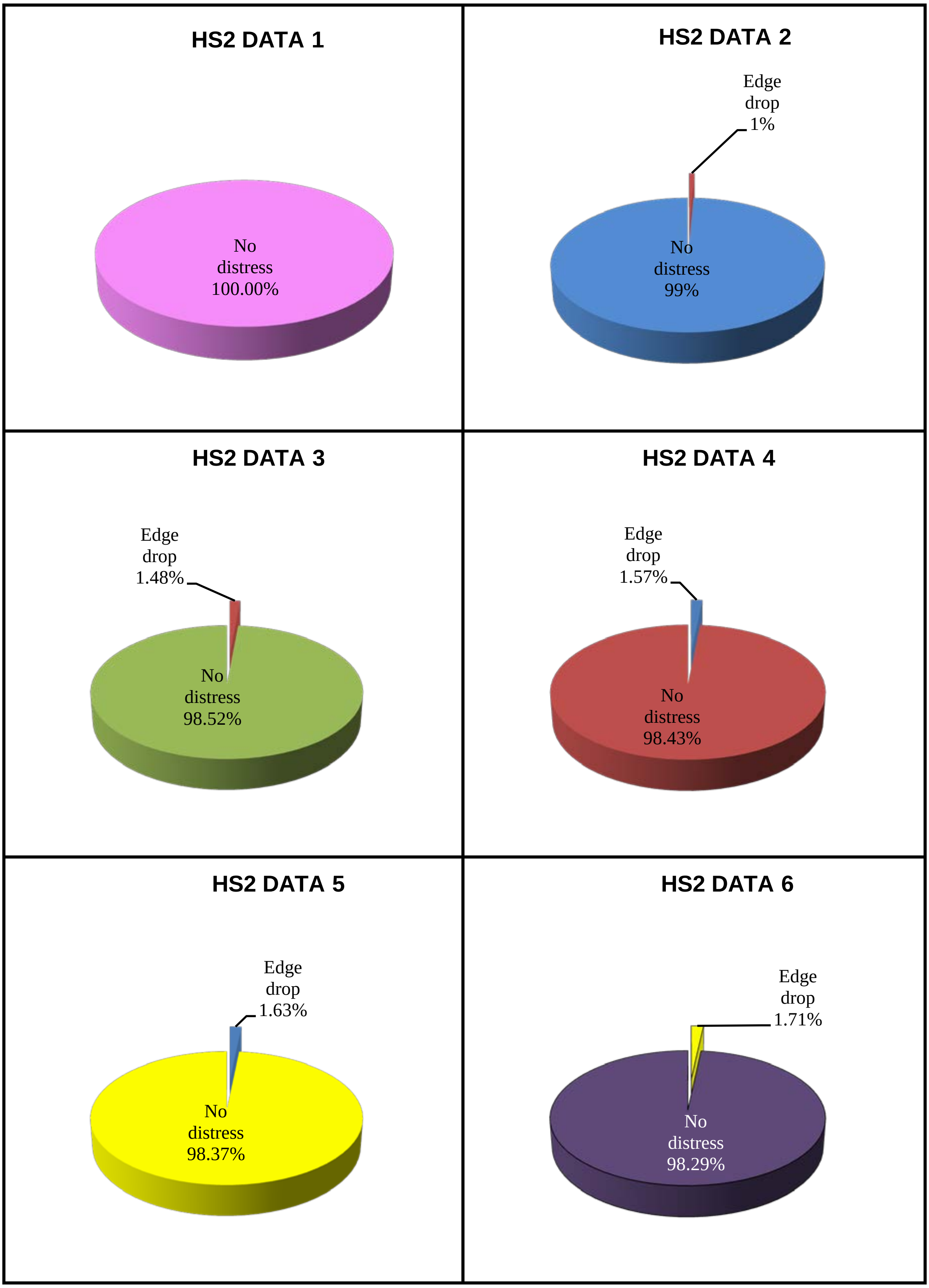

Fig. 6.28: Condition Survey - Varkala Kallambalam Road HS 2

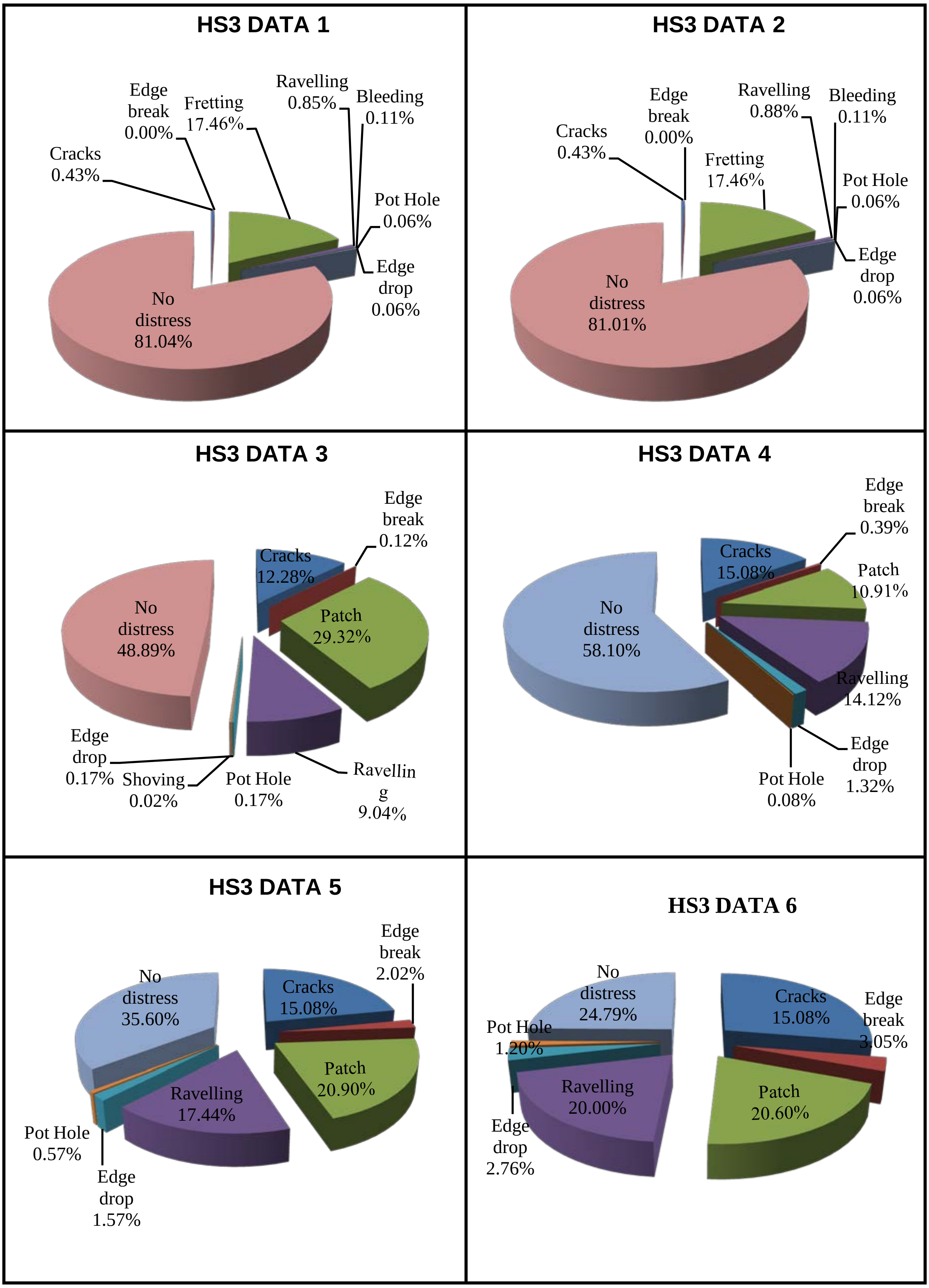

Fig. 6.29: Condition Survey - Varkala Kallambalam Road HS 3

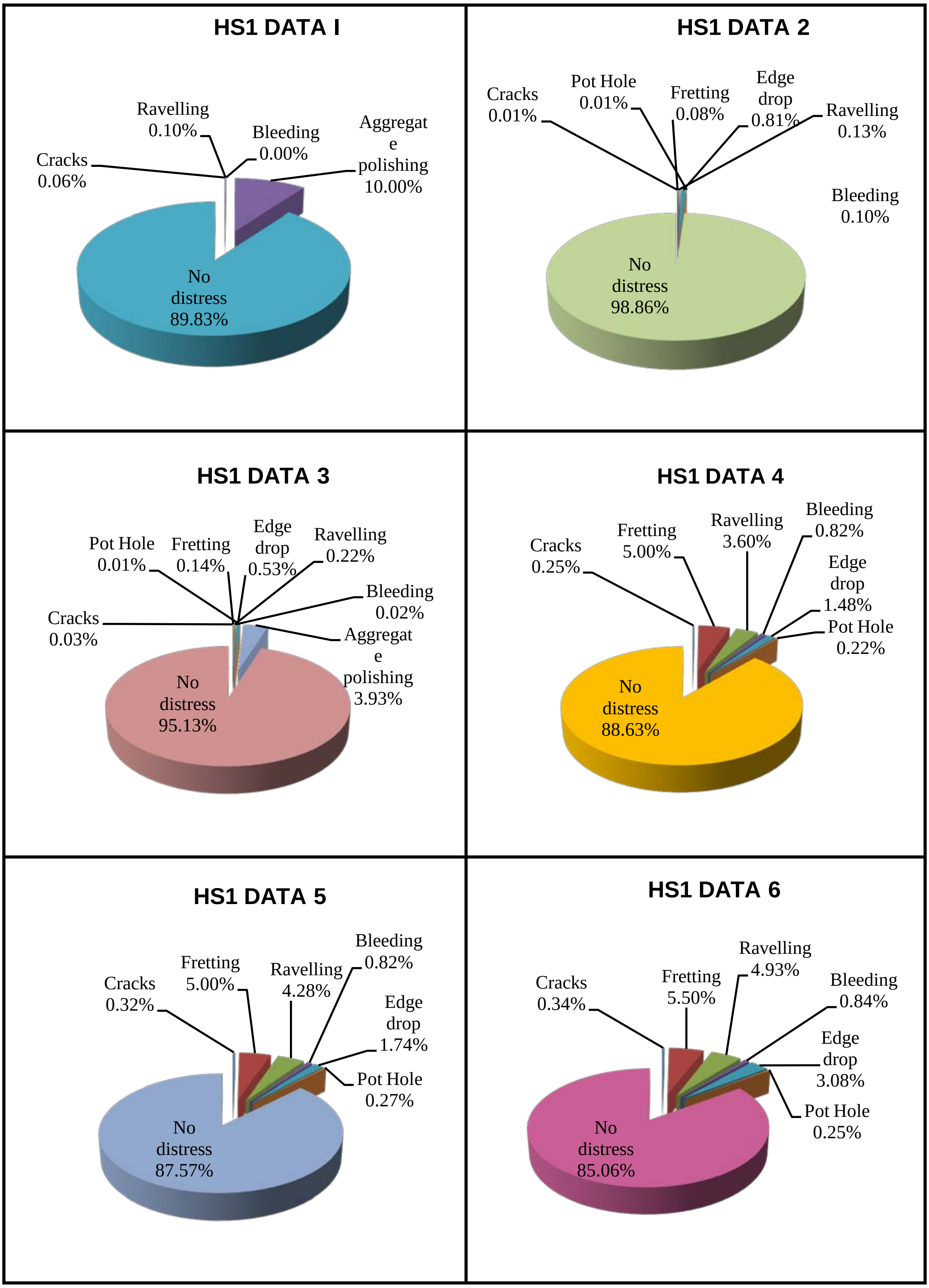

Fig. 6.30: Condition Survey – Mannanthala-Venjaramoodu Road HS I

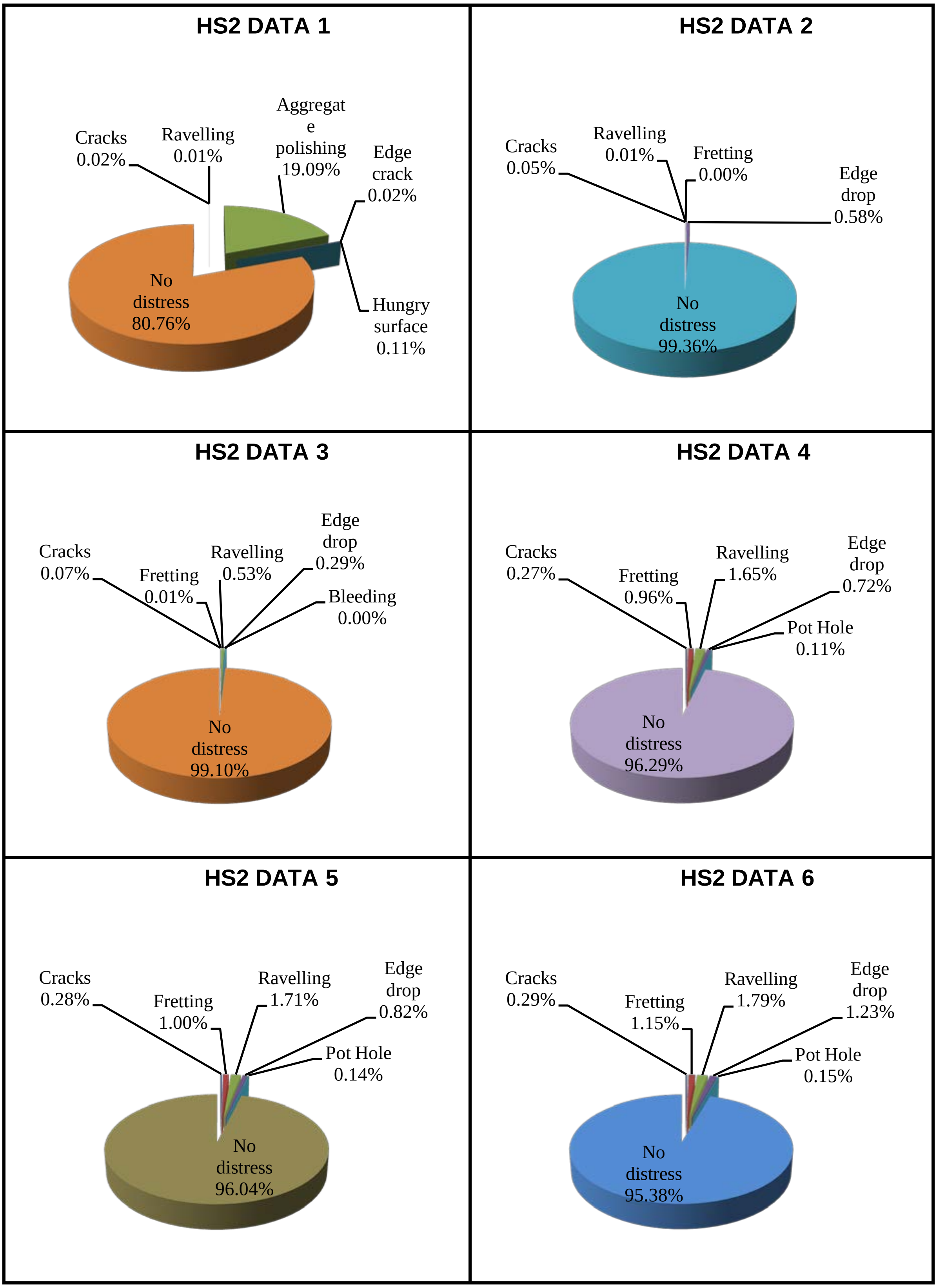

Fig. 6.31: Condition Survey – Mannanthala-Venjaramoodu Road HS 2

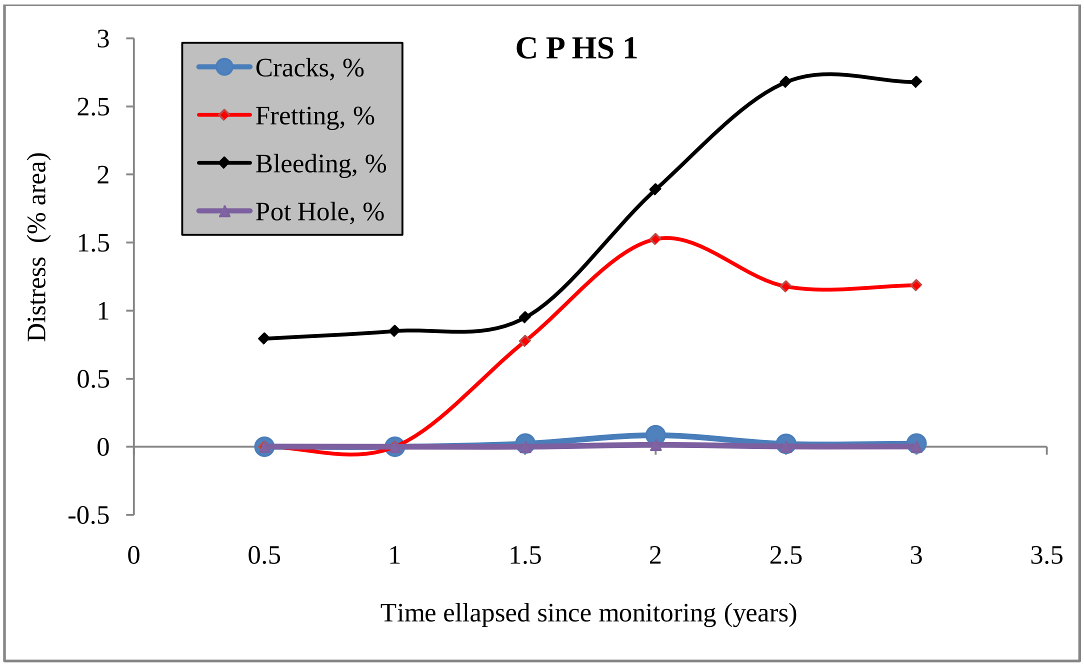

Fig. 6.32: Progression of major distress on Chavadimukku-Pallippuram Road HS I

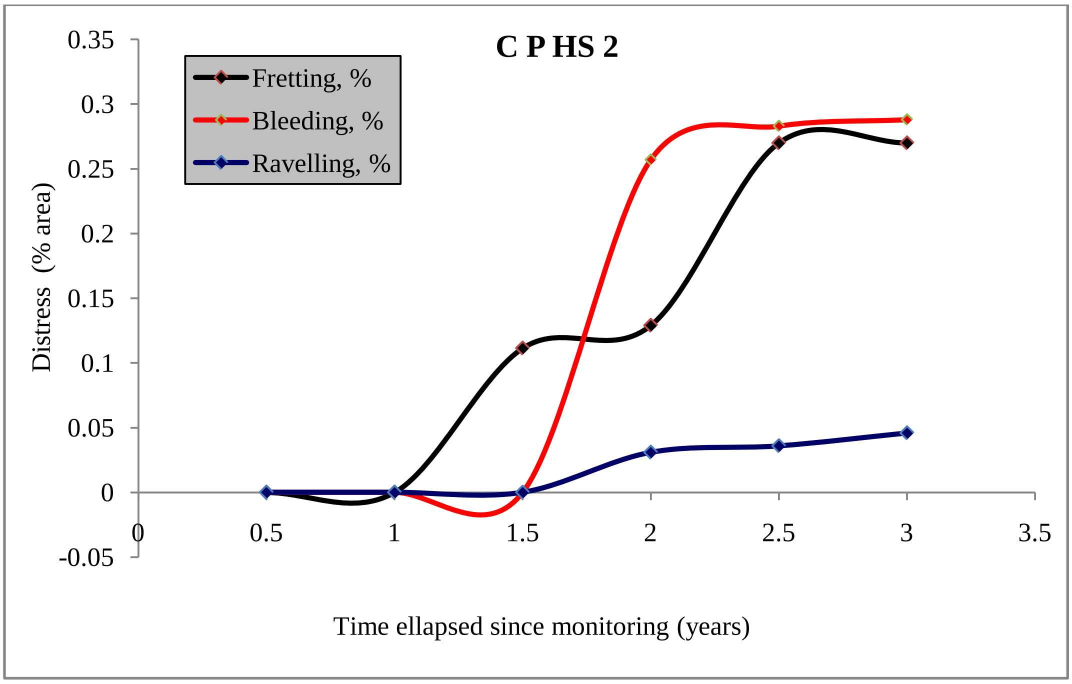

Fig. 6.33: Progression of major distress on Chavadimukku-Pallippuram Road HS 2

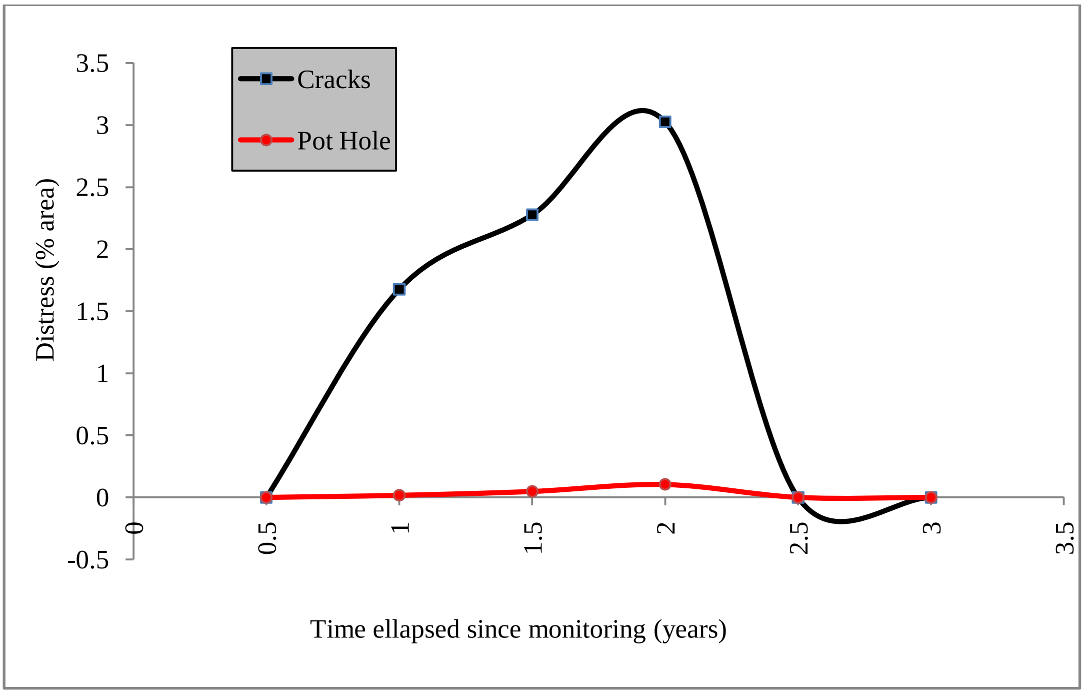

Fig. 6.34: Progression of major distress on Seaport Airport Road HS 2

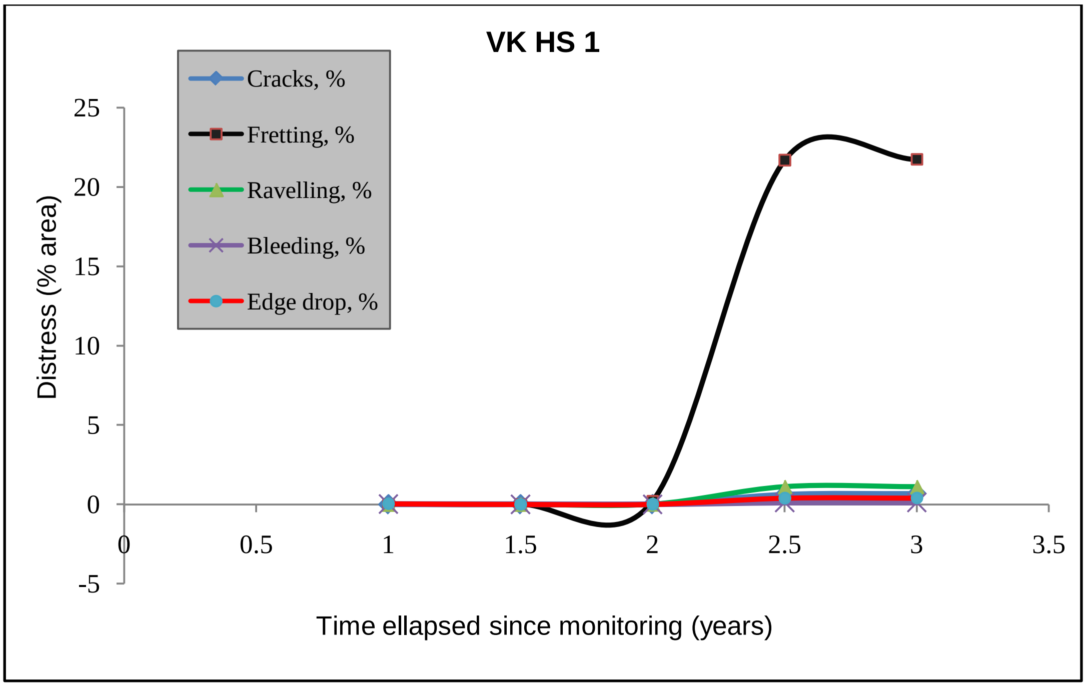

Fig. 6.35: Progression of major distress on Varkala - Kallambalam Road HS 1

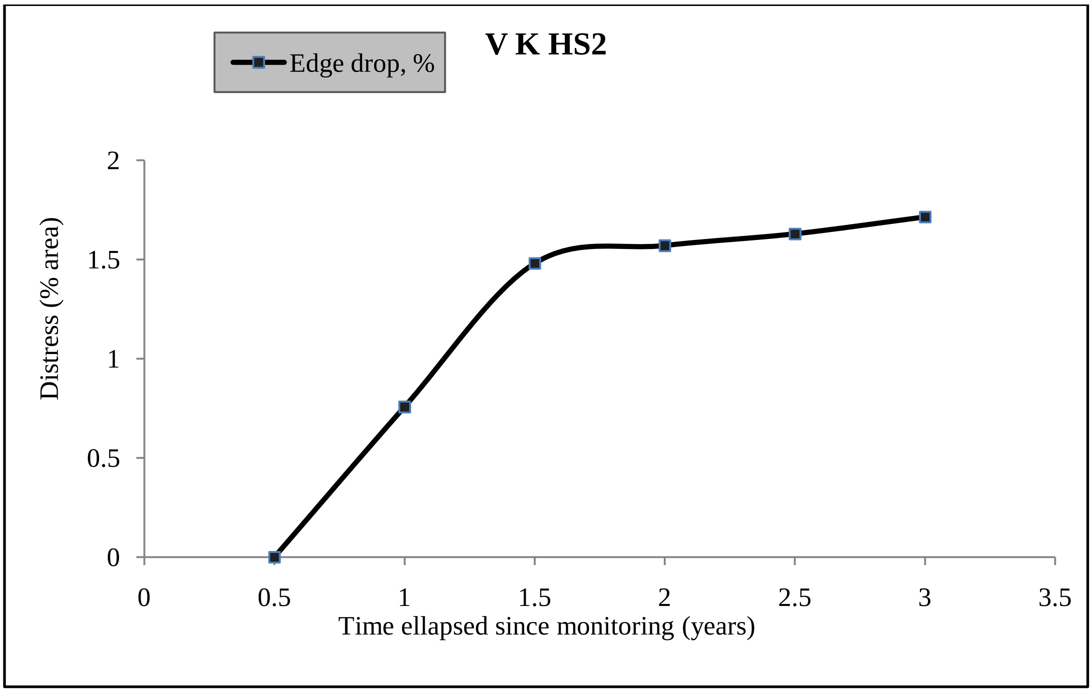

Fig. 6.36: Progression of major distress on Varkala - Kallambalam Road HS 2

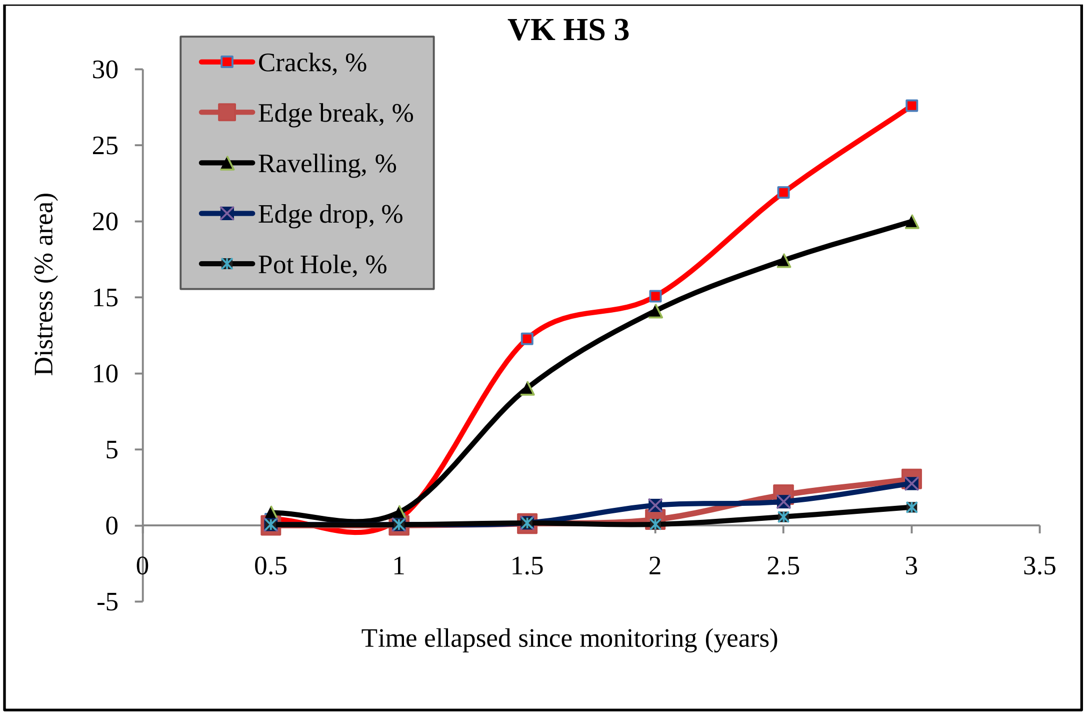

Fig. 6.37: Progression of major distress on Varkala - Kallambalam Road HS 3

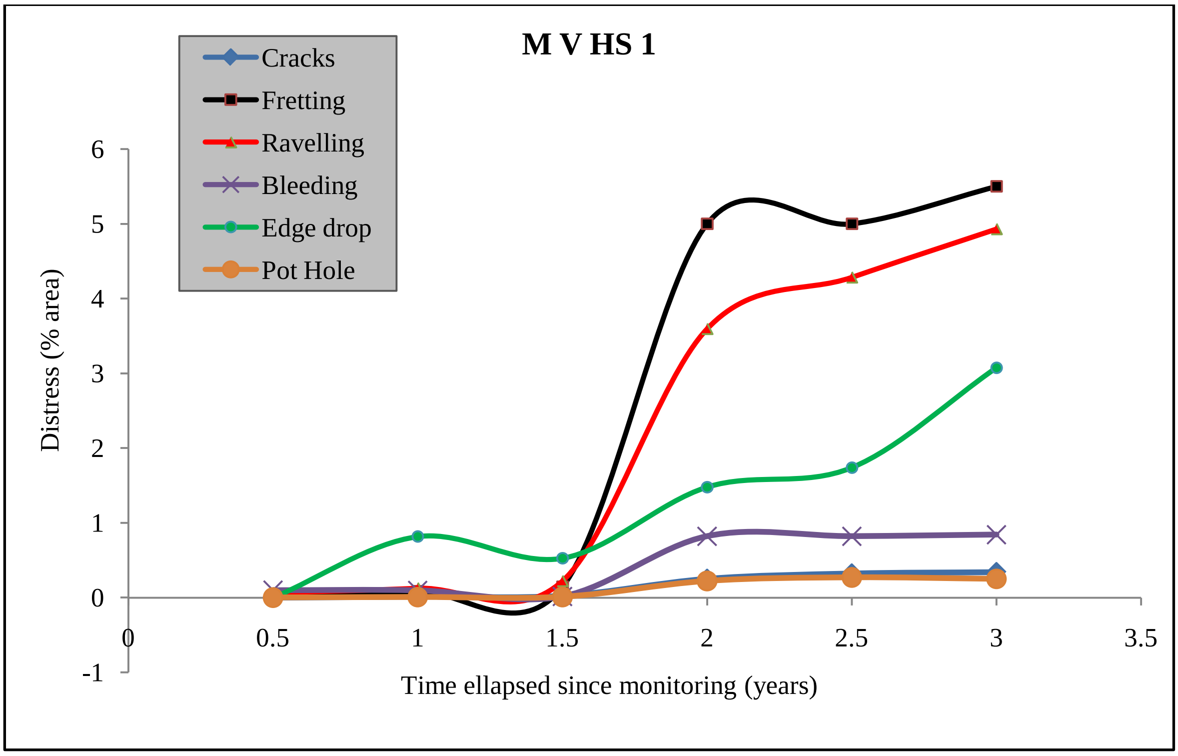

Fig. 6.38: Progression of major distress on Mannanthala-Venjaramoodu Road HS 1

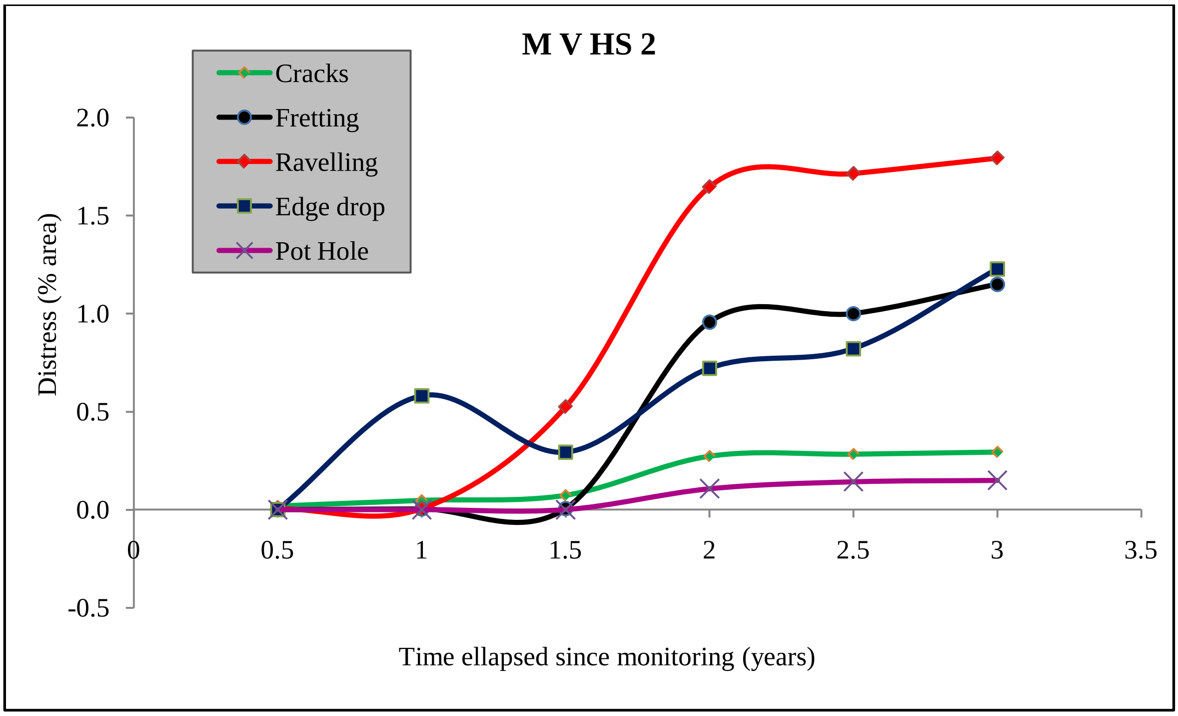

Fig. 6.39: Progression of major distress on Mannanthala-Venjaramoodu Road HS 2

6.7 SKID RESISTANCE STUDIES

The skid resistance of the study stretches were measured using Portable Skid Resistance Pendulum Tester and the results are represented as bar charts in **Fig. 6.40 to Fig. 6.45.** Three sets of data were collected for C P, V K, M V and S A roads. One set of data was collected for A K and K O roads.

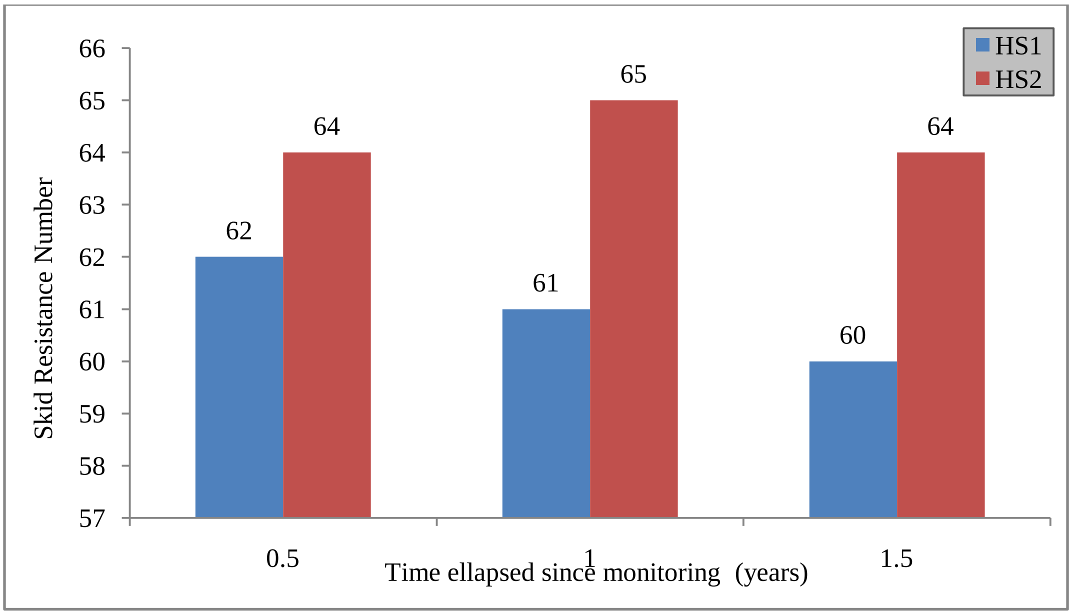

Fig. 6.40: Skid Resistance Number for Chavadimukku - Pallippuram Road

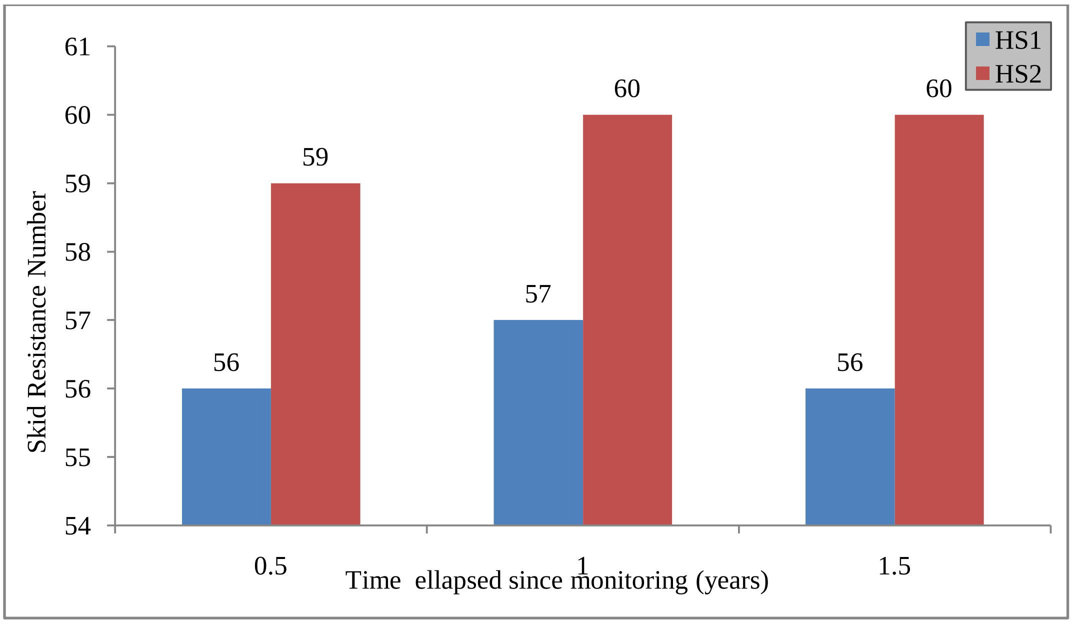

Fig. 6.41: Skid Resistance Number for Varkala – Kallamballam Road

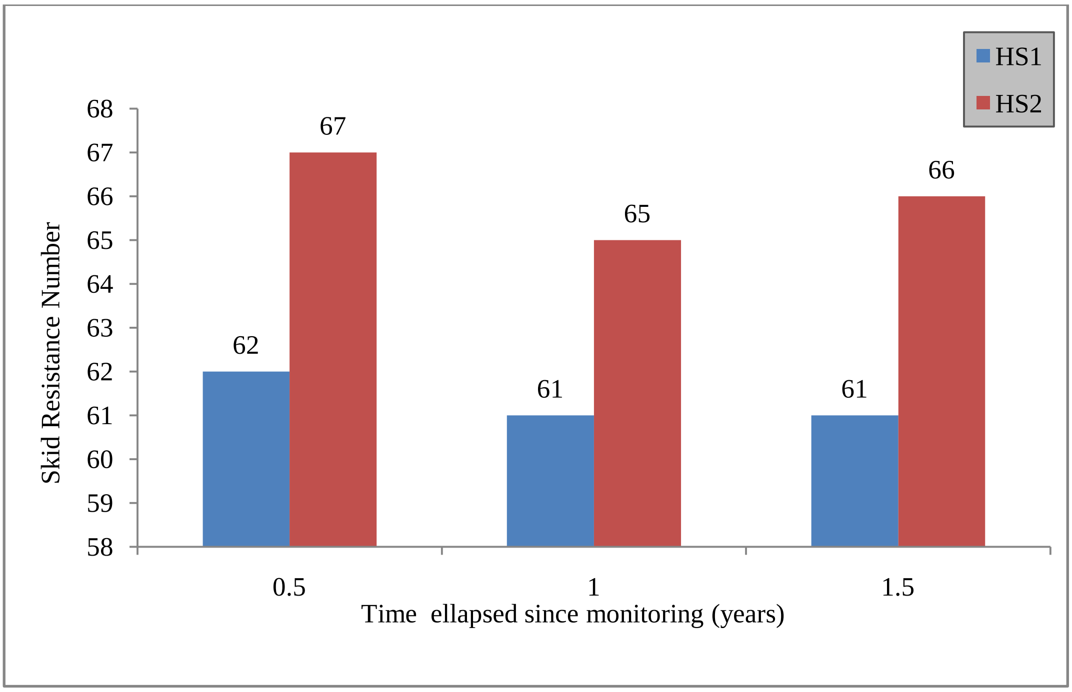

Fig. 6.42: Skid Resistance Number for Seaport-Airport Road

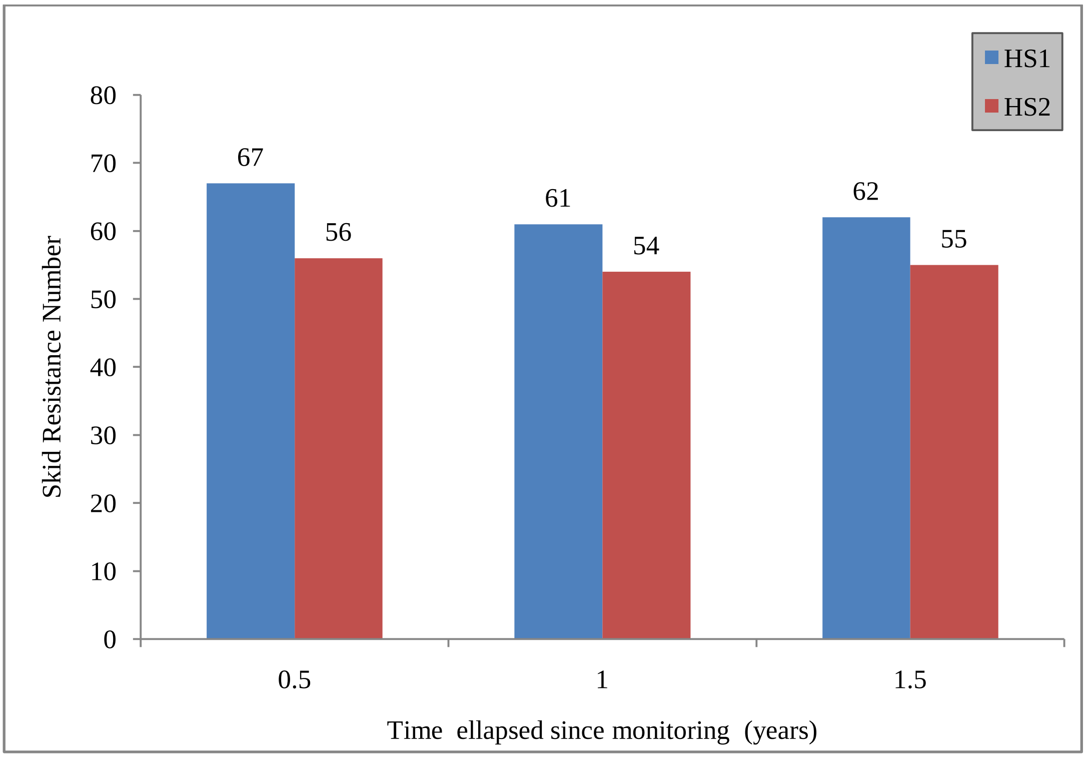

Fig. 6.43: Skid Resistance Number for Mannanthala-Venjaramoodu Road

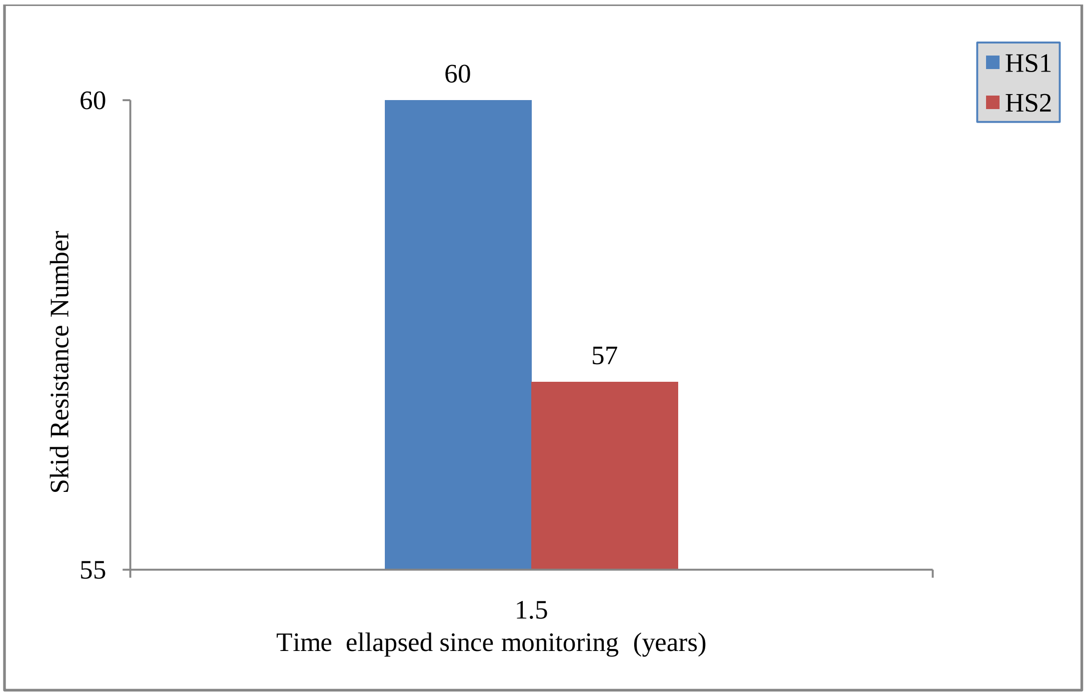

Fig. 6.44: Skid Resistance Number for Attingal - Kallamballam Road

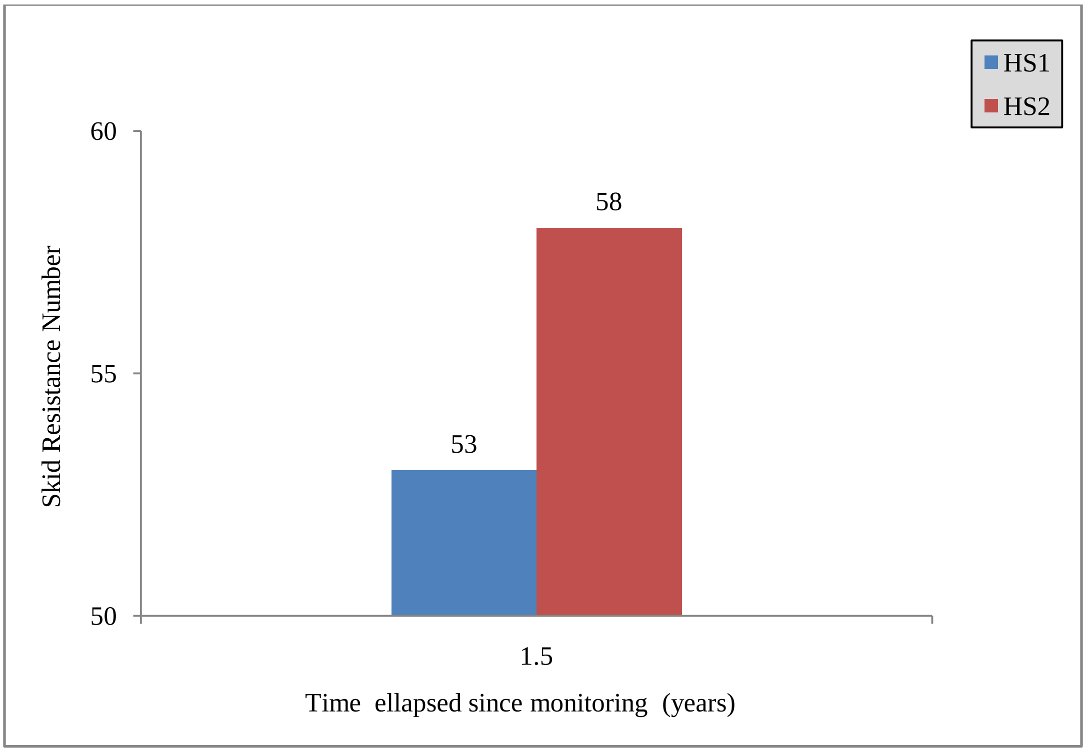

Fig. 6.45: Skid Resistance Number for Kazhakkuttom - Kovalam Road

6.8 DEVELOPMENT OF REGRESSION MODELS

To develop an effective model, extensive amount of data and sound statistical technique is required. Multiple linear regression models were developed as a step towards efficient technique in statistical analysis, but still it lacks in some directions. This does not account for the correlation of different factors associated with the model and biasness of the models towards some particular environmental conditions. These rating based models should also consider the user's perspective also as one of its factors to better judge its functional performance.

In the present study, using the periodic data sets collected for the study stretches through field investigations, linear and non- linear models were tried for Construction Quality, Raveling Progression, Roughness Progression, Pothole Progression, and Alligator Crack Progression. The influencing parameters considered were Construction Quality, Age of the pavement, Vehicle Damage Factor and Modified Structural Number. Construction Quality was derived as dependent on the Relative Compaction of the sub grade and Modified Structural Number. SPSS package was used as the tool and it was found that non- linear models gave best fit. The models generated using the data collected from the field in this research study are given below:

6.8.1 Construction Quality

Construction quality is considered as a function of Relative Compaction and Modified Structural Number. This was measured on a 1 to 5 score rating based on pavement layer thickness, material quality and compaction level achieved in the field. Specimen samples were extracted from the road and layer thickness measured which were further tested in the laboratory. Relative Compaction of the subgrade soil represents the compaction achieved in the soil with respect to the maximum achievable. Modified Structural Number represents the pavement composition and CBR value of the subgrade soil.

$$CQ = (RC)^{0.568} + (MSN)^{0.451} - 10.722 \qquad \text{------------------} \quad (6.1)$$

$R^2 = 0.76$

8.2 Raveling Progression Model

Raveling is the term used to represent the progressive loss of surface material of the pavement through traffic abrasion and/or weathering. The occurrence of raveling varies with construction methods & quality, specifications, properties of available materials, and local practice. Raveling is common in poorly constructed, thin bituminous layers such as surface treatment, but it is rarely seen in high quality hot-mix asphalt constructions. Raveling is influenced by the traffic characteristics, construction quality and age of the pavement.

$$RVP = (RV_i * AGE)^{0.339} + (CQ)^{-0.808} + (VDF)^{0.487} - 0.589 \quad \text{------------------} \quad (6.2)$$

$R^2 = 0.60$

6.8.3 Roughness Progression Model

The roughness is influenced by several components like cracking, disintegration, deformation and maintenance. The total incremental roughness is the sum of these components. In the model, Construction quality, MSN, Raveling and Potholes are considered.

$$RGP = (CQ * RV_t)^{0.547} - 2.342\,(MSN)^{-0.952} + 0.0672\,PH_i \quad \text{------------------} \quad (6.3)$$

$R^2 = 0.70$

6.8.4 Pothole Progression Model

Pothole progression arises from the formation of potholes due to cracking, raveling and the enlargement of existing potholes. The progression of potholes is affected by the time lapse between the occurrence and patching of potholes.

$$PHP = (CQ)^{-26.86} - 0.047 * MSN * AGE + (RVP)^{0.465} + (PH_i)^{0.081} \quad \text{-------------} \quad (6.4)$$

$R^2 = 0.92$

6.8.5 Alligator Crack Progression Model

Cracking is a major distress that develops in bituminous pavements. The principal factors which contribute to cracking of a bituminous pavement layer have been identified as fatigue and ageing. The propagation of cracking is accelerated through the increase of brittleness due to ageing, ingress of water, which can significantly weaken the underlying pavement layers and traffic loads.

$$ACP = (VDF)^{0.155} - 1.735 * (MSN * AGE)^{-0.238} \quad R^2 = 0.93 \text{ ------------ } (6.5)$$

6.8.6 Deflection Progression

Pavement deforms elastically under the wheel load application. It regains to the original state when the load is released. This deflection is called elastic deflection or rebound deflection. In pavement evaluation, deflection measured is rebound deflection. The amount of deflection measured under a wheel load is a measure of structural stability of the pavement system. Higher deflection values indicate weaker pavement structure, which may require higher overlay thickness or early strengthening.

Even though deflection is not a measure of pavement deterioration, it influences the rate of pavement deterioration. Performance and life of the flexible pavements are closely related to rebound deflection under the wheel loads. Hence, it is important to predict the deflection value. The magnitude of deflection or the elastic rebound of a flexible pavement due to a wheel load depends on the structural stability of the pavement system and also on the magnitude of the load. The rate of change of deflection depends on the initial deflection, strength of the pavement, vehicle loading etc.

$$\mathrm{DEF} = 0.358 \times \mathrm{DEF_i} + 0.009 \times \mathrm{e^{VDF}} - 0.002 \times \mathrm{e^{MSN}} + 0.653 \quad \text{------------- } (6.6)$$

$R^2 = 0.88$

Notations

DEF = Deflection

CQ = Construction Quality

RC = Relative Compaction

MSN = Modified Structural Number

RVP = Rate of Raveling Progression

RGP = Roughness Progression Rate

RV_i = Initial Raveling

VDF = Vehicle Damage Factor

AGE = Pavement Age since renewal

PH_i = Initial Pothole Area

PHP = Rate of Pothole Progression

ACP = Rate of Alligator Cracks Progression

6.8.7 Calibration and validation of the progression models

The parameters of regression models estimated and calibrated were found to be statistically significant by various statistical parameters like R^2, t-test, F-test. The internal validation method was adopted for validating the developed regression models. The models developed were used to predict corresponding values and these estimated values are compared with observed values. A paired t- test for means was used to evaluate if there exists any significant difference between observed and estimated values. The calculated t-values in all cases were found to be lower than the critical t values obtained from statistical tables at the 5 % level of significance and are shown in **Table 6.1.** Hence, it can be concluded that there exists no significant difference between observed and estimated values.

Table 6.1 Student's t-test for pavement performance models

Models	Relationship	Calculated absolute t- test value	Tabular t- test value
Construction Quality	$CQ = (RC)^{0.568} + (MSN)^{0.451} - 10.722$	0.039	2.262
Ravelling Progression	$RVP = (RV_i * AGE)^{0.339} + (CQ)^{-0.808} + (VDF)^{0.487} - 0.589$,	2.952	3.182
Roughness Progression	$RGP = (CQ * RV_t)^{-0.547} - 2.342 (MSN)^{-0.952} + 0.0672 PH_{i,}$	0.949	2.776
Pothole Progression	$PHP = (CQ)^{-26.86} - 0.047 * MSN* AGE +(RVP)^{0.465} +(PH_i)^{0.081}$,	0.344	2.776
Alligator Crack Progression	$ACP = (VDF)^{0.155} - 1.735 * (MSN * AGE)^{-0.238}$	0.051	2.776
Deflection Progression	$Def = 0.358 x DEF_i + 0.009 x e^{VDF} - 0.002 x e^{MSN} + 0.653$	0.452	2.776

Plot showing a comparison between the observed values (x-axis) and the predicted values (y-axis) on a linear scale of each model are shown in **Fig. 6.46 to 6.51.**

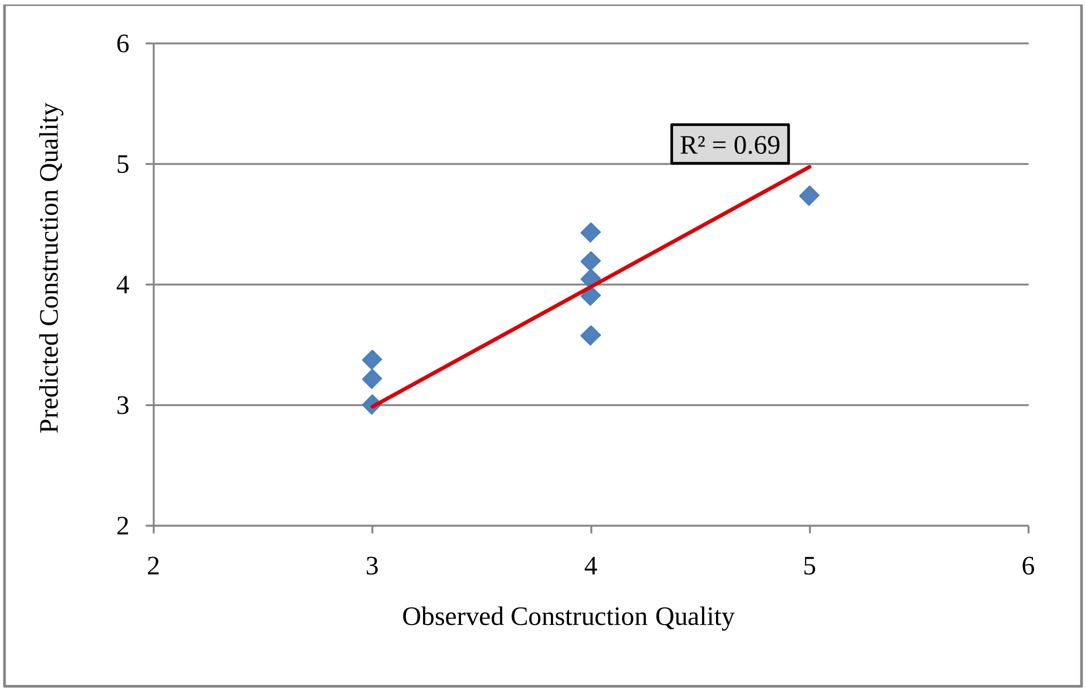

Fig. 6.46: Comparison of observed vs predicted Construction Quality

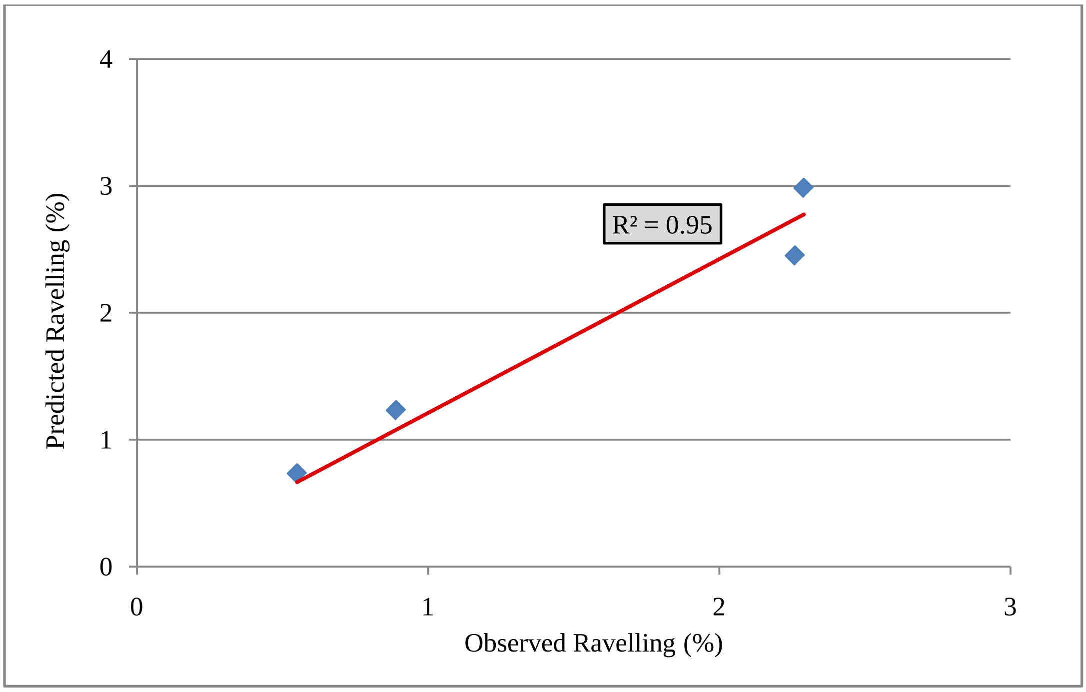

Fig. 6.47: Comparison of observed vs predicted Raveling

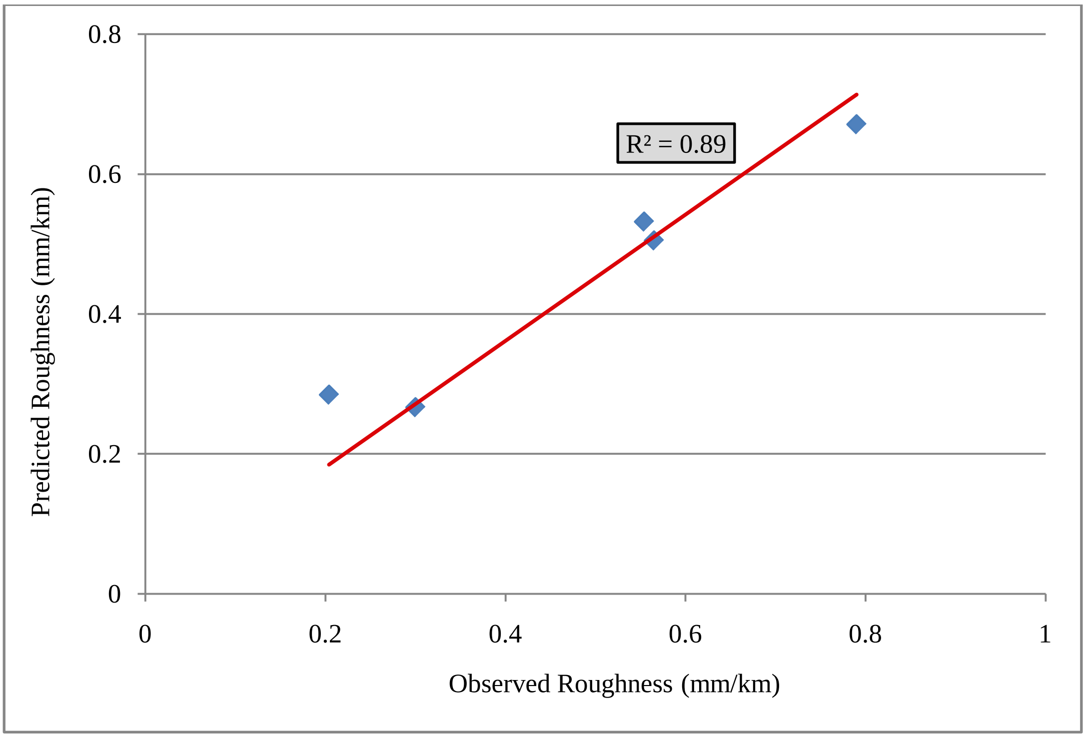

Fig. 6.48: Comparison of observed vs predicted Roughness

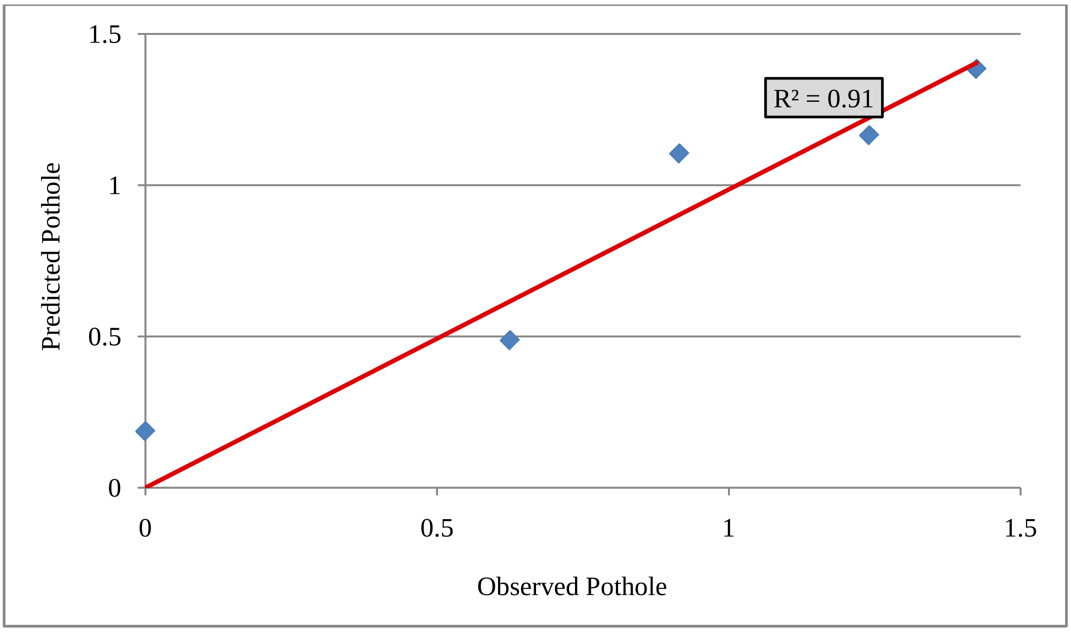

Fig. 6.49: Comparison of observed vs predicted pothole

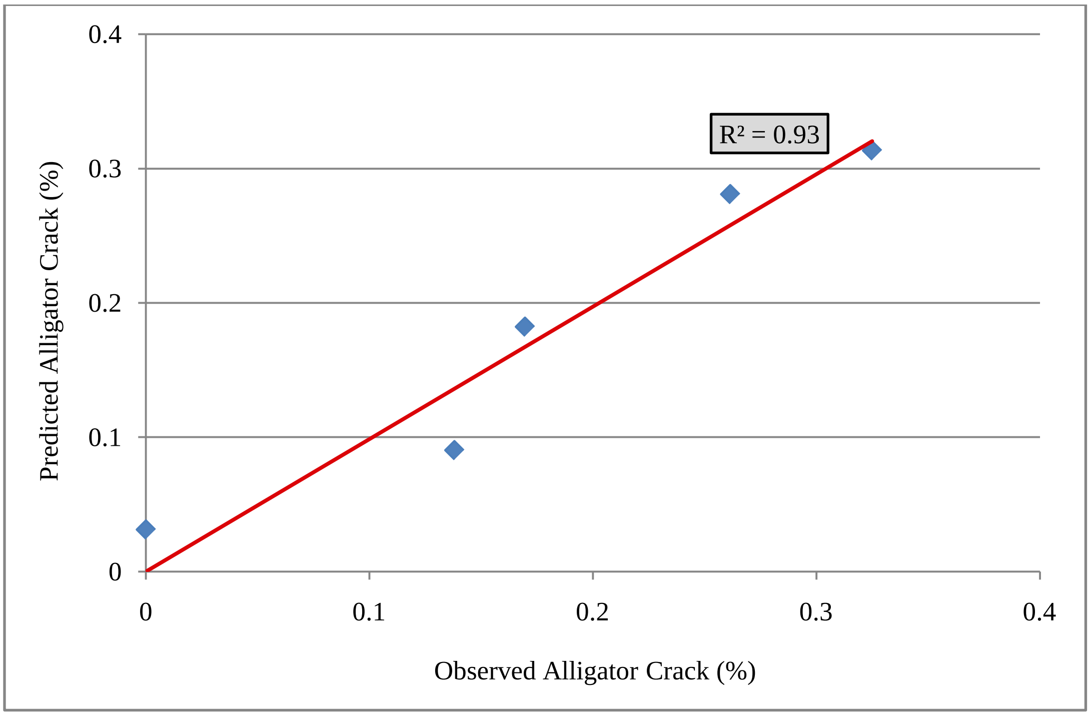

Fig. 6.50: Comparison of observed vs predicted Alligator Crack

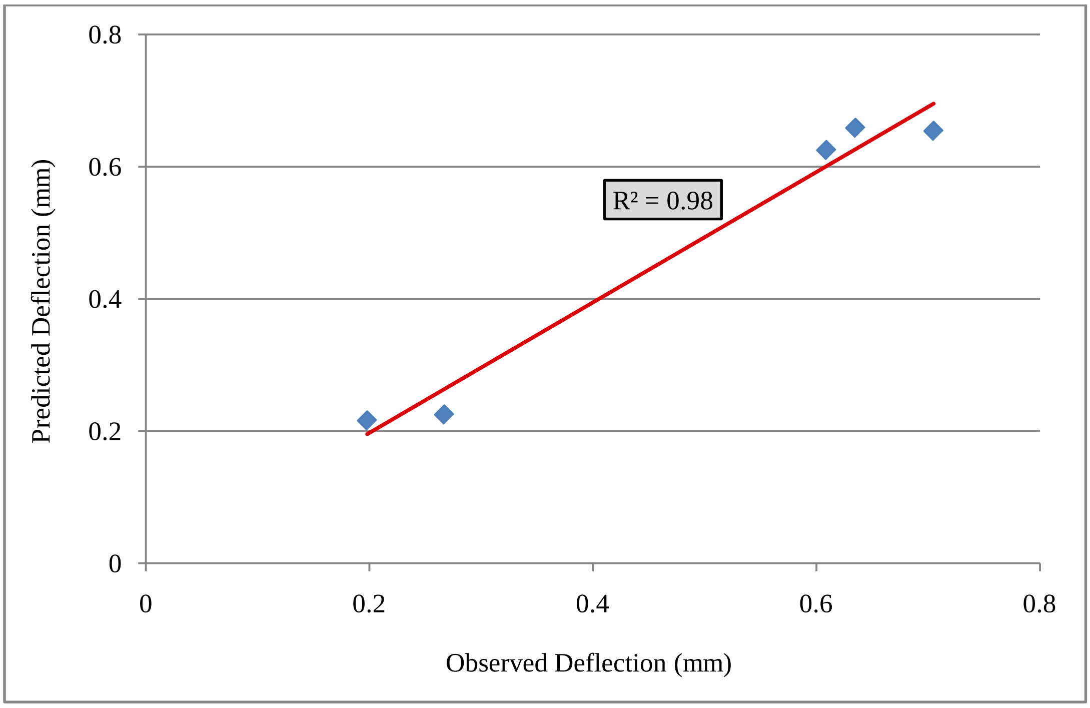

Fig. 6.51: Comparison of observed vs predicted Deflection

The models developed as part of the regression analysis were used in validating the relationship between the variables. The average prediction error (APE) and total prediction error (TPE) for the models were calculated and are tabulated in **Table 6.2.**

Table 6.2 Validation of the Regression Analysis

Models	Relationship	Average Prediction Error (%)	Prediction Error of Totals (%)
Construction Quality	$CQ = (RC)^{0.568} + (MSN)^{0.451} - 10.722$	0.717	0.099
Ravelling Progression	$RVP = (RV_i \text{ X } AGE)^{0.339} + (CQ)^{-0.808} + (VDF)^{0.487} - 0.589$	1.081	1.973
Roughness Progression	$RGP = (CQ \text{ X } RV_t)^{-0.547} - 2.342\ (MSN)^{-0.952} + 0.0672\ PH_i$	-0.278	-6.386
Pothole Progression	$PHP = (CQ)^{-26.86} - 0.047 \text{ X } MSN \text{ X } AGE + (RVP)^{0.465} + (PH_i)^{0.081}$	-2.659	2.802
Alligator Crack Progression	$ACP = (VDF)^{0.155} - 1.735 \text{ X } (MSN \text{ X } AGE)^{-0.238}$	-5.863	0.400
Deflection Progression	$Def = 0.358 \text{ x } DEF_i + 0.009 \text{ x } e^{VDF} - 0.002 \text{ x } e^{MSN} + 0.653$	-1.575	-1.518

The average prediction error (APE) for the progression models was found to vary between -5.9 % and 1.08 % and the total prediction error (TPE) for the models were found to vary between -6.4 % and 2.8 %. It may be observed in Table 6.2 that the 'Average Prediction Errors' and 'Prediction errors of Totals' in the estimation of variables were much lesser and reasonably accurate. Hence, the regression equations developed were assumed to be acceptable for future predictions.

6.9 APPLICATION OF HDM-4

The road deterioration framework developed for HDM-4 provides a single set of generic models whose coefficient values can be changed based on the surface layer and base type. (Odoki J B and Kerali H.G.R, 2000). HDM-4 provides PPMs for pavement distresses for rutting, potholes, cracking and roughness. The initiation phase is separated from the progression phase. The equations provided by HDM-4 are explained the sections below.

6.9.1 Crack Modelling

a. Initiation of all structural cracking

Crack initiation is considered to happen when 0.5% of the surface area of the carriageway is cracked. Initiation of all structural cracking depends on the construction, traffic, etc.

$$\mathrm{ICA} = \mathrm{K_{cia}}\ \left\{\mathrm{CDS}^2\ a_0 \exp\left[a_1\ \mathrm{SNP} + a_2\left(\frac{\mathrm{YE4}}{\mathrm{SNP}^2}\right)\right] + \mathrm{CRT}\right\}$$

------------- **(**6.7)

where,

ICA - Time of initiation of all structural cracks in years

CDS - Indicator of construction defects for bituminous surfacing

YE4 - Number of equivalent standard axles in millions/lane/year

SNP - Adjusted structural number of the pavement

CRT - Crack retardation time due to maintenance in years

b. Progression of all structural Cracking

The progression of all structural cracking is represented as given below:

$$\mathrm{dACA} = \mathrm{K_{cpa}}\ [\mathrm{CRP/CDS}]\ \mathrm{Z_A}\left[\frac{(\mathrm{Z_A}\, a_0 a_1 \delta\, \mathrm{T_A} + \mathrm{SCA}^{a1})}{a_1} - \mathrm{SCA}\right]$$

------------- **(**6.8)

The progression of all structural cracking commences when $\delta\ t_A > 0$ or $ACA_a > 0$

If $ACA_a > 0$, $\delta\ t_A = 1$

If $ACA_a > 50$ then: $Z_A = -1$ otherwise: $Z_A = 1$

ACA_a = MAX (ACA_a, 0.5)

SCA = MIN (ACAa, 100 – ACA_a)

dACA = incremental change in the area of all structural cracking during the analysis year in % of total carriageway area

ACA_a = Area of all structural cracking at the start of the analysis year

$\delta\ t_A$ = Fraction of analysis year in which all structural cracking progression applies

K_{cpa} = Calibration factor for progression of all structural cracking

c. Retardation of cracking progression

$$CRP = 1 - 0.12CRT \qquad \text{------------- (6.9)}$$

where,

CRP - Retardation of progression of cracking due to preventative maintenance

6.9.2 Ravelling

In the ravelling models, CDS is used as a variable for the construction defects indicator for bituminous surfacing,

a. Initiation of Raveling

Raveling is considered to happen on a given road section when 0.5% of the carriageway surface area is raveled. The initiation is given by the equation:

$$IRV = K_{vi}\, CDS^2\, a_0\, RRF \exp(a_1\, YAX) \qquad \text{------------- (6.10)}$$

where, IRV = Time to raveling initiation in years

CDS = Indicator of Construction defects for bituminous surfacing

YAX = Number of axles of all motorized vehicle types in the analysis year denoted as millions/lane/year

K_{vi} = Calibration factor for initiation of raveling

RRF = retardation factor for Raveling due to maintenance

b. Progression of Raveling

$$dARV = [K_{vp} / RRF][1/CDS^2]\, Z\ [Z\,(a_0 + a_1\, YAX)\, a_2\, \delta tv + SRV^{a2})^{1/a_2} - SRV \quad \text{------------- (6.11)}$$

Raveling Progression starts when δtv > 0 or $ARV_a > 0$

where:

If $ARV_a > 0$, δ tv = 1

If $ARV_a \geq 50$ then: z = -1, otherwise: z = 1

ARV_a = MAX (ARVa ,0.5)

SRV = MIN [ARV_a , (100 – ARV_a)]

dARV = Change the raveling area during the analysis year (% of total carriageway area)

ARV_a = Raveling area at the start of the analysis year (% of total carriageway area)

δ tv = Fraction of analysis year in which progression of raveling applies

Kvp = Calibration factor for progression of raveling

IRV = Time to raveling initiation (years)

6.9.3 Potholing

Potholes usually develop on a surface when it is either cracked, ravelled, or both. The pothole formation is accelerated by the presence of water. This happens due to the general weakening of the pavement structure and reduction in the resistance of the surface and base materials to disintegration. The potholing models use the construction defects indicator for the base (CDB), as a variable.

a. Pothole Initiation

Initiation of potholes due to cracking only is considered when the total area of structural cracking (ACW) exceeds 20%. Potholes initiated by ravelling arise when the ravelled area (ARV) exceeds 30%.

$$IPT = K_{pi} \text{ x } a_0 \left[\frac{(1+a_1 HS)}{(1+a_2 CDB)(1+a_3 YAX)(1+a_4 MMP)}\right] \quad \text{------------- (6.12)}$$

where**:**

IPT = Time between the initiation of Wide structural cracking or raveling and the initiation of potholes in years

HS = Total thickness of bituminous surfacing (mm)

CDB = Construction defects indicator for the base

YAX = Number of axles of all motorized vehicle types in the analysis year (millions/lane)

MMP = Mean monthly precipitation in mm/month

K_{pi} = Calibration factor for pothole initiation

b. Pothole progression

In this equation, a factor to account for the time lapse, TLF has been introduced as an indicator of the response time taken to patch the pothole.

$$dNPTi = K_{pp} \text{ x } a_0 \text{ x } ADIS_i (TLF)\left[\frac{(1+a_1 CDB)(1+a_2 YAX)(1+a_3 MMP)}{(1+a_4 HS)}\right]$$

------------- (6.13)

where,

$dNPT_i$ = Additional number of potholes per km due to distress type *i* (Wide structural cracking, raveling, enlargement) during the analysis year

ADISi = The percentage area of wide structural cracking at the start of the analysis year, or the percentage area of raveling at the start of the analysis year, or number of existing potholes per km at the start of the analysis year

TLF = Time lapse factor

dNPT = Total number of additional potholes per km during the analysis year

Kpp = Calibration factor for pothole progression

6.9.4 Roughness

The structural component of roughness relates to the deformation in the pavement under the shear stresses imposed by traffic loading. It is given by:

$$\Delta RI = K_{gm} \text{ x } a_0 \exp(m K_{gm} \text{ AGE3})(1 + SNPK_b) - 5 \text{ YE4} - [a_0 \text{ x } \Delta ACRA] + [a_0 \text{x } \Delta RDS] + [a_0(a_1 - FM)\{((NPT_a * TLF) + (\Delta NPT * TLF/2)^{a_2} - (NPT_a)^{a_2}\}] + [m K_{gm} RI_a] \quad \text{------------- (6.14)}$$

where,

dRI_s = Incremental change in roughness due to structural deterioration during the analysis year (IRI m/km)

*d*SNPK = Reduction in adjusted structural number of pavement due to cracking

$SNPK_b$ = Adjusted structural number of pavement due to cracking at the end of the analysis year

AGE3 = Pavement age since last overlay, reconstruction or new construction (years)

*d*ACRA = Incremental change in area of total cracking during the analysis year (% of total carriageway area)

*d*RDS = Incremental change in standard deviation of rut depth during the analysis year

YE4 = Annual number of equivalent standard axles (millions/lane)

FM = freedom to maneuver

CW = carriageway width (m)

AADT = annual average daily traffic (veh /day)

NPTa = number of potholes per km at the start of the analysis year

TLF = time lapse factor

m = Environmental coefficient

6.10 CALIBRATION OF HDM-4 DETERIORATION MODELS

6.10.1 Need for calibration

HDM or its relationships have been used in more than 100 countries, including both developed and developing countries. These countries have different technological, climatic and economic environments. A compendium of the countries where HDM had been applied was compiled as part of the International Study of Highway Development and Management Tools (ISOHDM).

The PPMs used in HDM-4 should be calibrated to reflect the observed rates of pavement deterioration for the roads where the models are applied. Since the model simulates future changes from current condition on the road system, the reliability of the results depend upon mainly on two primary considerations (Bennet and Paterson, 2000). The first one is the extent up to which the data provided to the model represent the reality of current conditions and influencing factors, in the terms understood by the model (Data Input). The second consideration is the accuracy up to which the predictions of the model fit in to the real behaviour and the interactions between various factors, with the different conditions to which it is applied (Calibration of the Output).

6.10.2 Procedure for Calibration

Calibration factors for Cracking, Ravelling, Potholing and Roughness models have been identified. To calibrate the HDM-4 deterioration models as applicable to local conditions, the predictions made by the models have been equated with actual observed field data. The data collected from Seaport Airport Road from 2006 to 2013 was used for the analysis. The deterioration factors, which gave closer relationship, were selected as the calibration factors.

The HDM-4 was run for the same loading, structural condition and distress as that of initial pavement condition. The progressions made by the two models were then plotted on a graph and the variations in the predictions made were found out. The calibration factors for Initiation and Progression were altered and the model was run to obtain the progression of distresses. It was assumed that the calibration factor for crack initiation is the same for all structural Cracking, Wide structural Cracking and Transverse Thermal Cracking. The same assumption was made for the calibration factor for cracking progression also. Calibration factors were determined for Cracking, Ravelling, Potholing and Roughness models as above. Default calibration factors (value = 1.00) were used for other models such as Edge-break, Rutting and Skid resistance. The respective calibration factors that were determined are presented in **Table 6.3**.

Pavement composition of the road is as given below:

BC	40mm
DBM	150mm
WMM	250mm
GSB	200mm
Subgrade	500mm

Table 6.3 HDM-4 Pavement Deterioration Models and Calibration Factors

Sl No	Model Description	HDM-4 Pavement Deterioration Models	Calibration Factors
1	Cracking Initiation	$ICA = K_{cia}\left\{CDS^2\, a_0 \exp\left[a_1 SNP + a_2\left(\frac{YE4}{SNP^2}\right)\right] + CRT\right\}$	K_{cia}=0.8
2	Cracking Progression	$dACA = K_{cpa}\ [CRP/CDS]\ Z_A[(Z_A\, a_0 a_1 \delta\, T_A + SCA^{a1})1/a_1 - SCA]$	K_{cpa}=0.69
3	Raveling Initiation	$IRV = K_{vi}\, CDS^2\, a_0\, RRF \exp(a_1\, YAX)$	K_{vi}=0.27
4	Raveling Progression	$dARV = [K_{vp} / RRF][1/CDS^2]\, Z\ [Z\,(a_0 + a_1\, YAX)\, a_2\, \delta tv + SRV^{a2})^{1/a_2} - SRV]$	K_{vp}=0.42
5	Pothole Initiation	$IPT = K_{pi}\ x\ a_0[\frac{(1 + a_1 HS)}{(1 + a_2 CDB)(1 + a_3 YAX)(1 + a_4 MMP)}]$	K_{pi}=0.45
6	Pothole Progression	$dNPTi = K_{pp}\ x\ a_0\ x\ ADIS_i(TLF)[\frac{(1 + a_1\, CDB)(1 + a_2 YAX)(1 + a_3\, MMP)}{(1 + a_4\, HS)}]$	K_{pp}= 0.8
7	Roughness Progression	$\Delta RI = K_{gm}\ x\ a_0 \exp(m\, K_{gm}\ AGE3)\ (1 + SNPK_b) - 5\ YE4 - [a_0\ x\ \Delta ACRA] + [a_0 x\ \Delta RDS] + [a_0(a_1 - FM)\{((NPT_a * TLF) + (\Delta NPT * TLF/2)^{a_2} - (NPT_a)^{a_2}\}] + [m\, K_{gm}\ RI_a]$	K_{gm}=0.4

6.10.3 Comparison of Actual and HDM Predicted Values

Progression of various distresses under Actual and Predicted conditions are shown graphically in **Fig. 6.52 to 6.57**. From the plots, it was observed that the variation between actual observed and HDM-4 predicted values were very less, indicating that the Pavement Performance prediction made by the calibrated HDM-4 software suits to the local conditions. Data from 2006 to 2013 was used for comparison.

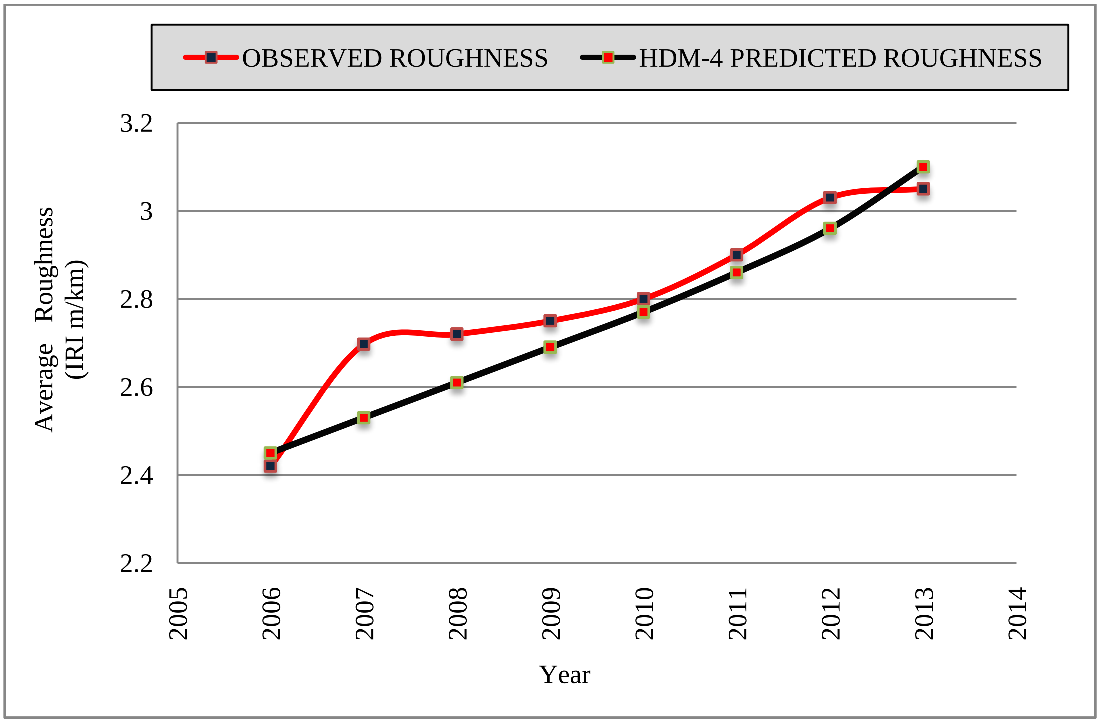

Fig. 6.52: Roughness Progression of S A 1

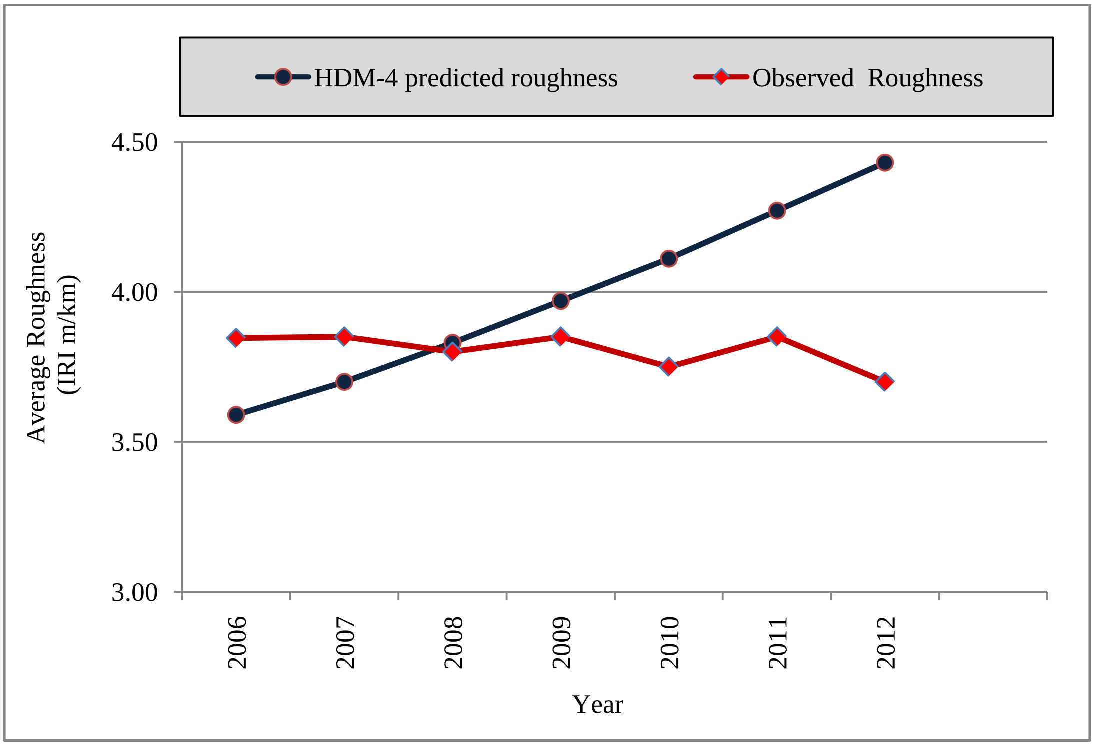

Fig. 6.53: Roughness Progression of S A II

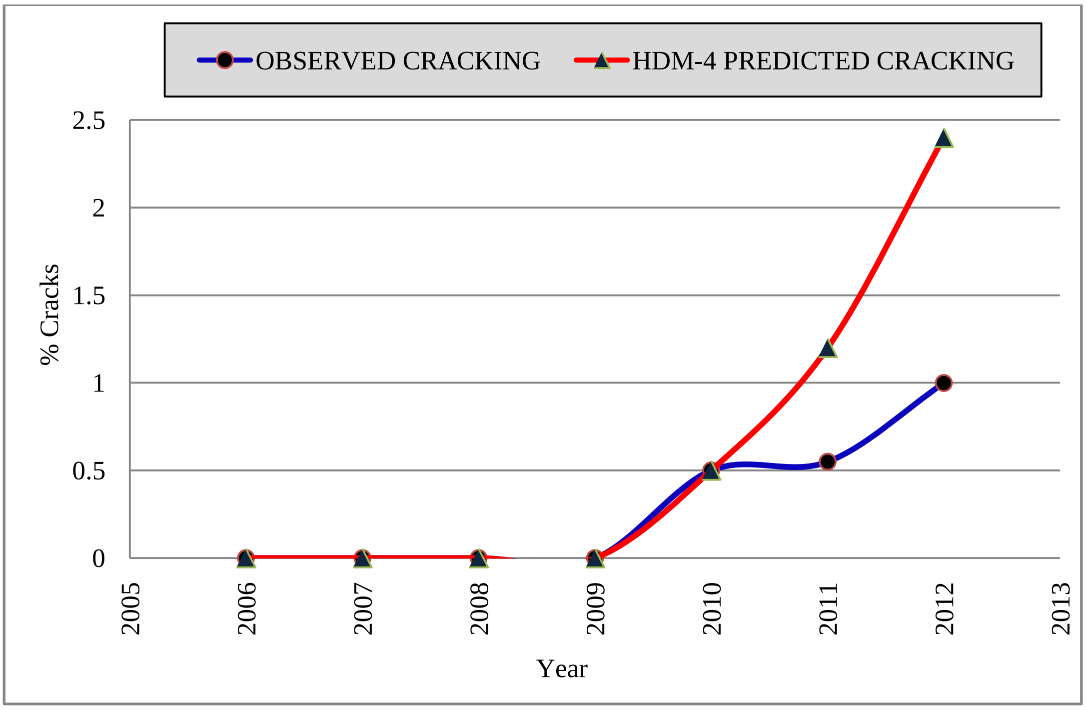

Fig. 6.54: Cracking Progression of SA-1

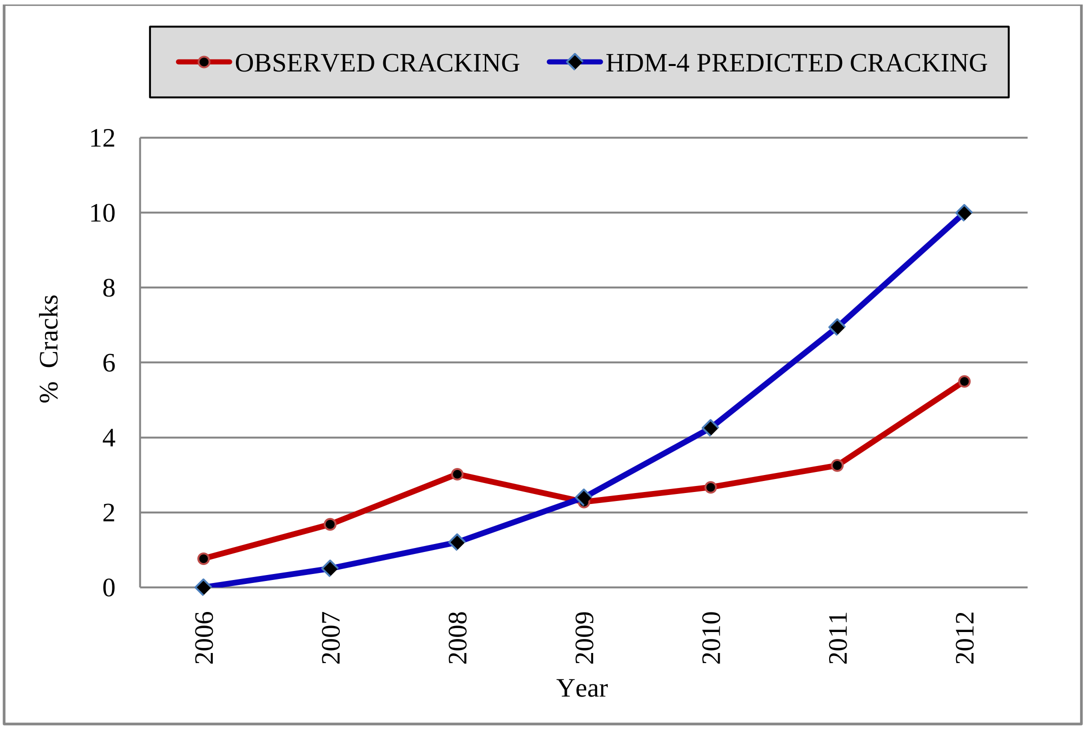

Fig. 6.55: Cracking Progression of SA-2

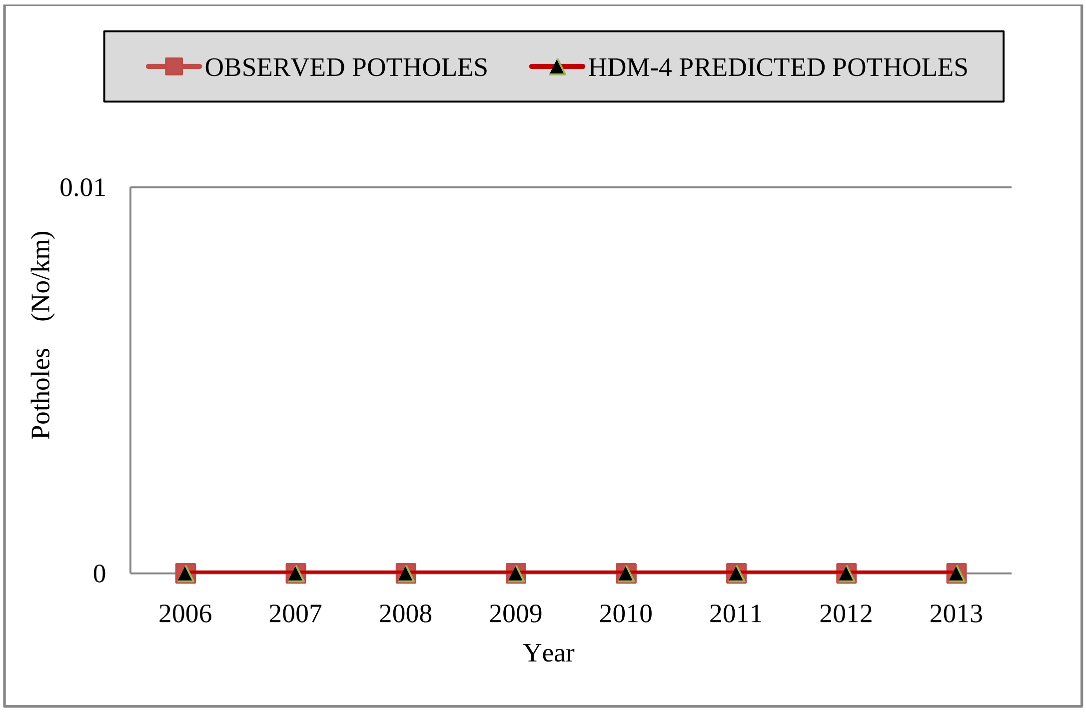

Fig. 6.56: Pothole Progression of SA-1

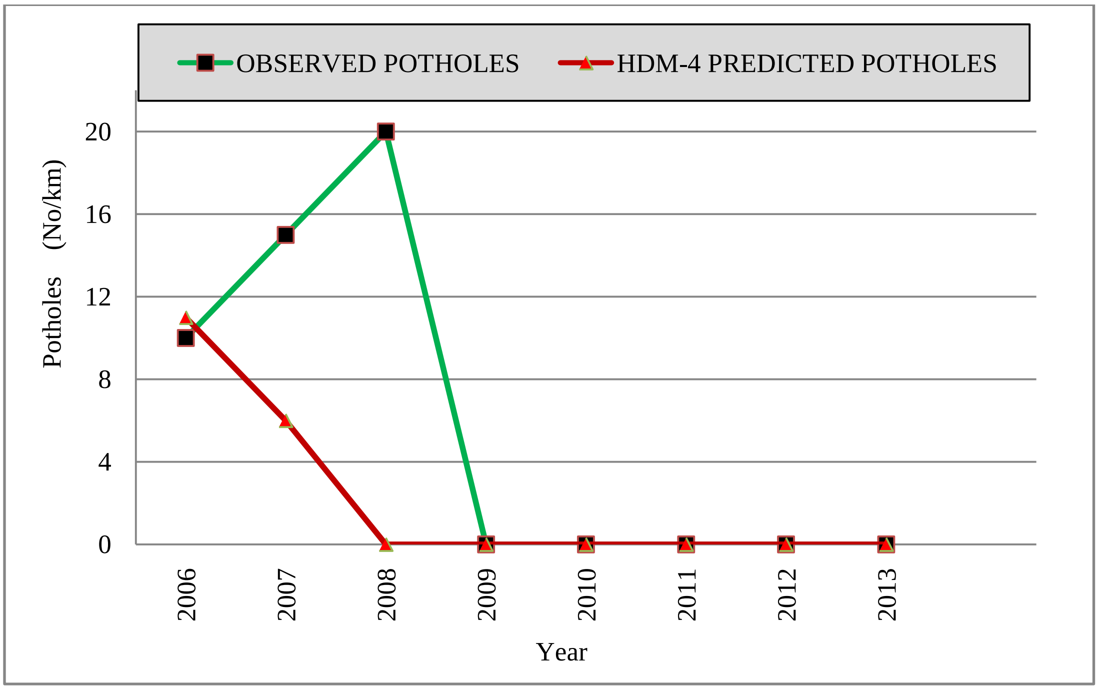

Fig. 6.57: Pothole Progression of SA-2

6.10.4 Validation

The validation of the calibrated pavement deterioration models was done to test the accuracy of these models, by comparing the predictions obtained by running the calibrated deterioration models with the values actually obtained on the selected pavement sections. Validation was done by the pavement condition data of Seaport-Airport road collected in 2013. The observed values of cracking area, ravelling area and roughness for pavement sections were compared with those predicted by the calibrated HDM-4 models. The variation of cracking area, ravelling area and roughness between observed and predicted values are shown in **Table 6.4**.

Table 6.4 Variation of Observed and Predicted Values

Section	Roughness (IRI m/Km)			Cracking (%Area)			Raveling(% Area)		
	Observed	Predicted	% Variation	Observed	Predicted	% Variation	Observed	Predicted	% Variation
SA-1	3.05	3.09	1.3	1.5	1.04	30.7	8	6.29	21.4
SA-2	3.85	3.79	1.6	6.1	8.52	39.7	7.5	7.85	4.7

From Table 6.4, it is seen that the variation of the actual and HDM predicted values for roughness is 1.3 percent and 1.6 percent only for the two sections. But in the case of cracking the variation is high. In the case of raveling, HS 1 showed high variation but HS 2 gave comparable values.

6.10.5 Analysis of Alternative Options

6.10.5.1 Case 1 - Improvement without Widening

In the present study, four options were considered for treatment as defined in **Table 6.5**. The first alternative, Base alternative was considered which is the minimum routine maintenance, which consists of crack sealing, and pothole patching only, until the reconstruction of the pavement section became inevitable. The overlay options considered as maintenance and rehabilitation options were 40 mm BC using ordinary Bitumen, 40 mm BC using Natural Rubber Modified Bitumen and Ultra Thin White Topping over Bituminous surface. No widening was considered at this stage.

Table 6.5 Proposed M&R Options without widening

Alternatives	Works Standard	Description of Work
Base Alternative	Routine maintenance	Crack Sealing
		Pothole Patching
		Drainage
Alternative 1	Overlay	40 mm Bituminous concrete using Ordinary Bitumen
Alternative 2	Overlay	40 mm Bituminous Concrete using Modified Bitumen (NRMB)
Alternative 3	Ultra-Thin White topping	100 mm thick CC Slab

a. Pavement Deterioration Summary for Case 1

The deterioration of the pavement section under various Maintenance & Rehabilitation options as indicated above for a period of 15 years were analyzed using HDM-4 and the deterioration summary is shown in **Table 6.6.** The distresses analyzed are roughness, cracks, raveling and potholes.

Table 6.6 Pavement Condition for alternative strategies

Year	AADT	Base Alternative				Alternative 1			
		Average Roughness IRI m/km	All structural Cracks %	Raveling %	No. of Potholes (No./km)	Average Roughness IRI m/km	All structural Cracks %	Raveling %	No.of Potholes (No./km)
2013	19,442	3.80	8.22	15.50	0	3.80	8.68	15.70	0
2014	20,790	4.03	11.07	28.25	0	3.80	4.34	7.85	0
2015	22,145	4.26	14.59	46.19	0	2.59	0.00	0.00	0
2016	23,592	4.51	11.30	60.28	20	2.74	0.00	0.00	0
2017	25,137	4.64	4.82	71.08	28	2.88	0.00	0.00	0
2018	26,788	4.88	7.73	78.73	35	3.03	0.50	0.00	0
2019	28,551	5.13	11.11	84.91	41	3.19	1.83	0.25	0
2020	30,310	5.39	14.77	84.50	47	3.37	4.01	2.33	0
2021	32,182	5.56	11.33	79.74	50	3.57	7.08	6.03	0
2022	34,175	5.54	4.46	85.68	50	3.79	10.59	13.22	0
2023	36,295	5.63	7.23	89.40	56	4.01	14.35	25.12	0
2024	38,553	5.73	10.46	88.79	62	4.23	12.21	41.88	0
2025	40,860	5.83	13.98	85.27	66	4.38	7.98	56.99	2
2026	43,309	5.93	10.22	80.68	67	4.61	11.13	69.02	3
2027	45,909	5.92	2.54	86.41	67	4.86	8.32	78.17	4

Year	AADT	Alternative 2				Alternative 3			
		Average Roughness IRI m/km	All structura Cracks %	Raveling %	No. of Potholes (No./km)	Average Roughness IRI m/km	Average Faulting mm	Spalled joints %	Cracked Slabs
2013	19,442	3.80	8.68	15.70	0	3.80	0.00	0.00	0.00
2014	20,790	3.80	4.34	7.85	0	3.80	0.00	0.00	0.00
2015	22,145	2.59	0.00	0.00	0	2.03	0.08	0.02	0.01
2016	23,592	2.75	0.00	0.00	0	2.07	0.09	0.04	0.04
2017	25,137	2.90	0.00	0.00	0	2.09	0.10	0.07	0.09
2018	26,788	3.05	0.50	0.00	0	2.09	0.11	0.11	0.15
2019	28,551	3.21	1.83	0.25	0	2.10	0.12	0.16	0.23
2020	30,310	3.40	4.01	2.33	0	2.11	0.12	0.22	0.32
2021	32,182	3.61	7.08	6.03	0	2.12	0.13	0.29	0.44
2022	34,175	3.84	10.59	13.22	0	2.12	0.13	0.37	0.57
2023	36,295	4.06	14.35	25.12	0	2.13	0.14	0.46	0.72
2024	38,553	4.30	12.21	41.88	0	2.14	0.14	0.55	0.89
2025	40,860	4.45	7.98	56.99	2	2.14	0.15	0.66	1.09
2026	43,309	4.69	11.12	69.02	3	2.15	0.15	0.77	1.32
2027	45,909	4.95	8.32	78.17	4	2.16	0.16	0.89	1.57

Roughness Progression

Roughness is the most useful indicator of the average condition or deterioration of the pavement section. The progression of roughness can be used to see that the maintenance works are correctly triggered according to the specified intervention criteria. The progression of Roughness with the alternatives are shown in **Fig. 6.58 & 6.59.**

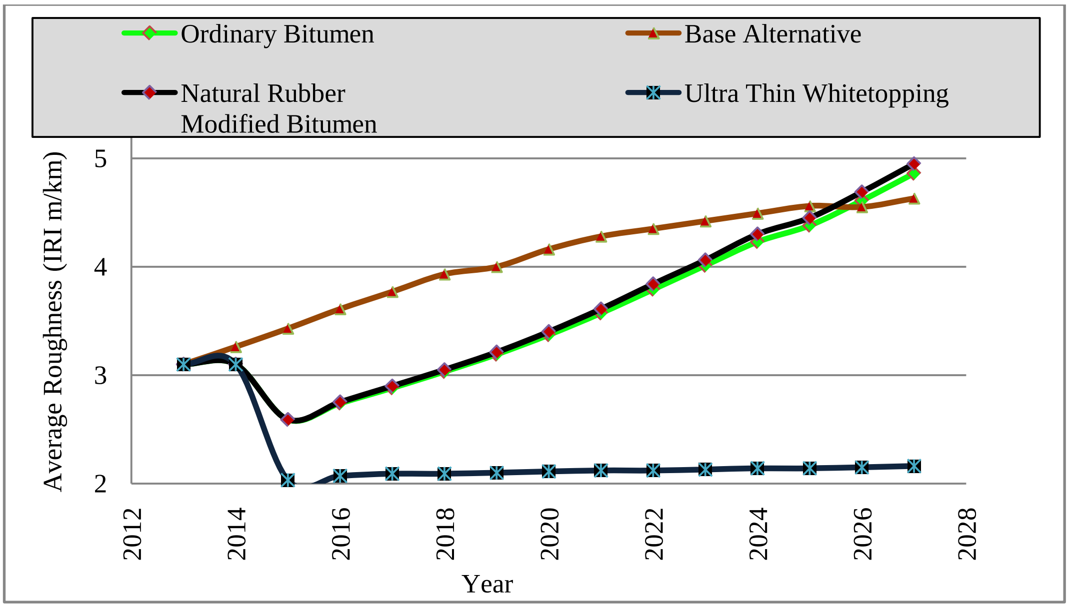

Fig. 6.58: Roughness Progression of SA-1 for Alternative options

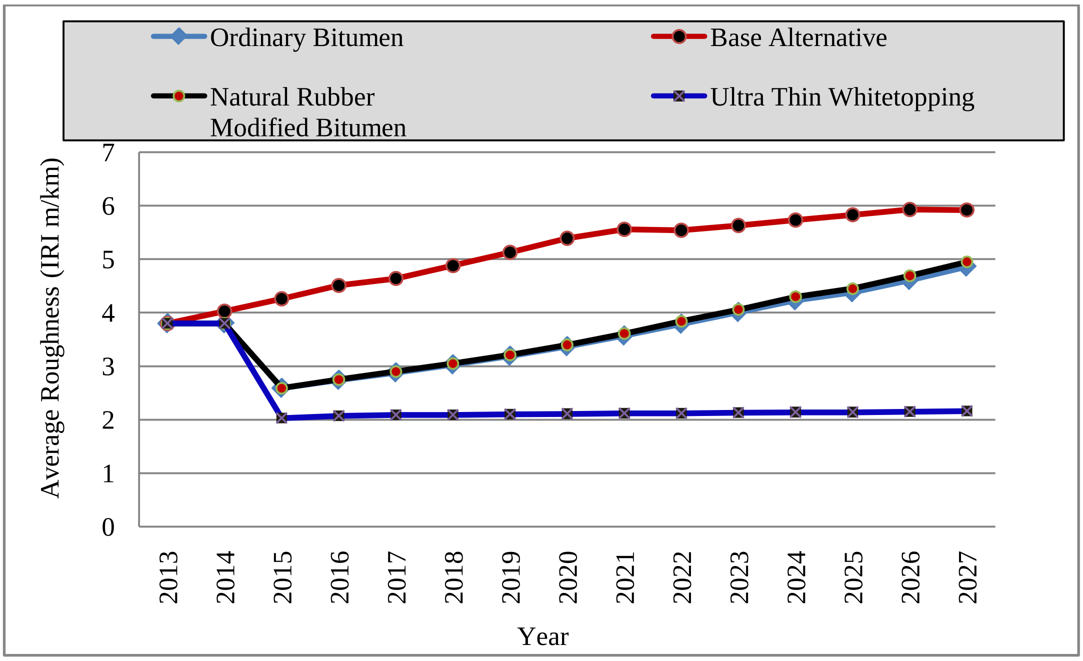

Fig. 6.59: Roughness Progression of SA-2 for Alternative options

Progression of distresses

The progression of Cracking, Pothole and Raveling are shown in the **Fig. 6.60 to 6.65.** From the deterioration summary, it was seen that even after 15 years, the riding quality of Ultra thin White topping is the most desirable one without any intervention. Routine maintenance cost is not required in this option.

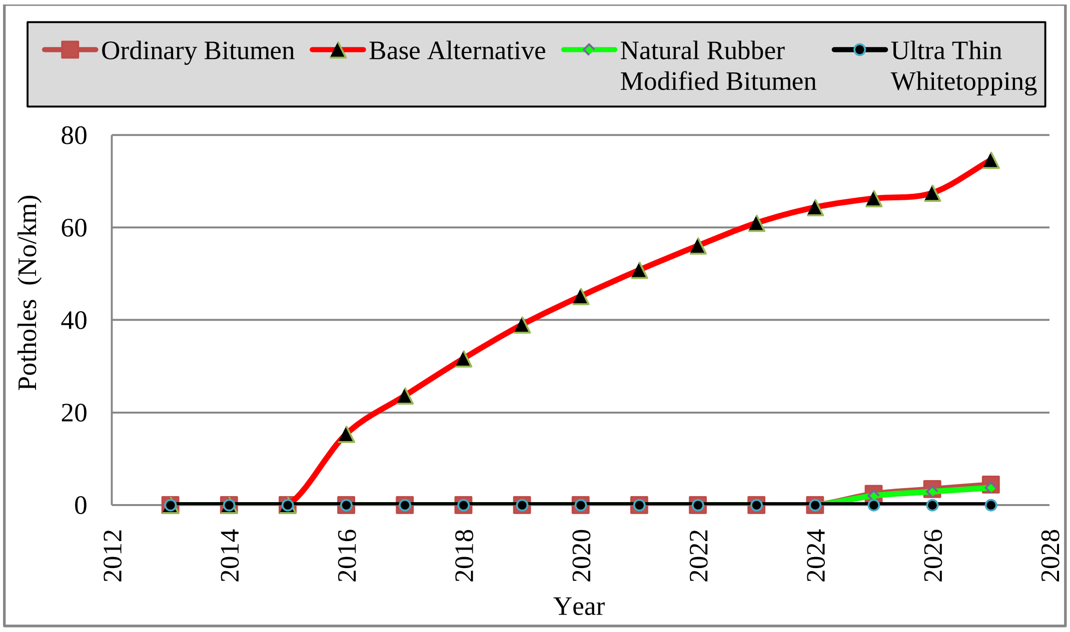

Fig. 6.60: Pothole Progression of SA-1 for Alternative options

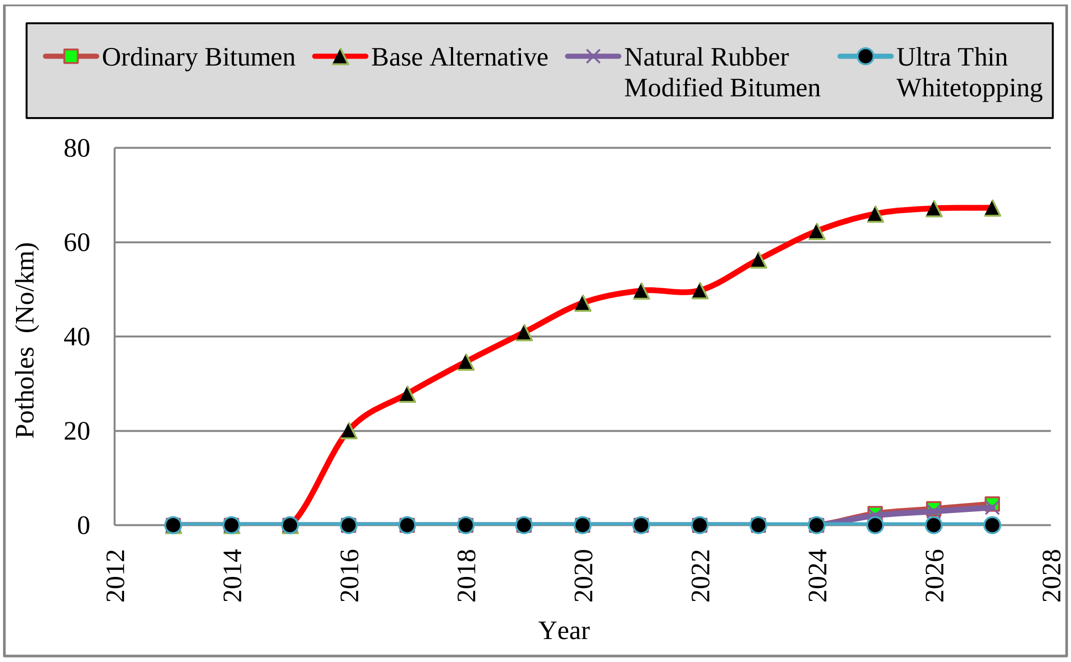

Fig. 6.61: Pothole Progression of SA-2 for Alternative options

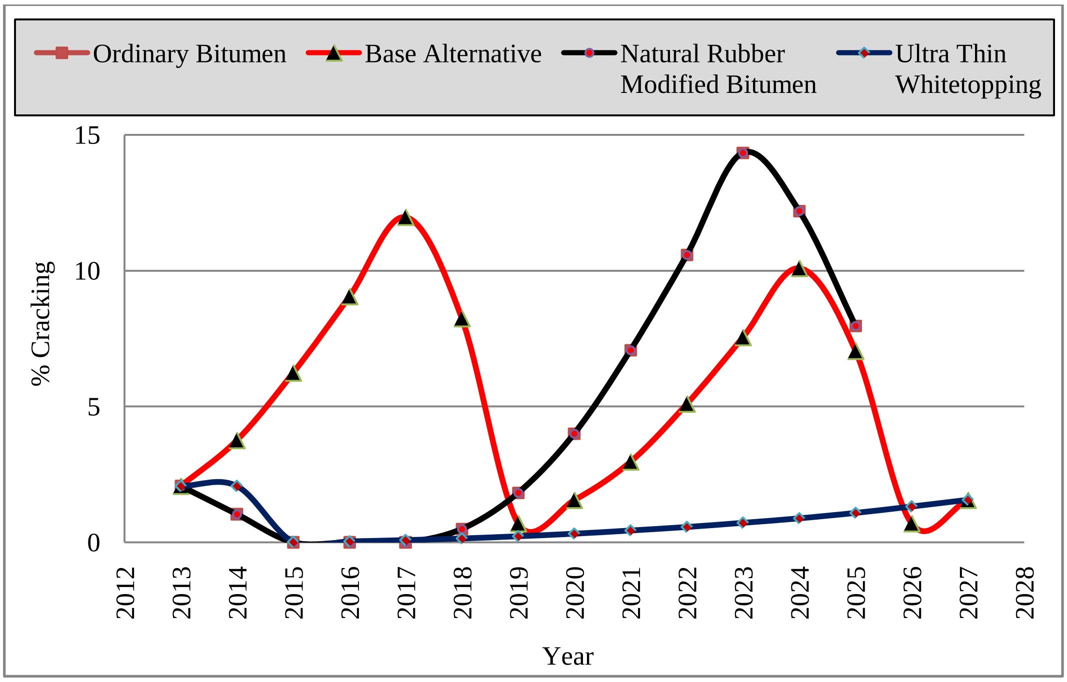

Fig. 6.62: Cracking Progression SA-1 for Alternative options

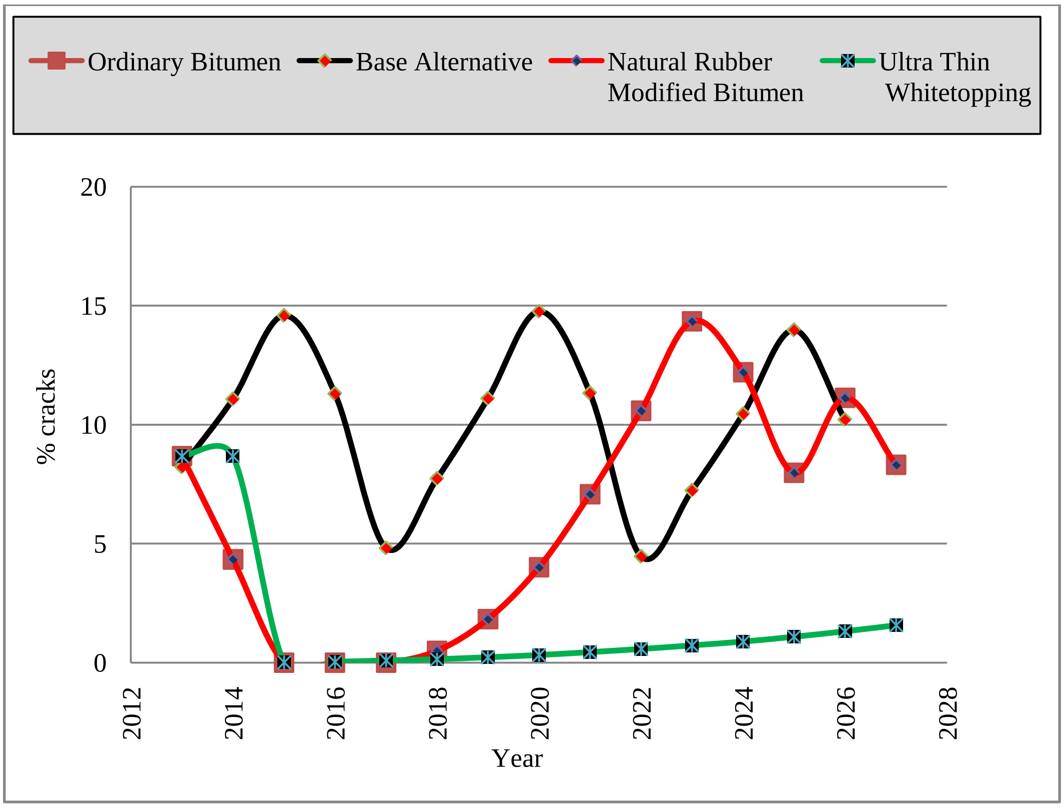

Fig. 6.63: Cracking Progression of SA-2 for Alternative options

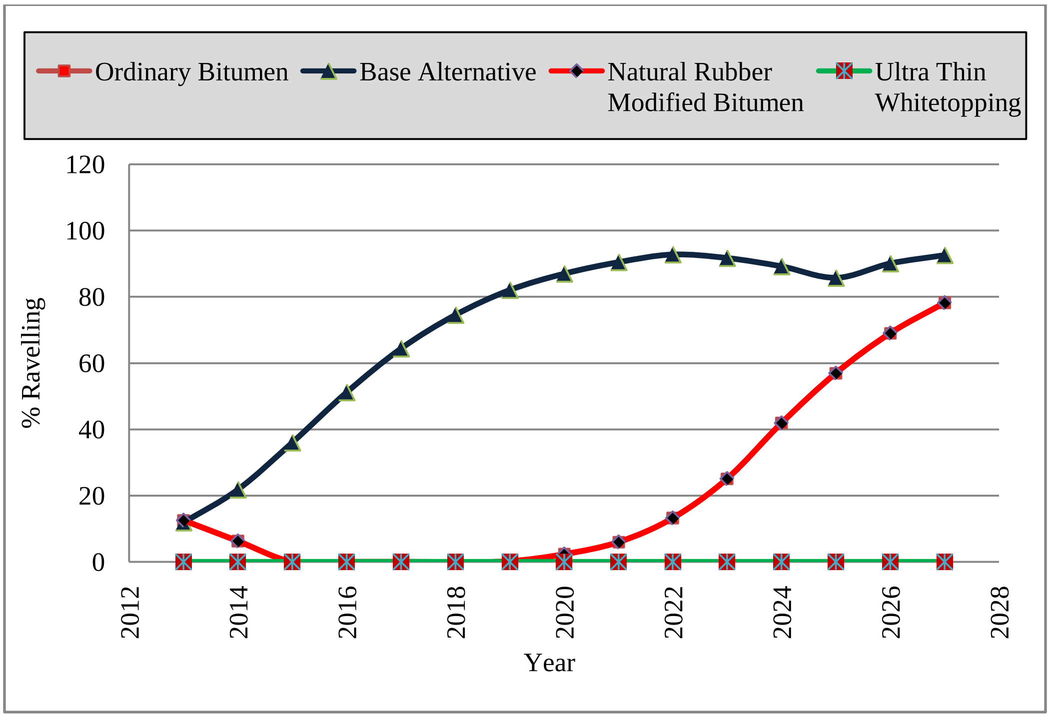

Fig. 6.64: Raveling Progression of SA-1 for Alternative options

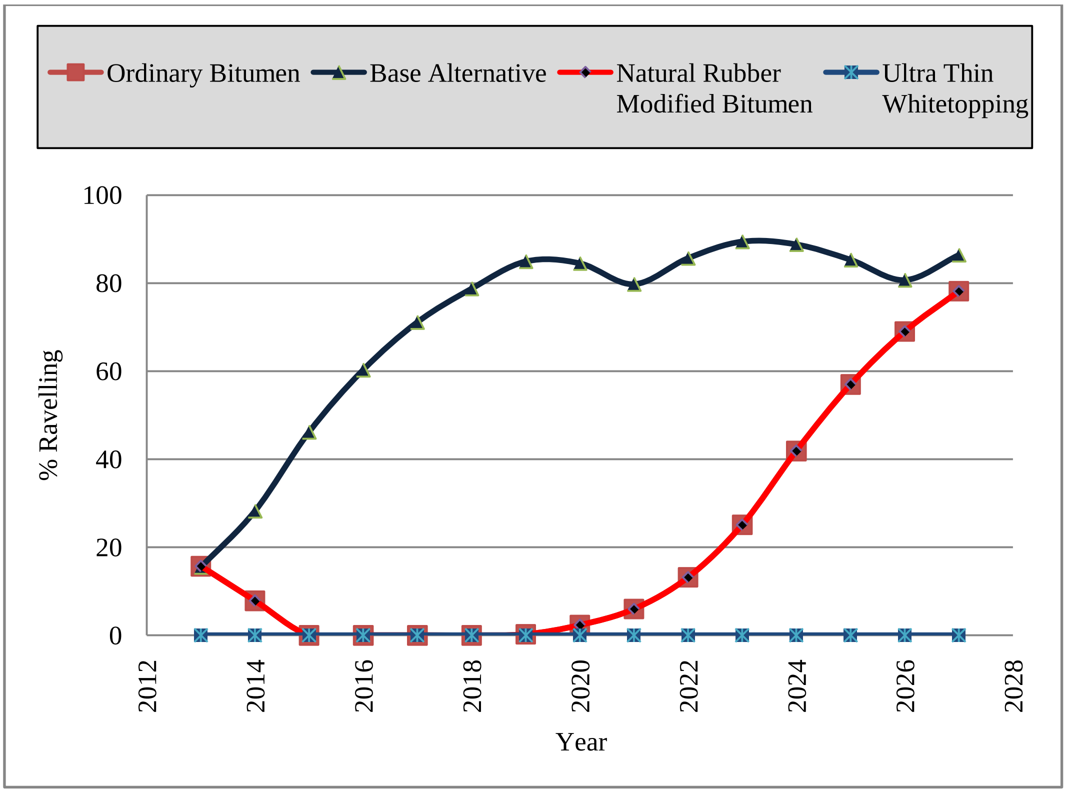

Fig. 6.65: Raveling Progression of SA-2 for Alternative options

b. Economic Analysis for case 1

To carry out the economic analysis, the Base Alternative was confirmed and a discount rate of 10 per cent was given. In the project analysis, Alternatives 1 to 3 were compared against the Base Alternative. Since speed is one of the criteria, which defines the level of service of the facility, the variation in average speed under various alternatives were derived from the analysis and are shown in the **Fig. 6.66 to 6.68**.

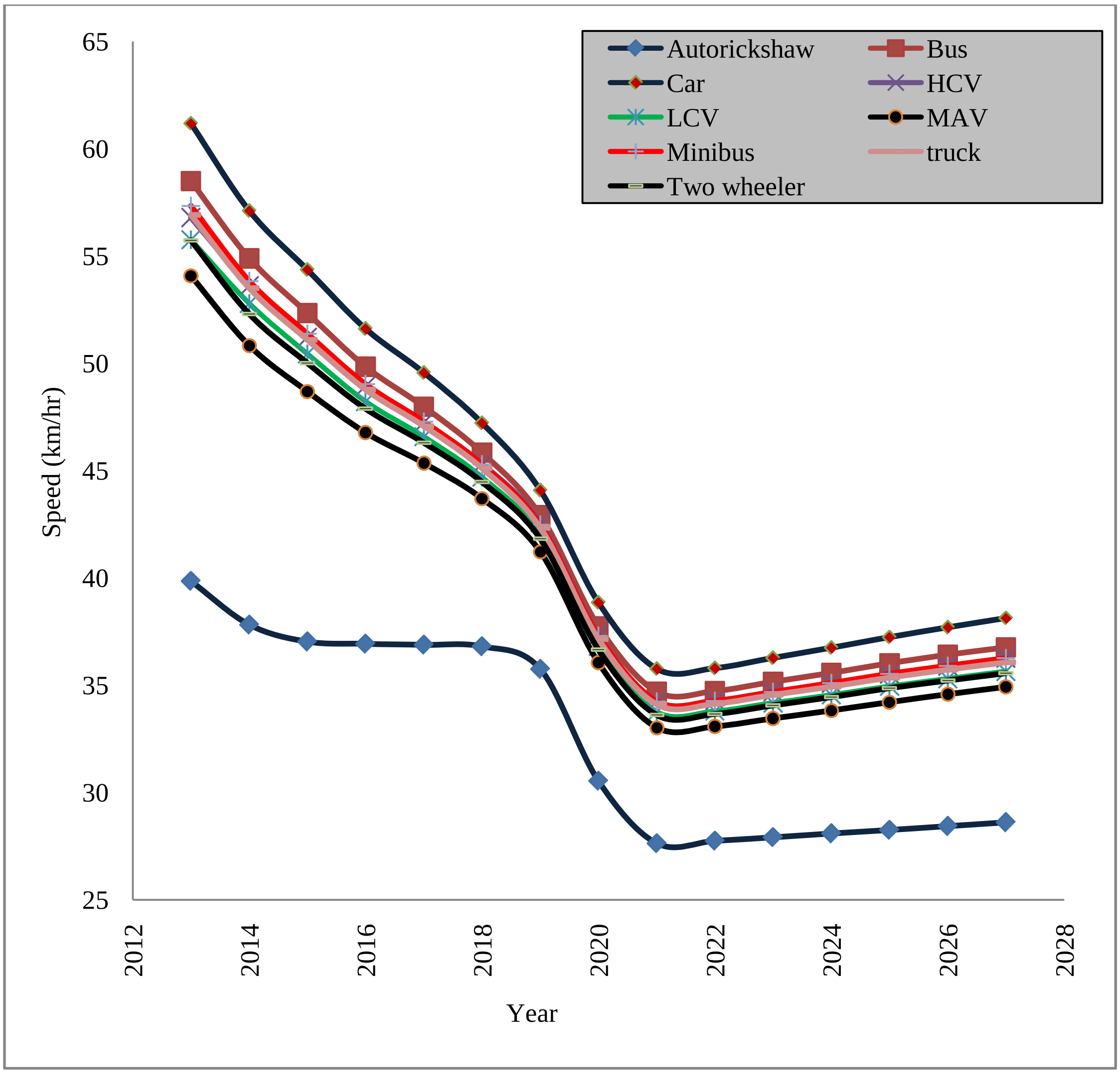

Fig. 6.66: Average Speed for BC with NRMB

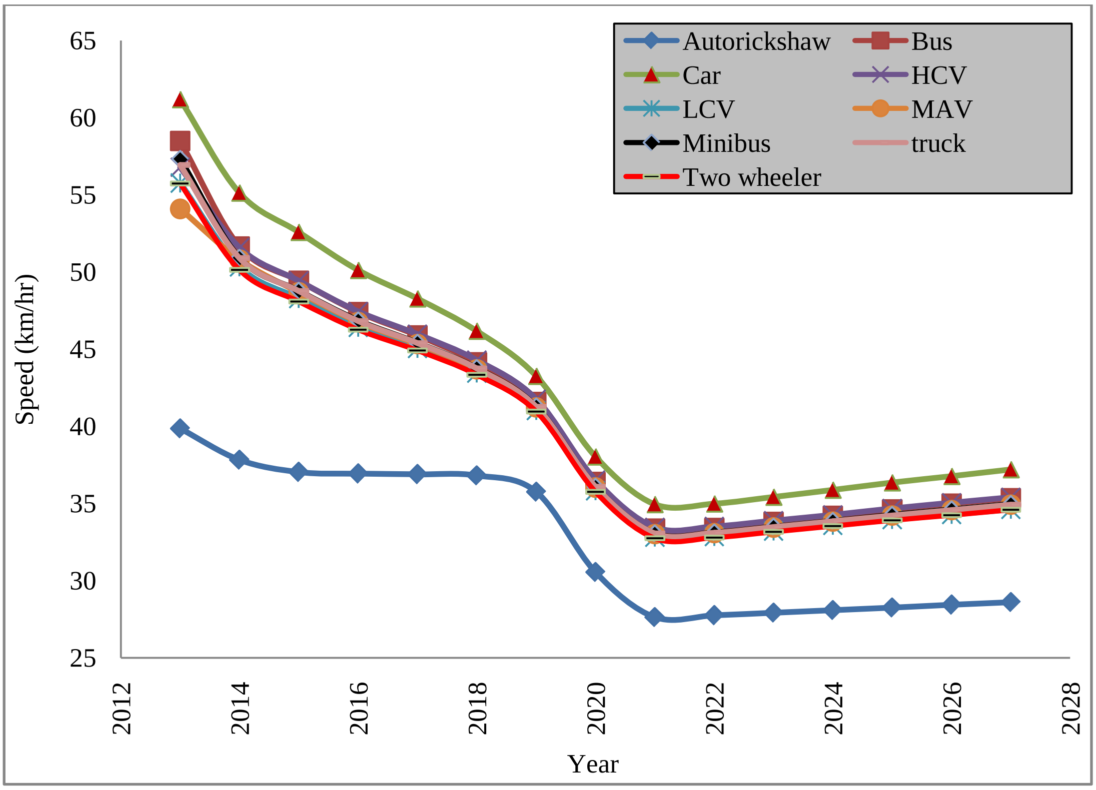

Fig. 6.67: Average Speed for BC Using Ordinary Bitumen

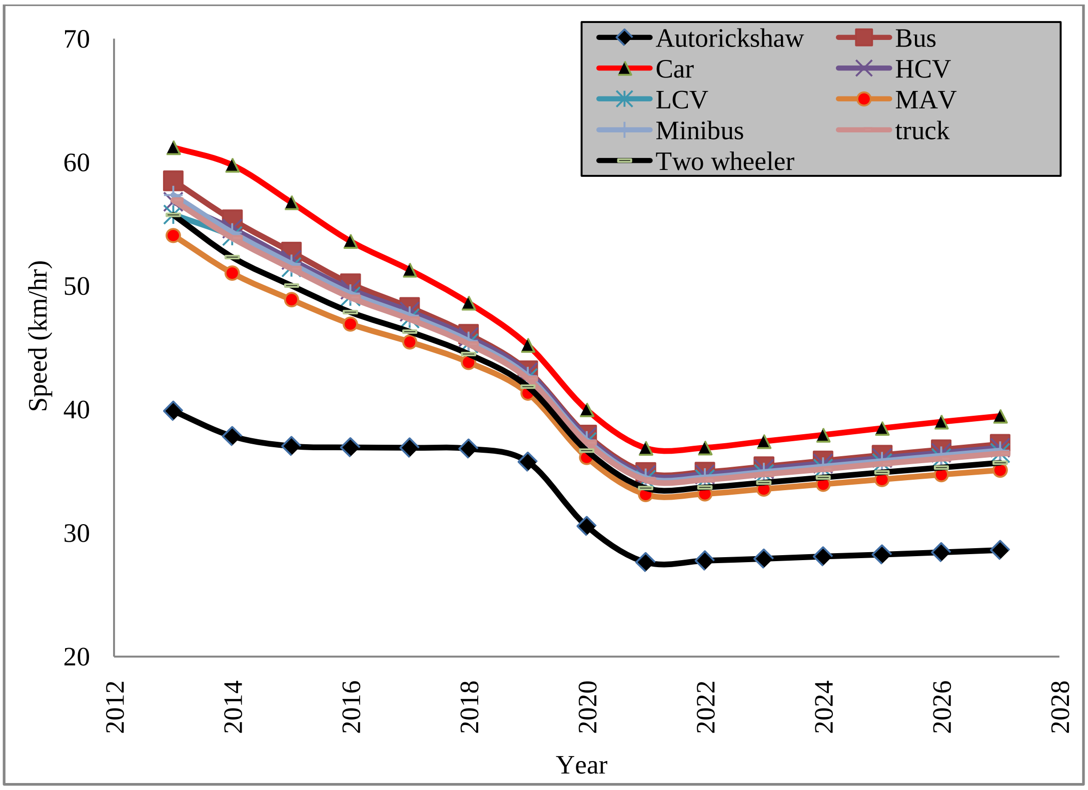

Fig. 6.68: Average Speed for Ultra Thin White Topping

The economic analysis for case 1 is given in the **Table 6.7**.

Table 6.7 Economic Analysis of Maintenance & Rehabilitation without Widening

Alternative	*Discounted Savings in VOC of Motorized Traffic	*Discounted Savings in Travel Time Cost of MT	*Discounted Net Economic Benefits, NPV	NPV/Cost ratio	Internal Rate of Return
Base Alternative	0	0	0	0	0
40mm BC	124.33	-16.31	58.09	1.163	20.8
40mm BC with NRMB	112.96	4.70	66.72	1.310	23.4
Ultra thin White topping	224.42	11.09	106.54	0.826	19.7

*(Indian Rupees (millions))

The above analysis indicated that there was not much savings in the Vehicle Operating Cost and Travel Time Cost. So the Net Benefits due to the overlay was very less which is not desirable. Due to the inadequate carriageway width, the traffic volume exceeded the capacity of the road, creating severe congestion and delay with reduced Level of Service of the road. In case of an accident or otherwise also, highly trafficked roads which is of under-capacity, easily become congested and this congestion causes increased costs to road users, industry and businesses, including road freight operators and their customers.

6.10.5.2 Case II - Improvement with Widening

From the above analysis, it was seen that the maximum speed that could be obtained is around 60 kmph in the first year of treatment and reduces to 40 kmph by 2027. Hence a second improvement strategy, i.e., pavement up gradation which includes pavement rehabilitation by providing a suitable overlay along with a partial widening of the carriageway width was considered, as given in **Table 6.8**. Widening up to 3 m was considered, so that the carriageway width will increase from 7.5 m to 10.5 m.

Table 6.8 Improvement with Widening

Alternatives	Works Standard	Description of Work
Base Alternative	Routine Maintenance	Crack Sealing
		Pothole Patching Drainage
Alternative 1	Overlay+ Partial Widening	40 mm B C + Routine maintenance
Alternative 2	Overlay+ Partial Widening	40 mm BC with mix using NRMB + Routine maintenance
Alternative 3	Ultra-Thin White topping + Partial Widening	100 mm thick CC Slab

Analysis of the three alternatives along with partial widening shows that there is considerable increase in the average speed of Motorized Vehicle fleet as shown in the **Fig. 6.69 to 6.71**. The average speed increases soon after the improvement, then gradually decreases and by 2027, a lane addition is needed to sustain the increasing traffic.

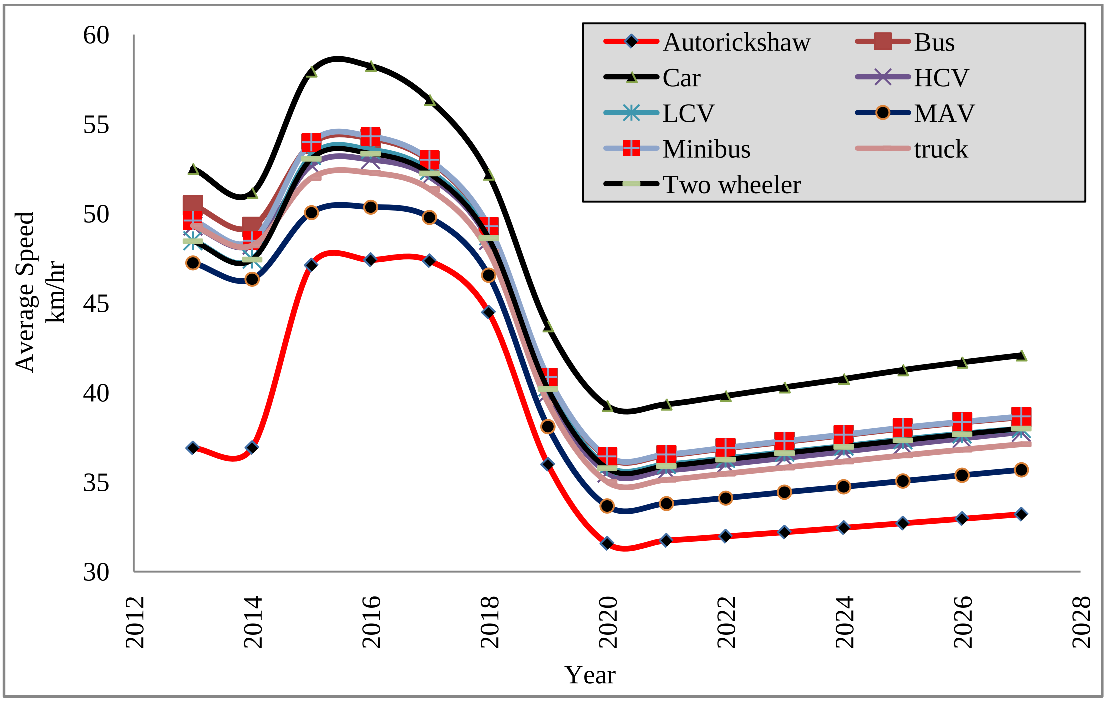

Fig. 6.69: Average Speed for BC overlay with Ordinary Bitumen and Widening

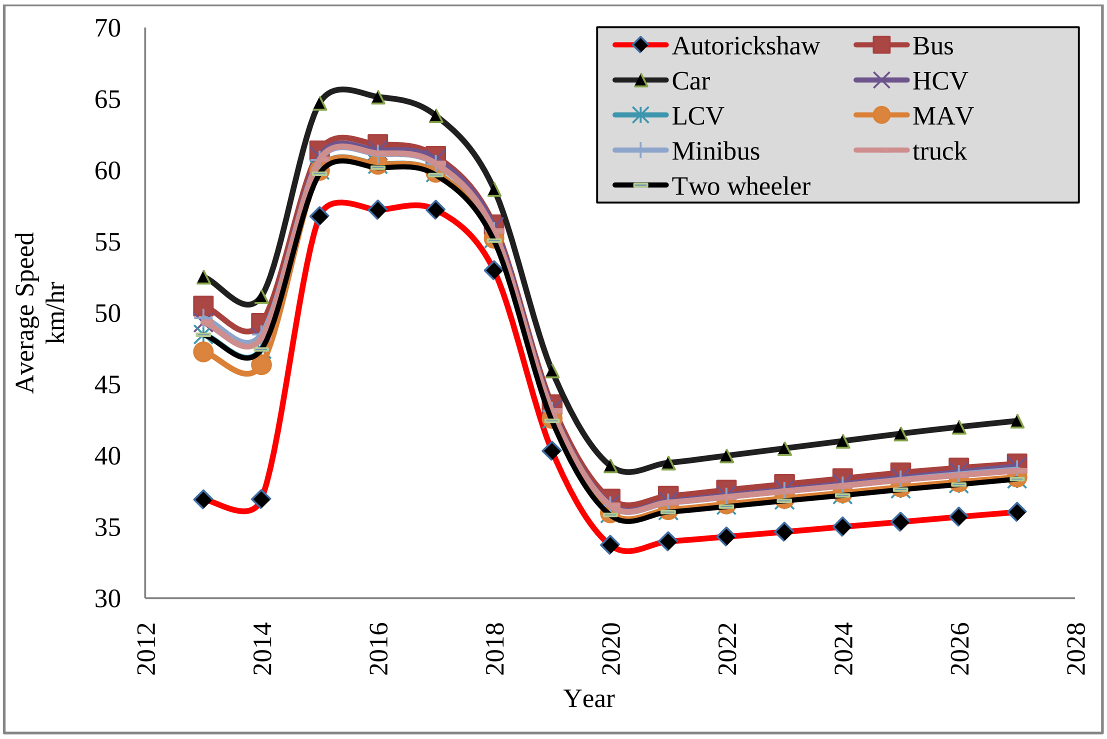

Fig. 6.70: Average Speed for BC overlay using NRMB mix and widening

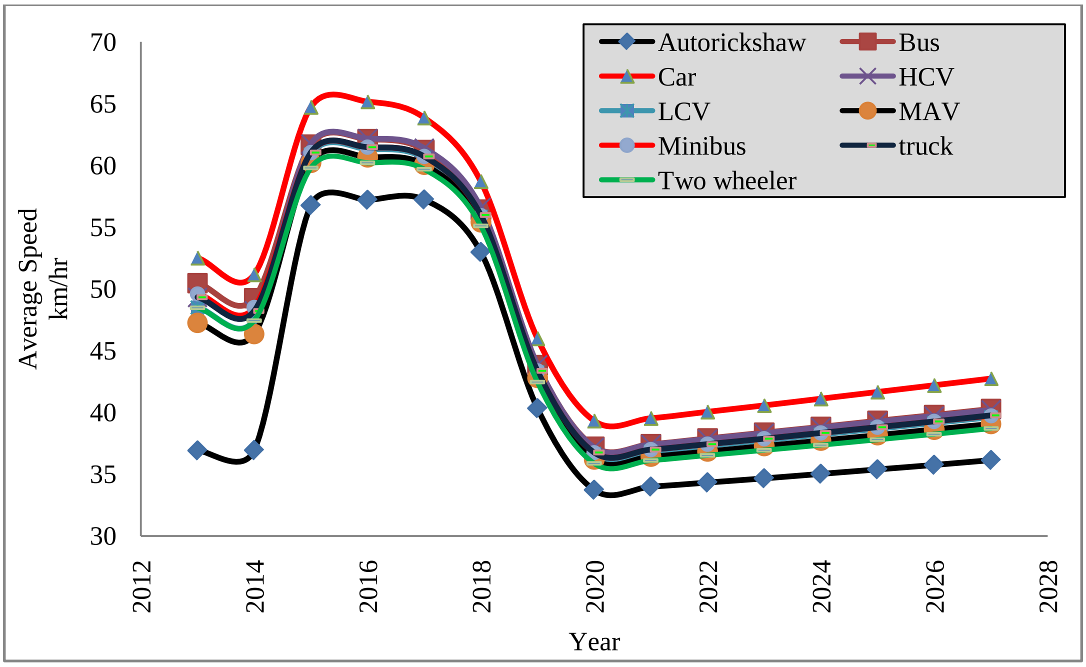

Fig. 6.71: Average Speed for Ultra Thin White topping and Widening

Economic analysis for Case 2

Summary of economic analysis for improvement with widening is shown in **Table 6.9.** From the Economic analysis, it was observed that Ultra-thin White topping has the maximum NPV/cost ratio and Internal Rate of Return, which clearly indicated that Ultra thin White topping is the most economical overlay option from among the alternatives considered in the present analysis. Hence, further work was attempted to develop sample charts for UTW design.

Table 6.9 Economic Analysis of improvement strategy along with widening

Alternative	Discounted Savings in Motorized Traffic VOC (Indian Rupees (millions)	Discounted Savings in MT Travel Time Cost (Indian Rupees millions)	Discounted Net Economic Benefits, NPV (Indian Rupees (millions)	NPV/Cost ratio	Internal Rate of Return
Base Alternative	0	0	0	0	0
40mm BC (Ordinary Bitumen)	173.94	977.16	822.93	2.51	40.4
40mm BC (Natural Rubber Modified Bitumen)	158.75	1205.66	1019.81	2.96	47.5
Ultra-thin White topping	343.98	1235.28	1203.07	3.20	48.2

6.11 PLOTTING OF DESIGN CHARTS FOR THIN CONCRETE OVERLAYS (ULTRA THIN WHITE TOPPING) ON ASPHALT LAYER

6.11.1 Design of UTW Systems

An empirical method to estimate the service life of the UTW based on thickness of the overlay, joint spacing, concrete mechanical strength and sub grade characteristics was developed by the Federal Highway Administration, U S A. Thickness of overlay, joint spacing, degree of bonding, thickness and condition of the remaining asphalt and pavement sub grade support conditions have been analytically proven to influence the stresses in the pavement.

James W Mack (1998) used the following equations for the maximum induced wheel load stress analysis on thin pavements based on the original work done by Westerguard.

Edge Stress $= 3(1+\mu)\ P/\ [\pi\ (3+\mu)\ h2]*[\ln Eh^3)/100ka^4) + 1.84 - 4\mu/3 + (1-\mu)/2 + 1.18\ (1 + 2\mu)\ (a/l)]$ ------------- **(**6.15)

Corner Stress $= 3P/(h)^2[1-(al2/l)^{0.6}$ ------------- **(**6.16)

where, μ = Poisson's Ratio

P = Total wheel load in kg

H = Thickness of slab in cm

L = Radius of relative stiffness

E = Elastic modulus of concrete in Kg/cm^2

k = Modulus of Sub-grade reaction

q = Tyre pressure in $Kg/\ cm^3$

S = Distance of two tyres in dual wheel assembly, in cm

a $= 0.8521*P/\ (q*\ \pi) + (S/\ \pi)*((P/\ (0.5227*q))^{0.5})^{0.5}$

6.11.2 Plotting of Design Charts for thin concrete overlays on Asphalt

Design charts were developed using the equations 6.26 and 6.27 for the thickness design of thin concrete overlays for wheel loads 3.2 t and 5.1 t, for different k values and the same is given in **Fig. 6.72** and **6.73.** The charts give the maximum stress for a particular slab thickness and different soil conditions incorporating different k values. IRC SP 76, 2008 gives design charts for axle load of 6 tonnes and 10.2 tonnes for k values ranging from 6 Kg per cu. cm to 45 kg per cu cm. In the present study, charts were plotted for wheel load of 3.2 T and 5.1 T, for k values ranging from 8 to 50 cm^3.

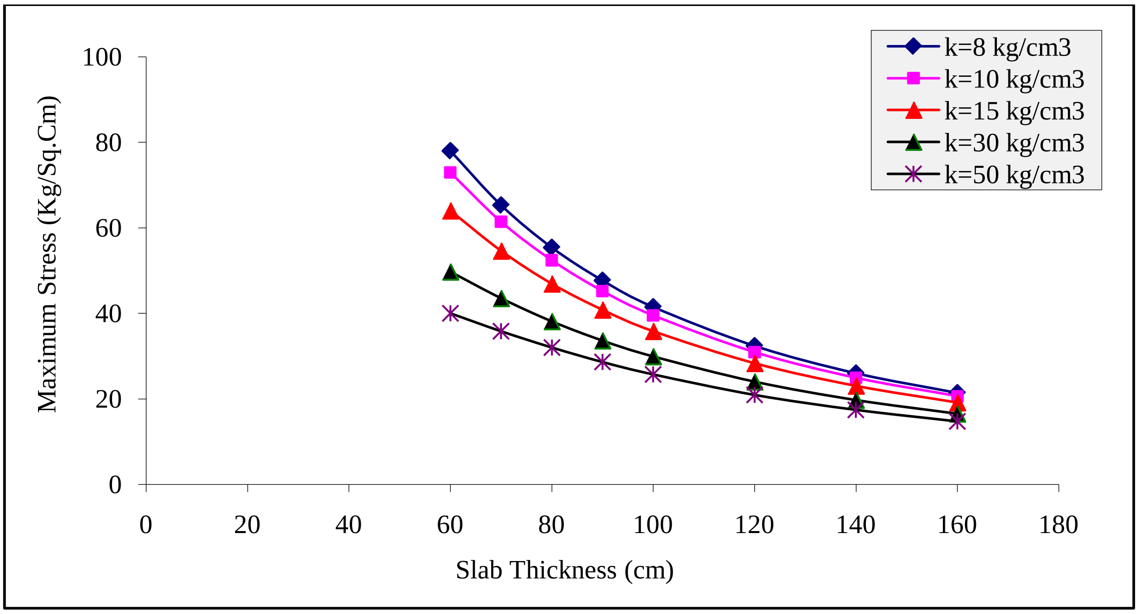

Fig. 6.72: Relationship between Wheel Load Stress and Slab Thickness for different k values for 3.2T wheel load

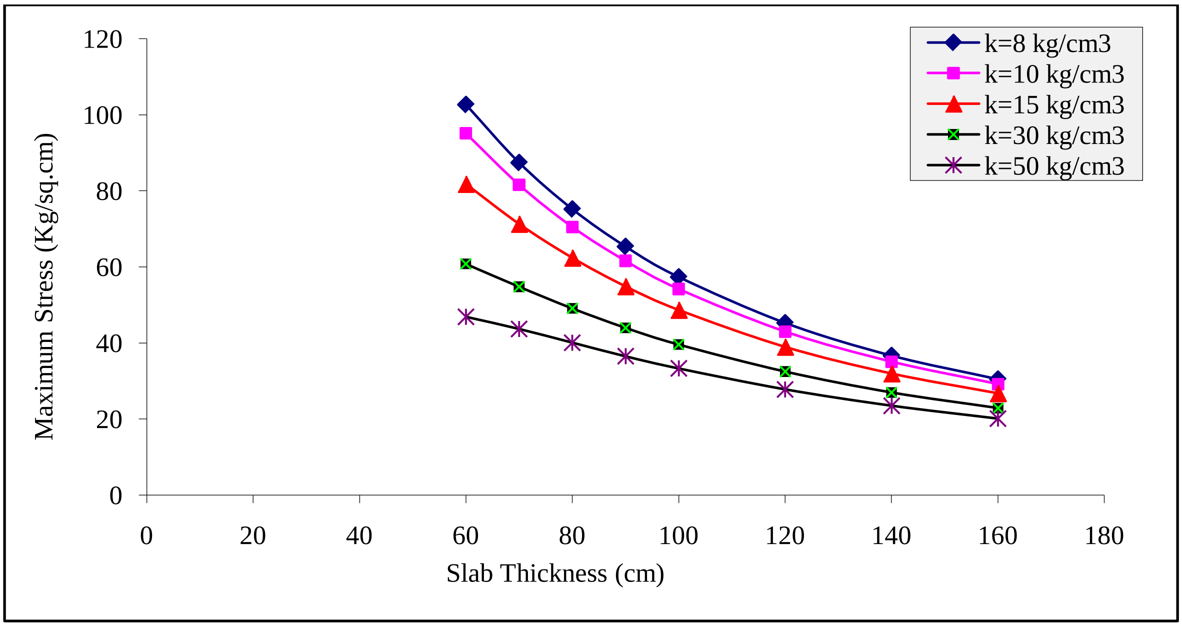

Fig. 6.73: Relationship between Wheel Load Stress and Slab Thickness for different k values for 5.1 T wheel load

6.12 APPLICATION OF FUZZY LOGIC

Model was developed for deflection using fuzzy rule based techniques and compared with the nonlinear regression model already developed. A fuzzy rule-based system was developed to represent the expert knowledge using fuzzy rules ‘if-then’. The fuzzy method that was used in the linguistic fuzzy model **(Mamdani method)** was adopted. The system parameters were represented by linguistic fuzzy sets and corresponding rules of the form 'If Age is Young and Pothole initiation is Very good……. then Pothole is Very good' are developed which represent the impact of the influencing factors on distress severity.

6.12.1 Fuzzy Rule based Deflection Progression model

The factors selected for prediction of deflection were Modified Structural Number, Vehicle Damage Factor and Initial Deflection. The membership functions were selected based on expert's opinion. A fuzzy rule based system was then developed. The influencing parameters were represented by linguistic fuzzy sets. The linguistic fuzzy sets were low, medium and high. The value range of each variable was divided in to three fuzzy sets with membership function as shown in the **Table 6.10**. The rule base contains 45 rules, which describe the effect of specific combinations. **(Annexure 1)**

Table 6.10 Fuzzy for Deflection Growth

Variable	Attribute	Fuzzy sets
VDF	Low	1/0,1/0,0/2
	Medium	0/1,1/2.5,0/4
	High	0/3,1/5,1/5
MSN	Low	1/2,1/2,0/4
	Medium	0/3,1/4.75,0/6.5
	High	0/5.67,1/7.5,1/7.5
Initial Deflection/ Deflection	Very good	1/0,1/0,0/0.4
	Good	0/0.2,1/0.5,0/0.8
	Fair	0/0.6,1/0.9,0/1.2
	Poor	0/1,1/1.3.0/1.6
	Very poor	0/1.4,1/1.7,1/2,1/2

6.12.2 Non linear Deflection Progression model

The non linear equation developed for deflection progression using SPSS is:

$$\mathrm{DEF} = 0.358 \times \mathrm{DEF_i} + 0.009 \times e^{\mathrm{VDF}} - 0.002 \times e^{\mathrm{MSN}} + 0.653$$

-- (Refer equation 6.6)

$R^2 = 0.879$, SE = 0.107

where,

Def = Deflection at time t in mm

$\mathrm{Def_i}$ = Initial Deflection

VDF = Vehicle damage factor

MSN = Modified Structural Number

6.12.3 Validation and comparison of the Models

The deflection growth model developed was validated with one set of data collected. The final set of data collected was used for the validation purpose. The reliability of the model was checked using T- test. The T-values observed at the 5 % level of significance is well below critical values and hence the models are reliable. Chi-squared test was more significant, but since the number of observations were below five, this test can't be done. The result of validation of the fuzzy rule based and regression model are shown in the **Tables 6.11 and 6.12**. The observed and expected values of deflection are shown in **Fig. 6.74.**

Table 6.11 Comparison of Actual and Regression Predicted Values

Sl.No	Stretch ID	Deflection (mm)		
		Actual	Regression Predicted	Fuzzy Predicted
1	KK- 1	0.82	0.80	0.81
2	KK- 2	1.10	1.10	1.19
3	KK-3	1.88	1.84	1.89
4	VK- 1	0.62	0.66	0.69
5	VK- 2	0.74	0.80	0.69

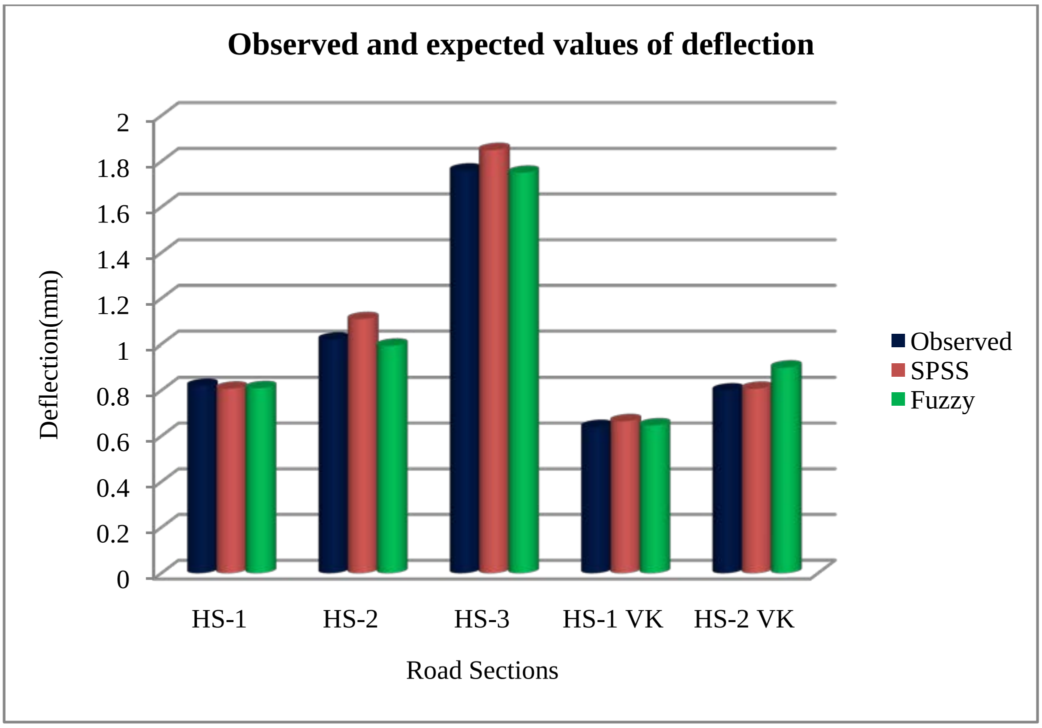

Fig. 6.74: Comparison of models for deflection

Table 6.12 T-test Results of Fuzzy Predicted and Regression values

Models	DF	Calculated values		Critical values
		Fuzzy	Regression	
Deflection Model	4	2.16	1.56	2.776

6.13 FUNCTIONAL CONDITION - RIDING COMFORT INDEX MODEL

The functional performance of a pavement is the ability of a pavement to serve its users in its primary function, which is to provide a safe and smooth riding surface. The most commonly used measure of functional performance of a pavement is its ride quality, which is commonly quantified in terms of the Present Serviceability Rating (PSR), Present Serviceability Index (PSI), Riding Comfort Index (RCI) and Ride Number (RN).

RCI is a function of pavement unevenness and is measured on a scale of 0 to 5. RCI of zero indicates well-constructed new pavement and five represents an extremely rough pavement. If the unevenness index value of a particular pavement stretch is known, the RCI value can be obtained directly. Initially the ride was measured subjectively assigning a scale of 1 to 10. The Portable Universal Roughness Device was used to measure ride quality from 1984 to 1998 using a scale of 1 to 10. Since 1997, IRI has been used as a measure of pavement roughness. The ride characteristics of a pavement can be objectively measured by commercially available equipment, which measures the longitudinal profile of the pavement surface. The profile data is then used to calculate an International Roughness Index (IRI). Roughness measurements are correlated to an assessment of ride quality as determined from **Table 6.13**. This ride quality indicator is the Riding Comfort Index (RCI). If the unevenness index value of a particular pavement stretch is known, the RCI value can be obtained directly.

Table 6.13 Riding Comfort Index Values

Unevenness Index (mm/km)	Riding Comfort Index (RCI)
< 2500	0
2500 - 3500	1
3500 - 5000	2
5000 - 7000	3
7000 - 10000	4
> 10000	5

6.13.1 RCI-Unevenness relationship of the study stretches

Analysis of data using regression analysis showed that the logarithmic regression model provided the best fit for the relationship between Riding Comfort Index and Unevenness Index. The logarithmic regression model takes the form,

$y = a \ln(x) + b$ where,

where,

a & b = Coefficients determined by method of least squares and

y = Riding Comfort Index, RCI

x = Unevenness Index (mm/km), UI

Fig. 6.75 to Fig. 6.87 shows the scatter plot diagram between Unevenness Index (x-axis) and Riding Comfort Index (y-axis) for the study stretches which was plotted with the data for all stretches together and for individual stretches.

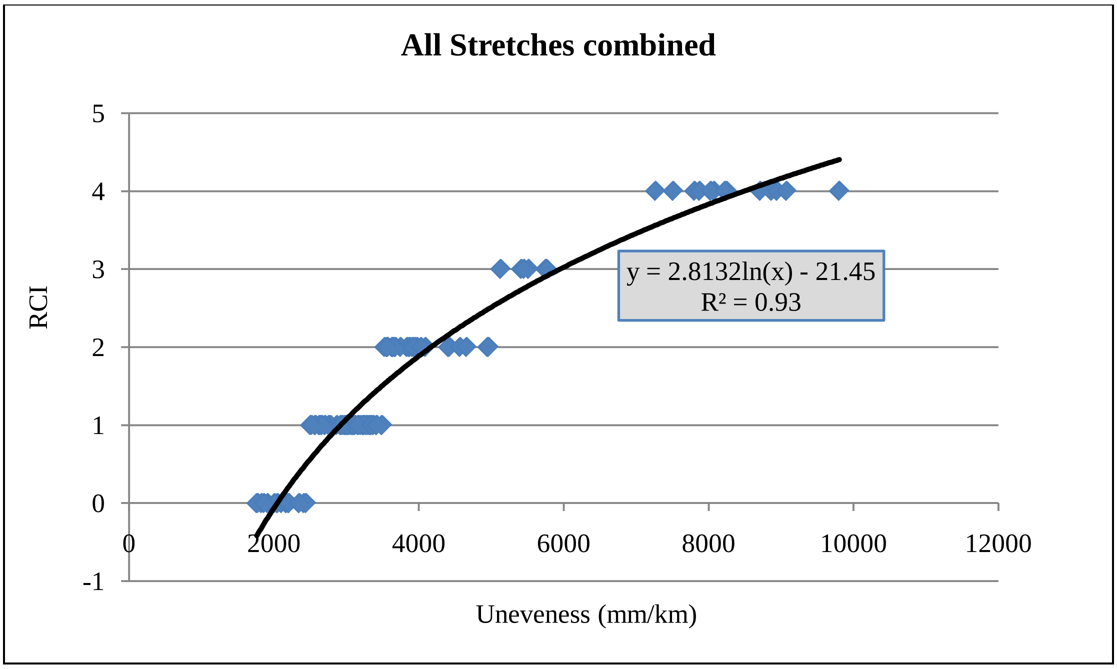

Fig. 6.75: Relationships between Unevenness and Riding Comfort Index (RCI) Combined

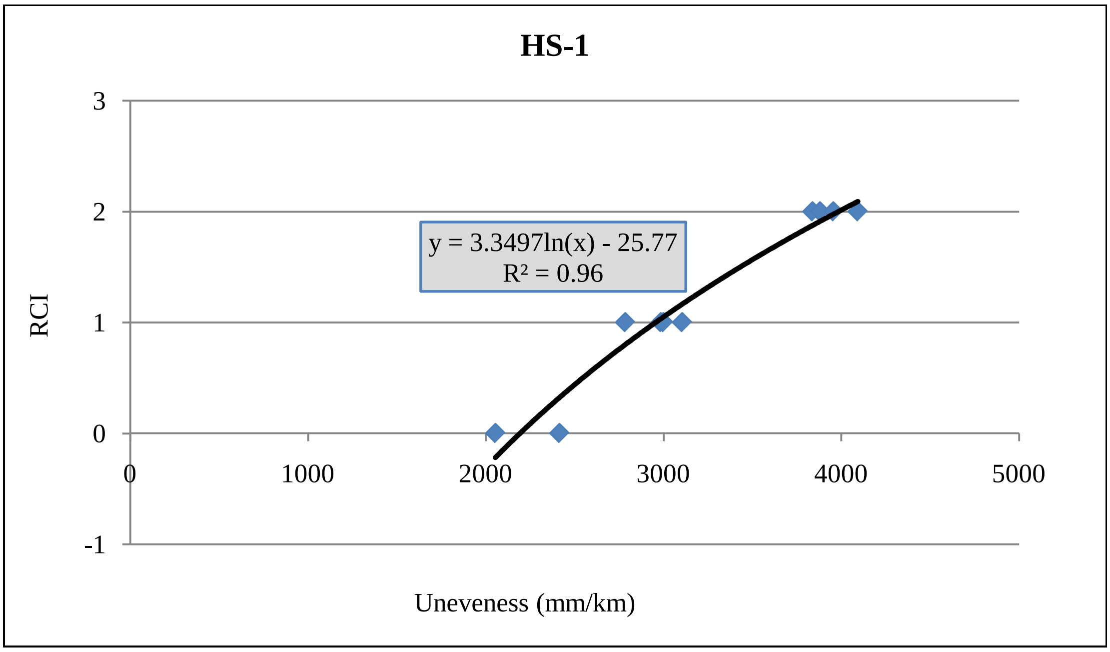

Fig. 6.76: Relationships between Unevenness and Riding Comfort Index (RCI) for Chavadimukku –Pallippuram HS-1

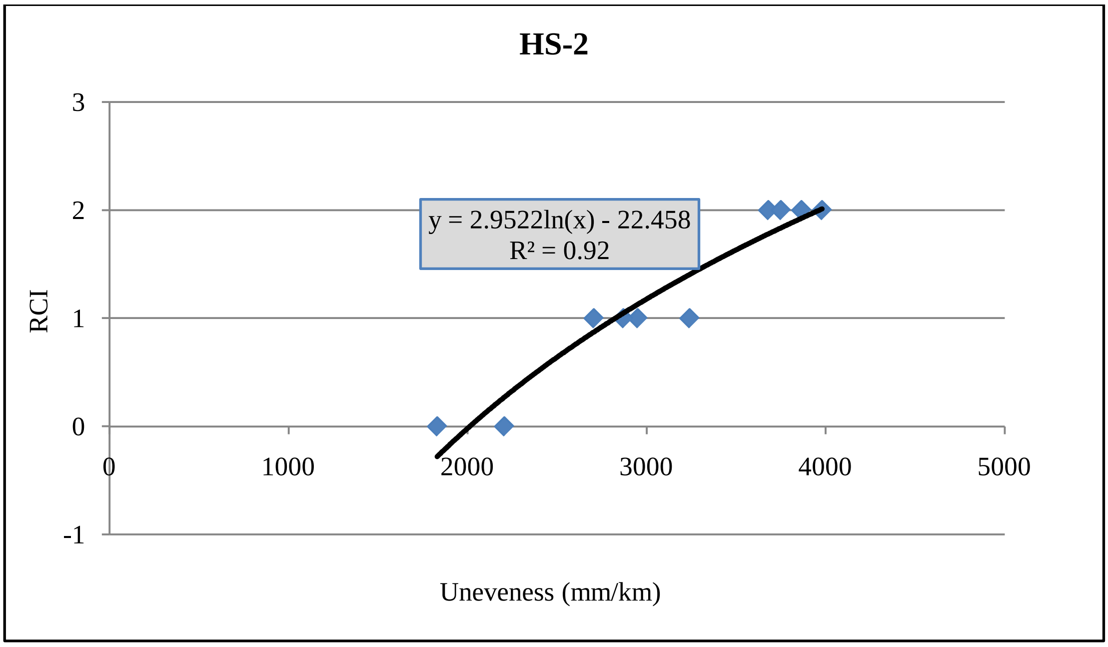

Fig. 6.77: Relationship between Unevenness and Riding Comfort Index (RCI) for Chavadimukku-Pallippuram HS-2

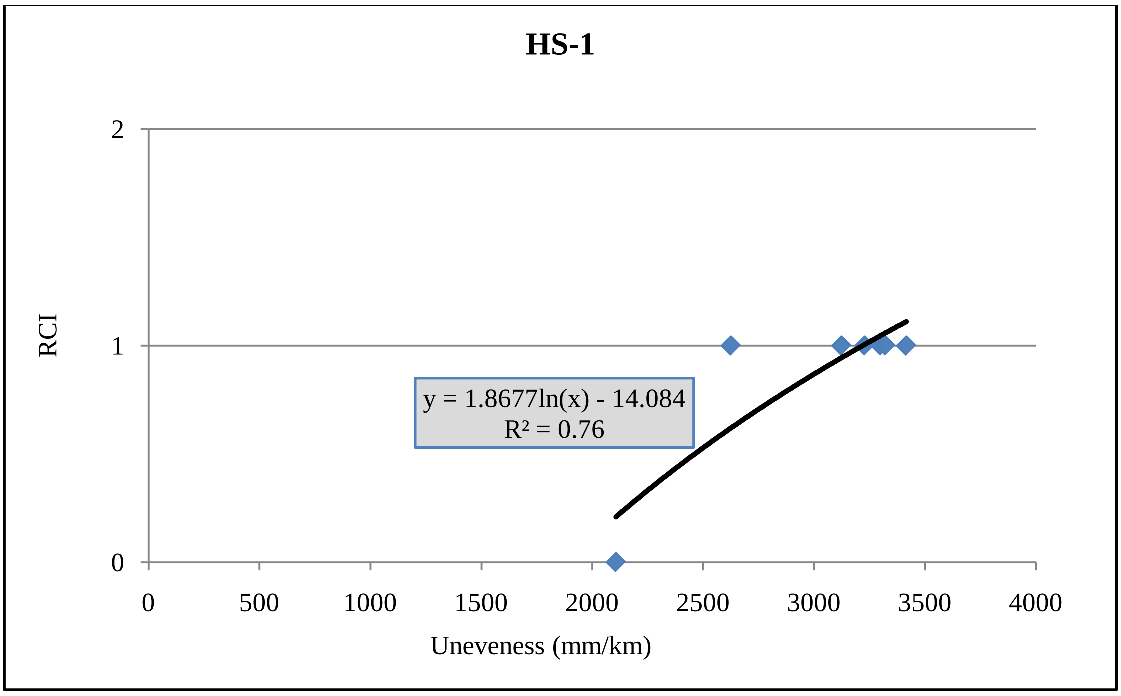

Fig. 6.78: Relationships between Unevenness and Riding Comfort Index (RCI) for Varkala-Kallambalam HS-1

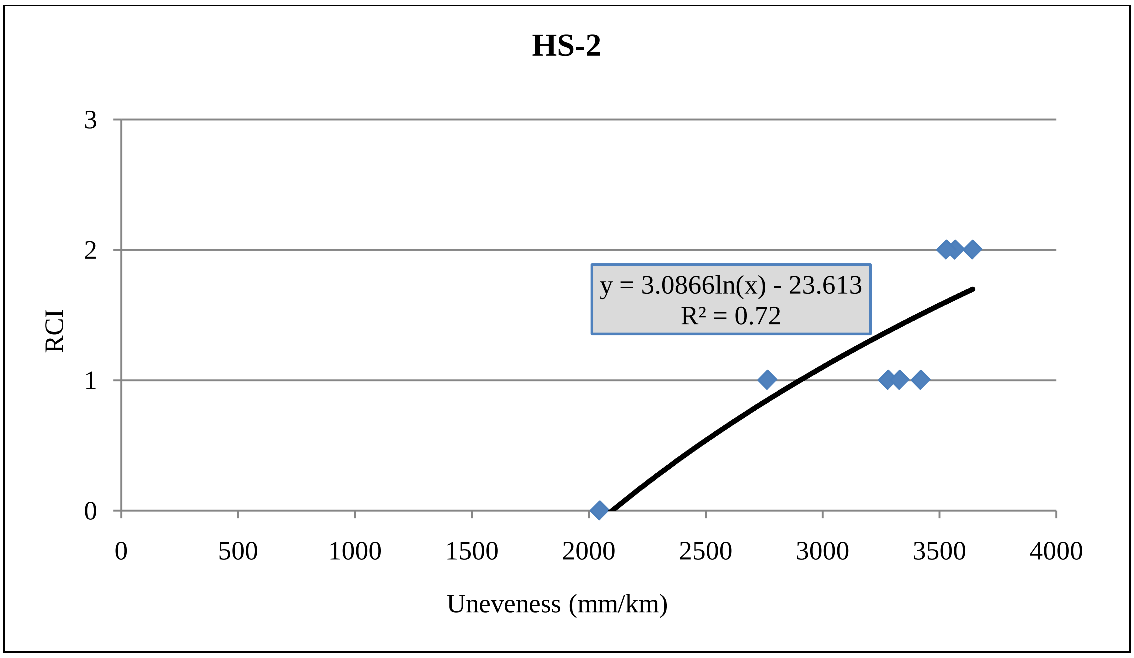

Fig. 6.79: Relationships between Unevenness and Riding Comfort Index (RCI) for Varkala-Kallambalam HS-2

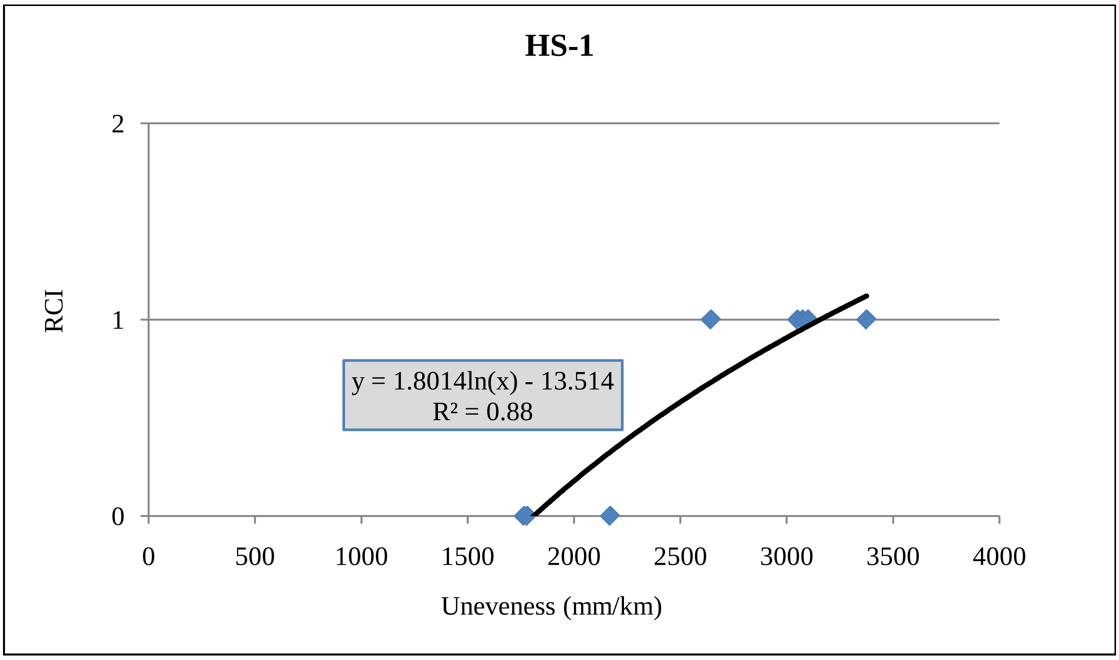

Fig. 6.80: Relationships between Unevenness and Riding Comfort Index (RCI) for Mannanthala-Venjarammoodu HS-1

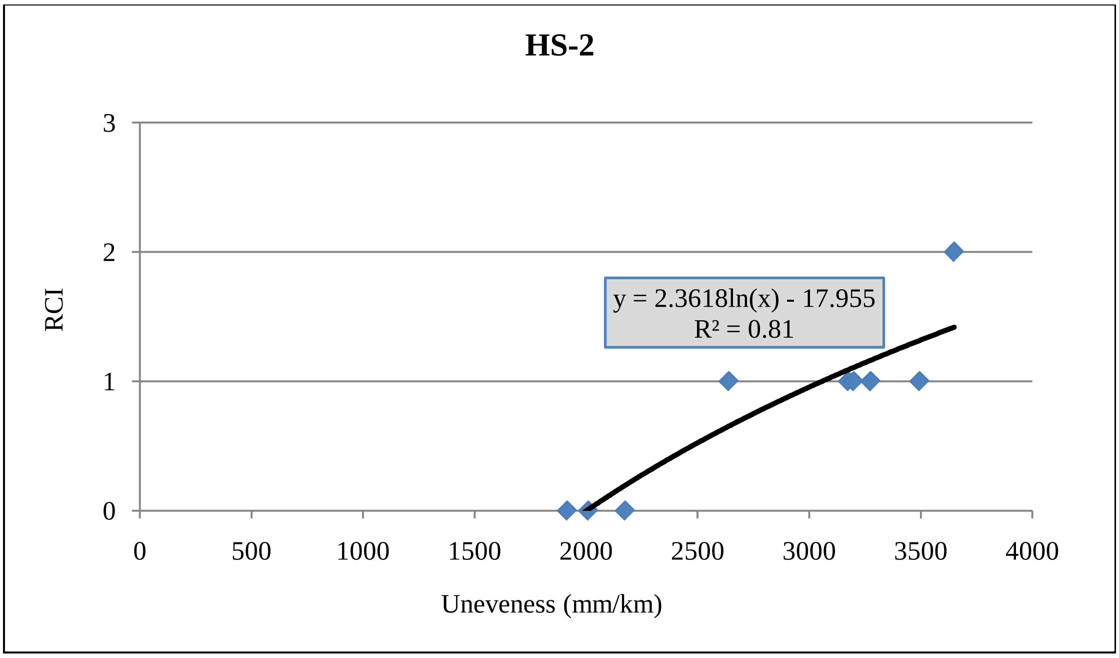

Fig. 6.81: Relationships between Unevenness and Riding Comfort Index (RCI) for Mannanthala-Venjarammoodu HS-2

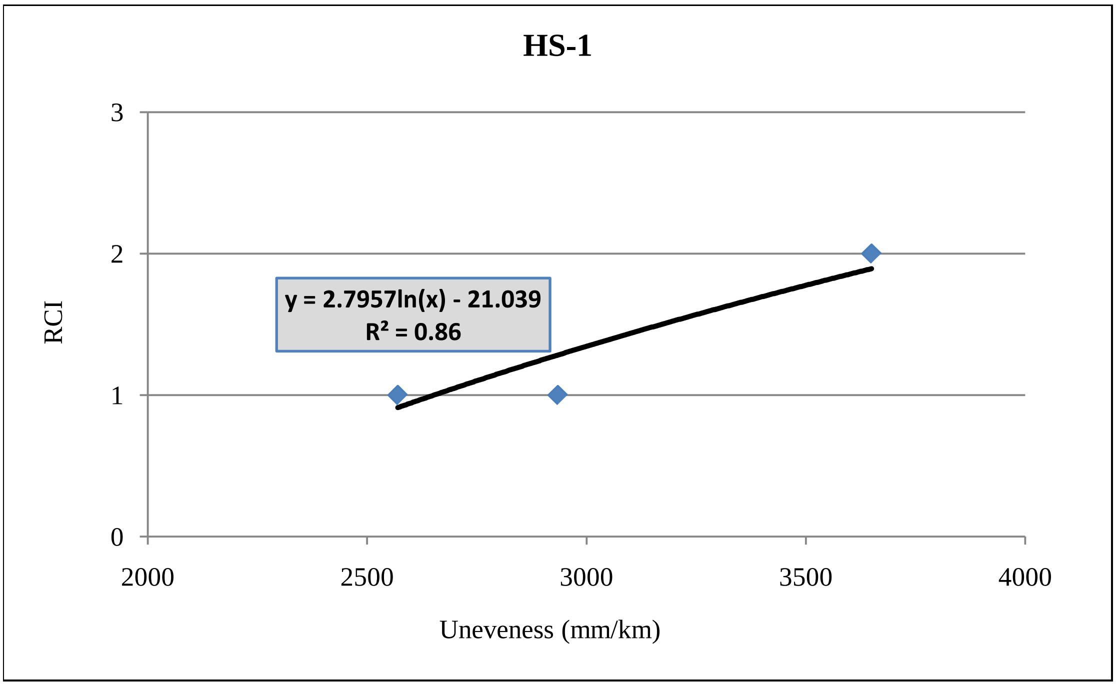

Fig. 6.82: Relationships between Unevenness and Riding Comfort Index (RCI) for Attingal-Kallamballam HS-1

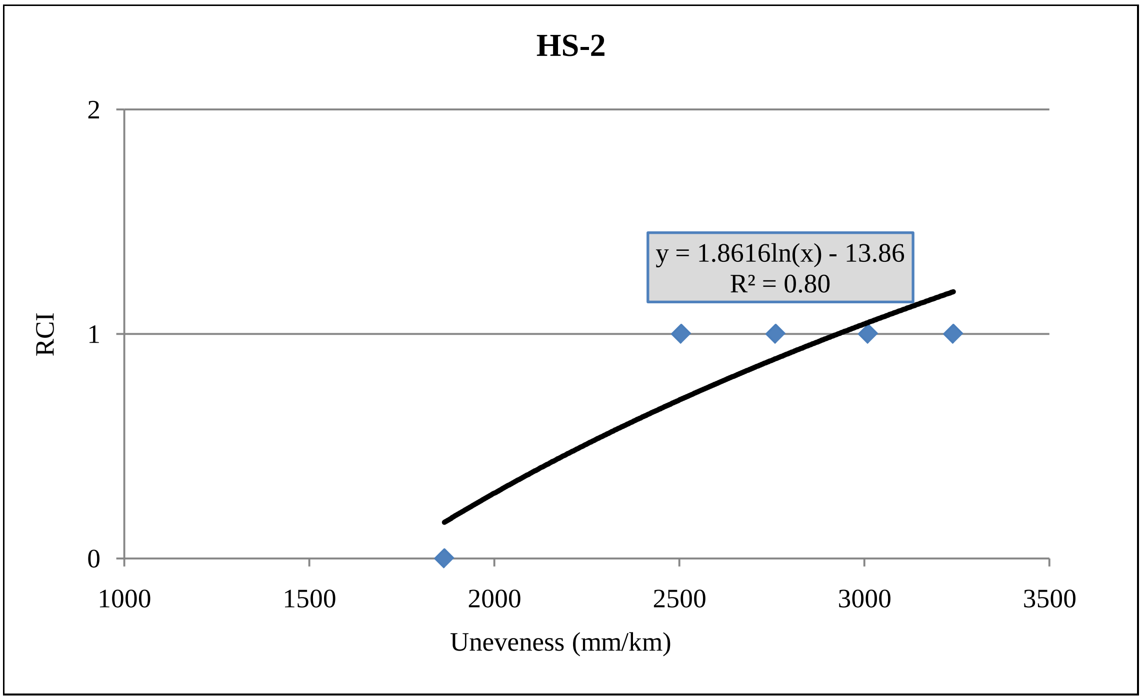

Fig. 6.83: Relationships between Unevenness and Riding Comfort Index (RCI) for Attingal-Kallamballam HS-2

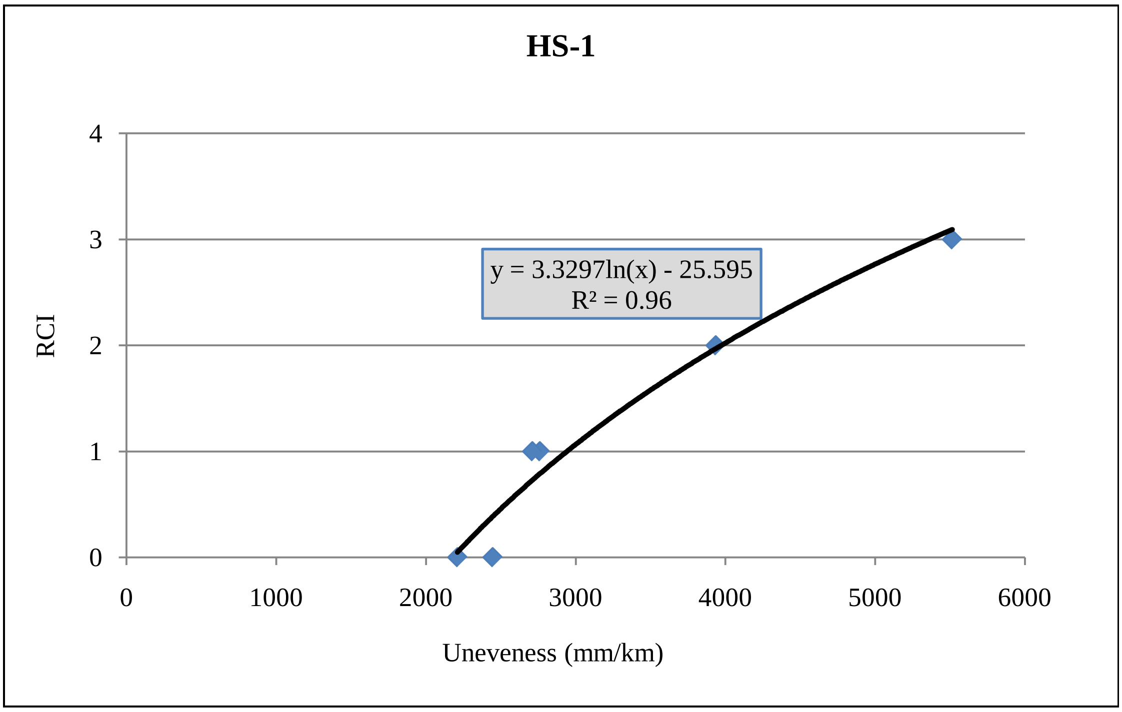

Fig. 6.84: Relationships between Unevenness and Riding Comfort Index (RCI) for Kazhakkuttam-Kovalam HS-1

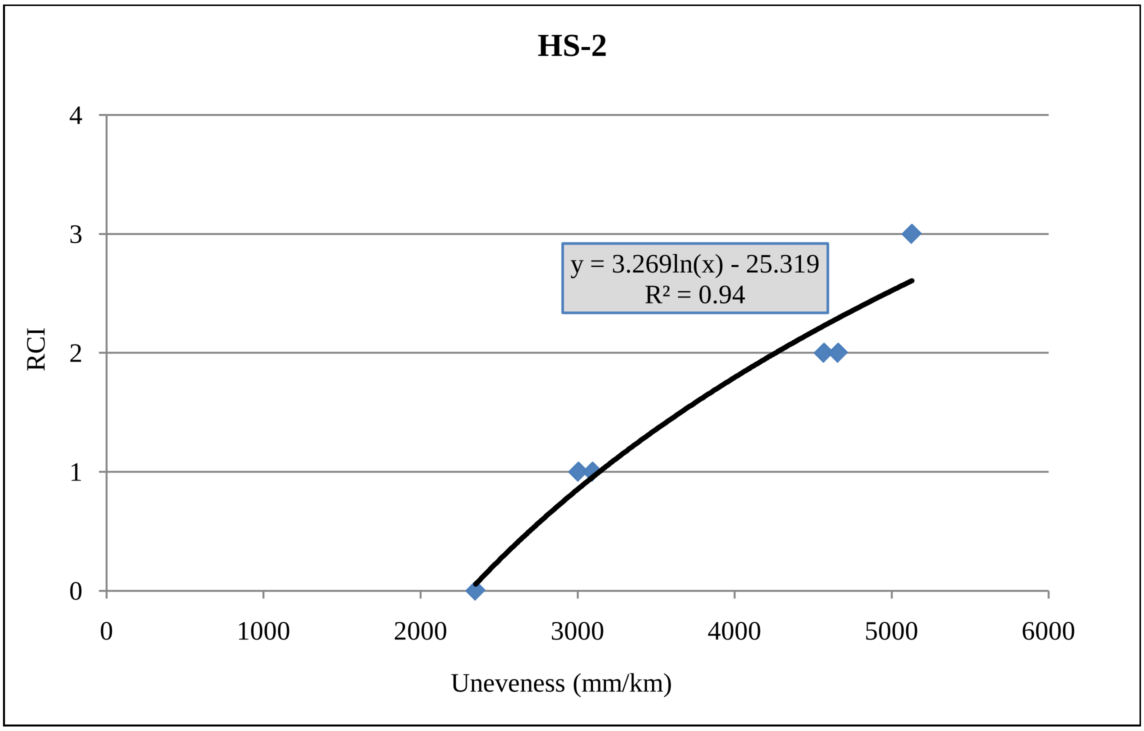

Fig. 6.85: Relationships between Unevenness and Riding Comfort Index (RCI) for Kazhakkuttam-Kovalam HS-2

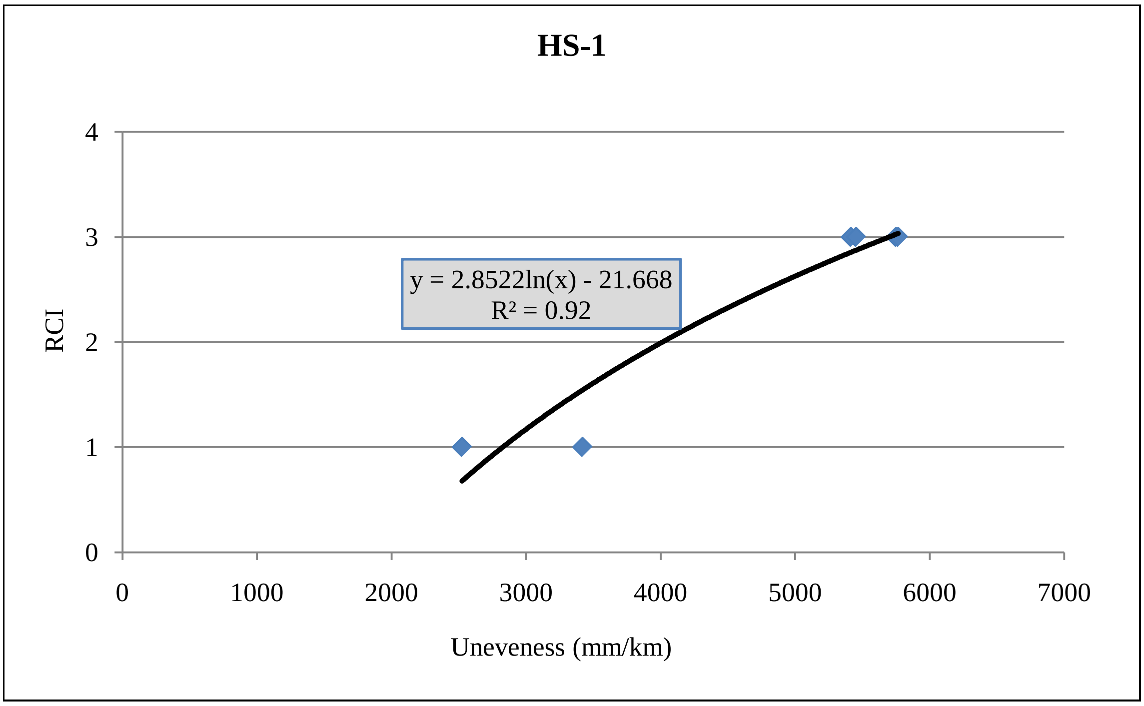

Fig. 6.86: Relationships between Unevenness and Riding Comfort Index (RCI) for Kesavadasapuram-Plamoodu Road

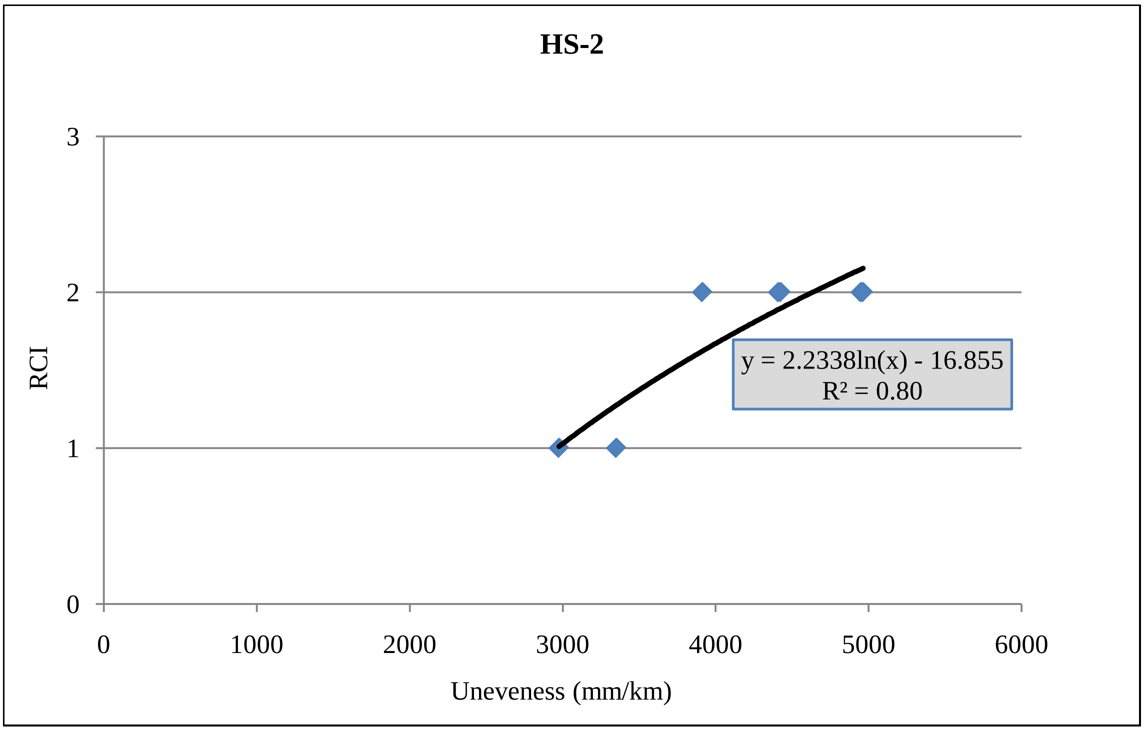

Fig. 6.87: Relationships between Unevenness and Riding Comfort Index (RCI) for Seaport-Airport Road

The logarithmic relationship between Unevenness Index and Riding Comfort Index is listed in **Table 6.14** along with Arithmetic mean, standard deviation and standard error for the relationships.

Table 6.14 Logarithmic relationship between Unevenness and Riding Comfort

Study stretch	Logarithmic relationship between Unevenness Index and Riding Comfort Index	R^2	Riding Comfort Index		
			Mean	Standard Deviation	Standard Error
Combined	RCI = 2.8132 ln(UI) – 21.45	0.93	1.50	1.13	0.10
Chavadimukku-Pallippuram (HS-1)	RCI = 3.3497 ln(UI) - 25.77	0.96	1.20	0.79	0.25
Chavadimukku - Pallippuram (HS-2)	RCI = 2.9522 ln(UI) - 22.458	0.92	1.20	0.79	0.25
Varkala - Kallambalam (HS-1)	RCI = 1.8677 ln(UI) - 14.084	0.76	0.86	0.38	0.14
Varkala - Kallambalam (HS-2)	RCI = 3.0866 ln(UI) - 23.613	0.72	1.25	0.71	0.25
Seaport - Airport Road (HS-2)	RCI = 2.2338 ln(UI) - 16.855	0.80	1.71	0.49	0.18
Mannanthala - Venjarammoodu (HS-1)	RCI = 1.8014 ln(UI) - 13.514	0.88	0.70	0.48	0.15
Mannanthala - Venjarammoodu (HS-2)	RCI = 2.3618 ln()UI - 17.955	0.81	0.80	0.63	0.20
Attingal - Kallamballam (HS-1)	RCI = 2.7957 ln (UI) - 21.039	0.86	1.25	0.50	0.25
Attingal - Kallamballam (HS-2)	RCI = 1.8616 ln(UI) - 13.86	0.80	0.80	0.45	0.20
Kazhakkuttam - Kovalam (HS-1)	RCI = 3.3297 ln(UI) - 25.595	0.96	1.17	1.17	0.48
Kazhakkuttam - Kovalam (HS-2)	RCI = 3.269 ln(UI) - 25.319	0.94	1.50	1.05	0.43
Kesavadasapuram - Plamood	RCI = 2.8522 ln(UI) - 21.668	0.92	2.33	1.03	0.42

6.13.2 Calibration and validation of models

The parameters of regression models estimated and calibrated are found to be statistically significant by various statistical parameters like R^2, t-test, F-test. The internal validation method was adopted for validating the developed regression models. A Student t-test for means is used to evaluate if there exists any significant difference between observed and estimated Riding Comfort Index values. The calculated t-values in all cases were found to be lower than the critical t values obtained from statistical tables at the 5% level of significance and are shown in **Table 6.15.** Hence, it can be concluded that there exists no significant difference between observed and estimated RCI values in all study stretches.

Scatter plot diagram showing comparison between observed values (x-axis) and predicted values (y-axis) of each model are shown in **Fig. 6.88** to **6.100.** A 45-degree reference line was also plotted.

Table 6.15 Student's t-test

Study stretch	Logarithmic relationship between Unevenness Index and Riding Comfort Index	t test values	
		Calculated absolute value	Tabular value
Combined	RCI = 2.8132 ln(UI) – 21.45	0.0113	1.9796
Chavadimukku-Pallippuram (HS-1)	RCI = 3.3497 ln(UI) - 25.77	0.0060	2.2622
Chavadimukku - Pallippuram (HS-2)	RCI = 2.9522 ln(UI) - 22.458	0.0019	2.2622
Varkala - Kallambalam (HS-1)	RCI = 1.8677 ln(UI) - 14.084	1.0734	2.4469
Varkala - Kallambalam (HS-2)	RCI = 3.0866 ln(UI) - 23.613	0.0001	2.3646
Seaport - Airport Road	RCI = 2.2338 ln(UI) - 16.855	0.7310	2.4469
Mannanthala - Venjarammoodu (HS-1)	RCI = 1.8014 ln(UI) - 13.514	0.0053	2.2622
Mannanthala - Venjarammoodu (HS-2)	RCI = 2.3618 ln(UI) - 17.955	0.0016	2.2622
Attingal - Kallamballam (HS-1)	RCI = 2.7957 ln (UI() - 21.039	0.9593	3.1825
Attingal - Kallamballam (HS-2)	RCI = 1.8616 ln(UI) - 13.86	0.0069	2.7765
Kazhakkuttam - Kovalam (HS-1)	RCI = 3.3297 ln(UI) - 25.595	0.0025	2.5706
Kazhakkuttam - Kovalam (HS-2)	RCI = 3.269 ln(UI) - 25.319	0.0022	2.5706
Kesavadasapuram - Plamood (HS-1)	RCI = 2.8522 ln(UI) - 21.668	0.0055	2.5706

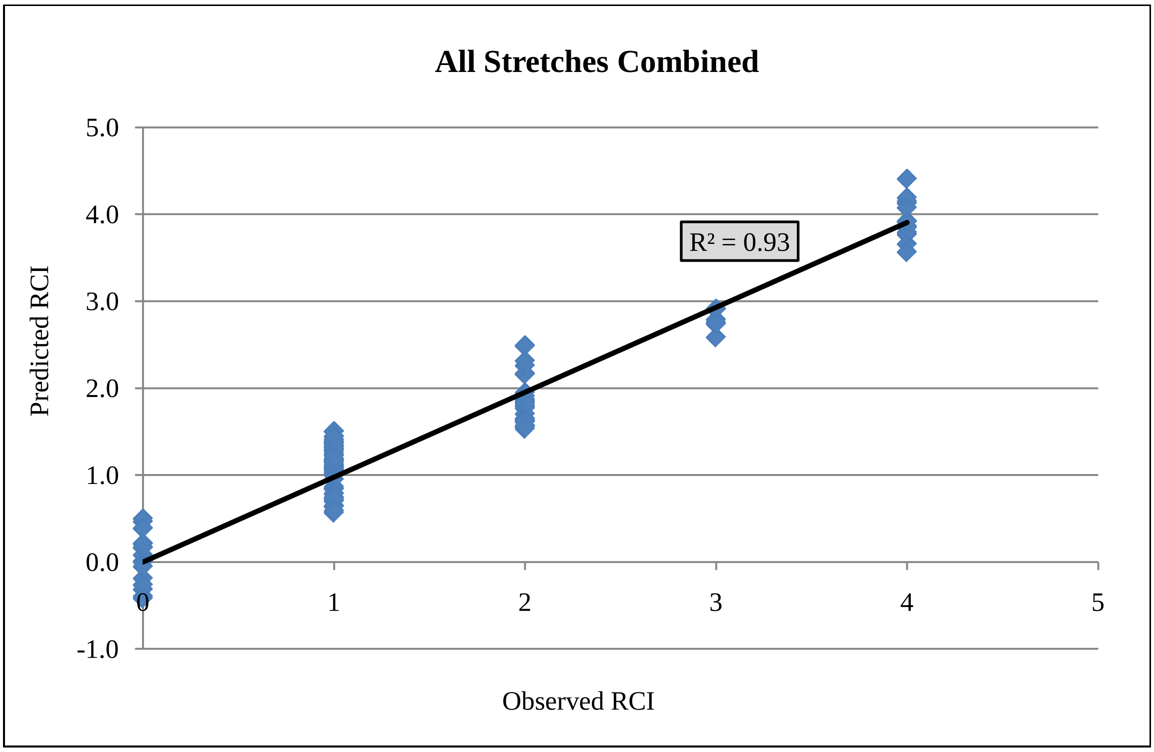

Fig. 6.88: Comparison of observed and predicted values for combined data

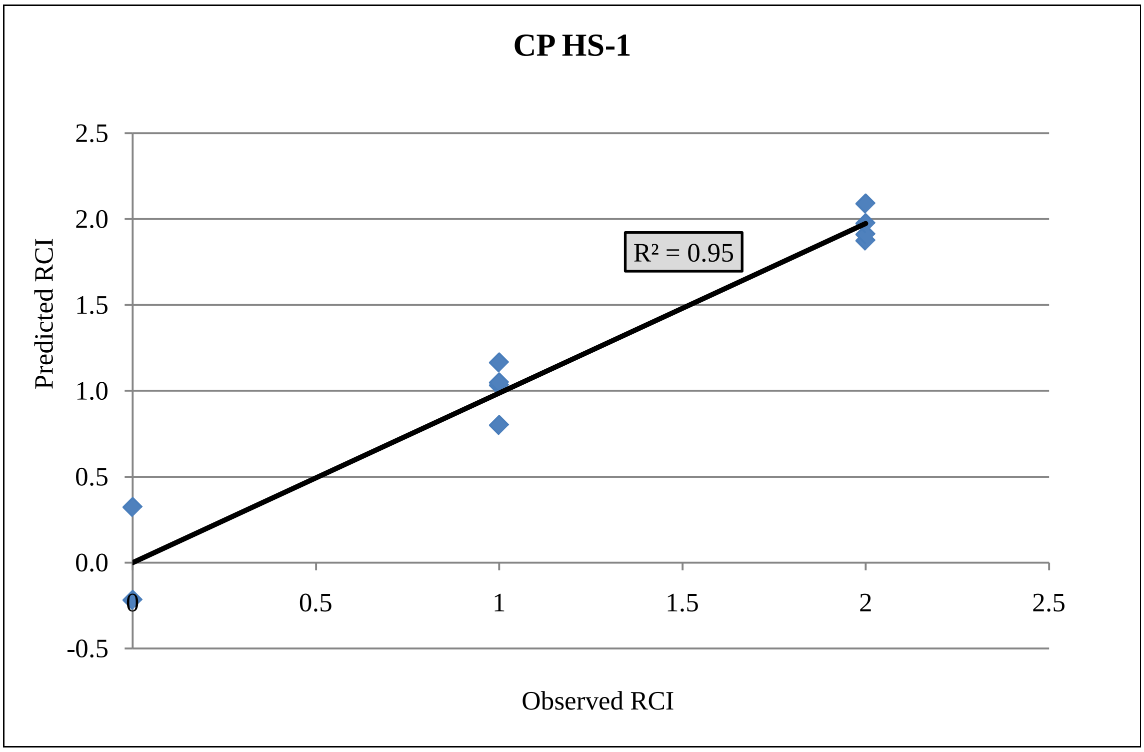

Fig. 6.89: Comparison of observed and predicted values of RCI for Chavadimukku-Pallippuram Road (HS-1)

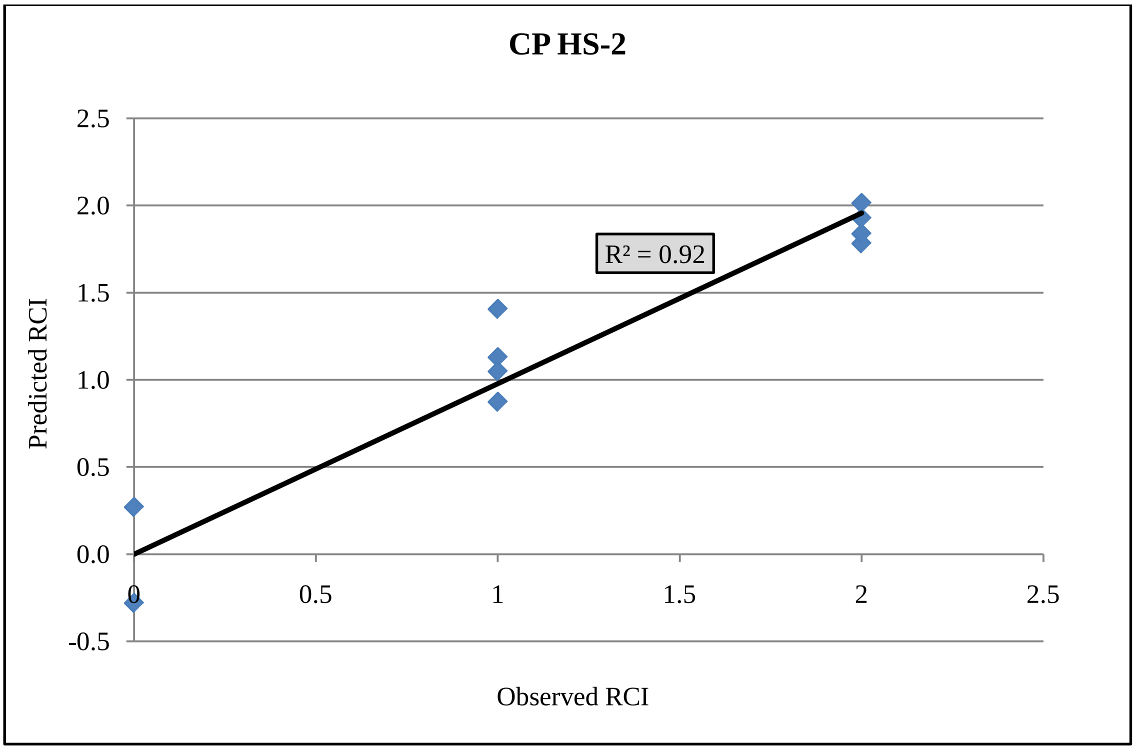

Fig. 6.90: Comparison of observed and predicted values of RCI for Chavadimukku-Pallippuram Road (HS-2)

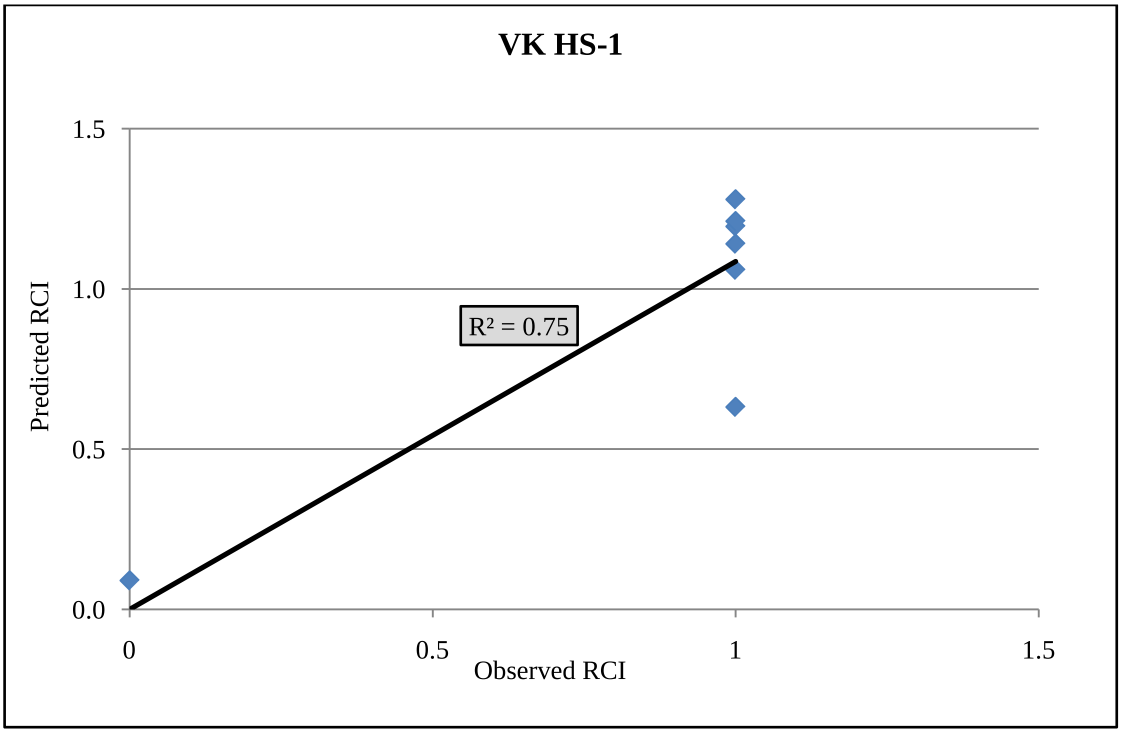

Fig. 6.91: Comparison of observed and predicted values of RCI for Varkala Kallambalam Road (HS-1)

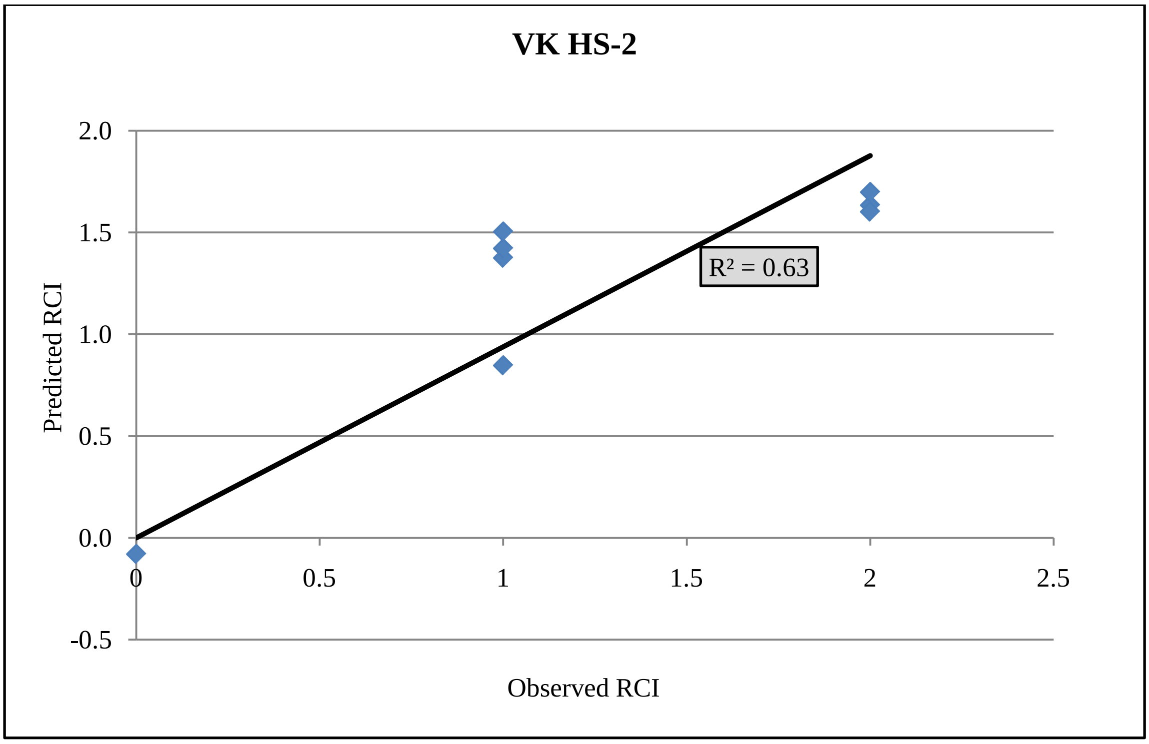

Fig. 6.92: Comparison of observed and predicted values of RCI for Varkala-Kallambalam Road (HS-2)

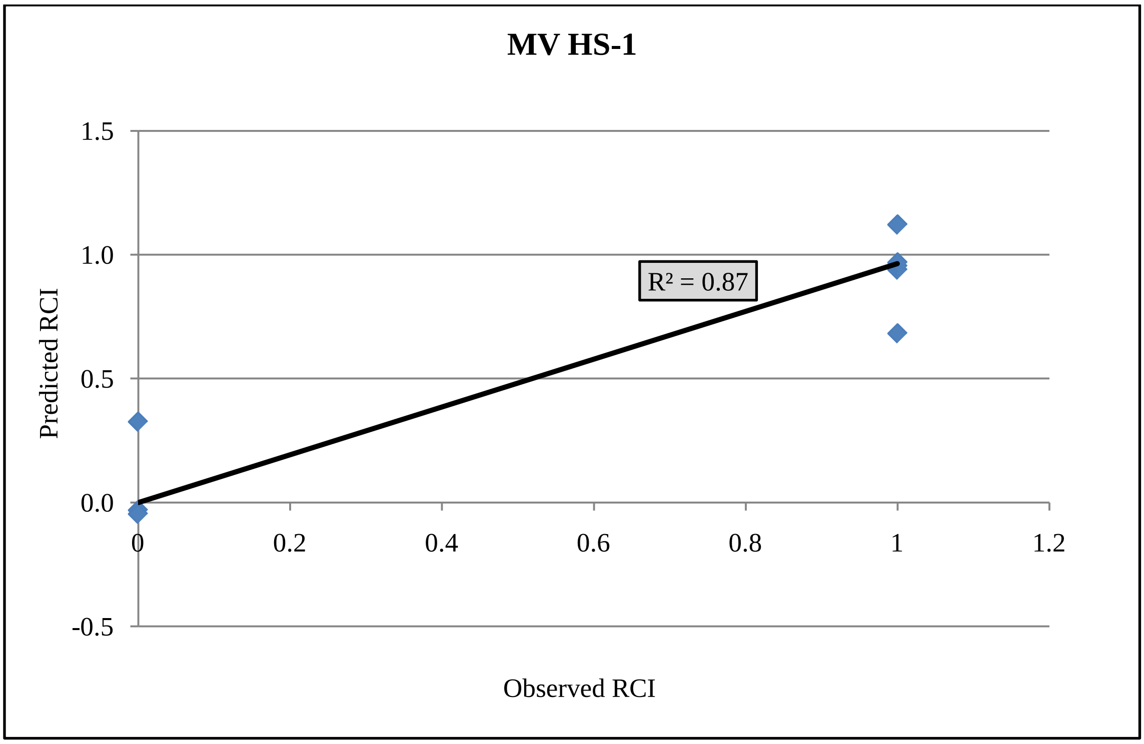

Fig. 6.93: Comparison of observed and predicted values of RCI for Mannanthala-Venjarammoodu Road (HS-1)

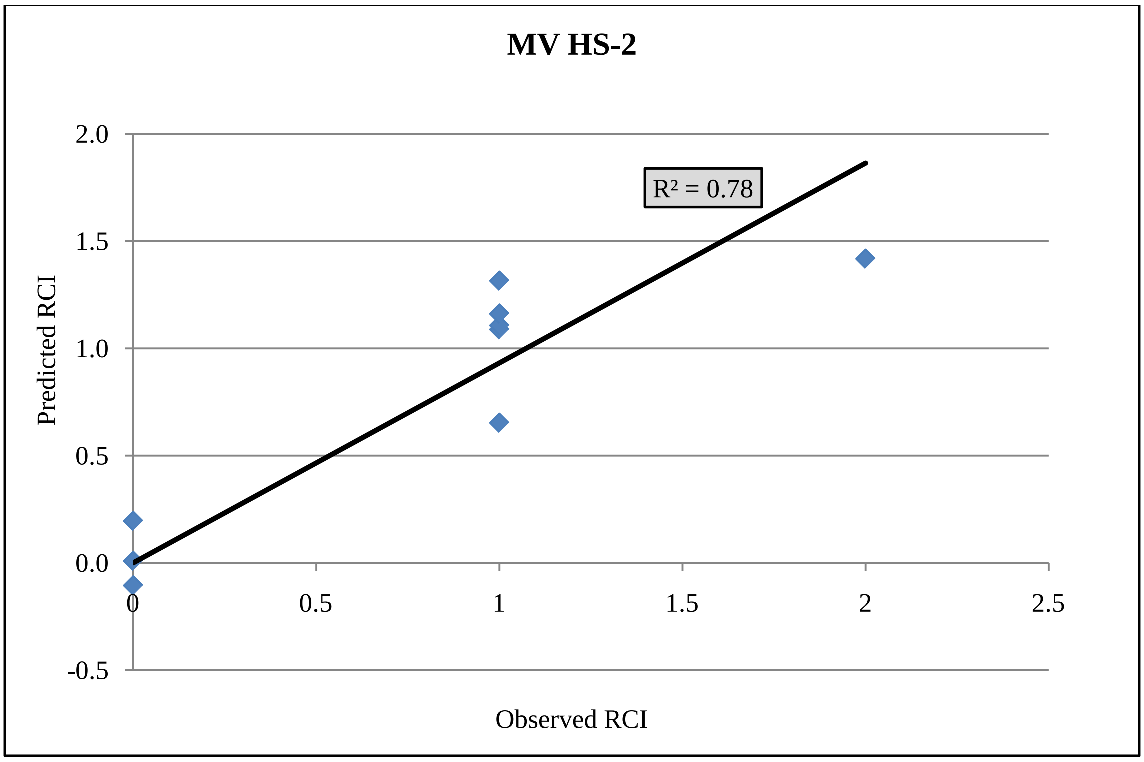

Fig. 6.94: Comparison of observed and predicted values of RCI for Mannanthala-Venjarammoodu Road (HS-2)

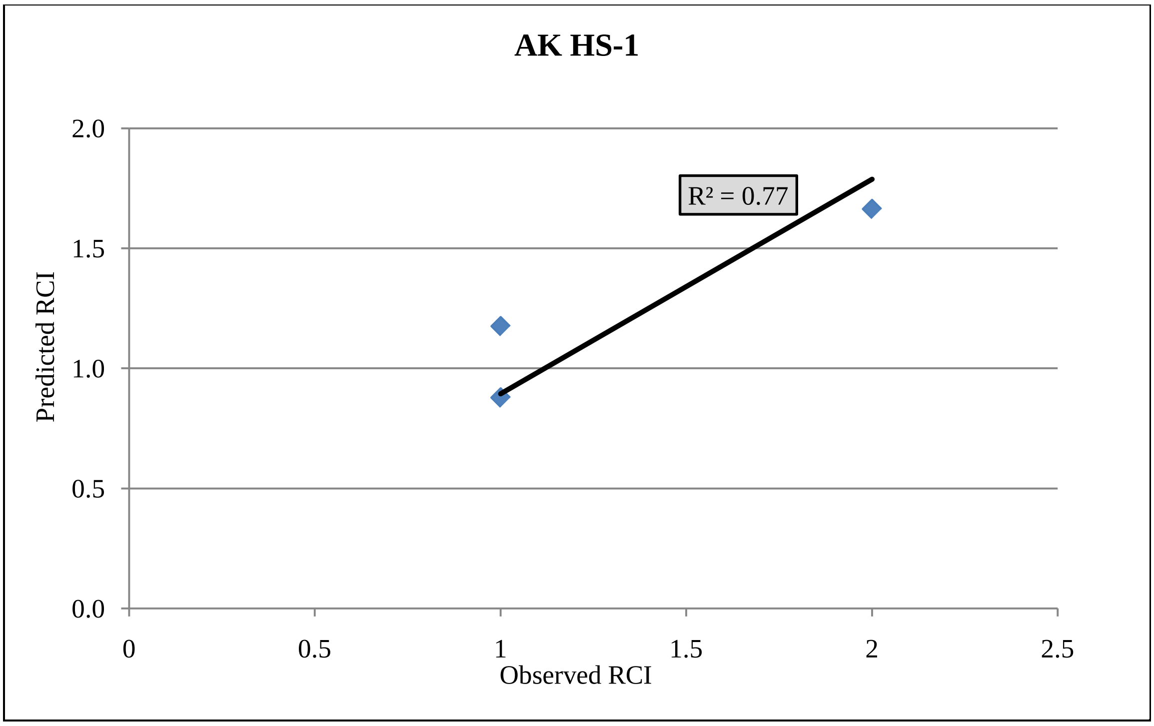

Fig. 6.95: Comparison of observed and predicted values of RCI for Attingal-Kallamballam Road (HS-1)

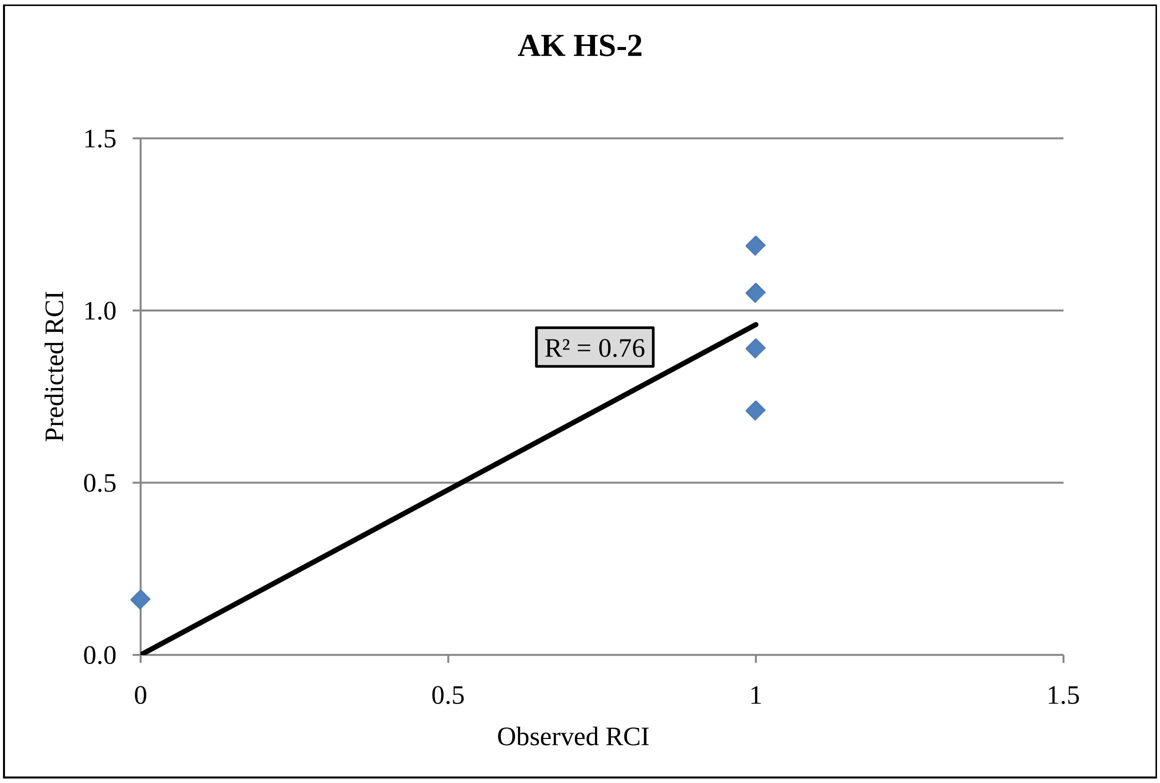

Fig. 6.96: Comparison of observed and predicted values of RCI for Attingal-Kallamballam Road (HS-2)

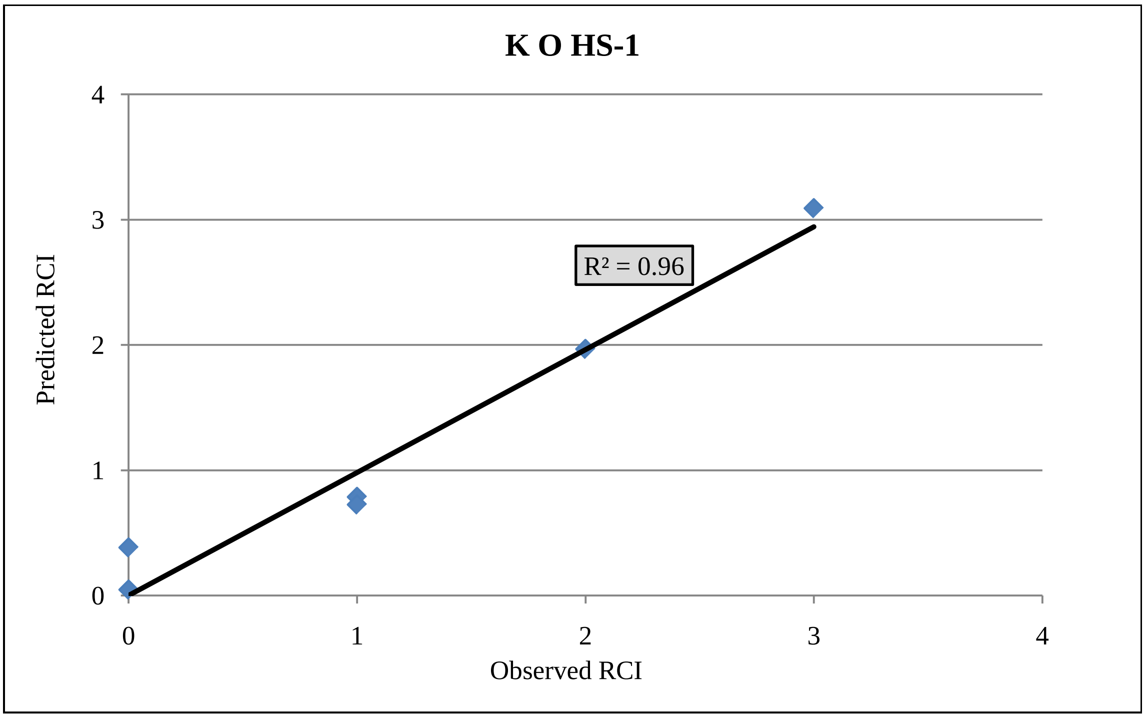

Fig. 6.97: Comparison of observed and predicted values of RCI for Kazhakkuttam-Kovalam Road (HS-1)

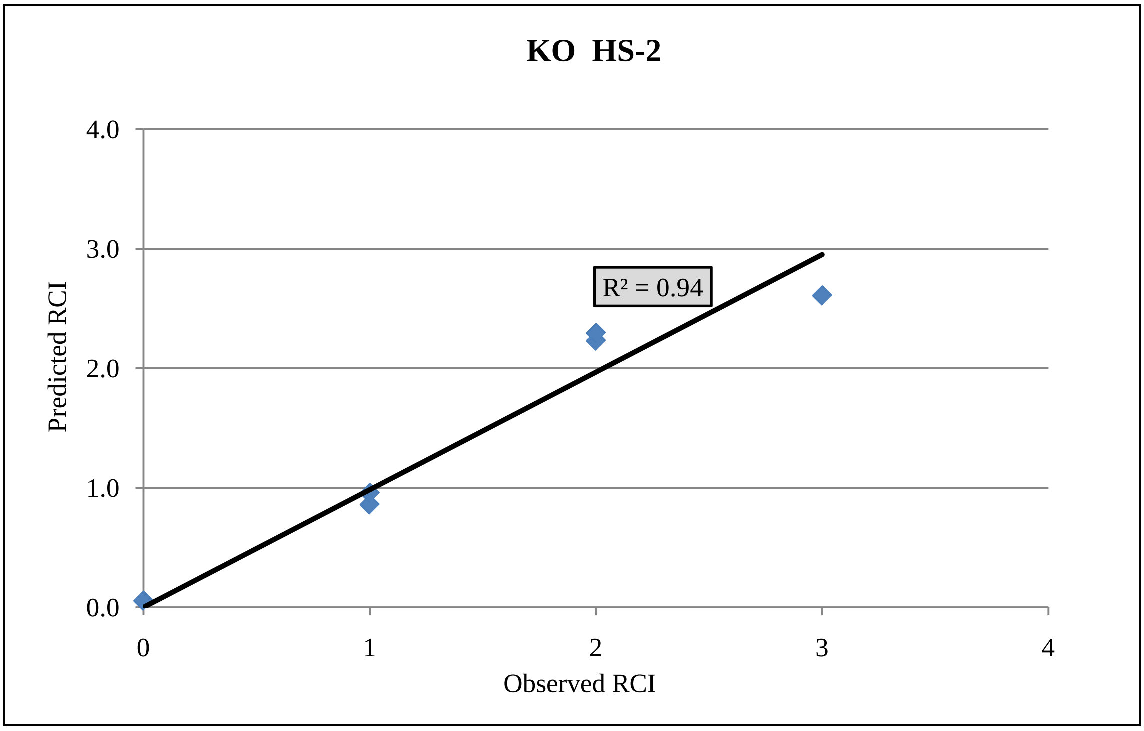

Fig. 6.98: Comparison of observed and predicted values of RCI for Kazhakkuttam-Kovalam Road (HS-2)

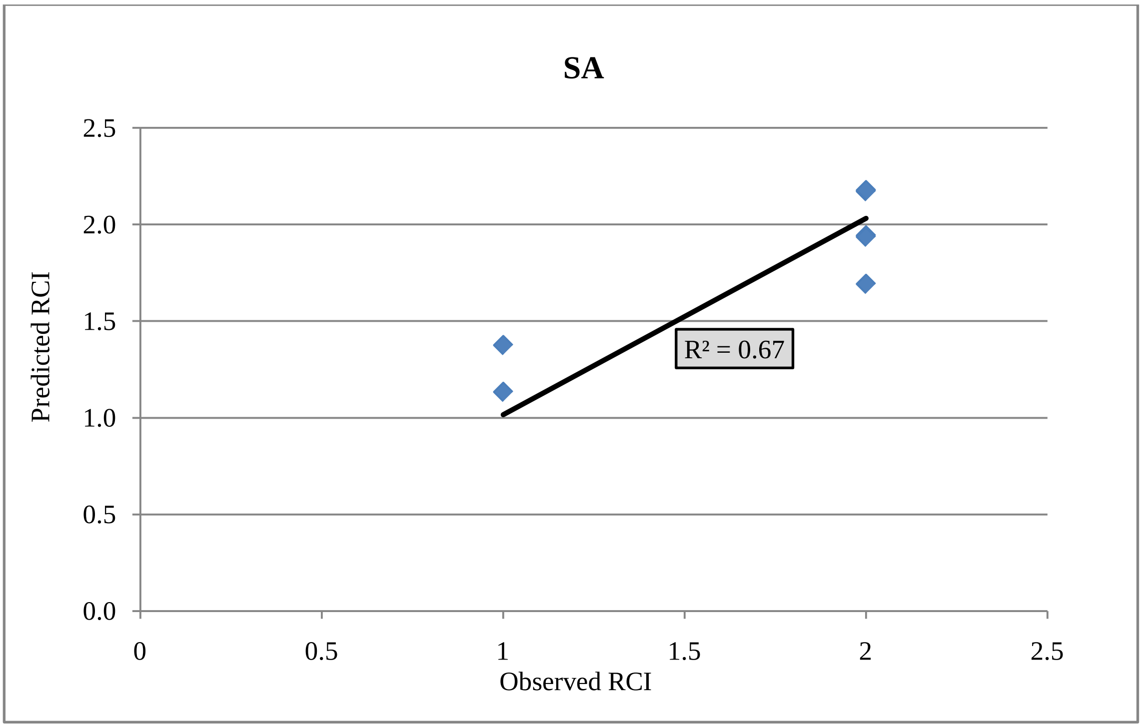

Fig. 6.99: Comparison of observed and predicted values of RCI for Seaport-Airport Road

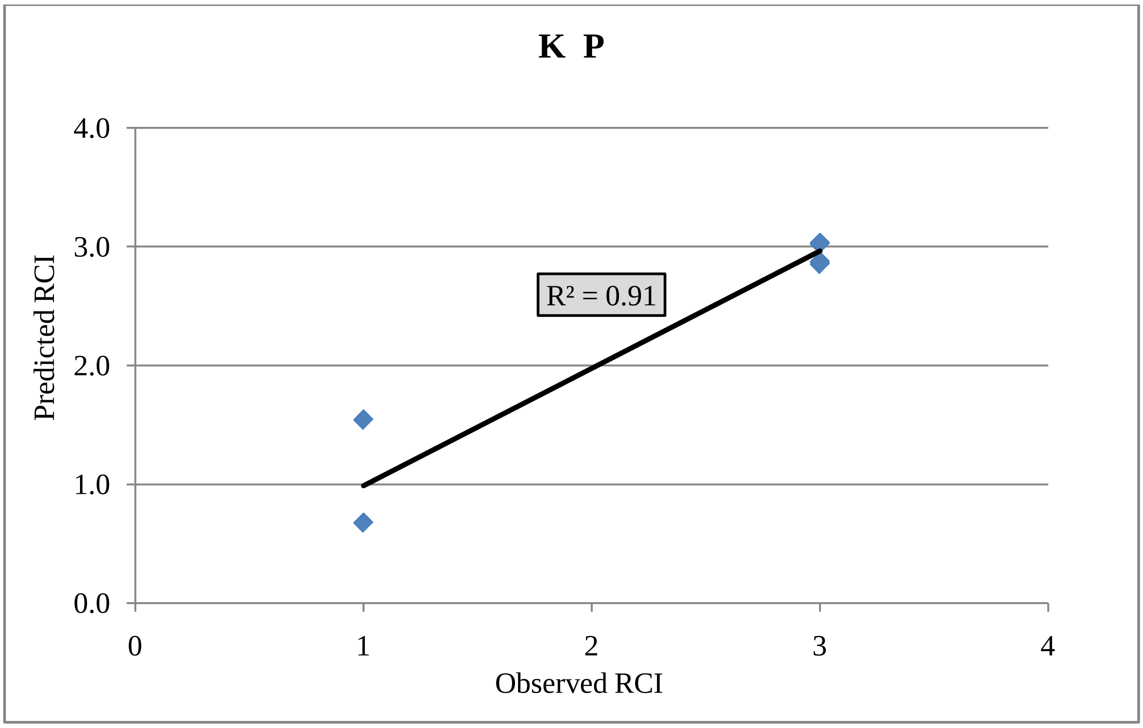

Fig. 6.100: Comparison of observed and predicted of RCI values for Kesavadasapuram- Plamood Road

The models developed as part of the regression analysis were used in validating the relationship between the variables, Unevenness and Riding Comfort Index (RCI). The average prediction error (APE) and total prediction error (TPE) for characteristic deflection were calculated and are tabulated in **Table 6.16**

The average prediction errors (APE) for Riding Comfort Index values were found to be varied between -6.663% and 4.562% and the total prediction error (TPE) for Riding Comfort Index values was found to be varied between -0.028% and 1.173%.

Table 6.16 Validation of the Regression Analysis

Study stretch	Relationship between Unevenness Index and Riding Comfort Index	Average Prediction Error (%)	Prediction Error of Totals (%)
Combined	RCI = 2.8132 ln(UI) – 21.45	2.558	0.02
Chavadimukku-Pallippuram (HS-1)	RCI = 3.3497 ln(UI) - 25.77	-0.395	-0.027
Chavadimukku - Pallippuram (HS-2)	RCI = 2.9522 ln(UI) - 22.458	2.922	0.011
Varkala - Kallambalam (HS-1)	RCI = 1.8677 ln(UI) - 14.084	3.069	0.013
Varkala - Kallambalam (HS-2)	RCI = 3.0866 ln(UI) - 23.613	1.828	-0.001
Seaport - Airport Road	RCI = 2.2338 ln(UI) - 16.855	4.562	1.173
Mannanthala - Venjarammoodu (HS-1)	RCI = 1.8014 ln(UI) - 13.514	3.563	-0.040
Mannanthala – Venjarammoodu (HS-2)	RCI = 2.3618 ln(UI) - 17.955	2.783	0.018
Attingal - Kallamballam (HS-1)	RCI = 2.7957 ln (UI() - 21.039	3.364	-0.003
Attingal - Kallamballam (HS-2)	RCI = 1.8616 ln(UI) - 13.86	-4.071	-0.077
Kazhakkuttam - Kovalam (HS-1)	RCI = 3.3297 ln(UI) - 25.595	-6.663	-0.021
Kazhakkuttam - Kovalam (HS-2)	RCI = 3.269 ln(UI) - 25.319	-1.084	0.015
Kesavadasapuram - Plamood	RCI = 2.8522 ln(UI) - 21.668	2.396	-0.028

It was observed in Table 6.16 that the ‘Average Prediction Errors’ and ‘Prediction errors of Totals’ in the estimation of variables are much lesser and reasonably accurate. Hence, the regression equations developed were assumed to be acceptable for future predictions.

6.14 SAFETY RELATED PERFORMANCE MODEL – RELATIONSHIP BETWEEN TEXTURE DEPTH AND SKID RESISTANCE

Analysis of data using regression analysis showed that second degree polynomial regression model provided the best fit for the relationship between texture depth and skid resistance. The polynomial regression model takes the form, $y = ax^2 + bx + c$, where,

a, b and c = Coefficients determined by method of least squares

y = Skid Resistance Number, SN,

x = Texture depth (mm), TD

The polynomial relationship between the variables, Texture depth (x) and Skid Resistance Number (y) for the three study stretches which showed acceptable fit are listed in **Table 6.17. Fig. 6.101 to 6.104** shows the scatter plot diagram showing the polynomial relationship between texture depth and skid resistance number.

Table 6.17 Polynomial relationship between Texture depth (x) and Skid Resistance Number

Study stretch	Polynomial relationship between Texture depth and Skid Resistance Number	R^2	Skid Resistance (mm)		
			Mean	Standard Deviation	Standard Error
Chavadimukku -Pallippuram	$SN = -3509.6\ TD^2 + 1688.1\ TD - 138.59$	0.72	62.67	1.97	0.80
Varkala - Kallambalam	$SN = -76.101\ TD^2 + 100.48\ TD + 26.882$	0.81	57.89	1.54	0.51
Seaport - Airport Road	$SN = -1765\ TD^2 + 1034.3\ TD - 84.349$	0.84	63.67	2.66	1.09
Mannanthala - Venjaramoodu	$SN = 5 \times 10^{-10}\ TD^2 + 121.69\ TD + 7.7421$	0.94	58.00	4.08	2.04

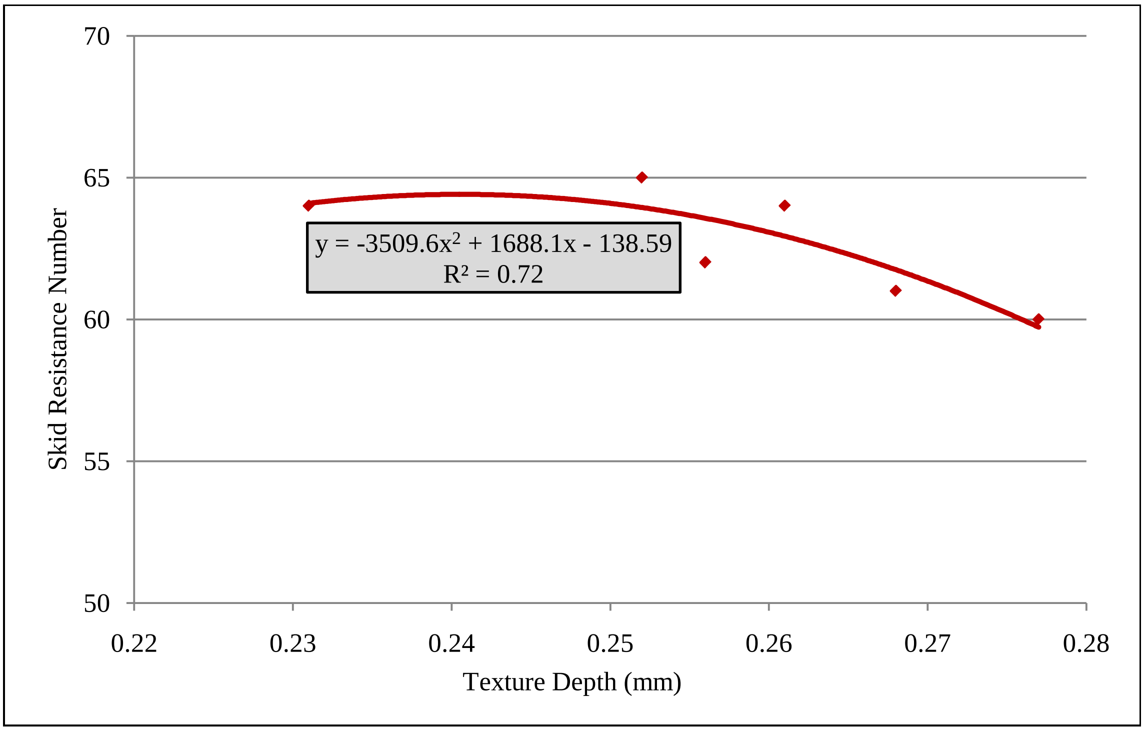

Fig. 6.101: Texture depth vs Skid resistance number for Chavadimukku-Pallippuram Road

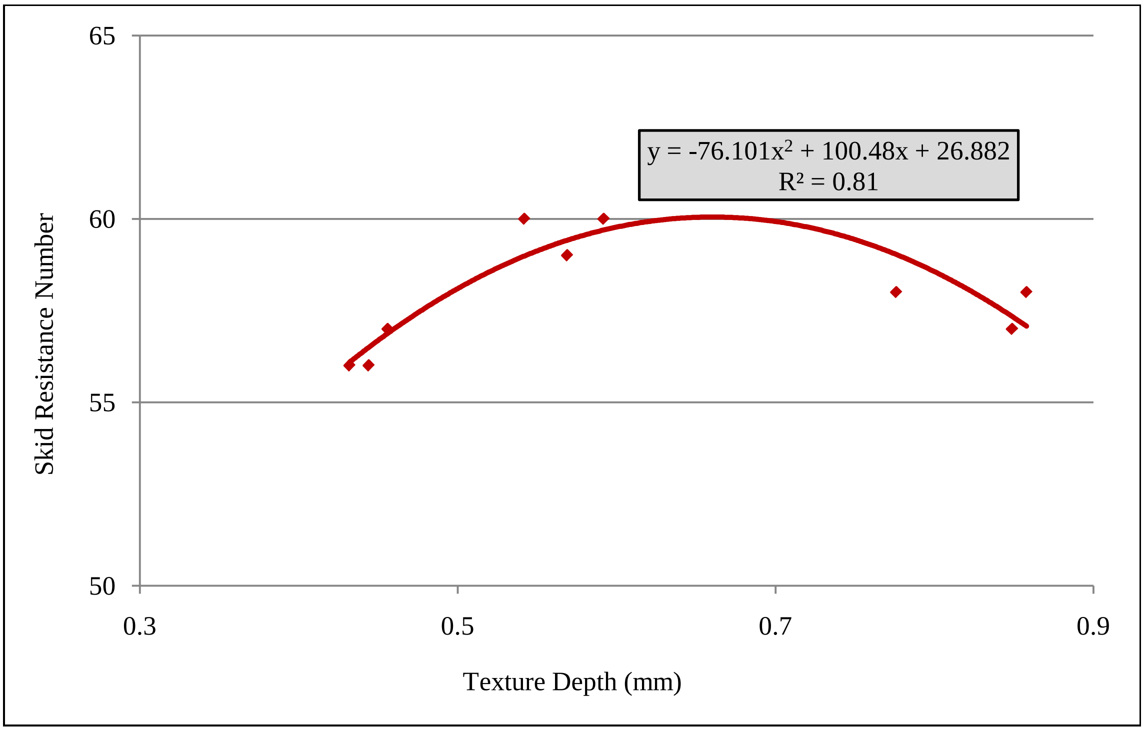

Fig. 6.102: Texture depth vs Skid resistance number for Varkala – Kallambalam Road

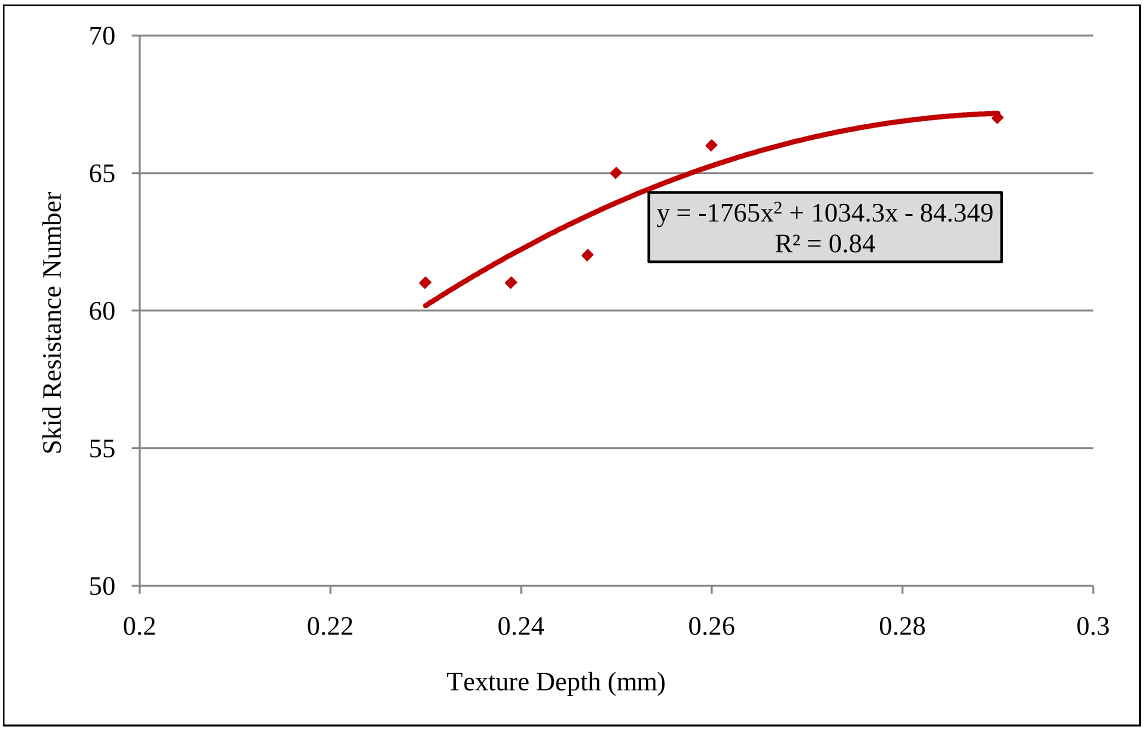

Fig. 6.103: Texture depth vs Skid Resistance number for Seaport –Airport Road

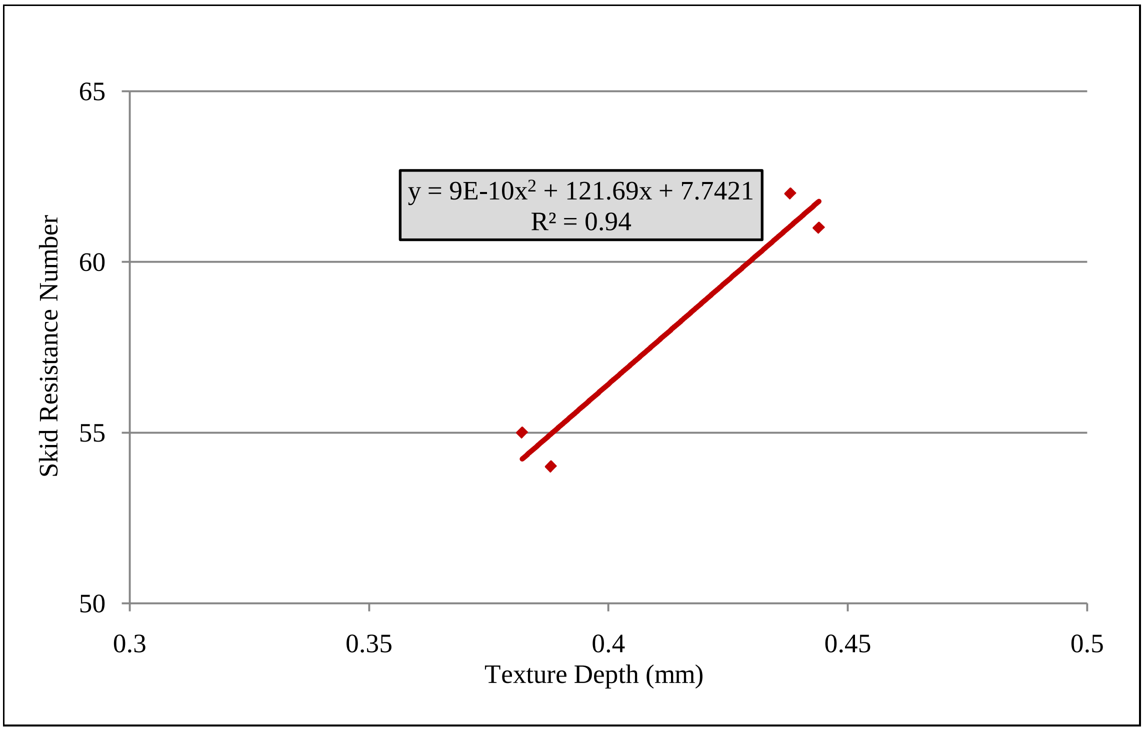

Fig. 6.104: Texture depth vs Skid Resistance number for Mannanthala-Venjaramoodu Road

6.14.1 Calibration and validation of regression models

The parameters of regression models estimated and calibrated are found to be statistically significant by various statistical parameters like R^2, t-test, F-test. The internal validation method was adopted for validating the developed regression models. Models developed in SPSS have been evaluated using Student t-test for comparing if there is any significant difference between observed and modeled values. The calculated t-values in all cases were found to be lower than the critical t- values obtained from statistical tables at the 5% level of significance and are shown in **Table 6.18.** Hence, it can be concluded that there exists no significant difference between observed and estimated skid resistance values in all study stretches.

Scatter plot diagram showing comparison between observed values (x-axis) and predicted values (y-axis) of each model are shown in **Fig. 6.105** to **6.108.** A 45-degree reference line is also plotted.

Table 6.18 Validation of relationship between Texture depth and Skid Resistance Number

Study stretch	Polynomial relationship between Texture depth and Skid Resistance Number	t test values	
		Calculated absolute value	Tabular value
Chavadimukku-Pallippuram	$SN = -3509.6TD^2 + 1688.1 TD - 138.59$	0.0076	2.5706
Varkala - Kallambalam	$SN = -76.101 TD^2 + 100.48 TD + 26.882$	0.0048	2.3060
Seaport - Airport Road	$SN = -1765 TD^2 + 1034.3 TD - 84.349$	0.0101	2.5706
Mannanthala - Venjaramoodu	$SN = 5x10^{-10} TD^2 + 121.69 TD + 7.7421$	0.0001	3.1825

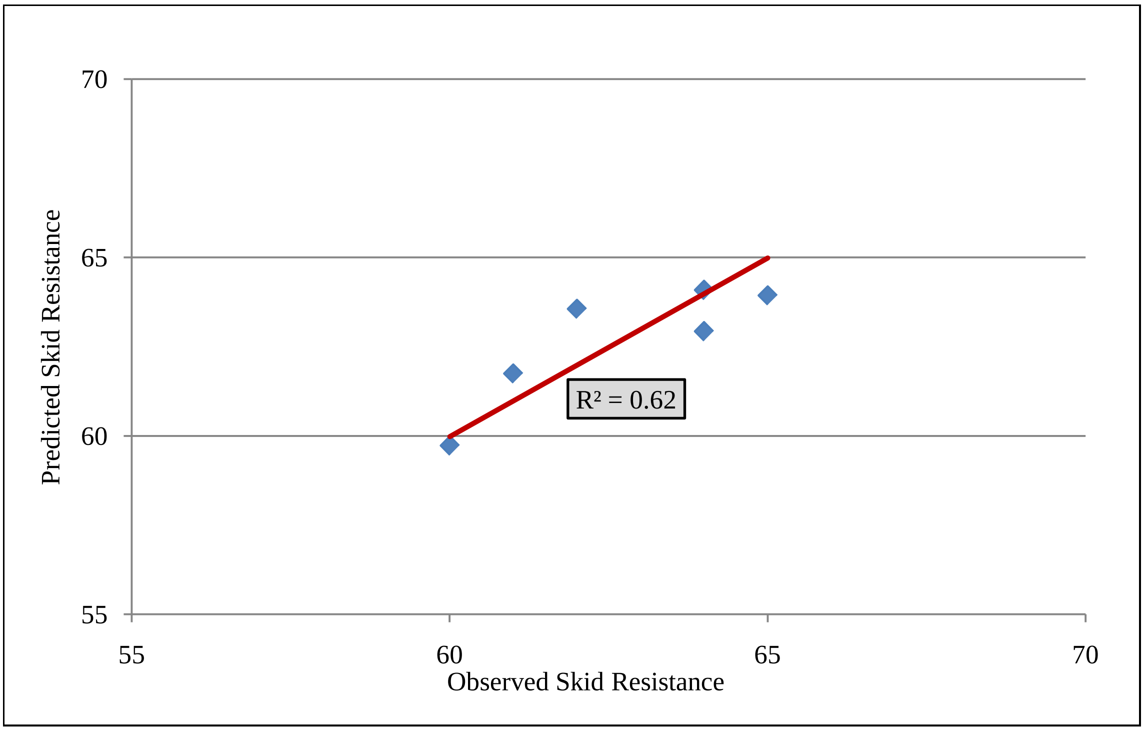

Fig. 6.105: Comparison of observed and predicted skid resistance values for Chavadimukku-Pallippuram

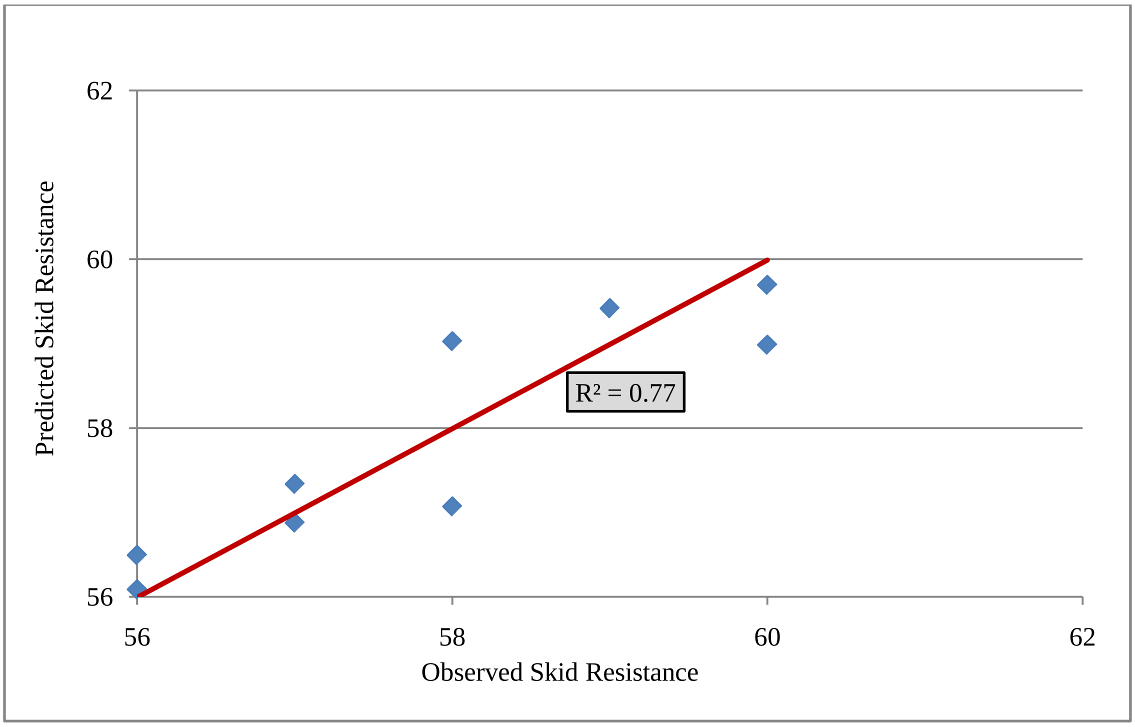

Fig. 6.106: Comparison of observed and predicted skid resistance values for Varkala-Kallamballam

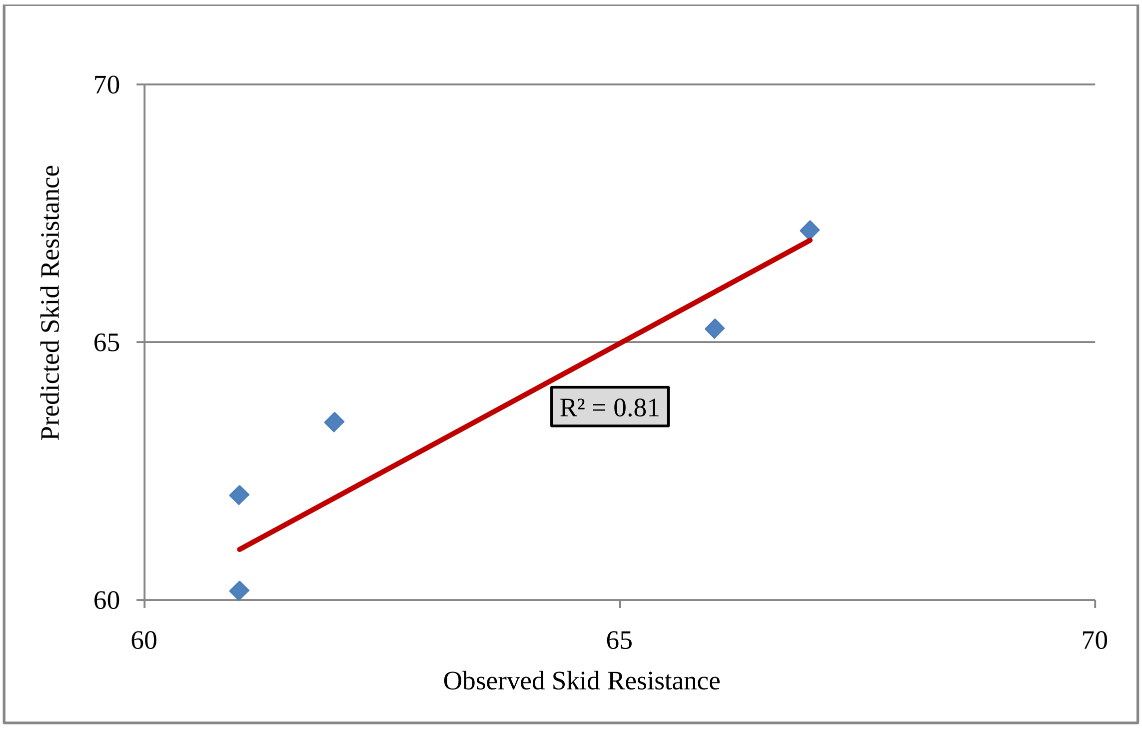

Fig. 6.107: Comparison of observed and predicted skid resistance values for Seaport-Airport Road

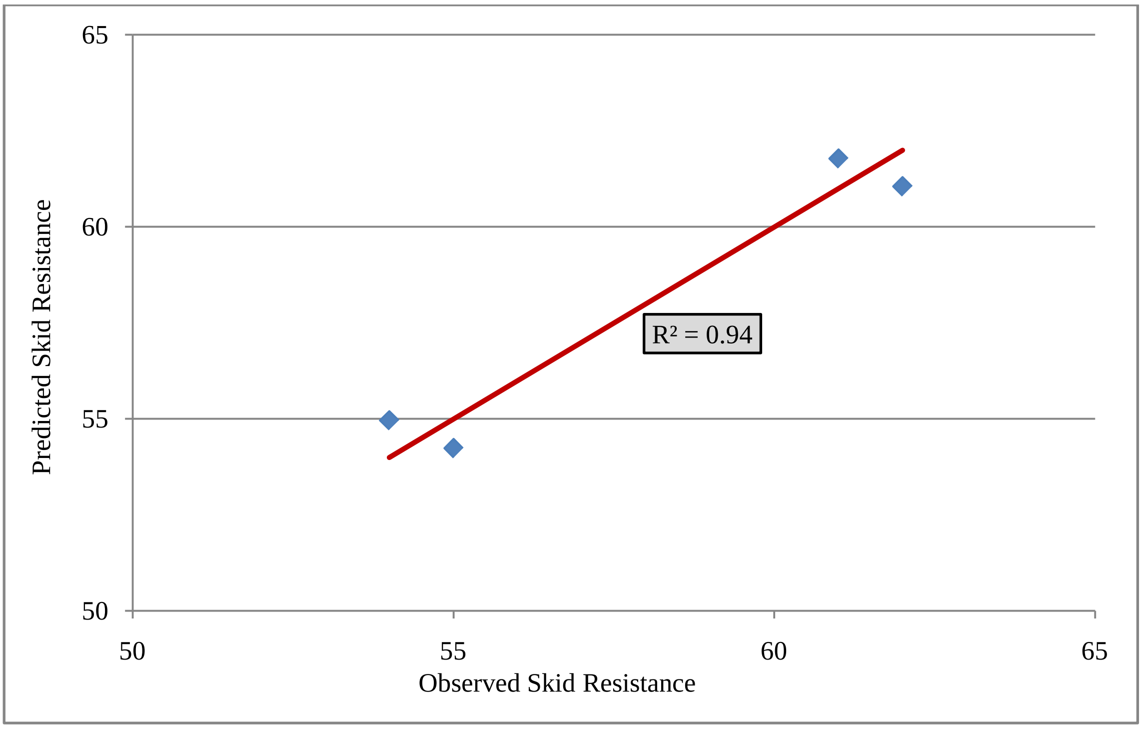

Fig. 6.108: Comparison of observed and predicted skid resistance values for Mannanthala-Venjaramoodu Road

The models developed as part of the regression analysis were used in validating the relationship between the variables, Texture Depth and Skid Resistance Number. The average prediction error (APE) and total prediction error (TPE) for Skid Resistance Number were calculated and are tabulated in **Table 6.19.**

The average prediction error (APE) for Skid Resistance Number were found to be varied between 0.010% and 0.023% and the total prediction error (TPE) for Riding Comfort Index values was found to be varied between -0.007% and -0.002%.

Table 6.19 Validation of the Regression Analysis

Study stretch	Relationship between Texture depth and Skid Resistance Number	Average Prediction Error (%)	Prediction Error of Totals (%)
Chavadimukku-Pallippuram	$SN = -3509.6\ TD^2 + 1688.1\ TD - 138.59$	0.017	-0.005
Varkala - Kallambalam	$SN = -76.101\ TD^2 + 100.48\ TD + 26.882$	0.010	-0.002
Seaport - Airport Road	$SN = -1765\ TD^2 + 1034.3\ TD - 84.349$	0.016	-0.007
Mannanthala - Venjaramoodu	$SN = 5 \times 10^{-10}\ TD^2 + 121.69\ TD + 7.7421$	0.023	0.000

It may be observed in Table 6.19 that the ‘Average Prediction Errors’ and ‘Prediction errors of Totals’ in the estimation of variables are much lesser and reasonably accurate. Hence, the regression equations developed were assumed to be acceptable for future predictions.

6.15 SUMMARY

The results of the periodic performance evaluation studies conducted on the study stretches are presented in this chapter. The observed progression of deflection, unevenness, distress, skid resistance and texture depth are represented in the form of graphs and charts. Non-linear models were developed in SPSS for Construction Quality, Raveling progression, Pothole progression, Alligator Crack progression and Deflection progression, incorporating the most influencing factors. Calibration of HDM-4 using the data for Sea Port - Air Port road (SA-1 and SA-2) for roughness, cracking, ravelling and pothole models were done and the same was validated and compared with the observed values. Various options of maintenance interventions with and without widening were tried in HDM-4. From the analysis, it was observed that overlay using Ultra Thin White Topping was most economical for the study road from among the alternative strategies considered in the analysis. Design charts for various k values of soil were developed for Ultra Thin White Topping. Application of Fuzzy Logic for pavement deflection modeling was attempted using the data for five road sections and compared with values predicted by SPSS models and observed values for cracking, pothole, roughness and deflection.

By plotting the Unevenness (UI) and Riding Comfort Index (RCI), equations were developed which will help in determining the functional performance of the road at any time. In order to have safety related evaluation of the performance of the study roads, plot of Texture Depth and Skid Resistance for the study roads were done to get best fit relationships. These models and relationships can be used for timely and appropriate maintenance interventions for the roads.

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Chapter 7

SUMMARY AND CONCLUSIONS

7.1 SUMMARY

Modeling pavement deterioration is an essential activity of the Pavement Management System. These models should be able to predict the performance of a facility. The distresses, which occur on the pavement surface, are considered as the major performance indicators of in-service pavements. A well formulated deterioration model should contain the appropriate variables that influence the deterioration process, physical principles that represent the deterioration mechanism and relevant statistical approach for evaluating the model. These models form an important component of the Pavement Management System. They should be able to do the financial planning, budgeting, pavement design and economic analysis. For accurate prediction of the pavement performance, recording of pavement cross section and its quality control is very important. These should be measured and analyzed carefully. Performance index based models takes account of all the variables affecting the performance. These models help in deciding upon the maintenance schedule also.

Various types of performance models, which predict the performance of the pavements, are available in the literature. In these models, the critical variables which affect the performance of pavements such as material properties, structural components, design approaches, environmental factors and maintenance effects are integrated. These factors, which affect the pavement deterioration, are investigated thoroughly and evaluated quantitatively. If regular maintenance is carried out, the roughness will get decreased and this should also be considered in pavement performance models. This can be incorporated in the form of cost incurred during maintenance activities like patching and sealing. Cracking, raveling and potholes initiation and progression models along with roughness and rutting models are also important to predict the performance of in-service pavements. Factors like material properties (stress, strain, resilient modulus, etc.), environmental factors and pavement thickness are responsible distresses in pavements in addition to the traffic it carries. Hence, the resulting performance prediction model takes the shape of the non-linear form. Among the other factors, initial condition of the pavement also plays a critical role in performance prediction. The structural make up of the pavement and materials also becomes a part of the performance models. It was observed that the reported models incorporated variables such as structural number, traffic etc.

It has been found that the properties of surface layers are mainly responsible for pavement distresses. Other factors include the properties and thickness of sub grade, sub base and base layer. The models also demonstrate that different deterioration mechanisms are associated with different indicators and are reflected through respective parameters. Relevant statistical methods are used to find out the correlation of different deterioration mechanisms and parameters across the models. The research done abroad has reported good performance prediction models but they have to be made adaptable to all conditions. Many roughness and rutting models developed are constrained to linear specifications and have not considered the effects of environment. Statistical techniques like model estimation are efficient only when finding the correlation among the models. Other techniques like Neural Network and Fuzzy Logic have been applied in predicting the performance of the pavements. Methods like genetic algorithm and cellular automata can also be applied in predicting the performance. Most of the models developed are restricted to particular site and environmental conditions and the parameter uncertainties across the sections are not included in the models. Presence of calibration procedure and site factors incorporating the local site and environmental conditions only will help the models to take the shape of a generalized one.

In the present research study, detailed literature review was done and pavement evaluation studies were carried out on selected road stretches in Kerala. The predominant parameters affecting the pavement performance have been identified as Modified Structural Number (MSN) and Vehicle Damage Factor (VDF). The main distresses identified on the study roads were raveling, potholes and cracking. For predicting any surface distress parameter, the traffic on the road and age of the pavement also play important roles. The present study addressed all the above aspects with particular focus on both urban and other roads. Simplified prediction models and relationships were developed for structural and functional condition and safety consideration of flexible pavements. These pavement models are based on the initial condition of respective parameter, initial strength of pavement and the increment in traffic loading and age. Non-linear regression models were developed for progression of cracking, raveling pothole, deflection and roughness. The models were validated with observed data.

The models developed in this study based on periodic field data collected from in-service flexible pavements represent the behavior of pavements during the study period of its life time. Width of the cracking, which indicates severity of distress level, has been considered in the development of crack area models. The models designed to forecast pavement changes are based essentially on statistical laws. Since these were derived from observations and measurements on different types of pavements in- service, they reflect the actual traffic and climatic conditions.

The predictive capabilities of various flexible pavement performance models proposed by different researchers in India were also examined. HDM-4 pavement deterioration models also were calibrated for the Kerala road conditions and validated using observed data. Various options of maintenance interventions with and without widening were tried in HDM-4.

By plotting the Unevenness (UI) and Riding Comfort Index (RCI), equations were developed which will help in determining the functional performance of the road at any time. In order to have safety related evaluation of the performance of the study roads, plot of Texture Depth and Skid Resistance for the study roads were done to get best fit relationships. Equations were derived to find the correlation between the observed deflection and structural strength of the pavement represented as Modified Structural Number (MSN) in urban conditions. The influence of Soil parameters and condition of the pavement on the measured deflection in the case of urban roads were also studied which can also be of help in designing the rehabilitation, strengthening and improvements.

To summarize, the modeling approach adopted and model estimation results presented in this study is expected to help in the performance evaluation, performance prediction and maintenance management of pavements more accurately. This enhanced accuracy can be translated into considerable savings by the highway agencies through systematic planning and efficient resource allocation. The relationships established between the pavement strength and pavement structure can help practicing engineers to arrive at the optimum overlay options based on the properties of the existing pavement composition and sub grade properties.

7.2 CONCLUSIONS

Based on the present study, the conclusions drawn are presented in three sections as given below:

7.2.1 Relationship of Pavement Strength and Pavement Composition

- The strength of the pavement represented by the measured deflection at the surface on mature soil sub grades is influenced by the sub grade soil properties and layer composition on an in-service pavement in urban conditions.

- There is good correlation between deflection and Modified Structural Number for various classes of Pavement Condition Index for sub grade soil types SM and SC.

- In the case of pavement in good condition, ie, for the PCI range 60-80, linear relationship give better correlation.

- In the case of soil type SM, power function relationship showed better correlation.

- In general, either power function or linear relationships can be used for finding the deflection in order to arrive at a suitable maintenance option.

- The parameters such as Field Dry Density, Field Moisture Content, Optimum Moisture Content, Maximum Dry Density, Atterberg Limits, CBR, Soil composition and the fraction of Silt & Clay of the subgrade soil have influence on the strength of the pavement.

- Field Dry Density and Plasticity Index of the sub grade soil influence the deflection, whereas Maximum Dry Density has got less impact when gravel and silt and clay fractions are considered as variables.

7.2.2 Pavement Performance Models

- Rutting is found to be absent on the study road stretches. The reasons that can be attributed for this are the absence of lane segregation and lane discipline.

- The regression models developed for deflection and roughness progression gave promising results for predicted values when validated with the observed values.

- HDM-4 calibrated to the site conditions also can be effectively used for performance prediction.

- For predicting roughness, the models developed using SPSS and HDM-4 showed promising results and hence can be used depending upon the user's choice. For roughness, the HDM-4 predicted values were less in the initial years, but showed closer values later.

- Regarding deflection, the regression and fuzzy models showed lesser values than the observed values, but closer values were obtained in the case of Fuzzy models.

- For cracking and pothole progression, HDM-4 predicted values were closer to the actual values.

- The models developed for Riding Comfort Index and Unevenness growth showed good correlation and hence are useful for studying the functional behavior of in-service flexible pavements. The values of UI at any critical RCI values can be determined from the RCI model. The Unevenness growth model is useful to estimate the unevenness after an anticipated traffic loading/ age, from an initial unevenness and deflection. The increment in vehicle operation cost of various pavements can be evaluated using this model.

- The relationship between skid resistance and Macro texture of the pavement developed in this study showed good correlation and can be used for the timely intervention from safety consideration.

- The developed models are simple and are useful for estimating the structural and functional behavior of flexible pavements with anticipated traffic loading. These models can be used to find the allowable traffic loading at different limiting values of deflection, crack area, RCI and UI. Thus, the phasing of maintenance/rehabilitation activities can be scientifically planned.

7.2.3 Overlay Options

- Different overlay options were tried in HDM-4 for the study roads to identify the best strategy. It has been found that Ultra Thin White Topping is a promising option.

- From the deterioration summary developed in HDM-4, it was observed that the riding quality of Ultra Thin White Topping is acceptable being the one without any intervention even after 15 years and the routine maintenance cost can be avoided.

- From the analysis, it was seen that due to the high increase in vehicular traffic and congestion, Level of Service of the study stretch became very low, thus demanding a Partial Widening in addition to overlay.

- Pavement up gradation with Ultra Thin White Topping was found to be the most economical since it has the highest Internal Rate of Return of 48.2, which is 19% higher than that of Ordinary Bituminous overlay.

- Ultra Thin White Topping showed the minimum rate of roughness progression which in turn leads to very low Road User Cost values thus providing 46% increase in the Net Economic Benefits than that of Ordinary Bituminous overlay. The functional efficiency of the pavement is also enhanced.

- Ultra Thin White Topping is suggested as an appropriate overlay option for roads similar to the study roads, which reduces the Life Cycle cost of the pavement.

7.3 SCOPE FOR FURTHER WORK

- Environmental factors affect the structural properties of the pavements, which are responsible for the deterioration of the pavements. But, these factors are quite uncertain in nature, vary from place to place and hence to be considered in analyzing the performance of the pavement to get more realistic results.

- Shoulder condition also is to be incorporated in the models.

- There are other benefits for different overlay options, which are not quantified, in the present study such as reducing the delays to traffic during maintenance of flexible pavement, increase in comfort and enhanced safety. These are to be incorporated in the analysis for better accuracy.

- The time series data collection is to be continued on the same roads for some more years to increase the predictive accuracy of the models developed.

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PUBLICATIONS BASED ON THE RESEARCH STUDY

1. ‘Design charts for thin concrete overlays on asphalt pavements’, Indian Highways, Vol. 37, No.1, 2009, 27-33.

2. ‘Field Performance Indicators for Natural Rubber Modified Bitumen in a Tropical Setting’, Indian Highways, Vol. 39, No.1, 2011, 47-57.

3. ‘A Case Study on Overlay Design using HDM-4’, International Journal of Innovative Research in Science, Engineering and Technology (IJIRSET), ISSN: 2319-8753, Vol. 2, Special Issue 1, 2013.

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6. ‘Pavement Distresses and Roughness modeling- A Case Study’, International Journal of Innovative Research in Science, Engineering and Technology (IJIRSET), ISSN:8753, Volume 2, Special Issue 1, 2013.

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Annexure 1

FUZZY RULES FOR MODELLING DEFLECTION

1. If (VDF is low) and (Initial_deflection is Very_good) and (MSN is low) then (deflection is Very good) (1)
2. If (VDF is low) and (Initial_deflection is Very_good) and (MSN is medium) then (deflection is Very good) (1)
3. If (VDF is low) and (Initial_deflection is Very_good) and (MSN is high) then (deflection is Very good) (1)
4. If (VDF is low) and (Initial_deflection is Good) and (MSN is low) then (deflection is Fair) (1)
5. If (VDF is low) and (Initial_deflection is Good) and (MSN is medium) then (deflection is Good) (1)
6. If (VDF is low) and (Initial_deflection is Good) and (MSN is high) then (deflection is Good) (1)
7. If (VDF is low) and (Initial_deflection is Fair) and (MSN is low) then (deflection is Fair) (1)
8. If (VDF is low) and (Initial_deflection is Fair) and (MSN is medium) then (deflection is Good) (1)
9. If (VDF is low) and (Initial_deflection is Fair) and (MSN is high) then (deflection is Good) (1)
10. If (VDF is low) and (Initial_deflection is Poor) and (MSN is low) then (deflection is Poor) (1)
11. If (VDF is low) and (Initial_deflection is Poor) and (MSN is medium) then (deflection is Fair) (1)
12. If (VDF is low) and (Initial_deflection is Poor) and (MSN is high) then (deflection is Good) (1)
13. If (VDF is low) and (Initial_deflection is Very_poor) and (MSN is low) then (deflection is Very poor) (1)
14. If (VDF is low) and (Initial_deflection is Very_poor) and (MSN is medium) then (deflection is Fair) (1)

15. If (VDF is low) and (Initial_deflection is Very_poor) and (MSN is high) then (deflection is Good) (1)
16. If (VDF is medium) and (Initial_deflection is Very_good) and (MSN is low) then (deflection is Very good) (1)
17. If (VDF is medium) and (Initial_deflection is Very_good) and (MSN is medium) then (deflection is Very good) (1)
18. If (VDF is medium) and (Initial_deflection is Very_good) and (MSN is high) then (deflection is Very good) (1)
19. If (VDF is medium) and (Initial_deflection is Good) and (MSN is low) then (deflection is Fair) (1)
20. If (VDF is medium) and (Initial_deflection is Good) and (MSN is medium) then (deflection is Fair) (1)
21. If (VDF is medium) and (Initial_deflection is Good) and (MSN is high) then (deflection is Good) (1)
22. If (VDF is medium) and (Initial_deflection is Fair) and (MSN is low) then (deflection is Poor) (1)
23. If (VDF is medium) and (Initial_deflection is Fair) and (MSN is medium) then (deflection is Fair) (1)
24. If (VDF is medium) and (Initial_deflection is Fair) and (MSN is high) then (deflection is Good) (1)
25. If (VDF is medium) and (Initial_deflection is Poor) and (MSN is low) then (deflection is Poor) (1)
26. If (VDF is medium) and (Initial_deflection is Poor) and (MSN is medium) then (deflection is Good) (1)
27. If (VDF is medium) and (Initial_deflection is Poor) and (MSN is high) then (deflection is Very good) (1)
28. If (VDF is medium) and (Initial_deflection is Very_poor) and (MSN is low) then (deflection is Very poor) (1)
29. If (VDF is medium) and (Initial_deflection is Very_poor) and (MSN is medium) then (deflection is Fair) (1)
30. If (VDF is medium) and (Initial_deflection is Very_poor) and (MSN is high) then (deflection is Fair) (1)

31. If (VDF is high) and (Initial_deflection is Very_good) and (MSN is low) then (deflection is Very good) (1)
32. If (VDF is high) and (Initial_deflection is Very_good) and (MSN is medium) then (deflection is Very good) (1)
33. If (VDF is high) and (Initial_deflection is Very_good) and (MSN is high) then (deflection is Very good) (1)
34. If (VDF is high) and (Initial_deflection is Good) and (MSN is low) then (deflection is Poor) (1)
35. If (VDF is high) and (Initial_deflection is Good) and (MSN is medium) then (deflection is Fair) (1)
36. If (VDF is high) and (Initial_deflection is Good) and (MSN is high) then (deflection is Good) (1)
37. If (VDF is high) and (Initial_deflection is Fair) and (MSN is low) then (deflection is Poor) (1)
38. If (VDF is high) and (Initial_deflection is Fair) and (MSN is medium) then (deflection is Fair) (1)
39. If (VDF is high) and (Initial_deflection is Fair) and (MSN is high) then (deflection is Good) (1)
40. If (VDF is high) and (Initial_deflection is Poor) and (MSN is low) then (deflection is Very poor) (1)
41. If (VDF is high) and (Initial_deflection is Poor) and (MSN is medium) then (deflection is Fair) (1)
42. If (VDF is high) and (Initial_deflection is Poor) and (MSN is high) then (deflection is Good) (1)
43. If (VDF is high) and (Initial_deflection is Very_poor) and (MSN is low) then (deflection is Very poor) (1)
44. If (VDF is high) and (Initial_deflection is Very_poor) and (MSN is medium) then (deflection is Very poor) (1)
45. If (VDF is high) and (Initial_deflection is Very_poor) and (MSN is high) then (deflection is Good) (1)

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