Deterioration Modelling of Flexible Pavements with Modified Bitumen in Wearing Course

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DOCTOR OF PHILOSOPHY
in **Civil Engineering**

by

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CERTIFICATE

This is to certify that the thesis titled ***"Deterioration Modelling of Flexible Pavements with Modified Bitumen in Wearing Course"*** submitted by **Sanjay Deori** to Indian Institute of Technology Guwahati, for the award of the degree of Doctor of Philosophy is a record of bona fide research work carried out by him under our supervision and guidance. The thesis work, in our opinion, has reached the requisite standard fulfilling the requirement for the degree of Doctor of Philosophy.

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DECLARATION

I, Sanjay Deori, hereby declare that this written submission represents my ideas in my own words and where others ideas and words have been included, I have adequately cited and referenced the original sources. I also declare that I have adhered to all principles of academic honesty and integrity and have not misinterpreted or fabricated or falsified any idea/ data/ fact/ source in my submission.

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ABSTRACT

India is developing her highway infrastructure through widening and rehabilitation of existing national highways and expressways under seven phases of National Highway Development Program (NHDP), consisting of a total length of 46,635 km. The first phase of NHDP consisted of Golden Quadrilateral, *i.e.* highways connecting four major metropolitan cities: Delhi-Mumbai-Chennai-Kolkata; and the second phase consisted of North-South corridor connecting Srinagar to Kanyakumari, East-West corridor connecting Silchar to Porbandar, and road connectivity to major ports of the country by linking them with key national highways. The NHDP road network is considered as a high speed road corridor of India.

A vast majority of the roads in India are flexible pavements consisting of bituminous courses underlain by base and sub-base layers on a compacted subgrade. Use of modified bituminous binders have increased manifold in the last decade in India, mainly in the bituminous concrete (BC) hot mix asphalt wearing course of flexible pavements. Because of their enhanced engineering properties, modified binders are preferred to conventional (unmodified) binders. Crumb rubber-modified bitumen (CRMB) and polymer-modified bitumen (PMB) are the most common modified binders used with the intent to improve the pavement performance against large variations in temperatures and continually increasing axle loads on Indian roads.

In light of the fact that the high-speed road corridors are vital assets for socio-economic development of the country, adoption of a scientific approach for their maintenance is imperative. Highway Development and Management (HDM-4) is one of the most useful and internationally recognised tools available for pavement performance and management analysis. The tool is developed by the World Bank and its use has been advocated by the government agencies in many developing countries, including India. At present, there is a strong need for well-calibrated and validated HDM-4 pavement deterioration models for the high-speed road corridors of India constructed with modified binders in the wearing courses for development of pavement maintenance and management strategies.

The main objective of this study is to perform deterioration modelling of flexible pavement sections with modified binders in the BC wearing course using HDM-4 tool. The road deterioration models inbuilt in HDM-4 predict the deterioration of the pavement over time with traffic, which is useful for decision making for highway administrators for planning pavement maintenance. However, the models need to be calibrated for local conditions, a process that requires time series data collected over several years through extensive field and laboratory scale investigations.

A total of 23 in-service flexible pavement road sections were identified from various sections of NHDP corridor. These 23 sections were divided into six homogeneous groups based on similarity in pavement composition and climatic conditions. These sections were spread throughout the country and covered different climatic and environmental zones. Extensive field and laboratory studies were carried out to gather time series data consecutively for three years (2011 to 2013) for all the 23 sections. The collected time series data included pavement distress data, traffic and axle load data, pavement crust composition data, pavement material characterisation data, temperature and rainfall data, and construction and maintenance history data.

The acquired time series data were used for calibration and validation of HDM-4 distress models. Calibration factors were obtained for various distress models, including cracking, ravelling, potholing, rut depth, roughness, texture depth, and skid resistance. Calibration factors were then validated through statistical comparison using the chi-square test between the data predicted by the calibrated models and the third year observed data. Validation results indicated that the differences between observed and predicted values for all distresses were not statistically significant. The study successfully produced calibration factors for different distress models for the six homogeneous groups consisting of 23 pavement sections. The calibrated and validated HDM-4 deterioration models can be put to use for prediction of distresses and the development of maintenance strategies for the Indian high-speed road corridor sections with modified binders.

Keywords: deterioration modelling; pavement maintenance; HDM-4; modified bituminous binders; NHDP; flexible pavements.

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(SANJAY DEORI)

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF PHOTOS

LIST OF TABLES

LIST OF ABBREVIATIONS

AADT	Annual Average Daily Traffic
AAE	Average Absolute Error
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
AB	Asphalt Base
AC	Asphalt Concrete
AM	Asphalt Mix
ANN	Artificial Neural Networks
AP	Andhra Pradesh
AP	Asphalt Pavement
ARRB	Australian Road Research Board
AS	Assam
ASTM	American Society for Testing and Materials
BB	Benkelman Beam
BC	Bituminous Concrete
BI	Bump Integrator
BPT	British Pendulum Tester
CBR	California Bearing Ratio
CRMB	Crumb Rubber Modified bitumen
CRRI	Central Road Research Institute
CSAL	Cumulative Standard Axle Loads
CTAB	Cement Treated Aggregate Base
CVPD	Commercial Vehicles Per Day
DBM	Dense Graded Bituminous Macadam
DBSD	Double Bituminous Surface Dressing
DMI	Distance Measuring Instrument
DOF	Degree of Freedom
EIRR	Economic Internal Rate of Return
ESAL	Equivalent Single Axle Load

ESALF	Equivalent Single Axle Load factor
EW	East-West
FEM	Finite Element Method
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
GB	Granular Base
GDP	Gross Domestic Product
GEIPOT	Brazilian Transportation Planning Agency
GIS	Geographical Information System
GJ	Gujarat
GPS	Global Positioning System
GQ	Golden Quadrilateral
GSB	Granular Sub-Base
HCM	Highway Cost Model
HDM-4	Highway Development and Management System-4
HDM-III	Highway Design and Maintenance Standards Model-III
HMA	Hot Mix Asphalt
IQL	Information Quality Levels
IRC	Indian Roads Congress
IRI	International Roughness Index
ISOHDM	International Study of Highway Development and Management
KA	Karnataka
Km.	Kilometer
LCPC	Laboratoire Centrale des Ponts et Chaussées
LL	Liquid Limit
LOE	Line of Equality
LTPP	Long-Term Pavement Performance
LVR	Low Volume Roads
MAE	Mean Absolute Error
M&R	Maintenance and Rehabilitation
MDR	Major District Roads
MH	Maharashtra

MIT	Massachusetts Institute of Technology
MORT&H	Ministry of Road Transport and Highways
MOS	Maintenance Optimization System
MPI	Maintenance Priority Index
MSN	Modified Structural Number
MSS	Mix Seal Surfacing
MT	Motorized Traffic
NCHRP	National Cooperative Highway Research Program
NE	National Expressway
NH	National Highways
NHAI	National Highway Authority of India
NHDP	National Highway Development Programme
NMT	Non-motorized Traffic
NPV	Net Present Value
NRRDA	National Rural Road Development Agency
NS	North-South
NSE	Nash-Sutcliffe Efficiency
NSV	Network Survey Vehicle
ODR	Other District Roads
OR	Ordinary Repairs
PC	Premix Carpet
PCC	Portland Cement Concrete
PCR	Pavement Condition Rating
PI	Plasticity Index
PIU	Project Implementation Unit
PL	Plastic Limit
PMB	Polymer Modified Bitumen
PMGSY	Pradhan Mantri Gram SadakYojana
PMMS	Pavement Maintenance Management System
PMS	Pavement Management System
PPS	Pavement Performance Study
PR	Periodic Renewals

PSI	Present Serviceability Index
PSR	Present Serviceability Rating
PWD	Public Works Department
R	Pearson’s Correlation Coefficient
RD	Road Deterioration
RIAM	Road Investment Analysis Model
RIS	Road Information System
RMSE	Root Mean Square Error
R^2	Coefficient of Determination
ROMDAS	Road Measurement Data Acquisition System
RSR	RMSE to Observations’ Standard Deviation Ratio
RSS	Residual Sum of Squares
RTIM	Road Transport Investment Model
RUCS	Road User Cost Study
RUE	Road User Effects
SB	Stabilized Base
SBSD	Single Bituminous Surface Dressing
SCRIM	Sideways Coefficient Routine Investigation Machine
SDBC	Semi Dense Bituminous Concrete
SEE	Social and Environmental Effects
SFC	Sideways Force Coefficient
SH	State Highways
SHRP	Strategic Highway Research Program
SN	Structural Number
SNP	Adjusted Structural Number
SR	Special Repairs
SRV	Skid Resistance Value
SSD	Sum of Squared Differences
SSR	Residual Sum of Squares
ST	Surface Treatment
TRB	Transportation Research Board
TRRL	Transportation and Road Research Laboratory

UP	Uttar Pradesh
URUCS	Updated Road User Cost Study
VDF	Vehicle Damage Factor
VOC	Vehicle Operating Costs
VR	Village Roads
WBM	Water Bound Macadam
WE	Work effects
WIM	Weigh in Motion System
WMM	Wet Mix Macadam
YAX	Number of Vehicle Axles
YE4	Annual Total Number of Equivalent Standard Axles

Chapter 1

INTRODUCTION

1.1 BACKGROUND

Transportation infrastructure plays a key role to accomplish the social, economic, and defence needs of a country. The modes of transportation are highways, railways, airways, marine, and pipelines. An efficient transportation system is of vital importance to the economy of any nation. For overall economic growth of a country, a good road transportation system is considered as the primary requirement. The road transport occupies a dominant and important position among all transportation systems due to its easy availability, flexibility of operation, door-to-door service, and reliability.

The major elements of the road transport system are pavements–mostly used by roadways, railways, and runways. The pavements comprise one-half of total highway expenditure, and the expenditure on road pavements continues to grow mainly due to vehicular loading for which maintenance and rehabilitation are periodically required. Huge national investment for maintenance is required to keep the pavements at satisfactory level of service, and also to ensure safe passage at design speed with low road user costs. Durability and serviceability of pavement after construction mainly depend on the maintenance provided as and when required, and if delayed, it will often involve extensive rehabilitation, and even reconstruction, costing many times more than timely maintenance treatment that could be carried out earlier. Timely maintenance of pavements reduces the deterioration considerably, reduces vehicle operating costs (VOC), decreases number of accidents, and increases the reliability of transport services. All this is reflected in the significant contribution made by the transportation sector to the gross domestic product (GDP) of the country. Many developed and industrialized countries report significant contribution of the transportation sector to their GDP. Thus, the transportation sector generates significant revenue to the government, and hence it is highly important to allocate funds for the development and maintenance of road network.

Investment has been made on pavement maintenance and up-gradation worldwide and billions will be spent annually to protect these assets through periodic rehabilitation and maintenance. Simple scientific mathematical models and flexible practical procedures of pavement management will result in budget savings. The variables such as materials, construction technology, axle loads, environmental conditions, performance, maintenance and economics, influence the design of pavements. Still, road maintenance is a challenge for governments due to its dynamic nature, surcharge loads, and expenditure. The deterioration of roads is influenced by many factors that include pavement structure, vehicle load, drainage, and climate. In city regions, minimising the vehicle operating cost and travel time can justify the investments on road network. Consequently, a scientific approach to manage the road network is inevitable.

1.2 INTRODUCTION TOPAVEMENT TYPES

The road is considered as one of the most important infrastructures for social and economic development of any country, and is directly related to the overall prosperity and well-being of its people. The Indian road infrastructure has different categories of roads in its network that include rural roads, district roads, state highways, national highways, and the expressways. There are mainly three types of pavements: flexible, rigid, and composite pavements. A flexible or asphalt[1] pavement includes an asphalt concrete[1] (AC) wearing course, an asphalt binder course underlain by one or more base and subbase layers which may or may not be stabilized. A rigid or Portland cement concrete (PCC) pavement consists of a Portland cement concrete slab, a cemented base and a granular subbase layer. A composite pavement is typically composed of an AC layer as wearing surface and a PCC slab as a base. Due to its high cost, this type of pavement is rarely used for a new construction. A typical flexible pavement and rigid pavement consists of surface course, and layers of granular base and/or subbase resting on a prepared subgrade soil as shown in Figure 1.1.

Majority of the roads and highways constructed globally, including India, are flexible pavements which utilize hot mix asphalt (HMA). HMA is a well-proven

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[1] Asphalt and bitumen are synonyms of each other and both terms have been used interchangeably in this thesis. Similarly, asphalt concrete and bituminous concrete refer to asphalt/bituminous mixes, and both have been used interchangeably.

construction material, which is long lasting and durable and is designed to accommodate wide range of climatic and traffic loading conditions for a variety of paving applications. The HMA layers are the major contributors to the structural capacity of the pavement. Granular layers and subgradealso have a significant impact on the structural condition of the pavement and contribute to pavement's permanent deformation. HMA pavements deteriorate with time due to traffic loading and environmental exposure.

Flexible pavements are generally designed using: empirical methods, mechanistic methods, or a combination of both. The empirical methods were developed based on the observed performance of roads actually tested with known pavement materials and structures subjected to certain traffic loads and environmental effects. The mechanistic methods are based on fundamental laws of physics and strength of materials. The mechanistic-empirical method calculates the response of pavement materials to vehicular loading, and then predicts the performance from these responses.

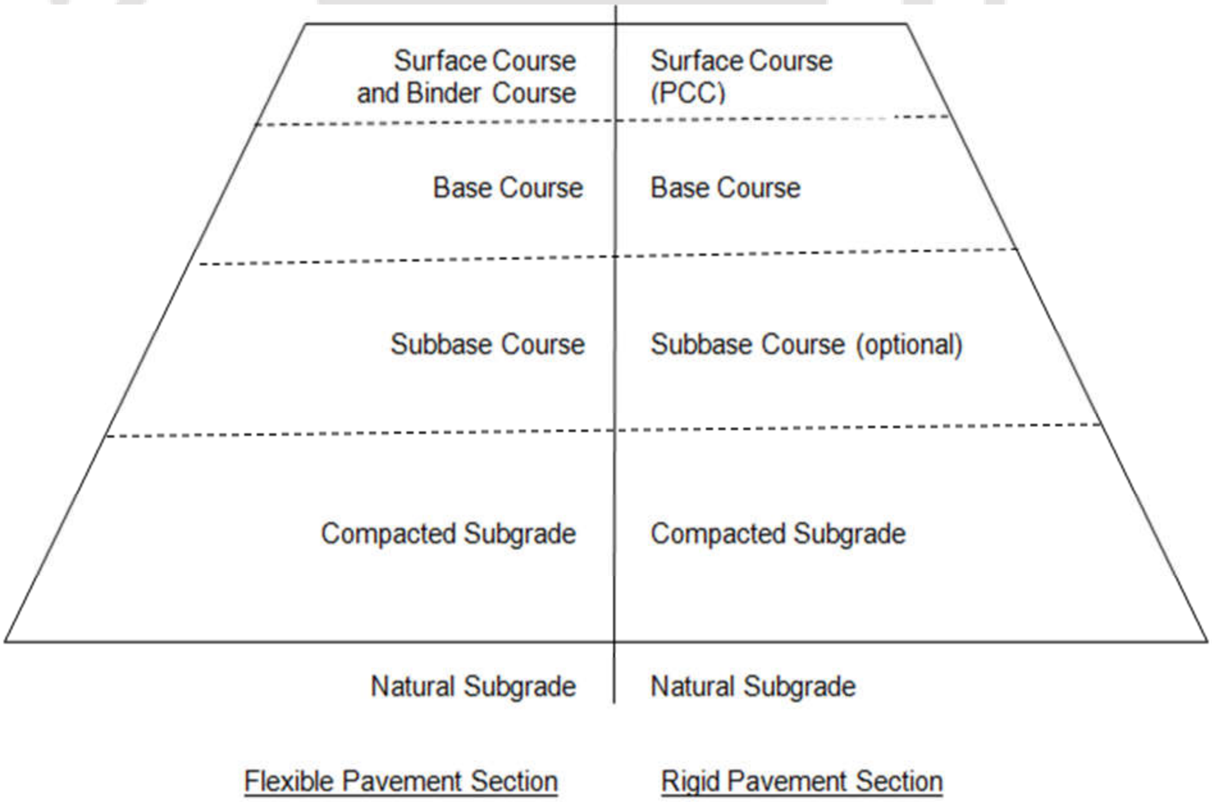


Figure 1.1 Typical sections of flexible and rigid pavements

1.3 INDIAN ROAD NETWORK

1.3.1 General

India is a vast country with twenty-nine (29) states and seven (7) union territories with a land-area of 3,060,500 square kilometers. The mainland comprises of four regions, namely, the great mountain zone, plains of the Ganga and the Indus, the desert region, and the southern peninsula having widely varying climate, terrain, and soil conditions. Transport is becoming an important part of India's economic growth and the unprecedented increase in India's road transport during the last few decades is likely to continue in the coming years as well. The national road development programslike National Highways Development Programme (NHDP) and Pradhan Mantri Gram Sadak Yojana (PMGSY, or the Prime Minister Rural Road Scheme) envisaged for development of the country involve large amounts of money, manpower, and materials. The objective of these programs is not only to build new roads but also to improve the existing road network and to maintain and rehabilitate them.

Presently, India has a road network of 5.31 million km, which is the second largest road network in the world. The quantitative density of the Indian road network is higher than Japan, United States, China, and Brazil. Nevertheless, qualitatively Indian roads consist of national, state and district highways, narrow and unpaved roads. About 61% of total length, *i.e.* 3.34 million kilometers of Indian roads are paved. Table 1.1 gives the category wise road length in the present road network of the country (MoRTH, 2017).

Table 1.1 Present road network of India (2017)

Sr. No.	Classification	Length (km)	Percentage
1.	Expressways	1455	0.03%
2.	National Highways	100,087	1.88%
3.	State Highways	167,109	3.14%
4.	Major District Roads	17,05,706	32.12%
5.	Village and Other Roads	3,337,255	62.83%
	Total Road Length	5,311,612	100.00%

Source: MoRTH (2017)

1.3.2 National Highways

National Highways (NH) are the main highways running through the length and breadth of the country connecting major ports, state capitals, large industrial and tourists centers in India. These highways constitute the primary system of road transportation and carry major road traffic across the country. The National Highways constitute about 2 percent of all the roads in India but carry about 40 percent of the total road traffic. In view of the importance of the national highways in the overall road network of the country, Government of India took a historic decision to widen and strengthen the existing NH's under the National Highway Development Programme (NHDP).The total length of National Highways in India has been continuously expanding from 58,112 km at the end of 9th Five-year plan (1997-2002) and has reached to 1,00,087kmat present in the 12th Five-year plan (2012-2017) (NHAI,2017).In the Fourth Twenty Year Road Development Plan for the period 2001-2021, it has been decided that the high-density traffic corridors of National Highways are to be converted to Expressways.

1.3.2.1 National Highway Development Programme (NHDP) – In India, road infrastructure is used to transport over 60 percent of total goods and 85 percent of total passenger traffic. Although quantitatively the total road length of India stands second in the world, but qualitatively just about 60 percent of roads are paved, and the highest-standard national highways and expressways merely occupy a share of 2 percent of the road network. This leads to poor connectivity and poor performance of the network and hence is impediment to the economic growth. The poor connectivity and poor performance of the road network system have been a major drag on the growth of national economy. Thus, in order to improve the economic development of the country, the Government of India has undertaken a massive National Highways Development Programme (NHDP) through widening and rehabilitation of existing national highways in the country. Since 1999, under the NHDP about 27,062 km length of National Highways has been completed from 2 lanes to 4/6 lanes, and presently10, 269km length is under implementation. The NHDP's focus is on developing international standards roads with facilities for uninterrupted flow of traffic and enhanced safety features, and to improve the riding quality of the country's major National Highway corridors. With the implementation of NHDP, capacity of heavy density corridors of the National Highway network has been increased. Under the seven phases of NHDP, highway network of India is planned to be improved mainly through the construction of Golden Quadrilateral (GQ) connecting four major metropolitan cities linking Delhi-Mumbai-Chennai-Kolkata, North-South (NS) corridor connecting Srinagar to Kanyakumari, East-West (EW) corridor connecting Silchar to Porbandar, and road connectivity of major ports of the country to National Highways. The high-speed road corridor network developed under the NHDP in Phase-I, Phase-II and Phase-III is presented in Figure 1.2.

Figure 1.2 Project map of NHDP (NHAI, 2017)

1.3.3 Expressways

Due to unprecedented growth of traffic in India, it is necessary to go in for expressways type facilities for high-density road corridors. The Government of India has planned to construct 18,637 km of expressways over a period of next 20 years. An expressway is a controlled-access highway providing rapid, unhindered and safe movement of high-speed traffic. The proposed Expressway network in India to be completed by the year 2020 is shown in Figure 1.3.India has already completed about 1455km length of six/eight lane controlled-access expressways. The construction programme is being implemented jointly by the Central and State Governments with the participation of the private sector. Some of the long awaiting Expressways have already been completed and are operational under the States road development agencies. The Mumbai-Pune Expressway in Maharashtra State, Ahmedabad-Vadodara Expressway in Gujarat State, Bangalore-Mysore Expressway in the Karnataka State, Yamuna Expressway in the State of Uttar Pradesh, Delhi-Gurgaon Expressway in the State of Delhi and Haryana, Delhi-Noida Expressway in the State of Delhi and Uttar Pradesh, Jaipur-Kishangarh Expressway in the state of Rajasthan are some of the major expressways connecting prominent cities in India.

1.3.4 Use of Modified Binders on Indian Highways

With heavier axle loads and rapidly increasing traffic volume, the traditional pavements with virgin bituminous binders have been losing their efficiency (Yildirim, 2007). This efficiency loss of traditional pavements has motivated the pavement engineers to search new methods/materials to improve the properties of pavements. The rheological weakness of conventional bitumen has generated interest in the use of various types of modifiers to enhance properties of conventional bitumen. In the past few decades, to improve the performance of the pavements, several efforts have been undertaken on modified asphalt materials.

Modified binders are bituminous binders whose properties have been modified through the use of different materials, mainly polymers. When added to the neat (unmodified) binders, these materials modify its chemical structure and/or physical and mechanical properties. These binders can either be manufactured at a separate plant, which is not at the worksite or by a special mobile unit immediately before use at the HMA production plant itself.

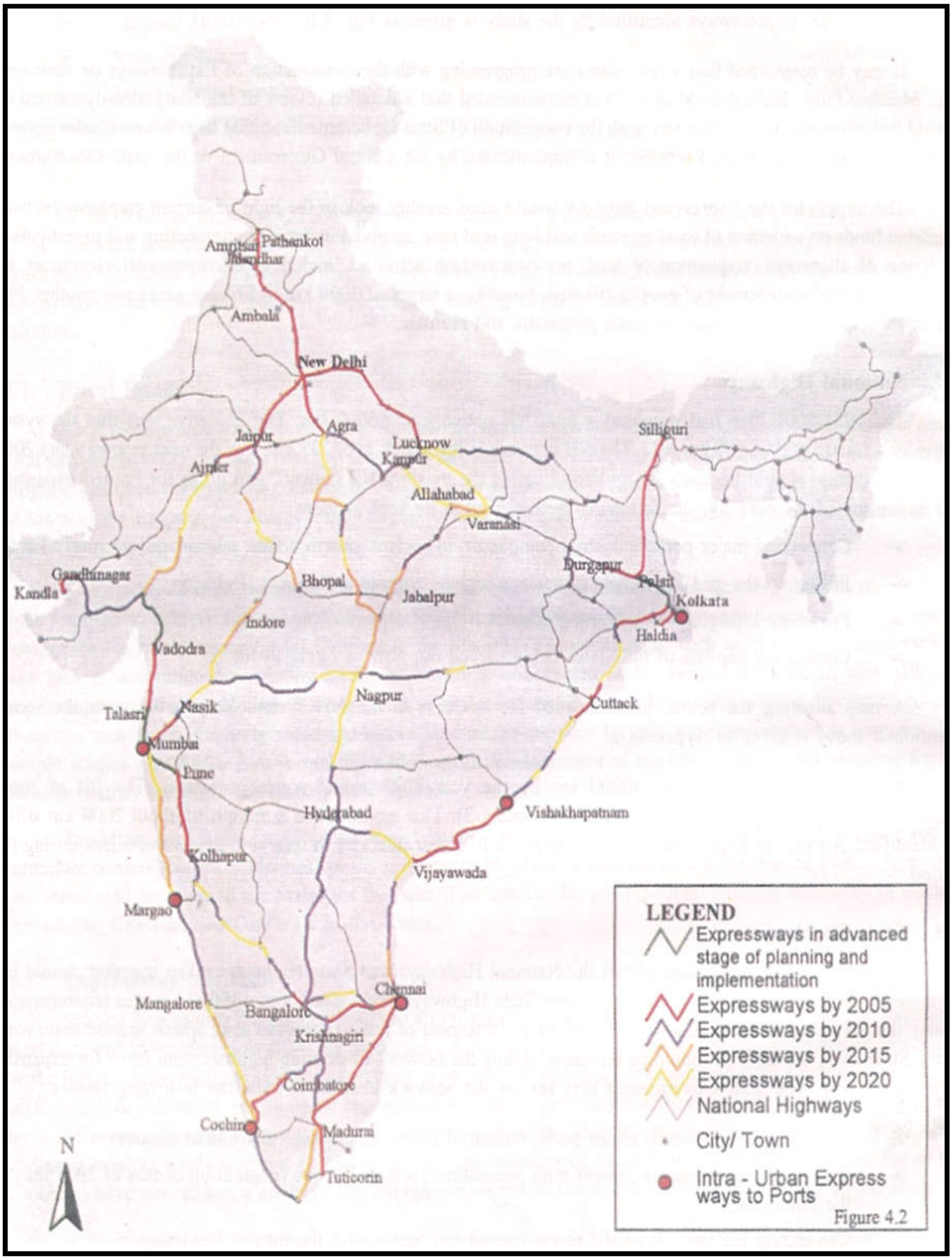


Figure 1.3 Proposed expressways network in India to be completed by 2020(MoRTH, 2014)

As asphalt binder is a viscoelastic material, its strength and durability are influenced by temperature and time. Under high temperatures, the stiffness of the bitumen reduces, while at low temperatures, flexibility reduces leading to thermal cracks. Therefore, both stiffness at high temperatures and flexibility at low temperatures are the important attributes of bituminous binders to be considered for rutting and cracking, respectively. Modifiers are typically added to the bitumen to accommodate the extreme fluctuations in weather conditions, and to resist heavy axle loads.

Use of polymers for bitumen modification has been increasing day-by-day. However, because of the high cost of these polymers compared to bitumen, the amount needed to improve pavement performance should be optimised to achieve the desired performance. The use of discarded vehicle tires in pavement construction was also one of the steps taken in this direction. Research on crumb rubber has been going on since last three decades. The characteristics of crumb rubber depend on the rubber type, asphalt composition, and size of rubber crumbs, as well as time and temperature of reaction. These factors have considerableeffect on pavement performance. Natural rubber latex produced from rubber plants is also used for modification of bitumen. The natural rubber latex is blended with slow and gradual addition of concentrated latex in melted bitumen.

With the continued socio-economic progress in the country, the traffic loads carried by the network have been growing heavier and heavier. To tackle the growing demand for long lasting pavement, modified binders are increasingly used in construction of bituminous surface course of national highways as well as expressways in India. During the past decade (2001-10), the use of modified bitumen for roads and airports, especially the use of crumb rubber modified bitumen (CRMB) and polymer modified bitumen (PMB), has increased significantly. During this period, quantity of modified bitumen used crossed 0.5 million tonnes per year. Currently, it is in the range of 12-15 percent of the total bitumen consumed on Indian roads. Nearly 80 percent of this consumption is in respect of crumb rubber modified bitumen. Latex rubber is used only to the extent of 15,000-20,000 tonnes per year, consumed mostly in and around the state of Kerala. The remaining 20 percent is shared between elastomeric and plastomeric thermoplastic polymers broadly in the ratio of 2:1, respectively (Kandhal and Dhir, 2011).

1.3.5 Maintenance of Indian Roads

Maintenance is provided to maintain or restore the functionality and structural integrity of pavements. If the pavement is not maintained, it will eventually deteriorate to a point where the only choice is reconstruction which is the costliest one, otherwise it can fail. Pavement maintenance operation can be broadly classified into three categories (MoRTH, 2011):

1. **Preventive Maintenance:** Preventive maintenance is applied before appearance of any serious deterioration and when pavement is still structurally sound. Preventive maintenance improves serviceability, condition and performance of the pavement. Preventive maintenance is generally planned and cyclic in nature. To delay the failures and deterioration in pavements and to improve the service life, preventive maintenance strategy is adopted.

2. **Corrective or Remedial Maintenance:** When the pavements show deficiencies that impact the free operation of vehicles, the corrective or remedial maintenance can be adopted. Corrective maintenance approach is generally reactive, not proactive, and is performed to rectify the deficiency and restore the deteriorated pavement near to original or acceptable condition usually with increase in its structural strength. It includes routine maintenance, rehabilitation/strengthening (structural overlays), and reconstruction activities.

   Routine maintenance activities are performed on a routine basis for operational reasons to restore the function of the pavement and includes: (i) ordinary repairs (OR) (examples: routine maintenance of road surface as well as maintenance of culverts, patch repairs, crack sealing, road side drainage, repairing of shoulders, road marking, removal of debris etc.), and (ii) periodic renewals (PR). Special Repairs (SR) for improving the curves or visibility and rectifying the localized road damages also come under the purview of corrective maintenance approach. Pavement reconstruction is required when a pavement has either failed or has become functionally obsolete and in which, the existing pavement structure is replaced by an equivalent or enhanced pavement structure.

3. **Emergency or Catastrophic Maintenance:** Emergency or catastrophic maintenance activities are generally necessary during an emergency situation to rectify the damages due to rain, floods, cyclones etc. on an immediate

basis, as required generally for safety reasons and/or to restore the function of the pavement to a minimum level of service (traffic worthy condition) while a permanent restoration is being designed and scheduled. Concrete pavement blow-ups, road washouts, avalanches, pavement subsidence, severe potholes etc. are such situations requiring catastrophic maintenance activities.

A maintenance treatment is called preventive, corrective or emergency depending on the condition of the pavement when the treatment is applied, rather than the type of treatment. Maintenance treatments like crack treatment, joint sealing, surface treatments (such as fog seals, slurry seals, seal coats, micro-surfacing, thin bituminous overlays etc.), and maintenance of drainage facilities etc. can be used either for preventive or corrective or emergency maintenance depending on the intended purpose.

In India, major emphasis has been placed on the corrective maintenance approach, so far. Traditionally, in this approach, the functional and structural conditions of the pavements are allowed to deteriorate to fair or poor level before taking steps to rehabilitate them. The aim of rehabilitation is to repair functional and structural damages and restore measurable pavement conditions, such as surface evenness, roughness, rutting and cracking to an acceptable level. In general, in view of limited funds and inter-se-priority, worst pavement section is chosen first for rehabilitation in order to curb its further deterioration; limit repair cost and time; avoid deterioration of pavement approaching to that stage in which only reconstruction option will restore the pavement condition; and of course, to improve public satisfaction and /or perception. As the pavement is selected for treatment nearer to failure level, the applied treatments are more expensive, more time-consuming to construct, and are generally short lived. Deferring the pavement maintenance beyond a certain point may endanger the structural capacity and integrity of the pavement structure. Therefore, the traditional practice of “corrective” and “worst first” maintenance approach is not only a costly and time-consuming activity with associated longer traffic disruptions and inconvenience to users but it is also unable to manage the existing pavement assets in an economic and optimizing manner under increased demands from traveling public and limited budgets.

1.4 PAVEMENT DETERIORATION MODELS

1.4.1 General

Pavements deteriorate with time due to the combined influence of traffic loads and environment, and the knowledge of these factors is essential in order to make predictions for future performance. For road users, the functional condition is important that determines the comfort, safety and user cost. The damages usually result in increased functional deterioration and are important for the highway engineer in planning the maintenance strategies needed for improvement.

It is complex phenomenon to model the pavement behaviour due to the fact that large variations have been found in material characteristics, quality of construction, traffic volume and loadings, environmental factors, maintenance inputs and so on. Although extensive research has been done on modelling of pavements behaviour/performance, the capabilities of available performance prediction models are still limited. This is because pavements seldom fail in a catastrophic way. Because of the inaccuracy in performance prediction, the selection of effective and appropriate maintenance strategies has not been correct sometimes. Pavement performance predictions, both the structural and functional, in conjunction with other inputs help in formulation of rational maintenance budgets and also the selection of best-suited maintenance strategies.

Any mathematical relationship expressing a particular mode of distress by some function of a directly measurable physical quantity or an index derived out of a measurable physical quantity (such as cracking index), is termed as ‘Deterioration Model’. The deterioration model relates a measure of distress due to traffic and environmental factors in terms of a structural response (stress or strain), load and repetitions, pavement composition and geometry, temperature and subgrade moisture.

Deterioration models are mainly classified as deterministic and probabilistic. Deterministic models predict a single value for the life of pavement or its level of distress and probabilistic models predict a distribution of such events and their reliability. The other type of classification of deterioration models is on the basis of analysis, and divides the deterioration models as: (i) mechanistic models that are based on the theoretical analysis in conjunction with experimental data, and (ii) empirical models – that are developed from regression analysis of historical data.

For many years, highway engineers and administrators, the world over, have relied on pavement performance models developed mainly in North America and Europe for planning, design, and rehabilitation of pavements. These models were developed based on the findings of a series of outstanding research projects taken up as a result of the extensive transportation development and paved highway network in these continents. However, these models have some inherent limitations and are not necessarily directly transferable for global application as they are developed from data for local conditions.

1.4.2 Use of Highway Development and Management (HDM-4) Model

Under the World Bank's Highway Design and Maintenance Standards Study, the total transport cost approach to life-cycle cost analysis has been developed and applied. The primary concern of World Bank was to assist developing countries in determining how to allocate scarce financial resources in the highway sector to the best economic advantage, what budgets should be allocated, what standards are appropriate and affordable for the highway network, and how to prioritize and select highway projects.

HDM-4 is one of the most useful and internationally recognised tools available for pavement performance and management analysis. HDM-4 system can be implemented to assist the highway agencies for establishing realistic levels of funding, and to set levels and priorities to maximize the effectiveness of expenditure on pavement maintenance activities. The road deterioration framework developed for HDM-4 is much more flexible and is able to handle a wider range of pavement types. The structured mechanistic-empirical road deterioration models inbuilt in HDM-4 attempt to model the complex interaction between vehicles, environment, pavement structure and surface. The HDM-4 pavement deterioration models thus predict the deterioration of pavement over time and under traffic loading, which is manifested by various kinds of distresses. The inbuilt HDM-4 pavement deterioration models need to be calibrated for local conditions before being used. Thus, the calibrated HDM-4 models are highly helpful to highway administrators and engineers for decision making and developing future pavement maintenance and rehabilitation strategies.

InHDM-4, the bituminous pavement deterioration is predicted through eight separate distress models, namely: cracking, raveling, potholing, edge repair, rutting, surface texture, skid resistance, and roughness. These can be considered under the following three categories:

- **Surface distresses**–comprise of cracking, ravelling, potholing and edge-break. The first three distress modes are characterized by two phases referred to as initiation and progression. The initiation phase is the period before distress of a given mode or severity develops. The progression phase refers to the period during which the area and the severity of distress increases. Edge-break is modeled only through its continuous progression.
- **Deformation distresses**–comprise of rutting and roughness. Deformation distress modes are continuous and represented by only progression equations. As it is partially dependent upon the surfacing distress, roughness is computed after the change of surfacing distress.
- **Surface texture –** comprises oftexture depth and skid resistance. Surface texture distress modes are continuous, and like deformation distress modes they are modeled only through their progression.

1.5 NEED OF THE STUDY

India is developing the national highway network through widening and rehabilitation of existing highways. Developing the road network may be considered as the most important activity for economic development of the country. But in contrary, without maintaining it scientifically, it will be a huge economic loss to the country. Therefore, there is a need for developing maintenance and improvement strategies for planning and budgeting, which is an important activity for the rational allocation of funds. HDM-4 tool has the ability to create maintenance strategies once the pavement deterioration models have been configured and calibrated for the local conditions. The calibration of these models is possible only when the inputs data required for the models are realistic and structured in an objective manner. Time-series pavement performance data are needed for the study which consequently would become the input data for calibration of HDM-4 pavement deterioration models. This clearly implies that the study section matrix has to be exhaustive/comprehensive and all physical and structural elements/parameters of a pavement section, located in different climatic/environmental conditions, should be included therein. The range of calibration factors, obtained for various pavement deterioration models of HDM-4, shall ultimately be used as inputs to run HDM-4 software for development and analysis of maintenance strategies for pavement sections.

Unprecedented growth of road traffic, high variations in pavement temperature and the need for long lasting pavements have increased the use of modified bitumen especially in wearing courses of many flexible pavement road sections. Crumb rubber-modified bitumen (CRMB) and polymer-modified bitumen (PMB) are most commonly used modified binders in India, since their use has shown improved mix performance properties than mixes with conventional (unmodified) bitumen. During the design life, bituminous road sections show different rates of initiation and propagation of distresses under varying traffic and climatic conditions.

The Government of India, through the NHDP, has launched a major scheme to upgrade, improve, and strengthen the existing National Highways network. This massive programme is being implemented through various phases of the NHDP. The current status under various phases of NHDP is given in Tables 1.2and 1.3.

For the purpose of this study, pavement sections from completed road network, as indicated in Tables 1.2 and 1.3, were considered as the high-speed road corridors. A number of sections were planned to be selected under the present study by considering different traffic and environmental (temperature-moisture) conditions towards finding out the calibration factors of pavement deterioration models built into HDM-4 tool.

In the present research study, an effort has been made at the country level to calibrate HDM-4 road deterioration models for the selected flexible pavement sections with modified binders in wearing courses. The different road distresses are modelled using HDM-4 tool for the newly constructed flexible pavement sections of Indian national highway network having modified binder in bituminous concrete (BC) mixes located in different regions of the country. Results of the study are useful for developing and budgeting suitable pavement maintenance management strategies for Indian national highways with different climatic conditions, pavement compositions, and traffic characteristics.

Table 1.2 High-speed corridors under NHDP

Phases of NHDP	Total Length (km)	Already 4/6 lane (km)	Under Implementation (km)
GQ	5846	5846	0
NS-EW Phase I& II	7142	6568	300
Port Connectivity	435	383	52
NHDP Phase III	11,809	7621	2161
NHDP Phase IV	13,203	4058	6050
NHDP Phase V	6500	2564	1428
NHDP Phase VI	1000	-	184
NHDP Phase VII	700	22	94
NHDP Total	46,635	27,062	10,269

Source: NHAI (2017); GQ: Golden Quadrilateral; NS-EW: North South- East West Corridor; NHDP: National Highways Development Program

Table 1.3 Expressways under NHDP

Sl. No.	Name of Expressway	Length (km)
1.	Ahmedabad- Vadodara Expressway	95
2.	Mumbai-Pune Expressway	93
3.	Gurgaon Expressway	40
4.	NOIDA-Greater NOIDA Expressway	30

Source: NHAI (2017)

1.6 OBJECTIVES OF THE STUDY

The main aim of this study is deterioration modeling of flexible pavement sections of national highways in India with modified binders in wearing course. The following objectives are framed to achieve the aim:

(i) Selection and classification of high-speed road corridor test sections of NHDP in India under different environmental and climatic conditions with modified binders in wearing course.

(ii) Collection of time series performance data for three years of different parameters including data for pavement distresses, traffic and axle load spectrum, pavement crust composition, pavement material characterisation, temperature and rainfall, and construction and maintenance history.

(iii) Determination of initiation and progression of different in-built Highway Development and Management (HDM-4) tool for high-speed flexible pavement sections with modified binders in wearing course.

(iv) Determination of calibration factors for progression of different distress models of HDM-4 tool for Indian conditions.

(v) Validation of calibration factors found for different in-built pavement distress models of HDM-4for high-speed corridor sections with modified binders in wearing course.

1.7 ORGANISATION OF THE THESIS

The thesis is organized in a manner closely reflecting the scope of work as defined in the previous section.

Chapter 1 defines the role of road transportation system in the economic development of any nation. It is also mentions the necessity of scientific tools and models for evaluation of road projects. It also introduces the current road network scenario in India, the available pavement types and, the importance and use of modified binders on highways. The various types of pavement maintenance and rehabilitation techniques have been highlighted. Different types of pavement deterioration models and the inbuilt Highway Development and Management (HDM-4) models are briefly discussed. The need for development of a scientific modelling for deterioration of different pavement distresses of Indian high-speed corridor network is presented. The aim and objectives of the present research study are finally framed.

Chapter 2 summarizes, by way of a comprehensive review of the literature, how the concept of pavement modelling has evolved across the world. Methods for developing pavement deterioration models are highlighted. This is followed by a discussion on input data needed for model formulation, factors influencing the model prediction capability, and measures to evaluate model performance. An extensive review of studies conducted on pavement deterioration modelling is presented, both from global and Indian perspectives. Comprehensive review of studies attempted with HDM-4 for pavement deterioration modelling is presented. Because of its universal recognition and applicability, the use internationally recognized HDM-4 system is justified to be used for the pavement deterioration modelling in this study.

Chapter 3 highlights the overview of theHDM-4 system. It introduces the role of HDM-4 and its different functions in highway maintenance and management process. The applications of HDM-4software tool for analysis of highway network are also briefly discussed. The chapter incorporates the detailed description of the relevant parameters of HDM-4, which would be used for analysis of Indian highway network. The various forms of inbuilt HDM-4 deterioration models of different distresses are presented. Various parameters of HDM-4 are described, which are relevant to the Indian geographical, climatic, soil, pavement, and traffic conditions. Various operational modules of HDM-4 including input data, technical models, and use of its three application modules of HDM-4 system are described. The role of HDM-4 in the development of deterioration models and overall methodology for calibration and adaptation of these models is also highlighted in this chapter.

Chapter 4 presents a detailed methodology for collection of data required for calibration of HDM-4 models for the selected pavement sections of Indian high-speed corridor network developed under the National Highway Development Programme (NHDP). It presents the ‘window’ based technique for identification and selection of the twenty-three (23) pavement sections at various locations in different environmental and climatic conditions on the national highway network. The methodology and instruments adopted for collection of time-series pavement performance data consisting of inventory data, pavement condition data, traffic data, and other necessary data on the selected test sections are presented in this chapter. Database has been developed for road network, vehicle fleet, road works standards and HDM-4 configuration to provide necessary inputs for executing the HDM-4 models.

Chapter 5 describes the detailed methodology adopted for calibration of HDM-4 deterioration models for Indian conditions. It is also highlights briefly the need of HDM-4 calibration, the different steps and the levels of calibration. It also presents the procedures adopted for determination of calibration factors for different distress initiation and distress progression models. The statistical analysis of calibration results and their interpretations are also shown in this chapter.

Chapter 6 discusses the validation of the calibrated HDM-4 deterioration models. The validation is carried out by observing the differences between distresses predicted by HDM-4 models and the distresses actually observed. The details of chi-square test conducted in the study, and the significance of the results obtained from validation of the calibrated HDM-4 pavement deterioration models are presented in this chapter. The results obtained are analyzed and discussed therein, and various aspects of use of pavement deterioration modelling have been also discussed.

A summary of the work and the conclusions drawn on the basis of the present study are presented in the Chapter 7. Some recommendations for further scope of research in this area are also given in this chapter.

Chapter 2

LITERATURE REVIEW

2.1 HISTORICAL BACKGROUND OF PAVEMENT DETERIORATION MODELS

Pavements are the most important for day-to-day transportation, socio-economic development, commerce, trade, and defence of a nation. Once constructed, road pavements deteriorate as a consequence of several factors, most notably due to traffic loading and environmental weathering. As a consequence, there has been a paradigm shift from new construction to the preservation and upkeep of the existing pavement infrastructure—a task accomplished through adequate and timely pavement maintenance techniques. Most countries all over the world are involved in the development of various pavement performance models for planning pavement maintenance strategies under different pavement performance studies. For many years, highway engineers and administrators relied on pavement performance models developed for planning, design, and rehabilitation of pavements. These models were developed based on the findings of a series of research projects based on general transportation development and paved highway network of various countries. However, most of the models lack geographical transferability, *i.e.,* these prediction models are not applicable globally as they were developed under regional conditions.

In 1969, Massachusetts Institute of Technology (MIT), under the sponsorship of World Bank, initiated the development of Road Transport and Research Investment Model (RTIM) for economic evaluation of investments in roads with low traffic volumes, evaluation of construction projects, and alternative maintenance procedures. This program concluded in 1971 and resulted in an integral relationship between costs of highway construction, maintenance, and utilization. The Highway Cost Model (HCM) developed in the first phase of this programme included the real correlations of road deterioration, costs, and benefits of the road users and number of maintenance policies. The results encouraged the World Bank to enter into an agreement with Transportation and Road Research Laboratory (TRRL), U.K., to correct the deficiencies of empirical model and to develop more accurate relationships.

TRRL initiated a study in Kenya in 1971 with the specific objective of measuring the characteristics of deterioration of roads and the operating costs of vehicles. It was completed in 1974 and a computer model “Road Transport Investment Model (RTIM)” was developed in 1975. The model calculated the total cost of a road and predicted the surface condition with passage of time (Hodges et al.1975).

In 1976, World Bank, MIT, and TRRL collaborated with the objective of developing a single model that would combine the HCM and RTIM models by eliminating the deficiencies of both. Another version that came into existence in 1982 was Road Investment Analysis Model (RIAM) (Parsley and Robinson, 1982), which utilised the structure developed during Phase-I of the MIT study and incorporated the results of the research carried out by TRRL in Kenya, together with the results of other technical studies in the highway engineering carried out since 1970. The equations developed under these studies correlated the effect of highway deterioration with its maintenance, and the World Bank later modified and expanded these models and designated it as Highway Design and Maintenance Standard Model (HDM) (Paterson, 1987).

In 1975, the Brazilian Transportation Planning Agency and United Nations Development Programme (UNDP) jointly carried out a highway research project in Brazil designated as “Research on the Interrelationship between Costs of Highway Construction, Maintenance &Utilisation”. One of the outputs of this project was a model to be used in the economic evaluation of highway investments and was designated as Highway Cost Model. The model was the result of incorporating the equations obtained from the study into the structure of the 1979 version of HDM with addition of features which included simulation of some maintenance operations on paved roads (GEIPOT, 1982).

The World Bank came out with the third version of HDM (HDM-III) in the year 1981. The HDM-III models were statistically estimated from data collected during a multi-year empirical study carried out in Brazil (GEIPOT, 1982). An international collaborative study known as ‘The International Study of Highway Development and Management (ISOHDM)’ was initiated in 1993 to extend the scope of HDM-III models. The statistical relationships were validated and extended using data from several other deterioration studies such as those from Kenya, the Caribbean, India, Texas, *etc.* These revised and improved models came to be known as HDM-4 road deterioration models.

2.2 PAVEMENT DETERIORATION MODELS

Deterioration model is a mathematical equation that predicts the future pavement performance as a function of present pavement conditions, pavement deterioration factors, and material properties. These models aim to explain the behaviour or performance of the pavement in the future using explanatory variables or factors including pavement structure, age, traffic loads, and environmental variables. The generated model helps for systematic planning and management of the roads, and for economic analysis and justification of expenditure for future maintenance activities.

Generally, the pavement deterioration models are developed by using one of the following methods (FHWA, 1990):

- Mechanistic Method
- Empirical Method
- Mechanistic-Empirical Method
- Probabilistic Method
- Bayesian Method

2.2.1 Mechanistic Method

The mechanistic method uses fundamental theories of pavement behaviour for the development of pavement deterioration models. It is based on the theory of mechanics. The method includes elastic layer theory and finite element methods. This method requires detailed structural information, which limits the accurate calculation of pavement stresses, strains, and deflections of the pavement sections for which comprehensive data are available.

2.2.2 Empirical Method

The empirical method is based on the results of lot of experiments or experience. This method requires large amount of data/observations to establish the relationship between the input parameters and output. The empirical method is usually based on the statistical analysis of locally observed deterioration trends, and may not be applicable globally.

2.2.3 Mechanistic – Empirical Method

The mechanistic–empirical method is the most popular method and has been widely applied for design of flexible pavements. This method consists of two steps: (i) determining the response of pavement materials under the applied loading and, (ii) predicting the pavement performance from these responses. The method encompasses a systematic database that includes the structural information, traffic loading and volume, and pavement condition data for each "homogeneous" section of the road (George et al., 1989).

2.2.4 Probabilistic Method

In this method, pavement condition is considered as a random variable with the probabilities associated with its values. The associated probabilities can be described by a suitable probability distribution function. Transition probability matrix (TPM) is developed and used to define the probability that a pavement in an initial condition state will be in some future condition state. TPM is developed for each combination of the factors that affect the performance of pavement. TPM is generally considered as a matrix that is obtained from expert views (Hass et al., 1994).

2.2.5 Bayesian Method

In this method, observed data and expert experience are combined using Bayesian regression analysis. In this analysis, the regression parameters are considered as random variables with associated probability distribution. Bayesian theorem can be expressed mathematically as (Thomas, 1993):

$$P(B \mid A) = \frac{P(A \mid B) \times P(B)}{\sum \left[P(A \mid B) \times P(B) \right]} \tag{2.1}$$

where,

P(A) = distribution of variants over all possible fraction variants

P(B) = prior distribution

P(B|A) = sampling distribution

P(A|B) = posterior distribution

2.3 DATA BASE AND INFLUENCING FACTORS

A pavement database includes a collection of pavement data that provide information adequate to support the models being developed. The major input parameters of pavement prediction models are: pavement age, pavement maintenance history, pavement condition data, and traffic loading effect. The key factors that affect the prediction models are characteristics of subgrade soil, environmental effects, and mechanical properties of pavement materials.

Maintenance, rehabilitation, and reconstruction data have a significant impact on the deterioration models. Unless maintenance, rehabilitation, and reconstruction data are not accounted, the models developed as a function of time will not be accurate (Ramaswamy and Akvia, 1990). Moreover, it is important to consider pavement condition and traffic loading during modelling phase. The traffic loading characteristics includes traffic volume, axle load, axle configuration, tyre pressure, repetition of axle load and vehicle speed.

In 1960s, based on empirical data obtained from the AASHO road test, the load equivalency factors were developed (AASHTO, 1986). On the other hand, the other load equivalency methods are based on pavement response to load (deflection, stress, *etc.*) or distress manifestations (fatigue cracking, rutting *etc.*) (Zhang et al., 2000).

There are several important factors that affect the prediction capability of models. Firstly, the natural soil normally used in road embankment and subgrade is an important factor for pavement design (AASHTO, 1993). The pavement design is mostly based on the soil CBR with respect to traffic loading considered during the designed life. Secondly, the layer thickness, type, and mechanical properties of each material used in pavement are the input parameters for prediction models. Thirdly, there is an impact of environmental conditions on model prediction due to the effect of moisture, temperature, rainfall, and their variations.

2.4 MODEL FORMS AND ACCURACY

In prediction of life cycles, pavement management systems or in the pricing of road use, the models need to predict the expected change of condition in future over a given period of time or under the transit of one extra axle load, when the current pavement condition is known (Paterson, 1987). Moreover, a priori conditions must also be met by prediction models. Such conditions limit the model form to those appropriate for the pavement condition measures being modelled as follows: initial state, initial slope, overall trend, variation in slope, final slope, and final state (Lytton, 1987). The first approach is to normalize the pavement condition to a dimensionless state so that, a pavement in its “new” or initial, state has a damage value of zero, and its “terminal” state has a damage value of one. Such predictive models are termed as damage functions (Paterson, 1987).

The most common tests for determining the precision and accuracy of the predictions models are standard error of estimate, the coefficient of determination, the residual analysis, correlation coefficient, F-test, analysis of variance (ANOVA) etc. (Smith, 1986; Sadek et al., 1996). The selection of test is based on the type of regression used. Regression analysis is used for analysing the causes of variation of a response variable and developing statistical relationships among variables. The coefficient of determination, well known as the R^2, is used by most engineer/researchers, especially in civil engineering to judge the accuracy or adequacy of the developed model.

2.5 INTERNATIONAL SCENARIO OF PAVEMENT DETERIORATION MODELS

Karan (1977) investigated pavement deterioration functions by means of Markov process modelling for maintenance of the pavement located in the Waterloo, Ontario. The variation of pavement performance deterioration with age was modelled by Markov process with constant transition probability matrices (TPM). Moreover, each element of the TPM was built based on the average opinions obtained through individual interviews and questioners.

The relationship between pavement distress and performance was established by Smeaton et al. (1980). In this study, the suitability of two widely used models of pavement performance was examined. Pavement models were formulated based on the following hypotheses: (a) the levels of various types of pavement distress behaviour and, (b) the pavement behaviour elements. The variability was investigated by postulating a deterioration mechanism for flexible AASHO road test sections.

In order to evaluate the pavement conditions, two parameters considered were: distress manifestation index and ride comfort rating. Generally, the distress was ratified from severe to slight in 5 categories. The formula to calculate the distress management index (DMI) is as follows (MTCO, 1980):

$$\mathrm{DMI} = \sum W_i (S_i + W_i) \tag{2.2}$$

where,

W_i = weighting value representing relative weight of a distress manifestation
S_i = severity weighing factor
D_i = density weighing factor

Phang and Stott (1981) carried research to examine the distresses prior to maintenance work on Brampton road test sections in Canada. At any time, the distress manifestation was assigned to a pavement, which was the summation of weighted values for condition (severity and extent) of each type and class. Unless weighted values for each type of distress were examined for the given time, it was difficult to determine whether certain types of distress led to rapid failure.

Ullidtz (1985) proposed analytical mechanics method for prediction of pavement condition and structural defects of rehabilitation. A method was introduced by Dynatest in Denmark for managing the pavement maintenance and rehabilitation. At a different project level, this method predicted feasibility for maintenance or rehabilitation.

Hajek et al. (1985) compared the prediction capabilities of 5 different models including PARS model (empirical, pavement classes), OPAC model (mechanistically derived), power curve (empirical, site-specific), sigmoid curve (empirical site-specific), and factored PARS model (Bayesian approach, site-specific). It was observed that the accuracy of the empirical site-specific models for sigmoid curve was better than the other types.

Bertelsen (1987) used the model developed by Public Roads Administration for Norway to predict the deterioration of longitudinal homogeneous roadway sections with layers of asphalt and unbound materials. Depending on climate, traffic and road data, the model was used to predict the damage and other parameters for the pavements.

Jackson et al. (1987) reported the findings from visual survey conducted by Washington State Department of Transportation (WSDOT) for every two years on their state highway network. An improved pavement management system was developed by WSDOT to predict pavement condition for each project. In case of flexible pavements, alligator cracking, longitudinal cracking, transverse cracking, and patching were identified during pavement evaluation. The final pavement condition rating (PCR) was a combination of visual rating and ride rating as shown in Equation 2.3:

$$PCR = \left(100 - \sum D\right)\left(1 - 0.3\left(\frac{CPM}{5000}\right)^{2}\right) \tag{2.3}$$

where,
ΣD is the sum of the detect values,
CPM is the counts per mile.

Lee et al. (1987) emphasized the need for simplified pavement performance models based on minimal amount of pavement data. Five models were developed for all conventional pavement types based on the parameters such as Present Serviceability Rating (PSR) using pavement age, cumulative equivalent single axle load (ESAL), and pavement structural number (SN). Moreover, anew calibration technique was incorporated into the models, which helped to predict the performance of both new and existing pavements.

The Foundation of Scientific and Industrial Research (SINTEF) of Norwegian Institute of Technology developed deterioration models for pavement performance prediction based on rutting and roughness. The factors such as climate, traffic and road data were found to influence the developed models that predicted the development of damage and pavement conditions (Bertelson, 1987).

Bourdeau (1990) studied a probabilistic approach for solution of many uncertainties and random factors playing role in the deterioration process of pavements under the effect of traffic. However, rational assessment of a pavement section was obtained by considering the design equation as a function of traffic loads and California bearing ratio (CBR). A sensitivity analysis showed that the CBR had a dramatic influence on pavement reliability.

Roberts et al. (1991) proposed a regression equation for Present Serviceability Index (PSI) as shown in Equation 2.4. The model evaluates the pavement serviceability based on the measurable pavement distresses.

$$\mathrm{PSI} = 5 - 1.91\log\left(1+\mathrm{SV}\right) - 1.38\left(\mathrm{RD}\right)^2 - 0.01\left(\mathrm{C}+\mathrm{P}\right)^{0.5} \tag{2.4}$$

where,
SV = slope variance
RD = average rut depth
C = pavement cracking in feet/1000 square feet of pavement surface
P = patching in square feet /1000 square feet of pavement surface

Saraf and Majidzadeh (1992) described the procedure to develop the distress prediction models for a network level PMS for Ohio Department of Transportation. Fourteen distress types were visually surveyed for the overlaid pavements. These distresses were grouped into four major groups. Models were developed to predict the distresses and pavement condition rating. It was reported that the developed models were able to predict the performance with reasonable accuracy for both individual pavement sections and for a group of pavements.

Johnson and Cation (1992) reported a study conducted in North Dakota in which performance curves for 42 different performance class pavements were presented. The original pavement data were categorized into traffic and structure groups and then analysed to develop performance curves. It was reported that the fourth degree polynomial showed best fit for distress and structural indices.

Harper and Mazidzadeh (1993) studied 13 types of pavement distresses, which were considered in the OHIO method. Pavement Condition Rating (PCR) reflects composite effect of various distress types, severity, and extent of all conditions. PCR was computed by subtracting the total deduct points from 100 as shown in Equation 2.5.

$$PCR = 100 - \sum \text{deduct } i \tag{2.5}$$

where,

deduct i= weight for distress x weight for severity x weight for extent

Al-Omari and Darter (1994) developed the relationships between present serviceability rating (PSR) and International Roughness Index (IRI) for flexible, rigid, and composite pavements. PSR was determined as the mean of user panel rating for rideability on the conventional scale from 0 to 5. Based on the data collected from six states including Louisiana, Michigan, New Jersey, New Mexico, Ohio, and Indiana, a relationship was developed between PSR and IRI for each state and for all the states together. It was observed that there was no significant difference between the models developed for different states. The final developed relationship between flexible, rigid, and composite pavements for all states together is shown in Equations 2.6–2.8.

$$PSR = 5 \times e^{(-0.24 \times IRI)} \tag{2.6}$$

$$PSR = 5 \times e^{(-0.272 \times IRI)} \tag{2.7}$$

$$PSR = 5 \times e^{(-0.293 \times IRI)} \tag{2.8}$$

where,

IRI is in millimeters per meter.

Li and Hass (1994) developed a methodology for evaluating the pavement condition in terms of strength index, distress ratio, riding quality index, and skid-resistance coefficient. Erlando and Chunhua (1994) presented the model developed by Minnesota Department of Transportation (MDOT), USA for prediction of future distress levels rather than the prediction of a composite index. Further, Michael (1994) presented the Customized Pavement Management System, which was developed for Port Orange, Florida. The identified types of distress were alligator cracking, longitudinal, and transverse cracking.

Chua et al. (1994) reported a structure for a pavement management system employing mechanistic performance sub-models for predicting pavement behaviour. This approach enabled the individual distress modes to be addressed in deciding about pavement management strategies.

Attoh-Okine (1994) evaluated the capabilities of artificial neural networks (ANNs) in predicting roughness progression in pavement from element deformation. Deformation was considered a function of modified structural number, incremental traffic loadings, extent of cracking and thickness of cracked layer, incremental variation of rut depth, and surface defects. Further, the surface defects were considered as function of changes in cracking, patching and potholing. In addition, the environmental and non-traffic related mechanisms taken were the function of pavement environment, time, and roughness.

Wang et al. (1994) developed TPMs for Arizona Department of Transportation by using a large number of observed pavement history data for categorized highways with several different initial pavement condition states. Further, Li et al. (1996) described the non-homogeneous Markov probabilistic modelling program. The model was developed to determine the pavement deterioration rates at various stages. The TPMs were considered as a time-related transition process. Each element of TPMs was determined based on a reliability analysis and Monte Carlo simulation technique.

Chen et al. (1995) investigated one of five indices developed for Pennsylvania models through Surface Distress Index (SDI). The distresses utilized to develop this model were excess asphalt, ravelling and weathering, block cracking, transverse and longitudinal cracking, edge deterioration, widening drop-off, and rutting. The developed SDI model is shown in Equation 2.9.

$$\begin{aligned} \text{SDI} = {} & 0.1(\text{Excess Asphalt}) + 0.13\ (\text{Ravelling and Weathering}) \\ & + 0.20(\text{Block Cracking}) + 0.25\ (\text{Transverse and} \\ & \text{Longitudinal Cracking}) + 0.05\ (\text{Edge Deterioration}) + \\ & 0.12\ (\text{Widening Drop-off}) + 0.15\ (\text{Rutting}) \end{aligned} \qquad (2.9)$$

Sadek et al. (1996) developed pavement deterioration models for Virginia’s interstate highways. Step-wise regression technique was used to develop the deterioration models. Parameters such as pavement age, thickness of overlay, average annual ESALs, and structural number were used in the regression analysis. Simple linear model, power model, and sigmoid model were developed. Goodness-of-fit statistics and ANOVA were used to check the accuracy of the models. It was reported that the power law model form showed better prediction for all pavement types except flexible pavements with no overlay. For the flexible pavements with no overlay, sigmoid model form was found to be more appropriate.

Hajek and Bradbury (1996) developed a pavement deterioration model using Bayesian statistical analysis approach for asphalt concrete surfaces containing steel slag. The developed model was based on the combined information derived from the observations of 79 existing projects. The developed model predicted the pavement performance in terms of a distress index, which was a function of age, asphalt content of the mix, and traffic volume. It was reported that the Bayesian statistical analysis approach improved the scope, reliability, and predictive power of the models.

Kerali et al. (1996) derived a model for rutting using parameters such as material properties, layer thickness, and aggregate types in Strategic Highway Research Program (SHRP) in association with Long-Term Pavement Performance (LTPP) experiment. It was observed that the material properties, layer thickness, and their combined effects had significant influence on the rutting characteristics.

Rezqallah (1997) reported model developed by Ministry of Transport (MOT) in the Kingdom of Saudi Arabia according to Ohio pavement condition system. Based on Present Serviceability Rating (PSR), the model was formulated. In case, there was more than one distress type, it was recommended to use a correction factor (C) of 0.70.

Shoukry et al. (1997) introduced fuzzy sets approach for assessing pavement condition and reported the pavement performance in terms of fuzzy distress index (FDI). FDI combines the structural distress with roughness to describe the overall status of the existing pavement section. It was reported that FDI model was an extremely flexible measure unlike PSI or PCI models for determining the overall pavement condition.

Li et al. (1997) studied the relationship between deterministic and probabilistic prediction models in pavement management. The deterministic pavement deterioration models were converted to probabilistic models, which can be used in pavement management to perform the following functions: (a) simulate the probabilistic behaviour of pavement deterioration in predicting pavement serviceability level in each year; (b) determine the required year(s) and a list of yearly rehabilitation or a maintenance priority program for each pavement section in the road network within the programming period; and (c) to provide information and inputs for dynamic pavement rehabilitation and maintenance priority programming of pavement management at the network level.

Forrai-Hernadi et al. (2000) described the pavement performance models in the form of generalized deterioration curves which were developed for different pavement conditions, where the pavement condition was a function of pavement age and traffic loading. These performance models were effectively utilized in PMS of Hungary.

Wang (2000) calculated the pavement performance by the model that depended on the pavement condition index for individual distress. The deduct-value concept was adopted in formulating the distress indices for asphalt pavements. The Pavement Distress Index (PDI) used for pavement evaluation was based on a different set of distress definition and measuring method. In the formulation of PDI, weighted function played a significant role as shown in Equation 2.10.

$$\text{PDI}=100-\sum\sum DP_{ij}\times W_{ij} \tag{2.10}$$

where,

DP_{ij} is a deduct value for distress type *i* and severity *j* and $W_{ij}$is an adjustment factor for multiple distress that varies with the proportion of the deduct value to the total summed deduct value.

The Pavement Condition Index (PCI) concept was developed for assessment of pavement condition by US Army Corps of Engineers (Shahin, 2002). The PCI index depended on pavement condition parameter, *i.e.* distress. The PCI is determined as shown in Equation 2.11:

$$\text{PCI} = C - \sum\sum a\left(\left(T_i, S_j, D_{ij}\right) \times D_{ij}\right) \times F \tag{2.11}$$

where,

T_i = distress type

S_j = severity level

D_{ij} = density of distress

C = constant (usually 100)

a = weighing factor

F = adjustment factor for multiple distress.

The pioneer research in development of mechanistic-empirical performance prediction models was given by Ullidtz (2002), Busch et al. (2005), and Hilderbrand (2006). The Mathematical Model of Pavement Performance (MMOPP) developed based on mechanistic-empirical approach was capable of predicting longitudinal roughness, rutting and fatigue cracking of a pavement consisting of bitumen or cement bound layer, a granular base and subbase layer and subgrade. The incremental-recursive procedure was used to simulate deterioration over time, where the output from one-time increment was used, recursively, as an input for the next step increment.

RRM (2007) reported a model for Riyadh PMS, which was combination of index of pavement distresses and pavement condition. Urban Distress Index was calculated based on pavement distress type, severity, and density. Performance and Economic Rating System estimated pavement performance in association with structural deterioration, rutting, roughness, skid resistance, and surface wear.

Abo-Hashema and Sharaf (2009) developed a decision system called maintenance unit (MU) through research conducted at Cairo University, Egypt. The developed MU system determined maintenance and rehabilitation activities on the basis of density of distress repair methods, and not the density of individual distresses.

Bekheet et al. (2008) studied the probabilistic and deterministic approach for pavement performance models in pavement management system. It was reported that the both probabilistic Markov process and deterministic models were comparable in project level and network level analysis.

2.6 PAVEMENT DETERIORATION MODELLING UNDER INDIAN SCENARIO

Central Road Research Institute (CRRI), New Delhi (CRRI, 1986) conducted a research study entitled ‘Pavement Performance Study (PPS)’ under the sponsorship of Ministry of Surface Transport (MOST), Govt. of India during 1986 to 1993 with the aim to develop total transportation cost model for Indian conditions. This study contained extensive time series data and pavement model relationships. They developed pavement deterioration models to predict different modes of distresses on pavements including cracking, ravelling, potholes, and roughness, which are most significant from the point of road maintenance and user cost. A software package ‘PDM’ was also developed as a convenient tool for making good use of the pavement deterioration models (CRRI, 1994). This study was further extended to specially designed and constructed pavement sections on in-service highways for more accurate data generation and refinement of models developed under existing pavement sections (Sood and Sharma, 1996).

Sharma (1986) carried out a study on pavement performance evaluation of some typical road sections at the University of Roorkee. In this study, PSI models were developed based on functional and structural aspects of pavements. This study concluded that pavement performance evaluation using serviceability techniques proved to be a ready to use multi-purpose maintenance tool.

Rao (1989) studied the performance of full depth granular pavements sealed with thin bituminous surfacing. Benkelman beam deflection was carried out for structural evaluation of pavements and adopted rutting and cracking as failure criteria. A relationship was developed to estimate average rut depth as a function of vertical subgrade strain and a cumulative number of standard axles as shown in Equation 2.12.

$$RD = -0.256 + 6.79 \times SVS + 3.08 \times ESAL \tag{2.12}$$

where,

RD is the mean rut depth (in)

SVS is the subgrade vertical strain (10^{-3})

ESAL is the equivalent single axle load

Krishna Murthy (1991) proposed the PSI model in terms of Unevenness Index (UI) in (cm/km). The relationship is given as under:

$$PSI = 315(UI)^{-0.822} \tag{2.13}$$

For calculating UI, following equation was used:

$$UI = (B/W) \times R \times 2.54 \tag{2.14}$$

where,

B is the bump reading from the field (after initial setting to zero)

W is the number of wheel revolutions

R is the number revolutions per km (460)

Mohanty (1992) analysed pavement performance data collected from several sections of rural roads having granular pavements with thin bituminous surfacing. A correlation was developed with analytical response of pavements for developing a performance based rutting criterion as shown in Equation 2.15.

$$N = 3.42 \times 10^{-11} \times \left(\frac{1}{\varepsilon_z} \right) \tag{2.15}$$

where,

N is the number of standard axle repetitions to cause 50 mm rutting,

ε_z is the vertical subgrade strain.

Jain et al. (1994) developed the structural and roughness deterioration models to assess the need for maintenance management of flexible pavements. These models were then calibrated and validated utilizing the data of other test sections. Reddy and Veeraragavan (1995) developed pavement maintenance management models for rural highways in India. The objectives were to determine the rate of structural deterioration of flexible pavements on rural highways measured in terms of pavement rebound deflection, and rate of progression of functional deterioration in terms of unevenness index (roughness).

Jain et al. (1996) carried out a study to determine the influencing parameters for efficient maintenance management of flexible pavements. Models were developed to predict deflection, rut depth, cracking and maintenance cost with time. A condition responsive maintenance strategy was recommended based on the field and laboratory studies.

Nagaraja et al. (1996) adopted a stochastic decisions process for PMS of 353 km of road length. TPM was developed by considering three variables namely maintenance, rehabilitation, and pavement condition. An effective strategy for pavements under different condition states for a period of five consecutive years was developed for the national, state and district roads. The three condition variables used were pavement unevenness index (UI), extent of cracking, and pavement rutting.

Reddy et al. (1999) obtained the relationship between rebound deflection and traffic repetitions in terms of cumulative standard axles. Deflection progression models were developed for four age categories of pavements based on initial deflection ranges. Statistical analysis was used to calculate the cumulative standard axle data corresponding to pavement ages. The developed rutting models were used to predict the rut depth at any time after overlay construction, whereas rut depth progression models were used to predict the progressed rut depth from an initial rut depth with an increment in traffic loading for in-service flexible pavements.

A mechanistic design method was developed by Das and Pandey (1999).This method was developed by correlating the bituminous pavements performance data collected from different parts of India to obtain critical stress-strain parameters. Rutting and fatigue failure criteria of flexible pavement design for Indian highways were established based on the fatigue and elastic properties of pavement materials, and field performance data. A thickness design chart was also developed for bituminous pavements.

Reddy and Veeraragavan (1999) investigated the effect of overloading and introduction of tandem axle trucks on pavement life. A deflection growth model was developed using historical data from six overlaid flexible pavement sub stretches with different initial deflection values. Ramesh and Veeraragavan (1999) developed pavement performance model and determined the life cycle cost for National Highways in Karnataka state.

CRRI (2000) developed maintenance management system (MMS) for rural roads in three districts of Maharashtra. The three districts Pune, Raigarh, and Yavatmal were selected on the basis of different geographical, topographical, and socio-economic status. Roughness and distress progression models were developed for paved and unpaved rural roads for prediction of distress and to make decisions for maintenance of rural roads.

Verma (2006) conducted study on 51 rural roads from eight districts of Uttarakhand state in India, which were selected based on traffic, terrain, rainfall, soil, climatic conditions, and age. Maintenance strategies were developed based on pavement condition index (PCI) and present serviceability index (PSI). Priority indices for maintenance, PCI based condition models, and PSI progression models were developed for rural roads.

Mathew et al. (2008) developed deterioration models for rural roads in India using ANN and regression techniques. Eight rural roads in Thiruvananthapuram district of Kerala were selected in this study. The details of each test section including pavement age, pavement thickness, subgrade strength, and severity of different distresses was collected. ANN and regression techniques were then used to develop the deterioration models for ravelling initiation and progression, pothole progression models, roughness progression, and edge failure. The predicted values for the developed models using ANN and regression techniques were compared with the actual observed values. It was concluded that the models developed using ANN technique were more suitable than regression technique due to inherent ability of the ANN to adjust to changing environment.

Sandra and Sarkar (2008) developed relationship between PSI and noticeable distress parameters commonly observed on Indian roads. SPSS software was used to develop a multiple linear regression model to find a relationship between reduced PSI values and pavement distress parameters for different classes of Indian highways.

Kumar and Patel (2009) conducted a study to develop pavement deterioration models for rural roads of India by conducting deflection and roughness measurements over 18 Pradhan Mantri Gram Sadak Yojana (PMGSY) sections of Uttarakhand and Uttar Pradesh. ANN and regression analysis were used to develop the models. The most realistic model was determined based on residual sum of squares (SSR) and coefficient of determination (R^2), and also considering logical relationship between input and output parameters. Polynomial relationship was found to be best fit between input parameters, such as, CBR of subgrade, pavement age, pavement thickness, and pavement condition indicators such as roughness and deflection. The best chosen models were validated using the paired t-test. Maintenance priority index (MPI) was also calculated for 18 PMGSY roads for prioritizing the road maintenance work using three parameters namely roughness, deflection, and traffic.

Rastogi et al. (2011) developed pavement performance models for low volume pavements in Uttarakhand and Uttar Pradesh states of India. Structural and functional responses were continuously measured for two years for the selected 18 sections. Statistical tools and ANN were used to develop the models. To select the best fit model, statistical performance indicators and logical relationships between input and output parameters were used.

Sandra and Sarkar (2012) developed a model to determine the relationship between roughness and commonly observed distresses on Indian roads such as rutting, ravelling, potholes, cracking, and patching. The model was developed by determining the extent and severity of distresses over 39.5 km length of road (consisting of National Highways, State Highways and Major District Roads) in Rajasthan state of India. The developed model between roughness and observed distresses is shown in Equation 2.16. The final relationship between International roughness index (IRI) and International roughness index due to distress only (IRI_D) is shown in Equation 2.17.

$$\begin{aligned} IRI_D\ (m/km) = {} & 0.0143\times RL_p + 0.0216\times RM_p + 0.0345\times RH_p + \\ & 0.0303\times PAL_p + 0.0418\times PAM_p + 0.0432\times PAH_p + \\ & 0.111\times PL_p + 0.151\times PM_p + 0.178\times PH_p + \\ & 0.0103\times CL_p + 0.0156\times CM_p + 0.0316\times CH_p + \\ & 0.0018\times RUL_p + 0.0023\times RUM_p + 0.0034\times RUH_p \end{aligned} \tag{2.16}$$

where,

IRI_D = IRI due to distresses only in m/km; RLp = low severity ravelling in % of area; RM_p = medium severity ravelling in % of area; RH_p = high severity ravelling in % of area; PAL_p = low severity patching in % of area; PAM_p = medium severity patching in % of area; PAH_p = high severity patching in % of area; PL_p = low severity potholes in % of area; PM_p = medium severity potholes in % of area; PH_p = high severity potholes in % of area; CL_p = low severity cracking in % of area; CM_p = medium severity cracking in % of area; CH_p = high severity cracking in %

of area; RUL_p = low severity rutting in metres per km; RUM_p = medium severity rutting in metres per km and RUH_p = high severity rutting in metres per km.

$$IRI\ (m/km) = A + IRI_D \tag{2.17}$$

where, *A* = 2.4 for NHs, 2.8 for SHs and 3.0 for MDRs.

2.7 USE OF HDM-4 IN INTERNATIONAL SCENARIO

Australia – The Queensland Department of Transport employed HDM-III as the investment analysis tool for the Queensland road network in concert with other in-house analysis control software for calibration of local models (Robertson and Charmala, 1994).The Queensland road network consisted of a hierarchy of state controlled roads representing the major state highways connecting developmental and arterial routes, and local roads and residential streets, which were administered by local authorities. Calibration efforts were concentrated on roughness and cracking performance of chip seal on granular pavements, and asphalt-surfaced pavements. It was observed that the default (Brazilian) calibrations produced partially acceptable results, but still deficient predictions of performance of Queensland pavements. Calibration of HDM-4 models was carried out for eight LTPP-Maintenance (LTPPM) sections: four in Victoria, two in Queensland, one in New South Wales, and one in Tasmania (Tepper and Martin, 1999).

Bangladesh–The Roads and Highways Department (RHD) of the Government of Bangladesh manages 20,854 kilometres of main arterial and subsidiary road network of the country. Management of this road network in an effective and efficient manner requires a working Road Maintenance Management System (RMMS). The Institutional Development Project (IDC 3) was initiated in 1994 as an effort towards developing an RMMS. Currently, an annual survey of condition of the road network was undertaken and the HDM-4 models were used for analysing this data in conjunction with the road database (Karim, 2001).

Colombia – HDM-4 deterioration models were used in Pavement Management System (PMS) in Bogota, Colombia. The management subsystem of the software was used to report the execution and investment plan. Here, the Maintenance Subsystem was used to define maintenance activities (Arguelles et al., 2011).

Brazil – Under the Brazilian National Highway Development Program, a systematic approach of pavement management started in 1983 for application on the paved federal road network, and later several states in Brazil gradually adopted the developed methodology. HDM-III model was used to simulate total life cycle costs and performance for road maintenance and rehabilitation alternatives. The data collected in Brazil were under a wide range of environmental conditions, and were used to develop sub models of HDM-III and to quantify the relationships between the road construction, maintenance cost and vehicle operating cost (Queiroz et al., 1992).

Chile – DeSolminihac et al. (2003) calibrated the structural cracking models in HDM-4 version 1.1, and compared the results from equivalent HDM-III models. HDM-4 version 1.1 for cracking models was recommended due to their operational advantages though the results of the HDM-III and HDM-4 calibrated models have similar values. Valdes et al. (2011) discussed the calibration of HDM-4 models used by the Chilean National Roads Authority. Calibration was carried out for the cracking, ravelling, potholes, rut depth, and roughness models of HDM-4 under Chilean conditions. Calibration factors were updated for various models in HDM-4 for asphalt pavements located in a diverse range of geographical areas in Chile.

Indonesia – Morosiuk et al. (1999) discussed the deterioration modelling of bituminous pavements of inter-urban roads of West Java in Indonesia withHDM-4 framework. Transport Research Laboratory (TRL) in the UK and the Institute of Road Engineering (IRE) in Indonesia evaluated the performance of road strengthening overlays in Indonesia under a comprehensive research programme. HDM-4cracking, rutting, and roughness relationships were calibrated against observed rates of deterioration.

Japan – Taniguchi and Yoshida (2003) discussed the results of calibrating HDM-4 rutting model on Japanese national highways and compared the HDM-4 rutting prediction model included in MLIT-PMS (Pavement Management System) developed by the Ministry of Land, Infrastructure, and Transportation (MLIT) of Japan. It was observed that the HDM-4 application was most reliable than MLIT-

PMS in both dense graded asphalt concrete pavement and porous asphalt pavement.

Kenya - Odoki (2016) conducted a study for the adaptation of HDM-4 for Kenyan roads. Configuration, calibration, and validation were the major activities involved in the study. Traffic-flow pattern, speed-flow types, accident classes, climate zones, road network aggregate data, road deterioration, work standards and effects, road user effects, and vehicle fleet were configured according to the Kenyan road scenario. Road deterioration models in terms of ravelling, cracking, edge-break, potholes, rutting, friction, roughness, and drainage were calibrated. Validation was performed for the developed deterioration models with the data not used for calibration. It was concluded that HDM-4 was a comprehensive decision support tool for predicting the pavement performance.

Malaysia – Every year, the Public Works Department of Malaysia (JKR) devotes considerable resources towards the routine and periodic maintenance of its 15000 kilometres of Federal road network. For that purpose, JKR uses a pavement management system called PAMS (Pavement Management and Appraisal Suite), which carries out project level analysis and then aggregates the outcome to form network level outputs. PAMS was developed and calibrated by JKR during the Malaysian National Axle Load Study. PAMS was microcomputer-based software and runs on DOS platform. This decision-support system was capable of performing whole-life costing of the road network under the user-specified environment, over an evaluation period of 2 to 20 years. It incorporates the pavement deterioration sub-model and the vehicle operating cost sub-model from HDM-III. The other sub-models within PAMS can internally determine pavement thickness, model road user costs, determine construction costs and assess the Net Present Value (NPV) for each maintenance policy selected (Onn-lai, 2001).

Namibia – The Namibian Road authority practiced a comprehensive and formal Road Management System over a period of ten years. To identify and prioritize rehabilitation, the system utilizes both the capabilities of HDM-4 and local experience. It was reported that the HDM-4 recommended very low long-term budget as compared to local experience models (Zyl and Tekie, 2008).

New Zealand – Bennet (2000) conducted a study with the objective to have a completed preliminary system integrated with the existing National Road Asset Management Program (RAMM). This consisted of a basic inventory and pavement condition database along with an algorithm for selecting maintenance treatments. The new PMS was built on the existing road management inventory system and existing funding framework. The software package DTIMS (Deighton Total Infrastructure Management System) along with a hybrid set of predictive models from HDM-III and HDM-4 were adopted for the development of PMS. The PMS would be used by over 70 different road-controlling authorities (city, district and state level) responsible for a network of more than 100,000 km of sealed and unsealed roads.

Pakistan – The Communications and Works Departments of the four provinces of Pakistan: Punjab, Sindh, North-West Frontier Province, and Balochistan were responsible for the maintenance of a total of over 90,000 km of roads. A comprehensive data collection program started in early 1997 and a road database was established for storage of data in every province. Design of the database was done in cooperation with international and local consultants and the implementation of the system was done locally to ensure sustainability in maintenance of the system. HDM-III was utilized as the tool for preparation of road deterioration and vehicle operating cost models (Vincent et al., 2000).

Philippines – The government of the Republic of Philippines introduced PMS considering local conditions. For the road network, the system consisted of automated pavement condition data collection technique, data management, data presentation techniques, and a pavement investment analysis HDM-III system comprising with an accurate and up-to-date locational reference system (Howard et al., 1994).

Portugal –The Maintenance Optimization System (MOS) of Portugal PMS used a global deterministic pavement performance prediction model. The PMS followed AASHTO flexible pavement design method. The new MOS (GENEPAV-HDM-4) used a similar optimisation model but the AASHTO pavement performance prediction model was substituted by the HDM-4 pavement performance prediction models to account for Portuguese local condition. From the results, MOS proved to be a valuable addition to the road engineer's toolbox (Jorge and Ferreira, 2012).

South Africa - Botswana was the first county in South Africa to actively pursue the inclusion of HDM-III pavement performance models into their road management system (Rohde et al., 1998). Subsequently, locally calibrated HDM-III performance models were included and used in the pavement management systems of several provinces in South Africa. In 1993, Gauteng province of South Africa decided to calibrate the HDM-III and HDM-4 pavement deterioration models of provincial PMS for Long-Term Pavement Performance Monitoring (LTPP) sections. Thirty-six (36) LTPP sections were selected and monitored annually. As a result of 15 years of yielded data from LTPP sections, calibration results were compared with actual PMS trends of the past. The final evaluated calibration factors for the 36 sections are shown in Table 2.1. It was found that the predicted network conditions using the calibrated models correlated well with the actual observed values (Rohde et al., 2002).

Sweden – Anita and Leif (2003) used the Swedish Pavement Management System (PMS) and HDM-4 for making decisions in the management of road infrastructure. The road user effects models included in Swedish PMS were based on the models in HDM-4 to some extent.

Table 2.1 Evaluated calibration factors (Rohde et al., 2002)

Distress Model	Calibration Coefficient
Cracking initiation (K_{ci})	0.62
Cracking progression (K_{cp})	0.26
Ravelling initiation (K_{vi})	0.58
Ravelling progression (K_{vp})	0.15
Pothole progression (K_{pp})	1.00
Rut progression (K_{rp})	3.00
Rut standard deviation progression (K_{rpe})	0.80
Riding quality (K_{ge})	1.00
Riding quality (K_{gp})	1.00

United Kingdom - Kerali et al. (1998) presented the newly incorporated features of HDM-4 over HDM-III. HDM-4 incorporated a wider range of technical relationships with three application tools for project level analysis, road work programming under constrained budgets, and for strategic planning of long term network performance and expenditure needs. The technical relationships for pavement deterioration for flexible and unsealed pavements, and vehicle operating costs were modified in HDM-4. The HDM-4 also incorporated new technical relationships to model accident costs, rigid pavement deterioration, energy consumption, traffic congestion, and environmental effects.

United States of America –Li et al. (2005) conducted a study to describe the application and calibration of pavement deterioration model of HDM-4 to the Washington State Department of Transportation's (WSDOT) road network. They reported that HDM-4 can be used to analyse WSDOT road network after successful calibration. However, the calibration factor incorporated in the HDM-4 was significantly different from the factors obtained by the authors. HDM-4 was not able to analyse Portland cement concrete pavements. They further concluded that WSDOT can efficiently use HDM-4 to predict the budget required based on the results obtained from the study.

Paterson and Bennett (1998) developed a general approach for calibration of HDM model to local conditions. Based on different levels of resources and time, calibration of HDM models was divided into three levels: basic application, calibration, and adaptation. The impact of model parameters and data on the output model were analysed based on four sensitivity levels. The four levels were divided based on impact elasticity, defined as the ratio of percentage change in a specific result to the percentage change in the input parameter, holding all other parameters constant at mean values.

Vietnam – HDM-4 system was used in pavement management system (PMS) of Vietnam for various projects such as highway capacity improvement project and road network improvement project (Hiep and Tsunokawa, 2005). To establish the strategic management plan for the national road network, the advanced HDM-4 tool was reported to be highly efficient.

2.8 INDIAN SCENARIO OF HDM-4 APPLICATION

Chakrabarti et al. (1995) discussed the calibration/adaptation of HDM-4 road deterioration and maintenance effects (RDME) models for Indian conditions. Calibration factors were derived for: cracking initiation and progression, ravelling progression, pothole progression, rut depth progression, and roughness progression. HDM-4 models and their output results of pavement deterioration were compared with the PPS models for selected national highway sections under Indian conditions. The calibrations factors determined in this study are shown in Table 2.2. Moreover, it was reported that the HDM was a robust tool, and more flexible to predict the deterioration for road pavements.

Roy et al. (2003) discussed the calibration of HDM-4 models and applications in Indian conditions. The HDM-4 road deterioration and works effects (RDWE) models were calibrated with the pavement performance study (PPS) models developed for Indian conditions. The calibration factors were determined for two types of pavement surfacing, *viz.*, bituminous concrete (BC) and premix carpet (PC).

Aggarwal et al. (2004) conducted a study to describe the use of HDM-4 for a national highway network in India to assist the engineers for maintenance works and also the authorities for fund allocation in making cost effective decisions for maintenance. Segments on five national highways *viz.*, NH-58, NH-72, NH-72A, NH-73, and NH-74 were considered for the study. NH segments were divided into 22 homogeneous pavement sections based on traffic volume, pavement condition characteristics, and pavement type and thickness. Pavement deterioration models of HDM-4 were calibrated as per local conditions. The average roughness value of the highway network was observed to increase from 3.2 m/km to 4.4 m/km IRI, which led to very high road user cost values. It was stated that the average roughness value of the highway network would not change much if the maintenance work was delayed by a year, however, if it was delayed by two years, the roughness value would sharply rise to 6m/km to 8 m/km.

Jain et al. (2005) used national highway sections located in the Uttaranchal and Uttar Pradesh states of India to calibrate HDM-4 pavement deterioration models. Cracking, ravelling, potholes, and roughness data were collected and used for calibration of HDM-4 pavement deterioration models. These models were validated using percent variability and coefficient of determination (R^2) to check the accuracy of the calibrated models. They observed variability between observed and predicted values in the range of 10.8 to 28.2% for cracking, 15.4 to 39.4% for ravelling, 0 to 66% for potholes, and 2.1 to 15.1% for roughness.

Table 2.2Calibration factors (Chakrabarti et al., 1995)

Distress Model	Calibration factors		
	HDM range	HDM default	Adopted
Cracking initiation	0.00-20	1.00	1.50
Cracking progression	0.00-20	1.00	1.50
Ravelling progression	0.00-20	1.00	1.00
Pothole progression	0.00-20	1.00	1.50
Rut depth progression	0.00-20	1.00	1.50
Roughness progression	0.00-20	1.00	1.50

Gedafa (2006) compared the flexible pavement performance using KENLAYER and HDM-4 software. Test sections located in Mumbai metropolitan region (MMR) of India were selected for this study. KENLAYER computer program was used for determining the damage ratio using distress models, while HDM-4 software was used for predicting the pavement performance using pavement deterioration models. It was observed that the results of rutting and cracking distress models in KENLAYER program were different from the rutting and cracking deterioration results from HDM-4 software. Results of the study showed that life of the pavement predicted by HDM-4 was less than that predicted by KENLAYER program.

Shankar et al. (2010) developed pavement deterioration models using HDM-4 tool for four rural road test sections identified in the Warangal district of Andhra Pradesh state. Road inventory data, pavement condition data, and traffic volume data were used for HDM-4 analysis for responsive and schedule maintenance.

Shah et al. (2012) used two approaches to evaluate the priority for effective pavement maintenance of urban roads. It was evaluated based on subjective rating and economic indicator. Subjective ranking was conducted using the maintenance priority index (MPI) and economic indicator was evaluated using the NPV/CAP (net present value/present value of agency cost) ratio for each pavement section with the help of HDM-4 software. 21 road sections were evaluated based on cracking, potholes, ravelling, rutting and patching. 67% of the road sections showed similar ranking from both the approaches. The authors concluded that MPI based method provided higher confidence compared to HDM-4 results.

Thube and Thube (2013) calibrated the pavement deterioration progression models of HDM-4 software for low volume roads (LVR) in India. Calibration was performed for cracking, ravelling, edge break, and pothole progression models for unbound base types of pavement composition. The optimum calibration factors were determined by varying the calibration factor from 0.1 to 20 at an increment of 0.1. The optimum calibration factor for each model was determined based on average absolute error (AAE), root mean square error (RMSE), and coefficient of determination (R^2) values.

2.9 SUMMARY

This chapter presented the comprehensive literature review on various methods used for developing pavement prediction models, pavement deterioration models, and HDM-4 application at both international and national level. Various pavement deterioration models have been developed all over the world for use in the development of PMS for respective countries. The latest state-of-the-art techniques like KENLAYER, ANN, regression techniques, and many other software based models were developed by various highway agencies to suit their local conditions, but they are not geographically transferable. HDM-4 is a tool that has the ability to create maintenance strategies once the pavement deterioration models have been configured and calibrated for the local conditions. In view of its international recognition and vast acceptability, especially in developing countries, HDM-4 has been recommended as a very suitable tool for pavement deterioration modelling.

Chapter 3

OVERVIEW OF HDM-4 PARAMETERS

3.1 HIGHWAY DEVELOPMENT AND MANAGEMENT (HDM) SYSTEM

3.1.1 General

Over the years, several pavement performance prediction models have been proposed and developed for road infrastructure management systems. In the last four decades, the pavement management systems have improved significantly due to the advances in computer technologies. At present, highway engineers and administrators have a series of software tools that allow them to make a better use of the available resources for pavement maintenance and rehabilitation. But many of these available tools and models have been developed keeping in view the requirements of a particular highway agency, or the conditions prevailing in a particular country, or a geographical region. Hence, these tools and models present a major drawback of universal acceptance and implementation. However, one of the widely accepted tools used for pavement management analysis is the Highway Development and Management (HDM-4) tool that allows the highway agencies to take rational decisions pertaining to pavement maintenance activities. World Bank developed HDM-4 software tool on the basis of data collected for road deterioration and road user cost models from Kenya, Brazil, Caribbean and Road User Cost Study of India and similar studies from many other countries, for evaluating the economic consequences of highway investments. HDM-4is quite flexible and is able to handle a wide range of pavement types. Thus, looking into its wider acceptance nationally and internationally, it was identified to be used in the present research study, to fulfil the research objectives through calibrating the inbuilt deterioration models for the multi-lane Indian high-speed road corridor network.

3.1.2 Background of HDM-4 Developments

The software based HDM-4 tool has been developed after series of research studies in various countries of the world, though it was initially used as a road appraisal model in the late sixties. Reputed research organizations and academic institutions of the world have contributed enormously to its development during the last four decades. Prominent amongst them are the Transport and Road Research Laboratory (TRRL), Laboratoire Centrale des Pontset Chaussées (LCPC), Massachusetts Institute of Technology (MIT), and University of Birmingham (UK). The historical development of theHDM-4 system in the last four decades is schematically shown in Figure 3.1.The key advantages of the HDM-4 are as under:

- HDM-4 is user-friendly system and can be used in a wide range of environments.
- Its applications have been designed to work with a wide range of data types.
- HDM-4 configuration provides the facility to customise system operation to reflect the norms that are customary in the environment under study.
- The default data and calibration coefficients can be defined in a flexible manner to minimise the amount of data that must be changed for each application of HDM-4.
- The default values supplied with HDM-4 are all user-definable and facilities are provided to enable this data to be modified.
- Import and Export functions, built into the modules, provide a mechanism for data transfer between existing databases and HDM-4 modules.

3.1.3 Objectives of the HDM-4 Development

Different series of HDM versions have been developed to properly organize the road maintenance and road rehabilitation according to the available budget. These series of versions are due to the need of fundamental revisions of various inbuilt models to consider the changed conditions of use, modern computing practices, and technical upgrades. Therefore, HDM tool has expanded considerably beyond normal project planning system and has proven to be a powerful tool for the road management and investment analysis (Kerali, 2000).

3.1.4 Enhancements in HDM-4

Early versions of HDM were based on simple empirical regression models based on field data collected from specific case studies. However, this prevented geographical transferability of these simple models. Later on, the transferability across different technological and climatic conditions was gradually introduced into the model. This was achieved through the use of structured mechanistic-empirical approach. By specifying default data sets, the local adaptation and calibration of HDM-4 models can be achieved. Novel approaches were developed in HDM-4 for applying field data and current knowledge of the technical problems and management needs of different countries (Odoki and Kerali, 2000).

3.2 HDM-4 MODULES

The overall structure of HDM-4 is detailed in Figure 3.2. Visual computer software languages like C++, and database tools that operate under Microsoft Windows operating system are used in HDM-4. It contains the folders of input data and outputs from three analysis tools *i.e.,* Strategy, Programme, and Project. The overall structure of HDM-4 is presented in Figure 3.2.

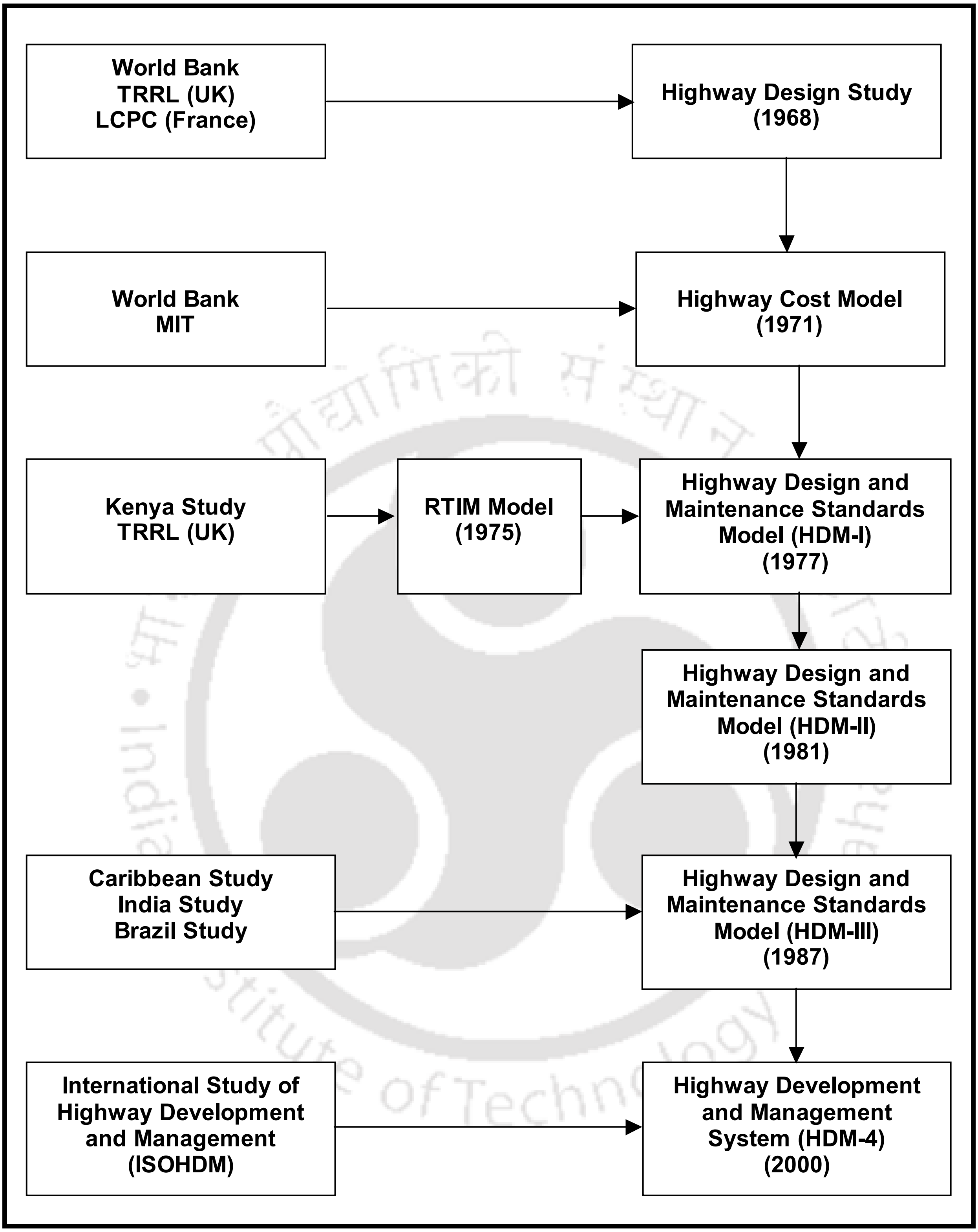


Figure 3.1 Historical development of HDM-4 tool (Kerali, 2000)

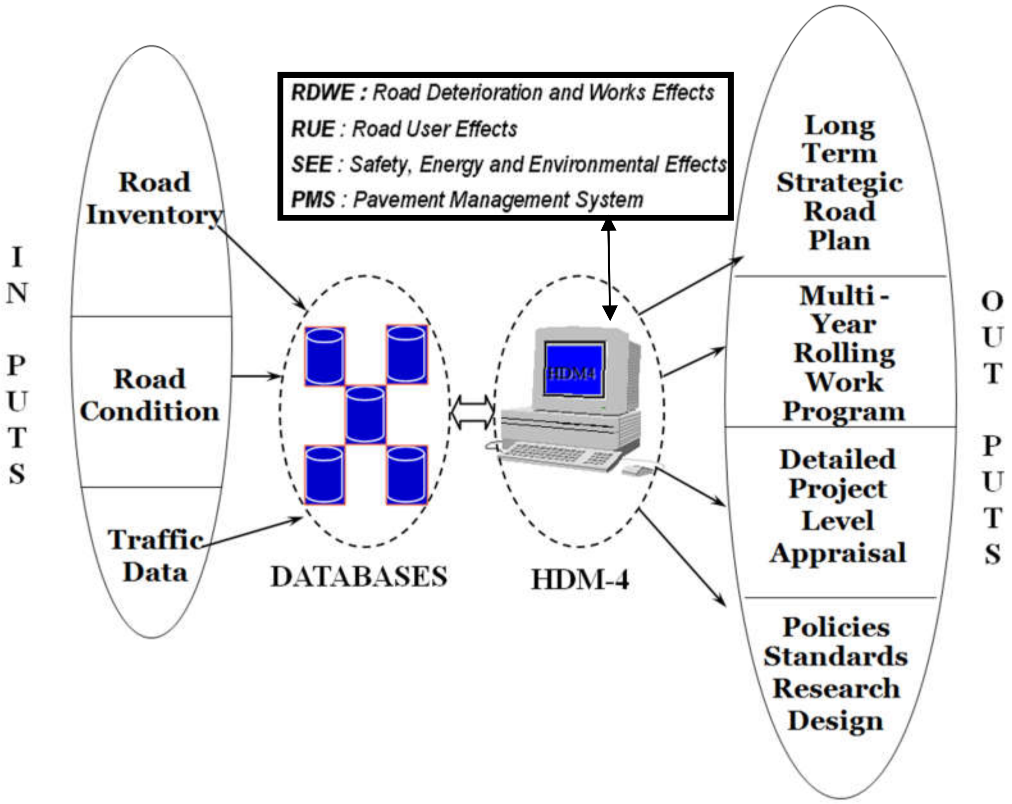


Figure 3.2 Overall structure of HDM-4

3.2.1 Input Data

Four data manager modules provide inputs to HDM-4 as detailed below:

- **Road network –** It contains physical characteristics of road sections.
- **Vehicle fleet –** It includes vehicle type and load, vehicle speed, operating cost, travel time cost, and other vehicle effects.
- **Road works standards –**It includes maintenance and improvement standards.
- **HDM-4 configuration –**It includes other data to be used in the applications as configuration.

3.2.2 Technical Models

The HDM-4 has four sets of models for analysis:

- Road Deterioration model (RD)
- Works Effects (WE)
- Road User Effects (RUE)
- Social and Environment Effects (SEE)

Since SEE and RUE models do not have direct relevance to deterioration study, therefore these models were considered, as it is, for the present research study.

3.2.3 Application Modules

The application modules of HDM-4 models are used for project analysis, program analysis, and strategy analysis (Kerali et al., 2000). These are described as follows:

3.2.3.1 Project analysis –The project analysis deals with the evaluation of one or more road project proposals. It examines road links or sections with user defined maintenance and rehabilitation treatments with associated costs and benefits which are the major factors of project analysis.

3.2.3.2 Programme analysis –It prioritizes road projects into a one-year or multi-year work programme under defined budget constraints. Further, it depends on maintenance, improvement or development standards that a road administration adopts. It also calculates the expenditure for each option.

3.2.3.3 Strategy analysis –This is medium to long term planning of funding needed for road network development and maintenance. This strategy is characterized on the basis of road class, surface type, pavement condition, traffic loading, etc. The important outcomes are estimation of medium to long-term budget requirements and also its forecasts of pavement maintenance. The main difference between strategy analysis and programme analysis is the way in which the road links and sections are identified.

3.2.4 Interface to External Systems

The HDM-4 system design is modular in structure to enable highway agencies to implement the HDM-4 application modules independently within their pavement management systems. The technical relationships can easily be calibrated to match local conditions by using HDM-4 configuration in addition to country specific default data (Kerali et al., 2000).

3.2.5 Life-cycle Analysis

HDM-4 can simulate the total life cycle conditions of a road. The life-cycle analysis comprises of costs to the road agency for maintaining and improving the road network and costs to road users for vehicle operation, passenger travel time, and road accidents. For each road section, the model simulates the road condition and data used for the maintenance. Subsequently, the economic benefits can be compared with this system (Odoki and Kerali, 2000).

3.3 ANALYTICAL FRAMEWORK

Under life cycle analysis, the trend of pavement performance can be predicted as shown in Figure 3.3. The pavement performance is represented by roughness (riding quality) and is measured in terms of the international roughness index (IRI). It imposes an acceptable limit to the level of deterioration when a maintenance standard is defined. As a result, capital costs and the total costs are included by highway agencies which depend on the standards of maintenance and improvement.

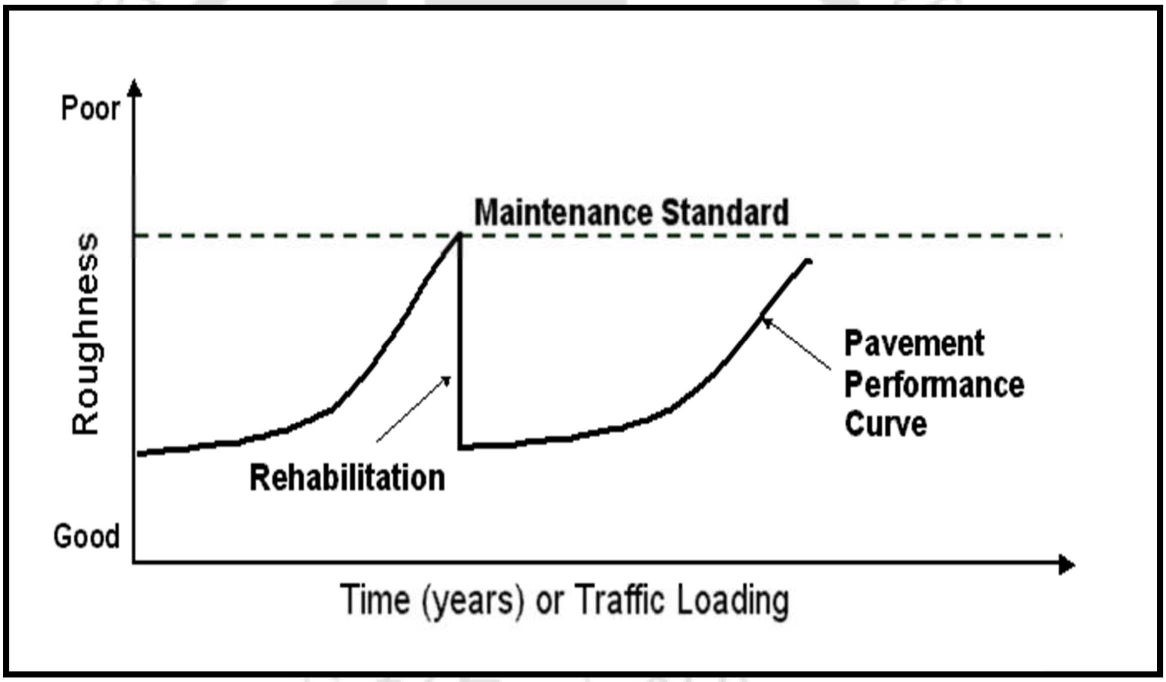


Figure 3.3 Predicted trends in pavement performance (Odoki and Kerali, 2000)

The road exisitng condition and road design standards generally are measured in terms of the road user costs, and other social and environmental effects. In addition, the road user cost models incorporated in the HDM-4 system includes (Bennett and Greenwood, 2001):

- Vehicle operation cost

- Cost of travel time
- Cost to the economy due to the road accidents

Moreover, it is necessary to ensure that the quantities of vehicle resource are predicted in the range of values observed in application.

3.4 MODELLING IN HDM-4

3.4.1 Modelling Concepts and Approach

HDM-4 includes relationships for modelling road deterioration (RD) and works effects (WE) for predicting annual road condition and for evaluating road works. Mechanistic models are based on fundamental theories whereas empirical models are usually based on statistical analyses. In the case of empirical model, calibration is needed according to local environmental conditions. A structured mechanistic-empirical approach has been adopted in HDM-4 for the development of RD and WE relationships.

3.4.2 Information Quality Level Concept

Figure 3.4 presents the concept of information quality levels (IQL) used in HDM-4. Detailed information at a low level (low-level data) can be summarized into fewer items in comparison to higher levels of IQL (high-level data) (Odoki and Kerali, 2000).

- IQL-1 - fundamental data obtained from research, laboratory, theoretical, and electronic studies.
- IQL-2 - a level of study useful for making project-level decisions
- IQL-3 -a simpler level of detail (simpler than IQL1 or IQL2)
- IQL-4 – data of planning
- IQL-5 - top level performance indicators

3.4.3 Homogeneous Sections

The concept of ‘Homogeneous Pavement Sections’ has been used where each section has almost uniform strength, geometry, traffic, and road condition characteristics over its entire length. The basic unit of economic analysis of road projects is the homogeneous pavement section, to which several investment options can be assigned for analysis. Also, in order to model pavement deterioration properly, it is required that homogeneous pavement sections in terms of physical attributes and condition should be identified so that a particular set of RD relationships can be applied. HDM-4 makes use of the concept of fixed sections to define homogeneous sections in all analyses (Odoki and Kerali, 2000).

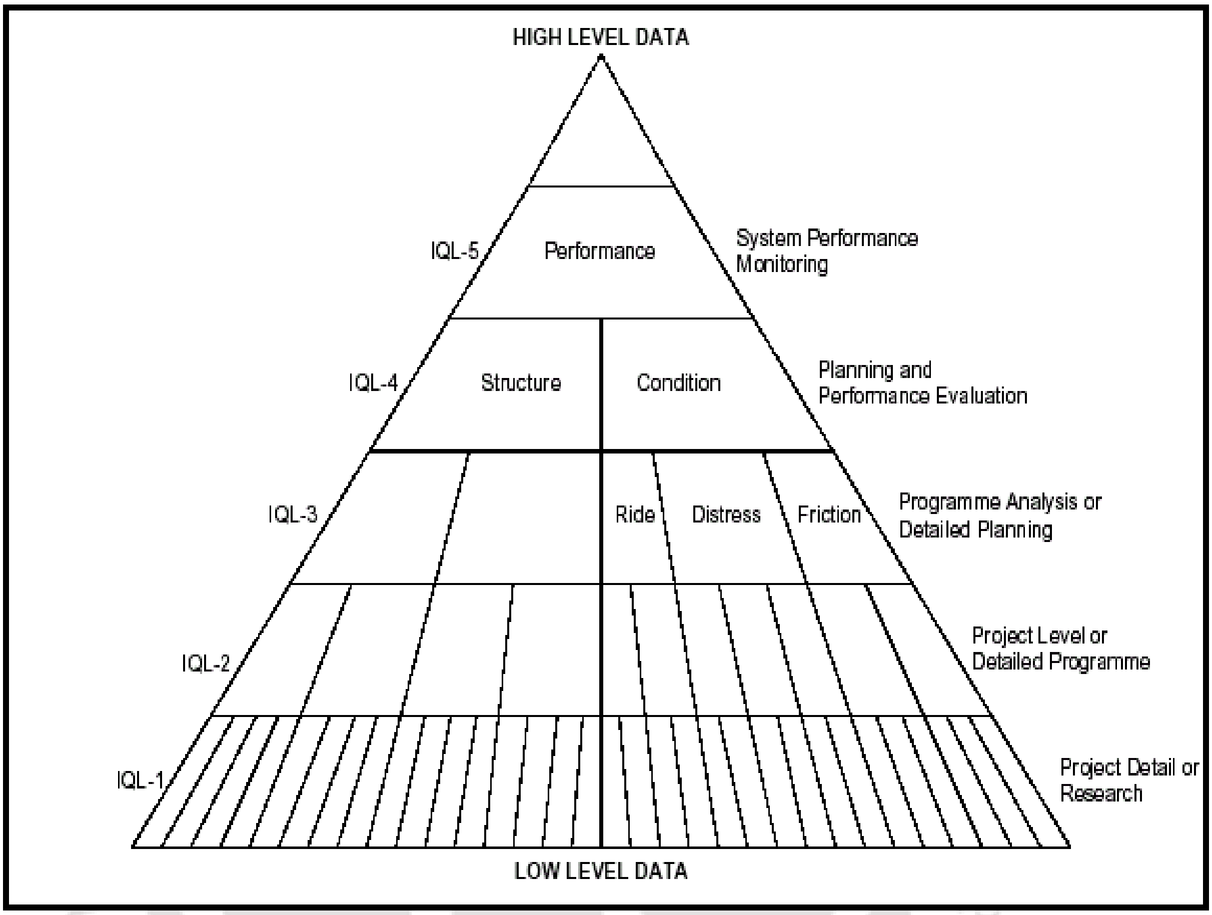


Figure 3.4 Information quality level concept used in HDM-4 (Odoki and Kerali, 2000)

3.4.4 Pavement Classification System

The HDM-4 pavement classification system is comprehensive to reflect the wider range of pavement types which can be separately modelled. It depends on the pavement surface and base type and this has been developed by providing a single set of generic models. The coefficients herein vary according to the local pavement conditions. The pavement classification system that forms the basis for defining the model framework for bituminous pavements is shown in Table 3.1. Since the performance of the pavement can be expected to vary according to materials, the coefficients can be associated with each combination of surface and base materials.

Table 3.1 HDM-4 bituminous pavement classification system

Pavement Type	Surface Type	Base Type	Description of Pavement Types
AMGB	AM	GB	Asphalt Mix on Granular Base
AMAB	AM	AB	Asphalt Mix on Asphalt Base
AMSB	AM	SB	Asphalt Mix on Stabilized Base
AMAP	AM	AP	Asphalt Mix on Asphalt Pavement
STGB	ST	GB	Surface Treatment on Granular Base
STAB	ST	AB	Surface Treatment on Asphalt Base
STSB	ST	SB	Surface Treatment on Stabilized Base
STAP	ST	AP	Surface Treatment on Asphalt Pavement

Source: Odoki and Kerali (2000)

3.4.5 Pavement Strength

Pavement comprises of different layers of materials which offer resistance under loads which may be heavy or light. Pavement strength is influenced by the existing environment and road deterioration. Meanwhile, the progression of deterioration likely relies on the residual pavement strength.

3.4.5.1 Structural number –The structural number concept is based on AASHO road test, which is essentially based on the measure of total thickness of the road layers, which is weighted according to the strength of each layer and calculated as given by Equation 3.1. Generally, pavements with high structural number will have a lower rate of deterioration under the same regime of traffic loading and environmental conditions.

$$SN = \sum_{i=1}^{n} a_i h_i \tag{3.1}$$

where,

SN = structural number of the pavement

n =number of pavement layers

a_i = strength coefficient of the ith layer

h_i = thickness of the ith layer, in inches

3.4.5.2 Modified structural number – To extend the scope of subgrade contribution to pavement strength, a modified structural number has been derived which is as follows (Equation 3.2) (Odoki and Kerali, 2000):

$$MSN = SN + 3.51\,(\log_{10} CBR_s) - 0.85\,(\log_{10} CBR_s)^2 - 1.43 \qquad (3.2)$$

where,

MSN = modified structural number of the pavement

SN = structural number of the pavement

CBR_s = in-situ CBR value of the subgrade, in %

3.4.5.3 Adjusted structural number–In HDM-4, pavement strength is characterised by the Adjusted Structural Number (SNP), an index representing the pavement strength with a weightage factor, which reduces the structural number with respect to old constructed pavements wherein numerous number of maintenance has been executed with time. It brings the concept of pavement having number of layers does not mean to have high strength (structural number) in comparison to new pavements. The adjusted structural number of a pavement is defined by an empirical relationship in which the thickness and strength of each pavement layer are combined.

3.4.6 Pavement Distress

The pavement deterioration manifests itself in various kinds of distresses and each of them has been modelled separately. The various pavement distresses, which have been modelled in HDM-4, can be classified into the following three categories which are surface distresses, deformation distresses, and surface texture.

3.4.6.1 Surfacing distress – This category of distress comprises of cracking, ravelling, potholing, and edge break. The first three distress modes are characterized by two phases referred to as initiation and progression. The initiation phase is the period before surface distress of a given mode or severity develops. The progression phase refers to the period during which the area and severity of distress increases. Edge break is modelled only through its continuous progression.

3.4.6.2 Deformation distress – This category of distress comprises of rutting and roughness. Deformation distress modes are continuous and represented by only progression equations. As they are partly dependent upon the surfacing distress, they are computed after the change of surfacing distress in the analysis year has been calculated.

3.4.6.3 Surface texture – This category of distress comprises of texture depth and skid resistance. Surface texture distress modes are continuous, and like deformation distress modes these are modelled only through their progression.

3.4.7 Distresses Modelled in HDM-4

A detailed description of the various kinds of distresses modelled in HDM-4 is given in the following sections (Morosiuk et al., 2001):

3.4.7.1 Cracking –. There are two types of cracking considered in HDM-4, explained in following sub sections.

- **Structural cracking:** Induced by traffic load and environmental effects.
- **Transverse thermal cracking:** Induced by large diurnal temperature changes or in freeze/thaw conditions, generally not occurring to Indian conditions.

Separate relationships are given for predicting each type of cracking initiation and then the rate of progression. At 0.5% of the carriageway surface area the crack initiation is said to occur.

3.4.7.2 Ravelling –Ravelling is a common distress in poorly constructed, thin bituminous layer such as surface treatment, but is rarely seen in high quality, hot-mix asphalt. Also, it is considered to be initiated on a given road section when 0.5% of the carriageway surface area is ravelled. Separate relationships are provided for ravelling initiation and progression.

3.4.7.3 Potholing –In HDM-4, potholing is expressed in terms of the number of pothole units of area 0.1 m^2. The volume of each of these pothole units is assumed to be 10 litres (*i.e.* 100 mm in depth).An upper limit of 10% is imposed on pothole area.

3.4.7.4 Rutting –The HDM-4 rut-depth model is based on four components of rutting described below:

- **Initial densification–** depends upon the degree of relative compaction of the base, sub-base, and subgrade layers, hence considered applicable only for one year after new construction.
- **Structural deformation–**associated with occurrence of cracking.
- **Plastic deformation–** associated with material properties such as softening point of the binder, and voids in the bituminous mix.

- **Surfaces wear–** due to use of vehicles with studded tyres in freezing climates, hence considered not applicable to Indian climatic conditions.

3.4.7.5 Roughness –Road roughness is the most important parameter in modelling pavement deterioration. The roughness model in HDM-4 consists of several components of roughness, which are described below:

- **Structural –** roughness due to the deformation in the pavement surface under the shear stress imposed by traffic loading.
- **Cracking –** this relates to roughness caused by cracked surface.
- **Rutting –** this relates to roughness caused by occurrence of rutting on the pavement.
- **Potholing –** this relates to roughness caused by presence of potholes on the surface.
- **Environmental –** roughness due to environmental factors, which mainly include temperature and moisture fluctuations.

The model used to convert roughness values between Bump Integrator (BI) and International Roughness Index (IRI) units is as follows (Odoki and Kerali, 2000).

$$BI = 630 \times IRI^{1.12} \tag{3.3}$$

3.4.7.6 Pavement surface texture –The pavement surface texture is modelled in HDM-4 as described below:

- **Texture depth –**It relates to macrotexture of pavement surface which reduces due to movement of vehicles.
- **Skid resistance–**It is strongly influenced by micro- and macro-texture which reduces due to wear and tear action of vehicles tyre. It is also a measure of the degree of polishing of a pavement surface or of the aggregate and the surface.

Definitions and a brief description of the measurement techniques for various distress measures used in HDM-4 road deterioration models are given in Table 3.2.

Table 3.2 Distress measurements in HDM-4

Measure	Definition
Area of distress	Sum of rectangular areas circumscribing manifest distress expressed as a percentage of the carriageway area
All cracking	Narrow and wide structural cracking inclusive
Narrow cracking	Interconnected or line cracks of 1-3 mm crack width
Wide cracking	Interconnected or line cracks of 3 mm crack width or greater
Indexed cracking	Normalized sum of narrow and wide cracking
Transverse thermal cracking	Unconnected cracks running across the pavement
Ravelling	Loss of material from wearing surface (sq. meter area)
Pothole	Open cavity in the road surface with at least 150 mm diameter and at least 25 mm depth
Edge-break	Loss of bituminous surface material from the edge of pavement
Rut-depth	Maximum depth under 2 m straight edge placed transversely across a wheel path
International Roughness Index (IRI)	Reference measure expressing roughness as average rectified slope statistic of the longitudinal profile of pavement
Mean Texture Depth	The average depth of the road surfacing determined from sand-patch test
Skid resistance	Resistance to skidding expressed by the sideways force coefficient (SFC)

Source: Morosiuk et al. (2001)

3.4.8 Key Variables Affecting Pavement Deterioration

There are several factors responsible for the development of distress that leads to the deterioration of a pavement structure. The key variables that affect the rate of pavement deterioration are as follows:

- Climatic and environment characteristics
- Traffic characteristics
- Pavement history

- Pavement structural characteristics
- Material properties
- Construction quality

3.4.8.1 Climatic and environment characteristics –It is well known that pavement strength changes during the course of a year due to climatic effects. Moreover, the climate in which a road is located has a significant impact on the rate at which the road deteriorates. The climatic factors are related to temperature and rainfall characteristics. The climatic characteristics, as per temperature and rainfall classifications used in HDM-4, are given in Tables 3.3 and 3.4 respectively.

Table 3.3 Temperature classification in HDM-4

Temperature Classification	Description	Temperature Range (^{o}C)
Tropical	Warm temperatures in small ranges	20 to 35
Sub-tropical - hot	High day, cool night temperature	-5 to 45
Sub-tropical - cool	Moderate day temperatures, cool winters	-10 to 30
Temperate - cool	Warm summer, shallow winter freeze	-20 to 25
Temperate - freeze	Cool summer, deep winter freeze	-40 to 20

Source: Morosiuk et al. (2001)

Table 3.4 Rainfall classification in HDM-4

Rainfall Classification	Description	Annual Precipitation (mm)
Arid	Very low rainfall, high evaporation	< 300
Semi-arid	Low rainfall	300 to 800
Sub-humid	Moderate rainfall or strongly seasonal rainfall	800 to 1600
Humid	Moderate warm seasonal rainfall	1500 to 3000
Per-humid	High rainfall, or many wet surface days	>2400

Source: Morosiuk et al. (2001)

3.4.8.2 Traffic characteristics –The primary traffic related variables that effect road deterioration include the number and types of vehicles using the road, and axle load characteristics of the different vehicle types. The results of economic analyses are quite sensitive to traffic data, and most benefits that justify road improvements arise from savings in road user costs. For the purpose of representing traffic characteristics both for project and network level analyses, pavement sections within a network must be also categorized according to the following factors:

- **Speed flow types –**Determine parameters for capacity, speed flow relationship, width effects, and passenger car space equivalents for each road type.
- **Traffic-flow pattern –**Commuter routes
- **Non-motorised transport factor –**It measures the effect of non-motorised transport (for example, bicycles, animal-drawn carts, pedestrians etc.) on motorised traffic speeds.
- **Roadside friction –**It measures the effect of roadside activity on traffic speeds; including the effects of land use, roadside stalls, bus stops, parking, access points etc.

3.4.8.3 Traffic data types– Traffic data types can be considered under the following headings:

- **Traffic categories–** Traffic is separated into three categories: normal, diverted, and generated, in order to assess benefits. Normal traffic is defined as the traffic that would pass along the project road if no investment took place, including normal growth. Diverted traffic is defined as traffic that changes from another route or transport mode to the project road, but still travels between the same origin and destination. Generated traffic is defined as additional traffic that occurs in response to the road investments.
- **Traffic composition –**The traffic composition is defined as the proportion of different vehicle types that use the road. Information on traffic composition is required for several analytical purposes, such as predicting pavement deterioration, estimating vehicle operating costs and travel time, and for carrying out economic analysis.
- **Traffic volume–**The traffic volume entered for each vehicle type is expressed as the annual average daily traffic (AADT) as given by Equation 3.4.

$$AADT = \frac{\text{Total annual traffic in both directions}}{365} \tag{3.4}$$

Volume constitutes the baseline flow for the analysis period. Seasonal variations in traffic flows are also to be accounted while estimating the AADT from traffic counts carried over shorter periods. Traffic volumes are then derived from the baseline AADT and the composition data. The computation of annual road user effect requires the AADT for each road section to be broken down by vehicle types. Equation 3.5 has been used to estimate the future traffic composition:

$$AADT_y = AADT_o\,[1 + p/100]^{(y-1)} \tag{3.5}$$

where,

$AADT_y$ = annual average daily traffic in year y (vehicles per day)

$AADT_o$ = annual average daily traffic in the base year (vehicles per day)

p = annual percentage increase in AADT

- **Traffic growth rates –**Uncertainties always exist in estimating initial traffic, but there is even greater uncertainty in forecasting future growth rates. In most situations, traffic growth has a major effect on the level of benefits obtained. Specifying different growth rates for each vehicle type/class can effect change in traffic composition over time. Several traffic growth periods may be defined, each with a minimum duration of one year. It is important to ensure that the defined traffic growth periods cover consistently each and every year of the analysis period.

3.4.8.4 Axle loading –The following measures of axle load are required to predict the impacts of traffic on pavement deterioration and maintenance effects:

- **Number of vehicle axles (YAX)–**The number of vehicle axles, YAX_k, traversing a given road section in a particular year is computed as the volume of traffic multiplied by the number of axles per vehicle of the type involved, as given by Equation 3.6.

$$YAX_k = \frac{T_k \times N_k}{ELANES \times 10^6} \tag{3.6}$$

The total number of all axles, YAX in a given year is obtained by summing YAX_k for all vehicle types, as given by Equation 3.7.

$$YAX = \sum_{k==1}^{K} YAX_k \tag{3.7}$$

where,

YAX = annual total no. of axles of all vehicle types (millions per lane)

T_k = annual traffic volume of vehicle type k

N_k = number of axles per vehicle type k

$ELANES$ = effective number of lanes for the road section

- **Number of equivalent standard axle loads (ESAL)–**The expression for calculating ESALF is given by Equation 3.8.

$$ESALF_k = \sum_{j=1}^{j_k} \left[\frac{AXL_{kj}}{SAXL_j} \right]^{LE} \tag{3.8}$$

where,

$ESALF_k$ = equivalent standard axle load factor for vehicle type k, in equivalent single axle loads

AXL_{kj} = the average load on axle j of vehicle type k

$SAXL_j$ = the standard axle load of axle group type j

j_k = the number of single axles per vehicle of type k

LE = axle load equivalency exponent (default value = 4.0)

The factor $ESALF_k$ is for an average overall vehicle of type *k*, loaded and unloaded, in both directions on the given pavement section. The annual total number of equivalent standard axles (millions per lane) is calculated as given by Equation 3.9.

$$YE4 = \sum_{k=1}^{K} \frac{T_k \times ESALF_k}{ELANES \times 10^6} \tag{3.9}$$

where,

$YE4$ = annual total number of equivalent standard axles (millions per lane)

T_k = annual traffic volume of vehicle type k

$ESALF_k$ = equivalent standard axle load factor for vehicle type k, in equivalent single axle loads

$ELANES$ = effective number of lanes for the road section

- **Cumulative traffic loading –**The cumulative number of equivalent standard axle loads since the last rehabilitation, reconstruction or new construction works is given by Equation 3.10.

$$NE4 = \sum_{y=1}^{AGE3} YE4_y \tag{3.10}$$

where,

NE4 = cumulative number of equivalent standard axle loads since the last rehabilitation, reconstruction or new construction (millions/lane)

$YE4_y$ = number of equivalent standard axle loads in year y (millions/lane)

AGE3 = number of years since last rehabilitation, reconstruction or new construction

- **Vehicle Damage Factor** (VDF) is a measure of the damage caused to the pavement by a heavy vehicle. It is a function of the axle configuration and axle load. The VDF for a single vehicle, and for a stream of vehicles is calculated, using the Equations 3.11 and 3.12, respectively.

$$VDF_k = \sum_{i=1}^{n} \left[\frac{AXL_i}{SAXL_i} \right]^4 \tag{3.11}$$

$$VDF = \frac{\sum_{k=1}^{z} VDF_k}{z} \tag{3.12}$$

where,

VDF_k = vehicle damage factor for vehicle k (ESAL/vehicle)

VDF = vehicle damage factor for a stream of vehicle (ESAL/vehicle)

AXL_i = the average load on axle *i*

$SAXL_i$ = the standard axle load of axle group type *i*

n = the number of single axles per vehicle of type *k*

z = number of vehicles in the stream

3.4.8.5 Pavement history –Pavement age variables are required in HDM-4 regarding pavement history. These variables are related to the previous maintenance, rehabilitation, and construction works carried out on the pavement, in terms of number of layers and their thickness. There are four variables defining the age of the pavement used in the models:

- **AGE1** is referred to as the preventive treatment age.
- **AGE2** is referred to as the surfacing age.
- **AGE3** is referred to as the rehabilitation age
- **AGE4** is referred to as the base construction age.

3.4.8.6 Pavement structural characteristics –These include measures of pavement strength, layer thickness, material types, construction quality, and subgrade stiffness. The deterioration model requires the thickness of new and old bituminous surfacing layers as input data. An original pavement that has not been resurfaced or overlaid since it was constructed/reconstructed is considered to have a new surfacing and no old surfacing. For a pavement that has been resurfaced or overlaid, the relationship given by Equation 3.13 applies:

$$HSOLD_2 = HSNEW_1 + HSOLD_1 - MLLD \qquad (3.13)$$

where,

$HSOLD_2$ = thickness of old surfacing after works, in mm
$HSNEW_1$ = thickness of most recent surfacing, in mm
$HSOLD_1$ = total thickness of previous underlying surfacing layers, in mm
MLLD = mill depth, in mm

3.4.8.7 Material properties –Different materials are used in different pavement layers. The strength requirements of the materials used decreases from top to bottom, with the strongest material being used in the surface course. The pavement performance does not depend solely on the characteristics of the materials in the individual layers but is also very much dependent on the interaction of these layers.

3.4.8.8 Construction quality –Poor construction quality of pavement results in greater variability in material properties and performance. The initiation and progression of certain distressesare more accurately attributed to problems in material handling, preparation, or construction than to structural weakness in the pavement. Therefore, an average level of construction defects is included in HDM-4 road deterioration modelling. The three construction defect indicators used are:

- **COMP–** Relative compaction of base, sub-base and subgrade layers.
- **CDS–** Construction defect indicator for bituminous surface.
- **CDB–** Construction defect indicator for base.

The values for the above three construction quality indicators, used in HDM-4, are given in Table 3.5.

Table 3.5 Values for construction defects indicators

Parameter	Construction Quality	
COMP	100%	Relative compaction for full compliance in all layers
	95%	Relative compaction for full compliance in all some layers
	85%	Relative compaction for poor compliance in most layers
CDS	0.5	Dry (brittle) surface condition normally about 10% below design optimal binder content
	1.0	Normal surface condition of optimal binder content
	1.5	Rich (soft) surface condition normally about 10% above design optimal binder content
CDB	0	No construction defects
	0.5	Poor gradation of material, aggregate shape and compaction
	1.5	Several construction defects

Source: Morosiuk et al. (2001)

3.5 DETERIORATION MODELS IN HDM-4

3.5.1 Features of HDM-4 Road Deterioration Models

The HDM-4 road deterioration models attempt to model the complex interaction between vehicles, the environment, the pavement structure and surface. The models used to predict the deterioration of bituminous pavements in HDM-4 have several common characteristics, such as (Morosiuk et al., 2001):

(i) individual types of deterioration are modelled rather than a composite index
(ii) the deterioration models are of the structured empirical form

(iii) deterioration models for a particular type of distress is interactive with other types of distress

The distress types, which are modelled in HDM-4, and the independent variables, which are used in the deterioration models, are given in Table 3.6.

Table 3.6 Types of distress and independent variables used in HDM-4 pavement deterioration models

Distress Mode	Distress Type	Independent variables			
		Pavement Strength	Material Properties	Traffic Loading	Environment
Cracking	Structural	*	*	*	*
	Transverse thermal		*		*
Disintegration	Ravelling		*	*	*
	Potholing	*	*	*	*
	Rutting – surface wear			*	*
	Edge break		*	*	*
Deformation	Rutting - structural	*	*	*	*
	Rutting – plastic flow		*	*	*
Profile	Roughness	*	*	*	*
Friction	Texture depth		*	*	
	Skid resistance		*	*	

Source: Morosiuk et al. (2001)

3.5.2 Model Forms of HDM-4 Road Deterioration Models

3.5.2.1 Modelling of cracks–Cracking is one of the important distresses in bituminous pavements in which fatigue and ageing are the main factors which contribute to cracking. There are two separate relationships for predicting the time to initiation and rate of progression of the cracking distress.

Initiation of all structural cracking–Initiation of cracking is said to occur on the pavement when 0.5% of the carriageway surface area is cracked. Initiation of all structural cracking depends on the type of base:

For bases except stabilized base *i.e.,* asphalt base, asphalt pavement and granular base:

a) For original surfacing (i.e. HSOLD = 0)

$$ICA = K_{cia}\left[CDS^{2}a_{0}\exp\left\{a_{1}SNP + a_{2}\left(\frac{YE4}{SNP^{2}}\right) + CRT\right\}\right] \qquad (3.14)$$

b) For overlays or reseals (i.e. HSOLD > 0) and for all surface materials other than soft bitumen mix or cold mix (CM), slurry seal (SL), and cape seal (CAPE):

$$ICA = K_{cia}\left\{CDS^{2}\left[MAX\left(a_{o}\exp\left[a_{1}SNP + a_{2}\left(\frac{YE4}{SNP^{2}}\right)\right] * MAX\left(1-\frac{PCRW}{a_{3}},0\right), a_{4}HSNEW\right)\right] + CRT\right\} \qquad (3.15)$$

where,

ICA = time to initiation of all structural cracks (years)

K_{cia} = calibration factor for initiation of all structural cracking

CDS = construction defects indicator for bituminous surfacing

YE4 = annual number of equivalent standard axles (millions/lane)

SNP = average annual adjusted structural number of the pavement

PCRW = area of wide cracking before latest reseal or overlay (% of total carriageway area)

HSNEW = thickness of the most recent surfacing, in mm

HSOLD = total thickness of previous underlying surfacing layers, in mm

CRT = crack retardation time due to maintenance, in years

a_0, a_1, a_2, a_3, a_4 = default model coefficients for initiation of all structural cracking

Progression of all structural cracking–For the progression of all structural cracking, the general form of the model is given as below:

$$dACA = K_{cpa}\left[\frac{CRP}{CDS}\right]Z_{A}\left[\left(Z_{A}a_{0}a_{1}\delta t_{A} + SCA^{a_{1}}\right)^{1/a_{1}} - SCA\right] \qquad (3.16)$$

where,

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

SCA = area of all structural cracking at the start of the analysis year

δt_A = fraction of analysis year in which all structural cracking progression applies

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

CRP = retardation of cracking progression due to preventative treatment, given by CRP = 1 – 0.12 CRT, where CRT is the crack retardation time due to maintenance, in years

a_0, a_1 = default model coefficients for progression of all structural cracking

3.5.2.2. Modelling of ravelling–Ravelling is a loss of surface material due to weathering and or traffic abrasion. The occurrence of ravelling varies based on the methodology followed in the construction of roads in various regions. It is a common distress observed in poorly constructed, thin bituminous surface layers and is rarely seen in good quality pavements. Similar to cracking, ravelling is also modelled in two different relationships, *viz.* initiation and progression.

Ravelling Initiation–Initiation of ravelling is said to occur on the pavement when 0.5% of the carriageway surface area is ravelled. The general model for ravelling initiation is given as below:

$$IRV = K_{vi} CDS^2 a_0 RRF \exp\left(a_1 YAX\right) \quad (3.17)$$

where,

IRV = time to ravelling initiation (years)

K_{vi} = calibration factor for ravelling initiation

CDS = construction defects indicator for bituminous surfacing

RRF = ravelling retardation factor due to maintenance

YAX = annual number of axles of all motorised vehicle types in the analysis year (millions/lane)

a_0, a_1 = default model coefficients for ravelling initiation

Ravelling Progression–For the progression of ravelling, the general form of the model is given as below:

$$dARV = \left[\frac{K_{vp}}{RRF}\right]\left[\frac{1}{CDS^2}\right] Z\left[\left(z\left(a_0 + a_1 YAX\right) a_2 \delta t_v + SRV^{a_2}\right)^{\frac{1}{a_2}} - SRV\right] \quad (3.18)$$

where,

dARV = change in area of ravelling during the analysis year (% of total carriageway area)

K_{vp} = calibration factor for ravelling progression

CDS = construction defects indicator for bituminous surfacing

SRV = min [ARV_a, $(100-ARV_a)$]

YAX = annual number of axles of all motorised vehicle types in the analysis year (millions/lane)

ARV_a = area of ravelling at the start of the analysis year (% of total carriageway area)

δt_v = fraction of analysis year in which ravelling progression applies

RRF = ravelling retardation factor due to maintenance

a_0, a_1, a_2 = default model coefficients for progression of ravelling

3.5.2.3 Modelling of Potholes–Potholes are surface distresses that occur due to cracking, ravelling, or both. The presence of water in cracking and ravelling accelerates the pothole formation by weakening the pavement structure and lowering the resistance of the surface and base materials to disintegration. For modelling the potholes, construction defects indicator (CDB) is used as a variable and the standard pothole unit is considered as 0.1 m^2 surface area or a volume of 10 liters (*i.e.* 100 mm depth). The relationships for cracking initiation and cracking progression are as follows:

Initiation of Potholes–Initiation of potholes due to cracking and ravelling arises when the total area of wide structural cracking (ACW) exceeds 20%, and when the total ravelled area (ARV) exceeds 30%, respectively. The time for initiation of potholes due to wide structural cracking is given by the following model:

$$IPT = K_{pi} \times a_0 \left[\frac{(1+a_1 HS)}{(1+a_2 CDB)(1+a_3 YAX)(1+a_4 MMP)} \right] \tag{3.19}$$

where,

IPT = time between the initiation of wide structural cracking and the initiation of potholes (years)

HS = total thickness of bituminous surfacing (mm)

CDB = construction defects indicator for the base

YAX = annual number of axles of all motorised vehicle types in the analysis year (millions/lane)

MMP = mean monthly precipitation (mm/month)

K_{pi} = calibration factor for pothole initiation due to wide structural cracking

a_0, a_1, a_2, a_3, a_4 = default model coefficients for initiation of potholing

The values of IPT for potholing due to cracking and ravelling should be calculated separately.

Progression of Potholes–Progression of potholes occurs from potholes due to cracking, ravelling, and the enlargement of existing potholes. The progression of potholes is affected by the patching policy assigned to the section. Due to each of these three distresses, the annual incremental increase in the number of potholes is calculated as:

$$dNPT_i = K_{pp} a_0 (ADIS_i)(PEFF_i)\left(\frac{ELANES}{2}\right)\left[\frac{(1+a_1 CDB)(1+a_2 YAX)(1+a_3 MMP)}{(1+a_4 HS)}\right] \tag{3.20}$$

where,

$dNPT_i$ = additional number of potholes per km derived from distress type *i* (wide structural cracking, ravelling, enlargement) during the analysis year

K_{pp} = calibration factor for pothole progression

$ADIS_i$ = the percentage area of wide structural cracking at the start of the analysis year, or the percentage area of ravelling at the start of the analysis year, or number of existing potholes per km at the start of the analysis year

$PEFF_i$ = patching policy factor for distress type *i*

ELANES = effective number of lanes for the road section which is based on the width of carriageway

CDB = construction defects indicator for the base

YAX = annual number of axles of all motorised vehicle types in the analysis year (millions/lane)

MMP = mean monthly precipitation (mm/month)

HS = total thickness of bituminous surfacing (mm)

a_0, a_1, a_2, a_3, a_4 = default model coefficients for progression of potholing

3.5.2.4 Modelling of Rutting–Rutting is defined as the permanent or unrecoverable traffic-associated deformation within pavement layers which, if channelized into wheel paths, accumulates over time and manifests as a rut (Paterson, 1987). Rut depth modelling is performed after the values of all the surface distresses (that is, cracking, ravelling, potholing and edge-break) at the end of the year have been calculated. The rut depth model is based on four components of rutting:

a) Initial densification
b) Structural deformation
c) Plastic deformation
d) Wear from studded tyres

Rutting due to Initial Densification–According to Paterson (1987), densification is the change in the volume of material as a result of the tighter packing of the material particles and sometimes also includes the degradation of particles into smaller sizes. The initial densification depends on the degree of relative compaction of the base, sub-base and selected subgrade layers; that is, COMP. The initial densification is given as:

$$RDO = K_{rid}\left[a_0\left(YE4 * 10^6\right)^{(a_1 + a_2 DEF)} SNP^{a_3} COMP^{a_4}\right] \quad (3.21)$$

where,

RDO = rutting due to initial densification (mm)

YE4 = annual number of equivalent standard axles (millions/lane)

DEF = average annual Benkelman beam deflection (mm)

SNP = average annual adjusted structural number of the pavement

COMP = relative compaction (%)

K_{rid} = calibration factor for initial densification

Structural deformation without cracking,

$$\Delta RDST_{uc} = K_{rst}\left(a_0 SNP^{a_1} YE4^{a_2} COMP^{a_3}\right) \quad (3.22)$$

Structural deformation after cracking,

$$\Delta RDST_{crk} = K_{rst}\left(a_0 SNP^{a_1} YE4^{a_2} MMP^{a_3} ACX_a^{\ a_4}\right) \quad (3.23)$$

where,

ΔRDST = total incremental increase in structural deformation in the analysis year (mm) ($\Delta RDST = \Delta RDST_{uc} + \Delta RDST_{crk}$)

$\Delta RDST_{uc}$ = incremental rutting due to structural deformation without cracking in the analysis year (mm)

$\Delta RDST_{crk}$ = incremental rutting due to structural deformation after cracking in the analysis year (mm)

SNP = average annual adjusted structural number of the pavement

YE4 = annual number of equivalent standard axles (millions/lane)

COMP = relative compaction (%)

MMP = mean monthly precipitation (mm/month)

ACX_a = area of indexed cracking at the beginning of the analysis year (% of total carriageway area)

K_{rst} = calibration factor for structural deformation

a_0, a_1, a_2, a_3, a_4 = default model coefficients for structural deformation

Rutting due to Plastic Deformation–The general plastic deformation model is given as:

$$\Delta RDPD = K_{rpd} a_0 CDS^{a_1} YE4 Sh^{a_2} HS^{a_3} \quad (3.24)$$

where,

ΔRDPD = incremental increase in plastic deformation in the analysis year (mm)

K_{rpd} = calibration factor for plastic deformation

CDS = construction defects indicator for bituminous surfacing

YE4 = annual number of equivalent standard axles (millions/lane)

Sh = speed of heavy vehicles (km/h)

HS = total thickness of bituminous surfacing (mm)

a_0,a_1,a_2,a_3 = default model coefficients for plastic deformation

Rutting due to Surface Wear–It is applied to environments where vehicles use studded tyres during freezing period and the model is given as follows:

$$\Delta RDW = K_{rsw}\left[a_0 PASS^{a_1} W^{a_2} S^{a_3} SALT^{a_4}\right] \tag{3.25}$$

where,

ΔRDW = incremental increase in rut depth due to studded tyres in the analysis year (mm)

PASS = annual number of vehicle passes with studded tyres in one direction (counted in thousand passes)

S = average traffic speed (km/h)

SALT = variable for salted or unsalted roads (2= salted; 1=unsalted)

W = road width (m) (carriageway plus total shoulder width)

K_{rsw} = calibration factor for surface wear

a_0,a_1,a_2,a_3,a_4 = default model coefficients for surface wear

3.5.2.5 Modelling of roughness–The roughness model consists of several components. The total incremental roughness is the sum of the roughness due to cracking, disintegration, deformation and maintenance. The structural component of roughness relates to the deformation in the pavement materials under the shear stresses imposed by traffic loading. It is given as per the following equations:

$$\Delta RI_s = K_{gs} a_0 \exp(mK_{gm} AGE3)(1 + SNPK_b)^{-5} YE4 \tag{3.26}$$

$$SNPK_b = MAX[(SNP_a - dSNPK), 1.5] \tag{3.27}$$

$$dSNPK = K_{snpk} a_0 \left\{MIN\left(a_1 ACX_a\right) HSNEW + MAX\left[MIN\left(ACX_a - PACX, a_2\right), 0\right] HSOLD\right\} \tag{3.28}$$

where,

ΔRI_s = incremental change in roughness due to structural deterioration during the analysis year (IRI m/km)

dSNPK = 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

SNP_a = adjusted structural number of pavement at the start of the analysis year

ACX_a = area of indexed cracking at the beginning of the analysis year (% of total carriageway area)

PACX = area of previous indexed cracking in the old surfacing (% of total carriageway area); that is, 0.62 (PCRA) + 0.39 (PCRW)

PCRW = area of wide cracking before latest reseal or overlay (% of total carriageway area)

PCRA = area of all cracking before latest reseal or overlay (% of total carriageway area)

HSNEW = thickness of the most recent surfacing (mm)

HSOLD = total thickness of previous underlying surfacing layers (mm)

AGE3 = pavement age since last overlay or reconstruction (years)

m = environment coefficient

K_{gm} = calibration factor for environmental coefficient

K_{snpk} = calibration factor for SNPK

K_{gs} = calibration factor for structural component of roughness

a_0, a_1, a_2 = default model coefficients for roughness

3.5.2.6 Modelling of texture depth–The texture of a pavement surface wears as a result of seasonal effects and the overall exposure to traffic. The progression of macrotexture model which has been incorporated into HDM-4 is given as follows:

$$\Delta TD = K_{td}\left\{ ITD - TD_a - a_0 ITD \log_{10}\left(10^{\left[\frac{ITD - TD_a}{a_0 ITD}\right]} + \Delta NELV \right) \right\} \quad (3.29)$$

where,

ΔTD = incremental change in sand patch derived texture depth during analysis year (mm)
ITD = initial texture depth at construction of surfacing (mm)
TD_s = texture depth at the beginning of the analysis year (mm)
$\Delta NELV$ = number of equivalent light vehicle passes during the analysis year (one heavy truck or heavy bus is equal to 10 NELV; light vehicles equal to 1)
K_{td} = calibration factor for texture depth
a_0 = default model coefficients for texture depth

3.5.2.7 Modelling of skid resistance–This is strongly influenced by the micro texture, which is a measure of the degree of polishing of a pavement surface or of the aggregate and the surface. The progression of skid resistance model is given as follows:

$$\Delta SFC_{50} = K_{sfc} a_0 MAX[0, \Delta QCV] \tag{3.30}$$

where,
ΔSFC_{50} = incremental change in sideway force coefficient during the analysis year, measured at 50 kmph
ΔQCV = annual incremental increase in the flow of commercial vehicles (veh/lane/day)
K_{sfc} = calibration factor for skid resistance
a_0 = default model coefficient for skid resistance progression

3.5.3 Overall Computational Procedure

The overall computational logic for modelling the deterioration of each road section, in each analysis year, can be summarized by the following steps:

(i) Initialize the road characteristics at the beginning of the year either from input data if it is the first year of analysis or the first year after construction, or otherwise from the results of the previous year's maintenance works.
(ii) Compute pavement strength parameters
(iii) Calculate the amount of change in each surfacing distress mode during the analysis year in the following order: cracking - ravelling - potholing
(iv) Check that the total damaged and undamaged carriageway surface area equals 100%, based on the limits defined for each distress mode.

(v) Determine the amount of each surfacing distress mode at the end of the year and the average value for the year.

(vi) Compute the change in each deformation distress mode during the year, and determine the amount of distress mode at the end of the year and the average value for the year.

(vii) Compute the change in each surface texture distress mode during the year, and determine the amount of distress mode at the end of the year and the average value for the year.

(viii) Store results for use in the Works Effects (WE) and Road Users Effects (RUE) model, and in the following analysis year.

3.6 ROAD WORKS EFFECTS

3.6.1 Roadworks

The term "roadworks" is used to hold any change in physical characteristics of a road; in addition to that it may include maintenance, such as cleaning road surface and construction of new carriageway. Benefit of roadworks can be almost immediate or longer term and arise from reduced society costs *i.e.,* of vehicle operation, adverse environmental effects and/or reduced cost to the road agency in future maintenance of the road (Morosiuk et al., 2001).

3.6.2 Roadworks Modelling

Roadworks modelling in the context of the HDM-4 system imply the following:

- Defining roadworks
- Timing of works
- Calculation of the physical quantities or the amounts of works
- Estimating the costs of works
- Resetting/changing one or more of the characteristics
- Resetting/changing the asset valuation of the road

Thus, the works effects (WE) module is used to estimate road agency resource needed for road preservation and development. These needs are expressed as physical quantities and the monetary costs of works to be undertaken.

3.6.3 Roadworks Classification

In HDM-4, road works are considered as a hierarchical structure of category, class and type. Each works type comprises several works activities or operations.

3.6.3.1 Works categories –Road works are divided under two categories:

- **Preservation** considers maintenance of the existing pavements by reducing the deterioration of roads.
- **Development** includes expansion of road network capacity, provide stronger pavement, and improve road geometric characteristics.

3.6.3.2 Works classes are classified as following:

- **Maintenance** mainly comprises of routine, periodic, and preventive maintenance. The main objective is to reduce the effect of deterioration.
- **Development** includes reducing traffic congestion and improving road safely. Also, new pavement construction and expansion of existing pavement come under this category.

3.6.3.3 Road Works types are described below:

- **Routine maintenance –**Routine maintenance works are divided into routine pavement maintenance, drainage maintenance and routine miscellaneous maintenance. The routine maintenance works on bituminous pavements mainly comprise of crack sealing and patching. Drainage maintenance works compriseof clearing side drains, clearing culverts, and culvert repairs. Routine miscellaneous works include all the works that are not modelled endogenously in HDM-4, for example, vegetation control, line-marking, road sign repairs, guard rail repair, etc.
- **Periodic maintenance –**The periodic maintenance works on bituminous roads comprise the following: resealing or resurfacing, preventive treatment, overlay, mill and replace inlays, and reconstruction.

3.6.4 Work Standards

Generally, a standard is a set of operations or works activities with definite intervention criteria to determine when and where to carry them out. Further, the intervention levels define the minimum level of service that is allowed. Standards are grouped into two types for input purposes in HDM-4, *i.e.* maintenance, and improvement standards. However, for a given pavement section, either maintenance or improvement standard will be effective in any analysis year.

3.6.5 Intervention Criteria

3.6.5.1 Intervention criteria can either be scheduled type or condition responsive which is as follows.

- **Time-based/Scheduled type –**By scheduling at fixed time intervals for maintenance and rehabilitation.
- **Condition responsive type –**Based on pavement condition, materials, pavement strength or traffic volumes and loading criteria.

3.6.5.2 Intervention parameters –There are many parameters that may be applied to restrict the use of different types of maintenance treatment. These parameters can be described in terms of traffic, pavement condition, and history.

3.6.6 Overall Computational Procedure

The overall computational procedure for modelling road works that is applied in each analysis year can be summarised by the following steps:

(i) Determine the works standard that is applicable in the given year. Only one maintenance standard can be applied to a road section in any analysis year.

(ii) Check intervention criteria and limits for maintenance works. A scheduled operation takes priority over a responsive operation of the same type.

(iii) Identify and apply the works activity at the top of hierarchy.

(iv) Compute the physical quantities of works.

(v) Compute works effects and reset modelling parameter values to reflect post-works road geometry, pavement structure, strength, condition and road use.

(vi) Calculate the costs of works by applying unit costs to the physical quantities.

(vii) Store results for economic analysis and for use in the following analysis year.

3.7 SUMMARY

This chapter presented the development of HDM-4 software tool, its operational modules, parameters needed for modelling pavement deterioration using HDM-4, and inbuilt model structures for various distresses. Considering its wider acceptance internationally, HDM-4has been identified to be used in the present research study for modelling the deterioration of the multilane high-speed road corridor network developed under NHDP in India. HDM-4 is one of the very few internationally recognised tools available for prediction of future pavement deterioration. The HDM-4 system is designed in a way that permits its integration with inbuilt deterioration models. In this chapter, various parameters of HDM-4 were described comprehensively, which are relevant to the Indian geographical, climatic, soil, pavement, and traffic conditions, for attempting pavement deterioration modelling.

Chapter 4

METHODOLOGY AND TIME SERIES DATA COLLECTION

4.1 INTRODUCTION

Since 1999, road network in India has undergone major development through construction, widening, and rehabilitation from existing two-lane to multi-lanes under the National Highway Development Programme (NHDP) and several other schemes such as the Pradhan Mantri Gram Sadak Yojana (PMGSY), Bharatmala, *etc*. Road network under NHDP, an ambitious programme of the Govt. of India, is considered as high-speed road corridor of India. Due to increasing aggressiveness of road traffic, variations in climatic conditions, and the need for long lasting pavements, use of modified binders has increased in the wearing courses of multi-lane highway network in the entire country. Crumb rubber modified bitumen (CRMB) and polymer modified bitumen (PMB)are the two most commonly used modified binders used in Bituminous Concrete (BC) wearing course of flexible pavements in India.

Pavements experience different rates of initiation and progression of distresses under varying traffic, climatic and environmental conditions. A study is strongly needed to observe the rate of different pavement distresses having modified bitumen in BC wearing course, and modelling of road distress deterioration for the flexible pavement road sections of Indian high-speed road corridor located in different parts of the country. Hence, a systematic methodology needs to be developed for prediction of future flexible pavement deterioration trends for adopting maintenance and rehabilitation strategies for multi-lane highway pavements that should incorporate all the processes involved in the formulation of pavement maintenance plans and programs.

The Highway Development and Management (HDM-4) is a support system for economic evaluations of road projects and decision making for highway managers to predict economic, social, and environmental impacts that might occur while making investment decisions. This is a powerful software tool developed by the World Bank for the analysis of pavement management and investment alternatives. The HDM-4 is adopted and used in various countries for their respective pavement management programmes, because it is developed and updated based on a well-established economic analysis framework; structured mechanistic-empirical models used in HDM-4 are derived from large-scale field experiments conducted worldwide; and provides a common framework for analysis of road management options (Bennet and Paterson, 2000). It is important that the HDM-4 system be configured and calibrated for local conditions before being used in any country. As HDM-4 is designed to be used in a wide range of environments, configuration of HDM-4 enables the facility to customize system operation to reflect the customary norms in the environment under study. The Ministry of Road Transport and Highways (MoRTH), Government of India, has also advocated the use of HDM-4 for total assessment of pavement deterioration and economic returns on investments in highway projects so that optimum utilization of funds may be achieved for India’s National highway network (MoRTH, 2001).

This chapter presents the methodology adopted in this study for modelling the pavement deterioration using HDM-4 for high-speed road corridor network of India.

4.2 OUTLINE OF DETERIORATION MODELLING METHODOLOGY

The following steps outline the methodology proposed for the pavement deterioration modelling for the high-speed road corridor network in India.

1. Identify the representative high-speed road corridor network for which the pavement deterioration modelling is to be attempted. This road corridor network should preferably be under the jurisdiction of one highway agency.
2. Develop a factorial study matrix consisting of various factors for identification and classification of test sections under different environmental and climatic conditions prevailing in India.
3. The selected test sections of road corridor network should be homogeneous pavement sections of lengths about 0.5 to 1.0km. The homogeneity of the

pavement sections should be ascertained in terms of traffic, pavement crust compositions, and climatic characteristics.

4. Prepare an inventory of all the pavement sections that includes data of items not likely to change in the near future. This includes type of soil subgrade, section length, carriageway width, shoulder width, drainage condition, temperature, and rainfall characteristics.
5. Collect the pavement condition data in terms of pavement surface distresses such as cracking, ravelling, potholes; pavement functional condition data such as rut depth, roughness, skid resistance, texture depth; and pavement structural condition data such as pavement surface deflection. These data should be collected on a time series basis.
6. Collect the data related to characteristics of the vehicle fleet using the highway network. Collect the traffic volume and axle load data to ascertain the traffic related characteristics of all the pavement sections.
7. Specify the type of maintenance and rehabilitation works previously undertaken by the highway agency on selected pavement sections. Choose the intervention criteria based upon the maintenance serviceability levels.
8. Calibrate the pavement deterioration models incorporated in HDM-4 for local conditions.
9. Validate the calibration factors through comparison of predicted and observed distresses for which the data have been collected in the next time series. Prepare a database of all the collected data in theHDM-4 system for use in its various application modules, such as ‘Project Analysis’, and ‘Strategy Analysis’. Use ‘Project Analysis’ application module of HDM-4 for carrying out project level pavement distress deterioration analysis for individual pavement sections.
10. Determine realistic and logical calibration factors for different distress models for pavement sections with modified bituminous binders used in wearing course.

The above steps included in the methodology are discussed in detail in the subsequent sections of this chapter.

4.3 IDENTIFICATION OF HIGH-SPEED ROAD CORRIDOR SECTIONS

In the present study, newly developed sub-networks of the multi-lane high-speed road corridor network were identified for the purpose of Pavement Deterioration Modelling. The sub-networks consist of Golden Quadrilateral (GQ), North-South and East-West Corridor, and Port Connectivity. The identified road corridors network were considered as representative of the whole multi-lane high-speed road corridor network developed under NHDP in the country, and are described in Table4.1.

Table 4.1 Selected high-speed road corridors for the present study

Sl. No.	High-Speed Road Corridor	Description
1.	Golden Quadrilateral (GQ)	Delhi – Mumbai – Chennai – Kolkata
2.	North-South (NS) Corridor	Srinagar – Kanyakumari
3.	East-West (EW) Corridor	Silchar – Porbandar (EW)
4.	Port Connectivity	Connectivity between national highways and major sea ports

4.3.1 Formulation of Section Matrix

The parameters to be covered for the development of study section matrix included the information about: pavement type, number of lanes and its surface condition, environment, temperature, moisture and traffic conditions prevailing in the country. The major parameters covered in the development of study section matrix are given in Table4.2.

4.3.2 Criteria for Preliminary Identification of Study Sections

The criteria adopted for preliminary identification of study sections (based on data/information available from records and inspection of sections on visual basis) were:

- The sections were located as part of new construction or construction of a new carriageway road section.
- The test sections were on rural highways and were uniform and homogeneous. One-kilometer length of test section having modified bitumen binder in wearing course was used for collection of time-series pavement

performance data towards determining the calibration factors and their validation. If the test section of one kilometer in length was not available, 500 m length of test section was used for calibration and validation under the study. It should be noted that the periodic pavement performance data were collected consecutively for three years for the complete road section (i.e. 1 km or 500 m).

Table 4.2 Major parameters covered in the study section matrix

Description	Types			Number of Cells
Pavement Type Classification	Flexible Pavement (having modified bituminous mix as wearing course)			1
Number of Lanes	Two (Two Way; Hilly sections)	Two (One way)	Three (One way)	3
Traffic Category (CVPD)	Low (<1500)	Medium (1500-4500)	High (>4500)	3
Terrain Classification (Cross country slope, %)	Plain (0 to 10)	Rolling (10 to 25)	Hilly (25 to 60)	3
Pavement Condition (Total surface distress, %)	Good (<5%)	Fair (5-15%)	Poor (>15%)	3
Moisture Classification (Annual rainfall, mm)	Semi Arid (300 - 800)	Sub Humid (800 -1600)	Humid (1500- 3000)	3
Temperature Classification (Air temp. range, °C)	Tropical (20 to 35)	Sub-tropical-Hot (-5 to 45)	Sub-tropical Cool (-10 to 30)	3
Total number of Cells (1*3*3*3*3*3*3)				729

- The selection of study sections was done in a manner to cover the flexible pavements having different structural composition with modified bitumen in the wearing course.
- The study sections were straight. Sections having cross roads (or near junctions), bridges and major cross drainage works were avoided.
- Sections on curves, in urban areas, and along ribbon development were completely avoided.
- Sections in special/problematic areas (marshy soils, collapsible soils, flood prone areas etc.) were avoided.

Once a section was identified, it was placed in various cells of the design matrix (presented in Table 4.2). In the present study, a ‘window’ based technique was used for the placement. This technique involved identification of road sections at different locations with modified binders in wearing course having varying surface distresses. Before the application of ‘window’ technique, the primary data/informationwere collected on the various test sections falling under different climatic and environmental conditions.

Since India has an extreme range of rainfall, soil characteristics, and atmospheric temperatures in different geological zones (which strongly affects the behavior of pavements), the test sections/locationswere identified initially through desk study, capturing different climatic and environmental zones of the country. The meteorological data for the whole country were obtained from Indian Meteorological Department. This included data concerning temperature, rainfall and other important features such as geological maps. The test sections so identified were finalized through detailed field studies for their inclusion in the design matrix. The Indian climatic and environmental zones differ slightly from the study section matrix since the section matrix is essential to be devised based on moisture and temperature classifications on HDM-4. Since the pavement deterioration models of HDM-4 are to be calibrated for local conditions, the range for Indian climatic and environmental zones/classificationswere suitably adjusted as per the requirements of HDM-4.

4.4 SELECTION OF TEST SECTIONS UNDER DIFFERENT CLIMATIC AND ENVIRONMENTAL CONDITIONS

4.4.1 Indian High Speed Road Network and Climate

The high-speed road corridor network developed under the NHDP in Phase-I, Phase-II and Phase-III are presented in Figure 4.1. India hosts basically six major climatic subtypes, ranging from desert in the west, to alpine tundra and glaciers in the north, to humid tropical regions supporting rain forests in the southwest and the island territories. Most of north-east India experiences a humid sub-tropical climate. Typical maps showing climatic zones of India, average annual temperature range, and average rainfall range are presented in Figures 4.2 to 4.4 respectively.

4.4.2 Climatic and Environmental Zones of High-Speed Road Corridors Network

For the present study, Golden Quadrilateral (GQ), port connectivity between NH-5 to Visakhapatnam Sea Port, North-South (NS), and East-West (EW) corridors were considered for selection of test sections. This network covers different climatic and environmental zones of the country and meets the requirement of study matrix.

The superimposition technique was used to classify the selected road sections under different climatic and environmental zones. This map of the road network (*i.e.* Figure 4.1) was superimposed on maps of climate, temperature, and rainfall obtained from Indian Meteorological Department. The three maps shown in Figure 4.2, Figure 4.3, and Figure 4.4were obtained after the superimposition, and they depict the road sections classified according to: (i) climatic zones, (ii) temperature zones, and (iii) rainfall zones. These maps helped in placement of road cells in different cells of the study section matrix.

Figure 4.1 High-speed road corridor network (NHAI, 2017)

4.5 PRELIMINARY DETAILS OF TEST SECTIONS

To strengthen the monitoring system of its on-going and completed works on day-to-day basis and to manage its road network, the NHAI has developed an online and real time system known as the ‘Road Information System (RIS)’. This system has modules such as location referencing, asset management, pavement management, bridge management, traffic management, accident management, toll management, performance management and environment management. It is interlinked with geographical information system (GIS) with other GIS features.

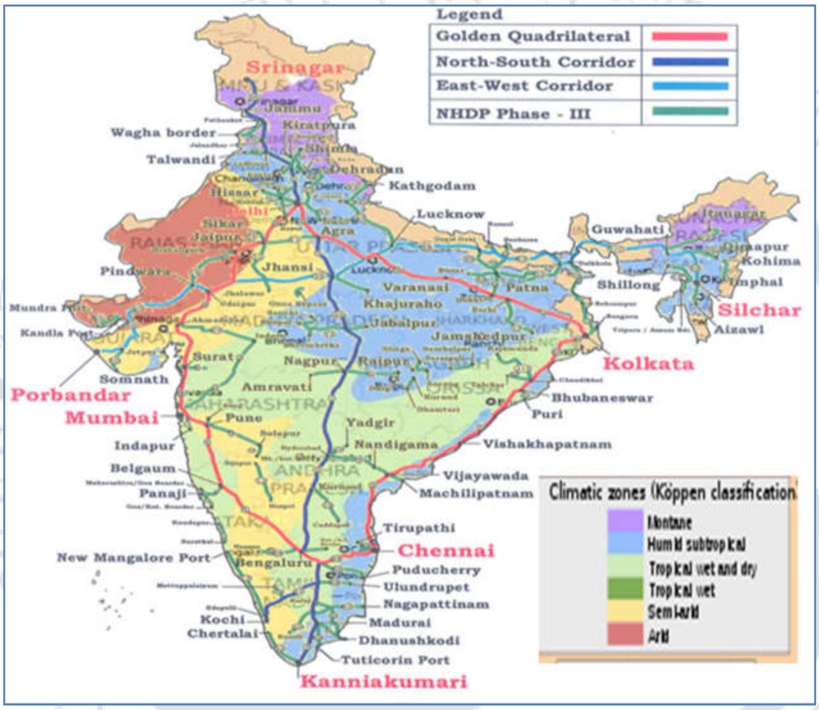


Figure 4.2 Map of selected road network in different climatic zones of India

(Courtesy: Survey of India, New Delhi)

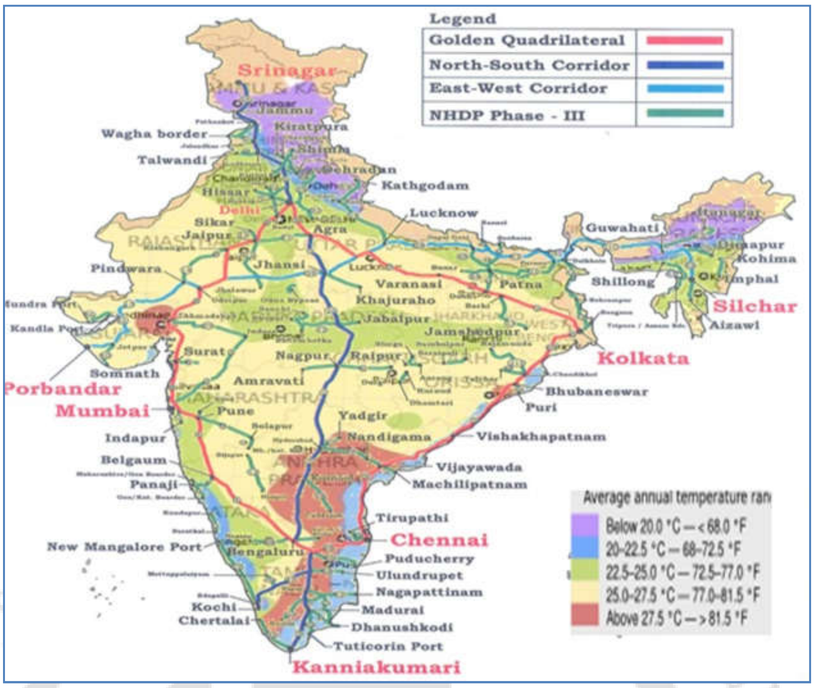


Figure 4.3 Map of selected road network in different temperature zones of India *(Courtesy: Survey of India, New Delhi)*

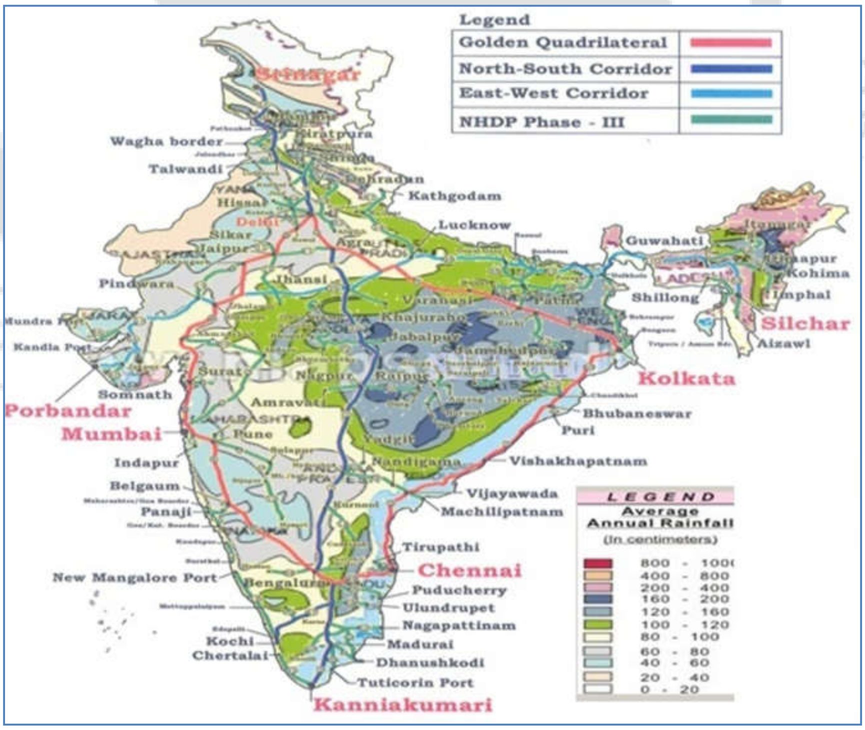


Figure4.4 Map of selected road network in different rainfall zones of India

(Courtesy: Survey of India, New Delhi)

It is thus possible to get information on crust composition through RIS for the test sections preliminary identified. Full details related to the structural and functional condition of pavement, traffic, etc. for about 5486 km of NHDP corridors is available on the RIS. The crust composition of test sections preliminary identified was found through RIS during the desk study. The information available on crust compositions were then matched with the locations of completed projects (4-lane dual carriageways).

The information on climate/environment, obtained through secondary sources, and superimposed with the road network of NHAI provides sound basis regarding locations of roads/road sections falling in the specific climate zone. These have been adjusted with the matrix required for calibration of HDM-4 deterioration models. The key information needed for the placement of roads/road sections in the given cells of the study section matrix, obtained through NHAI, RIS, and adjusted in the study section matrix Table 4.2. Some typical samples of road sections preliminary identified based on the data/information available from desk study are given in Table 4.3. The test sections were then subsequently finalized through studies done at the site.

4.5.1 Description of Selected High-Speed Road Corridor Networks

Golden Quadrilateral (GQ) is a network of national highways connecting India’s four top metropolitan cities, namely Delhi, Mumbai, Chennai, and Kolkata; forming a quadrilateral. This highway network also connects many other major industrial, agricultural, and cultural centres of India, some of which are Ahmedabad, Jaipur, Kanpur, Surat in the North, and Vijayawada, Visakhapatnam, and Bhubaneswar in the South. The GQ project was launched in 2001 as the first phase of NHDP. The overall length of the quadrilateral is 5846 km and consists of four/six lane highways and expressways.

North-South (NS) and East-West (EW) corridors comprise national highways connecting four extreme points of the country. The North-South (NS) corridor connects Srinagar, the capital city of northernmost state of Jammu & Kashmir in the Himalayas, and Kanyakumari in the south at the end of the Indian mainland at the meeting point of the Bay of Bengal and the Arabian Sea. The East-West (EW) corridor connects Silchar in the north-eastern state of Assam and Porbandar in the west, which is one of the major port cities on the Arabian Sea in Gujarat state. NS and EW corridors are included in the second phase of the NHDP and consist of 7300 km of four/six lane national highways and expressways.

Under the Port Connectivity scheme of the NHDP, it was planned to connect twelve (12) major sea ports in India with either the Golden Quadrilateral (GQ), or North–South (NS) and East-West (EW) corridor. The major ports connected are Jawaharlal Nehru Port (Maharashtra), Mormugoa Port (Goa), Cochin Port (Kerala), Kandla Port (Gujarat), New Mangalore Port (Karnataka), Tuticorin Port (Tamil Nadu), Chennai Port (Tamil Nadu), Visakhapatnam Port (Andhra Pradesh), Paradip Port (Odisha) and Haldia Port (West Bengal). The four lane road network of length of 435 km linking the major ports was developed under the Phase-1 and 2 of NHDP to improve the connectivity and to facilitate speedy cargo evacuation. The locations of all the identified test sections on the NHDP road corridor network are shown in Figure 4.5.

Table 4.3 Data available through RIS and NHAI adjusted with HDM-4 requirements

Sl. No.	High Speed Corridor Network	Description of the Road/Road Section	Data obtained through Secondary sources			Fitting of obtained data as per HDM-4 Classification		
			National Highway No.	Average Annual Temperature Range	Climate	Climate	Temp. Range	Annual Precipitation (mm)
1.	N-S Corridor	Nagpur-Hyderabad	NH-7	> 45.0°C	Semi Arid	Semi-Arid Subtropical-Hot	-5 to 45	300 to 800
2.	E-W Corridor	Nagaon - Guwahati	NH-37	20 - 22.5°C	Humid-subtropical	Humid Subtropical Cool	-10 to 30	1500 to 3000
3.	GQ	Ahmedabad - Vadodara	NE-1	Above 27°C	Semi Arid	Semi-Arid Subtropical-Hot	-5 to 45	300 to 800
4.	GQ	Allahabad - Varanasi	NH-2	Above 27°C	Sub-Humid	Sub-Humid Subtropical-Hot	-5 to 45	800 to 1500

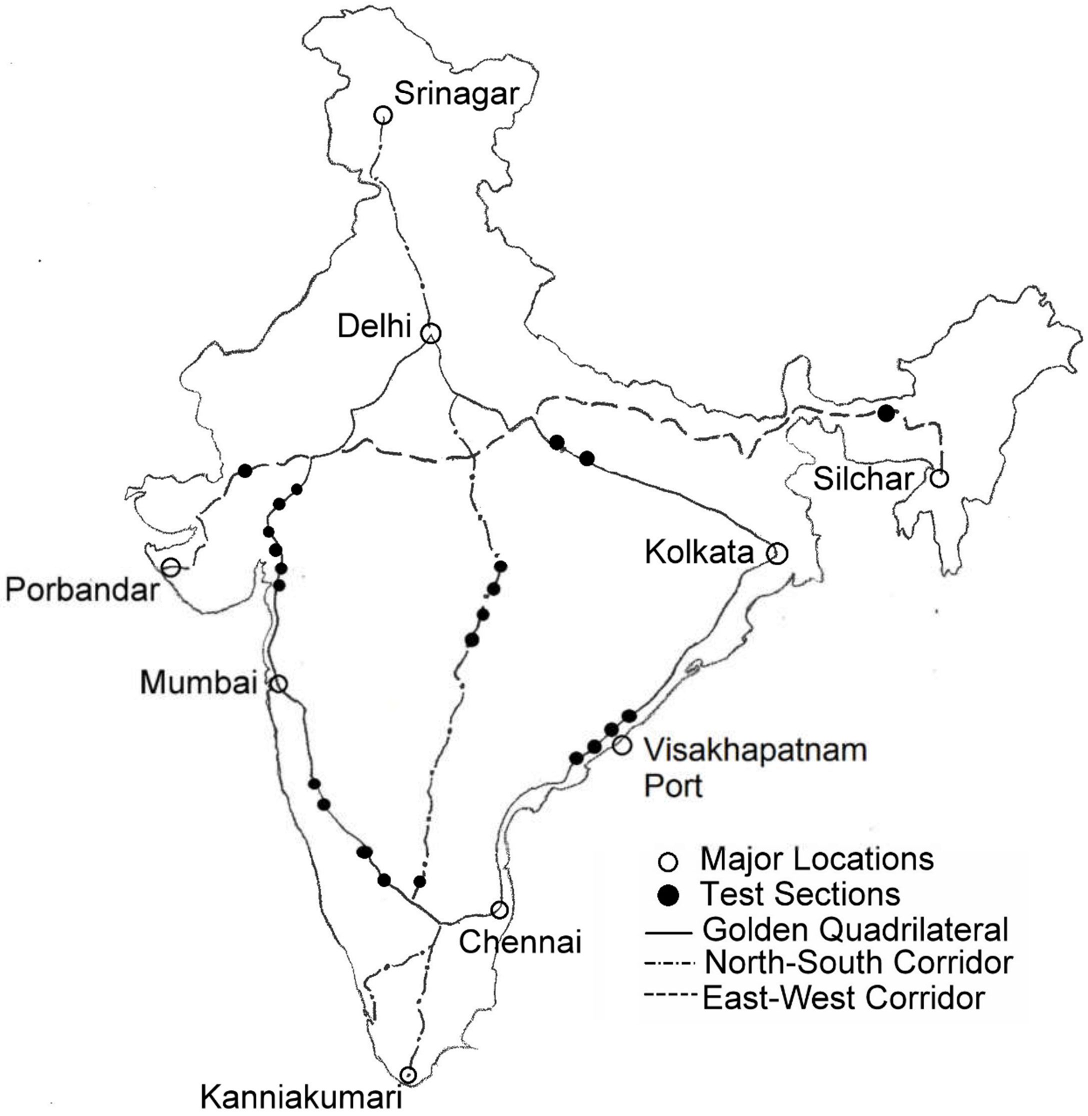


Figure 4.5 Locations of the selected 23 test sections on NHDP road network

4.5.2 Details of Pavement Sections

Total twenty-three (23) pavement sections were selected on the high-speed road corridors network at different locations. The selected pavement sections were homogenous within themselves but varied considerably from each other in terms of traffic volume, pavement geometry and climatic conditions (rainfall and temperature). The pavement sections had varying soil types, terrain type (plain and rolling), climatic conditions (rainfall and temperature), and traffic volume, as given in Table 4.4.All selected pavement sections were assigned a unique ‘Section ID’, *viz.* for Expressways as NE, or for National Highways as NH with number and the sections numberslocated in the respective state for their easy identification on the highway network. The ‘Section Name’ shows the length of each section in kilometre and ‘Section Description’ describes the selected road segments between two major cities.

4.5.3 Types of collected data

The study involved collection of a huge amount of data pertaining to the selected sections. It was aimed that the collected data should meet the requirements of the HDM-4 system directly or in its derived form. The process of data collection was classified under following four categories:

(i) Road network data
(ii) Vehicle fleet data
(iii) Maintenance and rehabilitation works data
(iv) Cost data

A detailed description of the collected data, the procedure of data collection, and the instruments used for the same are discussed in the subsequent sections.

4.6 ROAD NETWORK DATA COLLECTION

4.6.1 General

The road network data of the selected pavement sections were collected based on the data requirements of HDM-4. The data were obtained from the secondary sources such as the past records of Project Implementation Units (PIUs) of NHAI and relevant government publications. The PIUs function as in-charge of construction and maintenance of the respective sections of the selected highway network. The data were also collected from the selected pavement sections by carrying out field studies. The road network data included the locational data describing the position and geometry of the pavement section, and the attribute data describing the road characteristics or inventory associated with it. Data pertaining to the type of soil, terrain, traffic (volume and axle load data), pavement composition, and climate were also collected through field studies. The details of the surveys and data collected are given in following subsections.

4.6.2 Road network surveys

The following two types of surveys were conducted for data collection:

- Secondary Survey – Inventory data collection from NHAI offices

Table 4.4 Details of selected pavement sections of the identified high-speed road corridor network

Section ID	Section Name	Section Description	Soil Type	Terrain	Moisture Classification	Temperature Classification
NE-1GJ01	NE-1GJ [Km. 14-15]	Ahmedabad - Vadodara	Silty Sand	Plain	Semi-arid	Tropical
NE-1GJ02	NE-1GJ [Km.32 -31]	Vadodara - Ahmedabad	Silty Sand	Plain	Semi-arid	Tropical
NE-1GJ03	NE-1GJ [Km. 19 - 20]	Ahmedabad - Vadodara	Silty Sand	Plain	Semi-Arid	Tropical
NE-1GJ04	NE-1GJ [Km.15 - 14]	Vadodara - Ahmedabad	Silty Sand	Plain	Semi-arid	Tropical
NH-14GJ05	NH-14GJ [Km. 380.6 - 379.6]	Radhanpur - Deesa	Clayey Sand	Plain	Semi-arid	Tropical
NH-2UP01	NH-2UP [Km. 607 - 606]	Allahabad - Khaga	Clayey Sand	Plain	Sub-humid	Subtropical hot
NH-2UP02	NH-2UP [Km. 717 – 718]	Allahabad - Varanasi	Clayey Sand	Plain	Sub-humid	Subtropical hot
NH-37AS01	NH-37AS [Km. 177.3 - 178.3]	Nagaon - Guwahati	Silty Sand	Plain	Humid	Subtropical cool
NH-4KA01	NH-4KA [Km. 46 - 45]	Tumkur - Bangalore	Clayey Sand	Plain	Humid	Subtropical cool
NH-4KA02	NH-4KA [Km. 481.3 - 482.3]	Dharwad - Belgaon	Clayey Sand	Plain	Humid	Subtropical cool
NH-4KA03	NH-4KA [Km. 82 - 83]	Tumkur - Sira	Clayey Sand	Plain	Humid	Subtropical cool
NH-4MH01	NH-4MH [Km. 539 - 538]	Maharastra Border - Belgaon	Clayey Sand	Rolling	Semi-arid	Subtropical hot
NH-5AP01	NH-5AP [Km. 691.25 - 690.75]	Vizag - Srikakuram	Clayey sand	Rolling	Sub-humid	Subtropical hot
NH-5AP02	NH-5AP [Km.698.6 - 698.1]	Vizag - Srikakuram	Clayey sand	Rolling	Sub-humid	Subtropical hot
NH-5AP03	NH-5 AP [Km. 7.5 - 7.0]	NH-5 - Vizag Port	Clayey sand	Plain	Sub-humid	Subtropical hot
NH-5AP04	NH-5AP[Km.9.8 - 9.3]	NH-5 - Vizag Port	Clayey sand	Plain	Sub-humid	Subtropical hot
NH-7MH02	NH-7MH [Km. 84.2 - 84.7]	Hyderabad - Nagpur	Clayey sand	Plain	Semi-arid	Subtropical hot
NH-7MH03	NH-7MH [Km. 95.5 - 96.0]	Hyderabad - Nagpur	Clayey sand	Plain	Semi-arid	Subtropical hot
NH-7AP05	NH-7AP [Km. 41 - 40]	Nagpur - Hyderabad	Sandy Silt	Plain	Semi-arid	Subtropical hot
NH-7AP06	NH-7AP [Km. 51.4 – 50.4]	Nagpur - Hyderabad	Sandy Silt	Rolling	Semi-arid	Subtropical hot
NH-7KA04	NH-7KA [Km. 518 - 519]	Bagepalli - Bangalore	Clayey Sand	Plain	Humid	Subtropical cool
NH-8GJ06	NH-8GJ [Km. 433.7 - 434.7]	Palampur - Himmat Nagar	Clayey sand	Plain	Semi-arid	Tropical
NH-8GJ07	NH-8GJ [Km. 441.4 - 442.4]	Palampur – Himmat Nagar	Clayey sand	plain	Semi-arid	Tropical

Soil Type: as per IS classification - Clayey Sand (SC), Silty Sand (SM), Sandy Silt (ML)
Terrain: as per cross slope – Plain (< 10%), Rolling (10 - 25%), Hilly (> 25%)

- Primary Survey – Field data collection

The following secondary data were obtained from various PIU offices of the NHAI:

(i) Year of original construction and the specifications adopted
(ii) Crust thickness of each pavement layer
(iii) Maintenance inputs and the norms adopted
(iv) Traffic details at the time of design
(v) Year of strengthening and its specifications
(vi) Year and specifications of last renewal course
(vii) Temperature and rainfall data

The primary data were divided under the following heads:

(i) Inventory data
(ii) Structural evaluation (structural capacity) data
(iii) Functional evaluation (pavement condition and riding quality) data
(iv) Pavement material evaluation data

4.6.3 Definition of Road Network Elements

In order to customize HDM-4 for use in the study of pavement sections, the various road network elements have been defined as given below:

- **Road class** – As per functional hierarchy, all the pavement sections included in the selected highway network belong to the 'High-Speed Road Corridor' class of roads.
- **Referencing system** – The 'Kilometre-Node' referencing system was used for the locational positioning of data pertaining to pavement sections. Kilometre stones were used as nodes for indicating the beginning and end of a pavement section.
- **Section identification** – Each pavement section was assigned a unique name and ID, as shown in Table 4.4. The section ID includes the reference to the road class (NE and NH) and road designation number (1, 2, 3 etc.), and the respective state for easy identification of the sections. The 'Section Description' describes the selected road segments between two major cities connecting by the highway.
- **Basis of Fixed Length Section** – Criteria were made to fix the length of pavement section as one or half kilometre prior to identifying available bituminous

sections where modified bitumen was used in Bituminous Concrete (BC) wearing course. The 'Fixed-length Section' was used as the basis for road network representation in HDM-4 considering each homogeneous pavement sections. This section has been done on the basis of uniformity among all the selected pavement sections.

- **Speed flow type** – The speed flow type on various pavement sections were 'Four Lane Road' depending upon the capacity and the width of the carriageway. The capacity related characteristics of the 'Four Lane Road' speed flow type are shown in Figure 4.6 as per HDM-4.
- **Traffic flow pattern** – The traffic flow pattern in case of each pavement section is defined as of the type 'Inter City', as per the temporal distribution of traffic.
- **Climate zone**–Climate zones were defined on the basis of the temperature (mean annual temperature) and rainfall (mean annual precipitation) characteristics of the pavement sections study area. The temperature and rainfall characteristics of the study sections as per HDM-4are shown in Figure 4.7.
- **Geometry class** – The geometry class of each pavement section was defined in terms of the various parameters reflecting the horizontal and vertical curvature. Most of the sections were 'Straight' except for a few in hilly areas, which are 'Rolling and Gently Undulating'.

4.6.4 Inventory Data

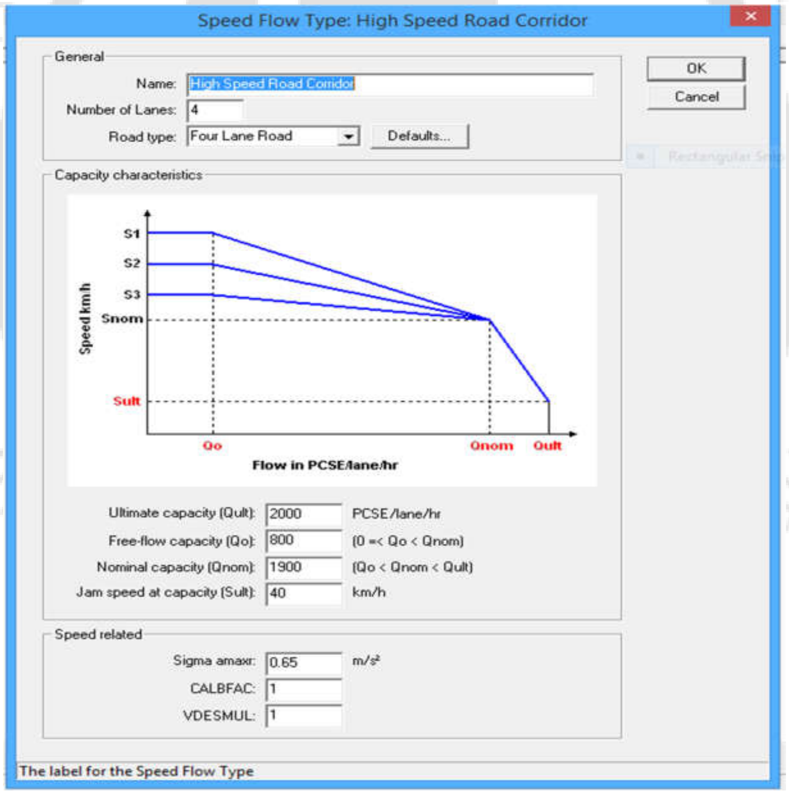
The inventory data includes the following details about the selected pavement sections:

(i) Name and Category of road
(ii) Carriageway width
(iii) Shoulder width
(iv) Drainage conditions
(v) Surface type and thickness
(vi) Pavement layer details

The above data were collected from visual inspection of the pavement sections, as well as from the construction and maintenance records of the NHAI PIUs in-charge of the maintenance of the respective pavement sections.

4.6.5 Structural Evaluation

The pavement structural strength in terms of the magnitude of pavement rebound deflection is an indicator of ability of the pavement to withstand traffic loads. Higher the rebound deflection, poorer is the structural capacity and performance. The practice hitherto is to use the non-destructive Benkelman Beam deflection method for evaluating the structural condition of the flexible pavement as per the procedure laid down in IRC: 81-1997. Alternative equipment such as Falling Weight Deflectometer (FWD) can also be used for determining the rebound deflection. However, use of FWD was quite expensive and also requires a longer time for transportation including various other arrangements and logistics support to carry out the deflection test, and therefore the structural capacity of the selected test sections was evaluated using the Benkelman Beam (BB) deflection method.



General

Name: High Speed Road Corridor

Number of Lanes: 4

Road type: Four Lane Road

Capacity characteristics

Ultimate capacity (Qult): 2000 PCSE/lane/hr

Free-flow capacity (Qo): 800 (0 =< Qo < Qnom)

Nominal capacity (Qnom): 1900 (Qo < Qnom < Qult)

Jam speed at capacity (Sult): 40 km/h

Speed related

Sigma amaxr: 0.65 m/s²

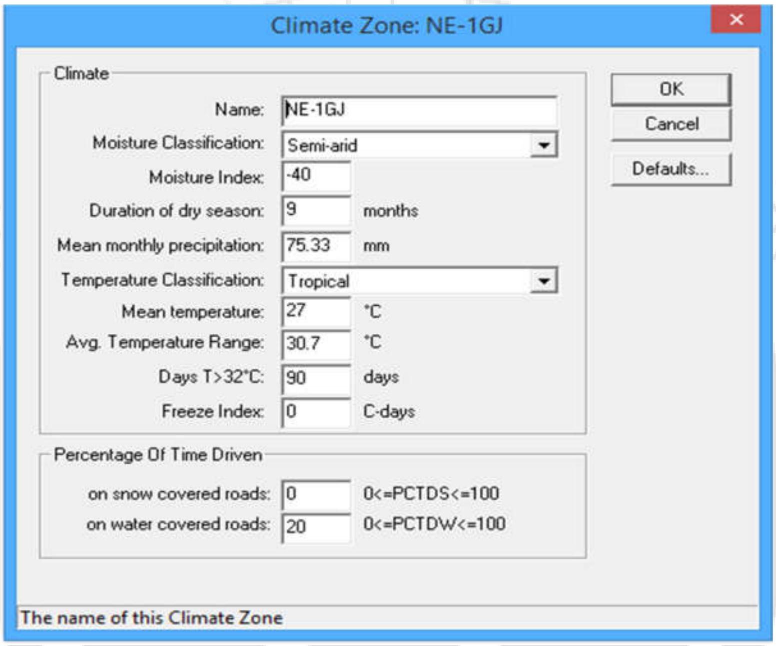
CALBFAC: 1

VDESMUL: 1

The label for the Speed Flow Type

Figure 4.6 Capacity related characteristics of the selected speed flow type ‘Four Lane Road’

4.6.5.1 Measurement of surface deflection using Benkelman Beam –The pavement surface deflection measurements were taken with Benkelman Beam as per the procedure specified in IRC: 81-1997. Permanent markings were left on the ground for future reference at all the identified sections. Test points were taken at a distance of 0.9 m from the edge of a double lane pavement. A standard axle load of 8162 kg on the rear axle of the loaded truck and a tyre pressure of 5.6 kg/cm^2 was maintained throughout the testing. Since the deflections measured by the Benkelman Beam are influenced by the pavement temperature and seasonal variations in climate, therefore pavement temperature, and soil subgrade details were also collected at all test sections for correcting the deflection values. Photo 4.1 shows the Benkelman Beam deflection test in progress at one location of pavement section NE-1GJ01 at Km.14.50.



Climate Zone: NE-1GJ

Climate

Name: NE-1GJ

Moisture Classification: Semi-arid

Moisture Index: -40

Duration of dry season: 9 months

Mean monthly precipitation: 75.33 mm

Temperature Classification: Tropical

Mean temperature: 27 °C

Avg. Temperature Range: 30.7 °C

Days T>32°C: 90 days

Freeze Index: 0 C-days

Percentage Of Time Driven

on snow covered roads: 0 0<=PCTDS<=100

on water covered roads: 20 0<=PCTDW<=100

The name of this Climate Zone

Figure 4.7 Temperature and rainfall characteristics of the selected climate zone 'NE-1GJ'

Photo 4.1 Benkelman beam deflection test at NE-1GJ01 at Km. 14.50

4.6.6 Functional Evaluation

Functional evaluation of pavements consists of collection of road data pertinent to surface distresses (crack area, ravelled area, and number of potholes), rut depth, roughness, texture depth, and skid resistance.

4.6.6.1 Surface distress measurements –The pavement surface condition survey was carried out by visual observations to evaluate the type and extent of distress developed at the surface, i.e. cracking, ravelling, patch work, potholes, depressions etc. The extent and type of distress developed were also measured in quantitative terms. The information on shoulder type, width, and condition, and drainage was also recorded. The details are presented as follows:

- **Measurement of crack area –**The pavement sections were divided into a number of representatives test sections of length 50m for crack measurement. The affected cracked area was marked in the form of rectangles in case of interconnected map and crocodile cracks. In case of single longitudinal and transverse cracks, the crack length was measured and affected width of the pavement surface across the length of crack was taken as 30 cm. Thus, crack area was expressed as percentage of total pavement area. Separate measurements were taken for cracks of width up to 3mm (narrow cracks) and width more than 3mm (wide cracks). In most of the test sections, narrow cracks were observed. Photo 4.2 shows surface condition being evaluated on the road section NE-1GJ03 at Km. 32.00.

Photo 4.2 Surface condition evaluation on pavement section NE-1GJ at Km. 32.00

- **Measurement of ravelled area** – Ravelling is the progressive loss of surface material by weathering and/or traffic abrasion. It is a commonly observed distress in poorly constructed, thin bituminous layers, such as surface treatments. The affected area was measured by considering area enclosed in regular geometric shapes such as rectangles, and then expressed as percentage of total pavement area. Photo 4.3 shows a view of severe ravelling on pavement section NH-8 at Km.434.00.

Photo 4.3 Ravelling on pavement sectionNH-8GJ at Km. 434.00

- **Measurement of pothole area –** Potholes are bowl shaped holes of varying sizes in a surface layer or extending into base/sub base course. The pothole area was measured in terms of sq. m. Depth of each pothole was also measured to convert into volume of potholes. The volume was then converted into number of standard pothole units of ten litre volume each. The pothole measurements were finally expressed as number of pothole units per km length of the pavement section as stipulated in HDM-4 data requirements. Photo 4.4 shows a few potholes observed on pavement section NH-14GJ at Km.381.00.
- **Use of Automated Road Survey System (Network Survey Vehicle)** –Network survey vehicle (NSV) is a multi-component modular automated road survey system used for road asset inventory. This system consists of state-of-the-art laser beam assembly mounted on the front and rear side of the vehicle, an accelerometer and gyroscope, a GIPSI-trac for geometry, digital cameras for image capturing, a global positioning system (GPS), distance measuring

instrument (DMI), and a computer based data acquisition system. Photo 4.5 shows the NSV with different components. The following data were collected using the NSV at highway speeds upto 100 km/h:

- Longitudinal profile (International Roughness Index)
- Transverse profile (rut depth)
- Pavement texture in terms of mean profile depth
- Cross slope, gradient, and horizontal curvature
- GPS coordinates (X,Y,Z) *viz.* longitude, latitude, and altitude using DGPS with Google map support
- Pavement surface imaging for pavement distress
- Video imaging for roadside furniture/road asset
- Real time in-vehicle data acquisition software for display and collection of data for all parameters simultaneously
- Post processing software for data analysis and report preparation

Photo 4.4 Potholes on pavement section NH-14GJ at Km. 381.00.

The laser profiler assembly mounted on the front of the survey vehicle, provides pavement profile and vertical acceleration data. The following tests were conducted for calibration of laser profiler as per the “User Manual Hawkeye 2000 Series” before use:

a) Calibration of laser sensors

b) Laser profiler bounce test

c) Laser profiler straight edge calibration
d) Laser profiler straight edge confirmation test

- **Calibration of laser sensors** – This calibration computes calibration constants for each of the individual lasers on the profiler assembly by measuring, firstly, the distance to a plate on the ground, and secondly, the distance to the top of an accurate gauge block placed over the plate. The supplied gauge block has dimensions of 25x 50x 80mm. The 50 mm dimension is used for calibration while the 25mm and 80mm dimensions are used to check linearity. This procedure describes measuring first with the calibration plate (0 mm) then with the 50 mm block. The recommended consistency for calibrations is that the measured height should be within the range of ± 0.5 mm.

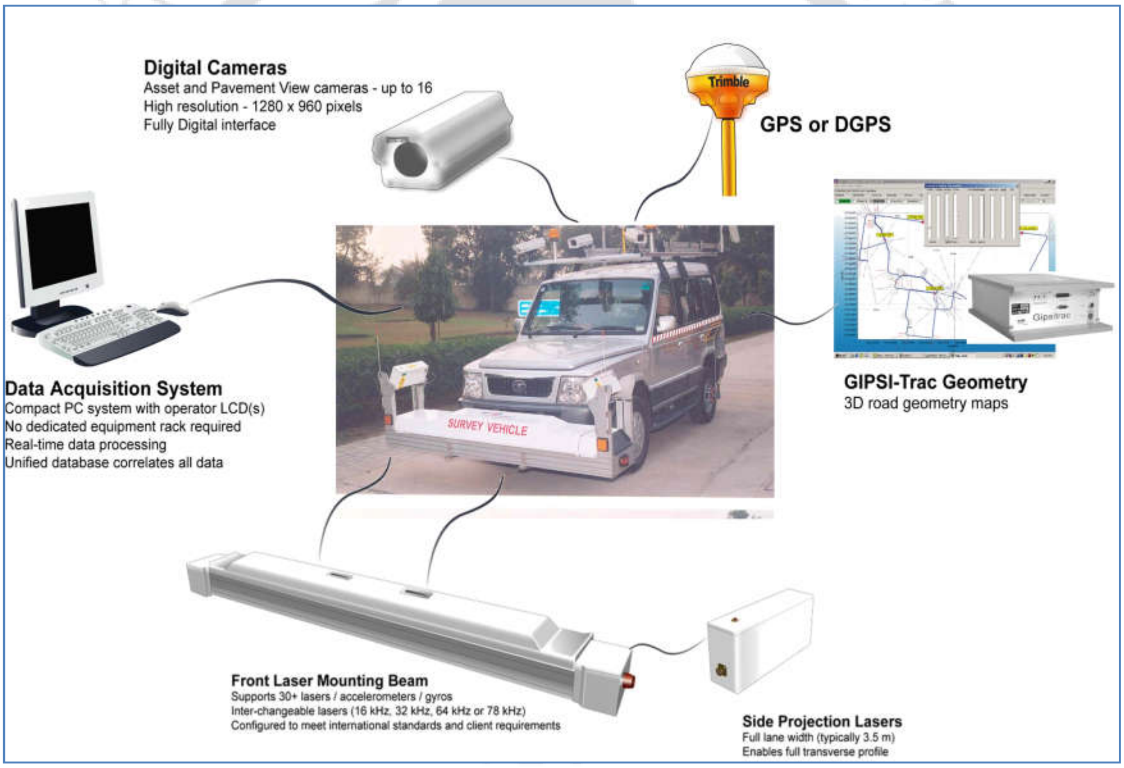


Photo 4.5 Network survey vehicle (NSV) with different components

- **Laser profiler bounce test** – An accelerometer is a device to measure the motion or vibration by converting physical movement into an electrical signal suitable for measurement, recording, analysis and/or control. The bounce test verifies that the laser profiler accelerometers are correcting for vertical movement of the survey vehicle and calibrate the roughness measurements prior to a new survey. This test is generally conducted in two phases – the still vehicle phase, and the bounce phase. Each phase is approximately 10 seconds long. During the still vehicle phase, the vehicle must not be moved. Immediately after end of the phase, the bounce phase begins where the vehicle must be bounced pushing down on the laser profiler beam for full period of the phase at the intervals of 0.5 sec and 1.5 sec. To check the bounce test calibration, the bounce wave shapes on a graph should be sinusoidal and the IRI values should be acceptable. If the IRI is greater than the acceptable level, the test should be repeated until an acceptable level is reached.

- **Laser profiler straight edge calibration** –The straight edge calibration will calculate relative offsets against a straight surface. This test confirms the operation of all lasers. The calibration is essential prior to rutting and transverse profile surveys. In this calibration, on a level surface, the front wheels of the survey vehicles shall drive onto the ramps. The purpose is to raise the front of the vehicle so that the straight edge, when fitted, is clear of the ground. The straight edge should be hooked in hangers on to the front of the profiler assembly so that it is below the lasers and correctly aligned. As the laser sensors are in operation, the straight edge calibration graph will show the current laser offsets. The current offsets are live and should be fluctuating over a very small range.

- **Laser profiler straight edge confirmation test** – The straight edge confirmation test measures whether there is any bow or bend in the straight edge used for the calibration test. This test should be performed after a successful straight edge calibration and the requirements are the same as for the calibration. After the activation of lasers, the graph will show the measurements of the beam in mm. Because the laser offsets have just been calibrated, all lasers should be very close to 0 mm.

4.6.6.2 Rut depth measurements –The transverse deformation across the wheel path is defined as the rut. Rut depth was measured using the NSV across full lane width (typically 3.5m) taken as per AASHTOPP38 standard. A laser beam was mounted on the front of NSV providing a high-resolution transverse profile every 250± 0.5mm accuracy at travel up to highway speeds.

4.6.6.3 Roughness measurements –The roughness is the deviation of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage. The roughness was also measured using the NSV upto a width of 3.5m of the pavement surface as per ASTM E950 standard. The accuracy of longitudinal profile measurement was ±0.5mm and the wavelength was 100mm to 100m. The measured roughness is expressed as International Roughness Index (IRI).

4.6.6.4 Texture depth measurements – The microtexture and macro texture deviations of pavement surface from a true planar surface are called pavement texture. In general, microtexture determines the maximum skid resistance afforded by a dry pavement, while macrotexture determines the drainage ability and therefore how microtexture will be effective when the pavement is wet. Most skidding related accidents occur on wet pavements. The changes in macrotexture due to wear and compaction resulting from traffic have important safety as well as economic consequences since rolling resistance is a function of texture. Texture depth was measured using the sand-patch test. The average texture depth of the road surfacing is expressed as the quotient of a given volume of standardized material (sand) and the area of that material spread in a circular patch on the surface being tested.

4.6.6.5 Skid resistance measurements –The ‘Portable Skid Resistance Tester’ also known as ‘British Pendulum Tester’ was used for measurements of skid resistance of pavement surface. The characteristics of the instrument are such as to simulate the sliding conditions between vehicles with patterned tyres, braking with locked wheels on wet pavement surface at a speed of 50 kmph. The quantity measured with the portable tester is termed skid resistance value (SRV).

The SRV is converted to ‘Sideways Force Coefficient’ (SFC, measured with Sideways Coefficient Routine Investigation Machine - SCRIM), using the relationship given by Equation 4.1 (Kennedy et al., 1990).

$$\text{SFC} = 0.01 * \text{SRV} \tag{4.1}$$

4.6.7 Evaluation of Pavement Materials

4.6.7.1 Field evaluation – The destructive technique of testing was adopted for field evaluation of test sections. Test pits of size approx. 1.0 x 1.0m were made at suitable locations representing the nearby pavement sections with similar crust compositions. The following tests were conducted:

- Layer thickness of the most recent surface course and old surface courses
- Layer thickness of bituminous and granular base and subbase
- Field dry density and field moisture content of the granular base, subbase, and soil subgrade

For characterization of in-situ materials in the laboratory, representative subgrade soil, subbase and base materials' samples were collected from the test pits. Photos 4.6 and 4.7 show test pit observation and field density test being conducted. The bituminous core samples were collected from the bituminous surface course and bituminous base course using core cutting machine for density and sieve analysis. Photo 4.8 shows the progress of full-depth core cutting in a test section.

Photo 4.6 Observation of test pits

4.6.7.2 Laboratory evaluation– Detailed evaluation of the subgrade soil samples, collected from the field was done in the laboratory, in accordance with Indian standards and specifications. The following tests were carried out for each soil sample:

- Atterberg limits (liquid limit and plastic limit)

- Modified proctor density and optimum moisture content
- CBR (soaked at field conditions)

Photo 4.7 Field density test

Photo 4.8 Full-depth core cutting of bituminous layers

4.6.8 Road Network Database

Calibration and local adaptation of the HDM-4 pavement deterioration model through the calibration factors needs the availability of good quality quantitative time series data on the occurrence of distresses for different pavement sections and traffic combinations under local conditions. To calibrate a pavement deterioration model, it is necessary to collect a group of distress data that serve to represent the real performance curve. It is necessary to measure data continuously for an extended period of time for each selected section for validation of the calibrated deterioration models which can be later compared for best agreement between the model's predictions and the observed field data.

This study involved generation of a huge amount of field data, because of the large size of the highway network. It was aimed that the collected data, directly or in its derived form should meet the requirements of HDM-4 system. Time series pavement performance data were collected for three (3) years consecutively for all 23 pavement sections selected. Time series data collected during the year 2011, 2012, and 2013 for each pavement section, are given in Tables 4.5 to 4.13. These data items were also stored in the road network database created for the identified highway network. This road network database has been named as 'high-speed road corridor network' for all future references and uses.

4.7 VEHICLE FLEET DATA

4.7.1 Categories of Vehicles

A typical traffic flow on all categories of roads in India, including national highways, comprises of both motorised (MT) and non-motorised (NMT) vehicles. However, pedestrians were also considered under the category of NMT. But in the case of access controlled expressways, pedestrians and slow moving MT and NMT vehicles were not allowed for plying. The following vehicle fleets were identified:

- For MT Vehicle -
  - (i) Two Wheeler
  - (ii) Three Wheeler
  - (iii) Car/Jeep (Big)
  - (iv) Car /Jeep (Small)
  - (v) Bus (BUS)
  - (vi) Minibus (LPV)
  - (vii) Light Commercial Vehicle (LCV)
  - (viii) Medium Truck (3 Single Axle)
  - (ix) Normal 2 Axle Truck (6 Wheel)
  - (x) Heavy Multi Axle Truck (Single Axle)
  - (xi) Heavy Multi Axle Truck (Tandem Axle)
  - (xii) Heavy Multi Axle Truck (Tridem Axle)
- For NMT Vehicle-
  - (i) Bicycle
  - (ii) Cycle Rickshaw
  - (iii) Pedestrians

The basic vehicle fleet data items, which are required to be specified for each vehicle type are incorporated in the vehicle fleet database created in HDM-4. This vehicle fleet database has been named as 'Vehicle Fleet' for all future references and uses.

4.7.2 Traffic Volume Counts

Traffic surveys were conducted manually, for 48 hours round the clock, by engaging an adequate number of skilled enumerators for motorised and non-motorised vehicles at all locations. The annual average daily traffic (AADT) of motorised vehicles obtained for the time series (for years 2011, 2012, and 2013) of all the pavement sections are given in the Tables 4.5, 4.9, and 4.11. The compositions of motorised vehicles for the year 2011 are given in Table 4.13.

4.7.3 Axle Load Survey

The equivalent standard axle load factor is defined as the number of applications of a standard 80 kN dual-wheel single axle load that would cause the same amount of damage to the road as one application of the axle load being considered. The axle load surveys of commercial vehicles were conducted using the weigh-in-motion system at all locations of test sections. The average vehicle damage factor (VDF) values obtained are given in Table 4.14.

Table 4.5 Inventory data of all selected pavement sections of high-speed road corridor network (Time-Series-1)

Section ID	Section Name	Link Name	Speed Flow Type	Traffic Flow Pattern	Climate Zone	Section Length (km)	CW Width (m)	Motorised AADT	AADT Year
NE-1GJ01	NE-1GJ[Km. 14-15]	Ahmedabad – Vadodara	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	8387	2011
NE-1GJ02	NE-1GJ[Km. 32 -31]	Vadodara - Ahmedabad	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	8304	2011
NE-1GJ03	NE-1GJ[Km. 19 - 20]	Ahmedabad – Vadodara	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	8387	2011
NE-1GJ04	NE-1GJ[Km. 15 - 14]	Vadodara - Ahmedabad	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	8304	2011
NH-14GJ05	NH-14GJ[Km. 380.6 - 379.6]	Radhanpur-Deesa	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	3715	2011
NH-2 UP01	NH-2 UP[Km. 607 - 606]	Allahabad - Khaga	Four Lane Road	Inter-city	Sub Humid Subtropical hot	1.0	7.0	4789	2011
NH-2UP02	NH-2UP[Km.717 - 718]	Allahabad - Varanasi	Four Lane Road	Inter-city	Sub Humid Subtropical hot	1.0	7.0	4906	2011
NH-37AS01	NH-37AS[Km.177.3 - 178.3]	Nagaon - Guwahati	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	4316	2011
NH-4KA01	NH-4KA[Km. 46 - 45]	Tumkur - Bangalore	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	16789	2011
NH-4KA02	NH-4KA[Km. 481.3 - 482.3]	Dharwad - Belgaon	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	5088	2011
NH-4KA03	NH-4KA[Km. 82 - 83]	Tumkur - Sira	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	11456	2011
NH-4MH01	NH-4MH[Km. 539 - 538]	Maharastra Border - Belgaon	Four Lane Road	Inter-city	Semi- Arid–Subtropical hot	1.0	7.0	5029	2011
NH-5AP01	NH-5AP[Km. 691.25 - 690.75]	Vizag - Srikakuram	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	6771	2011
NH-5AP02	NH-5AP[Km. 698.6 - 698.1]	Vizag - Srikakuram	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	6771	2011
NH-5AP03	NH-5 AP[Km. 7.5 - 7.0]	NH-5 - Vizag Port	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	5682	2011
NH-5AP04	NH-5AP[Km. 9.8 - 9.3]	NH-5 - Vizag Port	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	5682	2011
NH-7MH02	NH-7MH[Km. 84.2 - 84.7]	Hyderabad-Nagpur	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	0.5	7.0	4288	2011
NH-7MH03	NH-7MH[Km. 95.5 - 96.0]	Hyderabad - Nagpur	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	0.5	7.0	4288	2011
NH-7AP05	NH-7AP[Km. 41 - 40]	Nagpur - Hyderabad	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	1.0	7.0	5614	2011
NH-7AP06	NH-7AP[Km. 51.4 – 50.4]	Nagpur - Hyderabad	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	1.0	7.0	5614	2011
NH-7KA04	NH-7KA[Km. 518 - 519]	Bagepalli - Bangalore	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	11375	2011
NH-8GJ06	NH-8GJ[Km. 433.7 - 434.7]	Palampur - Himmat Nagar	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	7577	2011
NH-8GJ07	NH-8GJ[Km. 441.4 - 442.4]	Palampur- Himmat Nagar	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	7577	2011

Note: CW: Carriageway; AADT: Average annual daily traffic

Table 4.6 Observed condition data on all pavement sections of high-speed road corridor network (Time-Series-1)

Section Name	Condition Year	Roughness IRI (m/km)	Cracking Area (%)	Ravelled Area (%)	Potholes (no./km)	Average Rut Depth (mm)	Texture Depth (mm)	Skid Resistance (SFC50)	Benkelman Beam Deflection (mm)
NE-1GJ [Km. 14 - 15]	2011	2.3	2	2.3	0	2.2	0.70	0.60	0.49
NE-1GJ [Km. 32 - 31]	2011	2.5	7	5	0	3.5	0.90	0.65	0.37
NE-1GJ [Km. 19 - 20]	2011	2.5	1	1	0	2.7	0.70	0.65	0.24
NE-1GJ [Km. 15 - 14]	2011	2.5	1.5	1	0	2.2	0.90	0.65	0.35
NH-14GJ [Km. 380.6 - 379.6]	2011	2.7	27.07	23	3	2.4	0.80	0.65	0.56
NH-2UP[Km. 607 - 606]	2011	2.5	7	4	2	3	0.80	0.75	0.55
NH-2UP[Km. 717 - 718]	2011	2.8	1	2	0	2.7	0.65	0.65	0.67
NH-37AS[Km.177.30 -178.30]	2011	2.5	0	0	0	2	0.70	0.65	0.64
NH-4KA [Km. 46 - 45]	2011	3.2	4.57	2	0	2.6	0.70	0.60	0.75
NH-4KA [Km. 481.3 - 482.3]	2011	3.7	5	2	0	2.1	0.80	0.65	0.70
NH-4KA [Km. 82 - 83]	2011	3.9	19.86	15	2	3.6	0.80	0.65	0.75
NH-4MH [Km. 539 - 538]	2011	3.0	0	0	0	2.1	1.10	0.70	0.80
NH-5AP [Km. 691.25 - 690.75]	2011	2.5	1.16	0	0	1.6	0.55	0.70	0.68
NH-5AP[Km. 698.6 - 698.1]	2011	2.8	0.66	0	0	2.7	0.60	0.70	0.75
NH-5 AP[Km. 7.5 - 7.0]	2011	2.9	10.87	8	1	1.9	0.75	0.80	0.72
NH-5AP[Km. 9.8 - 9.3]	2011	2.9	11	9	1	1.7	0.70	0.75	0.88
NH-7MH [Km.84.2 - 84.7]	2011	2.5	0	0	0	2	0.68	0.60	0.75
NH-7MH [Km. 95.5 - 96.0]	2011	2.4	0	1	1	2	0.74	0.68	0.65
NH-7AP [Km. 41 - 40]	2011	2.9	0	0	0	1.8	0.80	0.67	0.50
NH-7AP [Km.51.4 - 50.4]	2011	2.6	0	0	0	2.5	0.73	0.63	0.43
NH-7KA [Km.518 - 519]	2011	2.4	1	0	0	1.9	0.70	0.70	0.75
NH-8GJ [Km. 433.7 - 434.7]	2011	2.6	2.38	1	0	1.9	0.80	0.64	0.62
NH-8GJ [Km. 441.4 - 442.4]	2011	2.6	0	1	0	3.1	1.00	0.70	0.51

Table 4.7 Pavement data collected from all sections of high-speed road corridor network (Time-Series-1)

Section Name	Composition of Pavement Sections (thickness in mm)							Current Surface Thickness (mm)	Previous Surface Thickness (mm)	Last Constn. Year	Last Rehab. Year	Last Surf. Year	Last Prev. Treat. Year
	Subgrade	GSB	WBM	CTAB	WMM	DBM with Binder Grade	BC with Binder Grade						
NE-1GJ [Km. 14 - 15]	600	300	150	150	150	150(60/70)	50(CRMB)	10	50	Oct. 2003	2003	2009	2009
NE-1GJ [Km. 32 - 31]	600	300	150	150	150	150(60/70)	50(PMB)	10	50	Oct. 2003	2003	2009	2009
NE-1GJ [Km. 19-20]	600	300	150	150	150	150(60/70)	50(CRMB)	10	50	Oct. 2003	2003	2009	2009
NE-1GJ [Km.15 - 14]	600	300	150	150	150	150(60/70)	50(CRMB)	10	50	Oct. 2003	2003	2009	2009
NH-14GJ [Km. 380.6 - 379.6]	500	200	-	-	250	160(60/70)	50(CRMB)	-	50	Oct. 2008	2008	2008	2008
NH-2 UP[Km. 607 - 606]	500	300	-	-	250	170(60/70	50(CRMB)	-	50	July 2005	2005	2005	2005
NH-2UP[Km. 717 - 718]	500	300	-	-	250	170(60/70)	50(CRMB)	-	50	Oct. 2005	2005	2005	2005
NH-37AS[Km. 177.3 - 178.3]	500	200	-	-	300	120(60/70)	40(PMB)	-	40	Feb. 2010	2010	2010	2010
NH-4KA [Km. 46 - 45]	500	200	-	-	300	120 (60/70)	40(CRMB)	40	40	Aug. 2003	2009	2009	2009
NH-4KA [Km. 481.3 - 482.3]	500	200	-	-	300	120 (60/70)	40(CRMB)	-	40	Nov. 2008	2008	2008	2008
NH-4KA [Km. 82 - 83]	500	200	-	-	300	120(60/70)	40(CRMB)	-	40	Sept. 2004	2004	2004	2004
NH-4MH [Km. 539 - 538]	500	200	-	-	275	125(60/70)	40(PMB)	-	40	June 2004	2004	2004	2004
NH-5AP [Km. 691.25 - 690.75]	500	300	-	-	250	170(60/70)	50(CRMB)	-	50	Dec.2004	2004	2004	2004
NH-5AP[Km. 698.6 - 698.1]	500	300	-	-	250	170(60/70)	50(CRMB)	-	50	Dec.2004	2004	2004	2004
NH-5 AP[Km. 7.5 - 7.0]	500	300	-	-	250	170(60/70)	50(CRMB)	-	50	Dec.2006	2006	2006	2006
NH-5AP[Km. 9.8 - 9.3]	500	300	-	-	250	170(60/70)	50(CRMB)	-	50	Dec.2006	2006	2006	2006
NH-7MH [Km. 84.2 - 84.7]	500	200	-	-	275	125(60/70)	40(PMB)	-	40	April2008	2008	2008	2008
NH-7MH [Km. 95.5 - 96.0]	500	200	-	-	300	140(60/70)	50(CRMB)	-	50	April2008	2008	2008	2008
NH-7AP [Km. 41 - 40]	500	200	-	-	300	140(60/70)	50(CRMB)	-	50	March2008	2008	2008	2008
NH-7AP [Km. 51.4 -50.4]	500	200	-	-	300	140(60/70)	50(CRMB)	-	50	June 2009	2009	2009	2009
NH-7KA [Km. 518 - 519]	500	200	-	-	300	120(60/70)	40(CRMB)	-	40	Dec 2007	2007	2007	2007
NH-8GJ [Km. 433.7 - 434.7]	500	200	-	-	250	160(60/70)	50(CRMB)	40	50	Aug. 2004	2009	2009	2009
NH-8GJ [Km.441.4 - 442.4]	500	200	-	-	250	160(60/70)	50(CRMB)	-	50	Aug.2004	2004	2004	2004

Note: GSB- Granular Sub-Base; WBM-Water Bound Macadam; CTAB- Cement Treated Aggregate Base; WMM- Wet Mix Macadam;DBM- Dense Graded Bituminous Macadam; BC- Bituminous Concrete; Constn. - Construction; Rehab.- Rehabilitation; Surf.- Surfacing;Prev. – Preventive; Treat. - Treatment

Table 4.8 Laboratory test results of collected subgrade soil samples on all pavement sections

Section Name	Maximum Dry Density (gm/cc)	Optimum Moisture Content (%)	Atterberg Limits (%)			CBR (%) at Field Dry Density (Soaked)
			LL	PL	PI	
NE-1GJ [Km. 14 - 15]	1.82	8.20	0	0	NP	6.2
NE-1GJ [Km. 32 - 31]	1.82	8.20	0	0	NP	6.2
NE=1GJ [Km. 19-20]	1.82	8.20	0	0	NP	6.2
NE-1GJ [Km. 15 - 14]	1.82	8.20	0	0	NP	6.2
NH-14GJ [Km. 380.6 - 379.6]	2.04	8.80	34	17	17.0	6.2
NH-2 UP[Km. 607 - 606]	2.12	10.50	28	18	10	6.3
NH-2UP[Km. 717 - 718]	2.12	10.50	28	18	10	6.3
NH-37AS[Km. 177.3 - 178.3]	1.87	12.80	0	0	NP	8.0
NH-4KA [Km. 46 - 45]	1.86	10.60	40	28	12	6.1
NH-4KA [Km. 481.3 - 482.3]	2.13	10.20	40	24	16	6.2
NH-4KA [Km. 82 - 83]	2.13	8.00	34	24	10	6.3
NH-4MH [Km. 539 - 538]	2.10	17.00	44	27	17	6.1
NH-5AP[Km. 691.25 - 690.75]	2.03	9.00	30	24	6	7.0
NH-5AP[Km. 698.6 - 698.1]	2.03	9.00	30	24	6	7.0
NH-5 AP [Km.7.5 - 7.0]	1.96	12.30	26	16	10	6.8
NH-5AP[Km.9.8 - 9.3]	1.94	12.00	24	14	10	7.2
NH-7MH [Km.84.2 - 84.7]	2.09	9.60	0	0	NP	7.9
NH-7MH [Km. 95.5 - 96.0]	2.09	9.60	0	0	NP	7.9
NH-7AP [Km. 41 - 40]	1.91	15.00	38	32	7	7.0
NH-7AP [Km. 51.4 -50.4]	1.96	10.00	44	36	8	6.5
NH-7KA [Km. 518 - 519]	2.14	8.20	0	0	NP	7.3
NH-8GJ [Km. 433.7 - 434.7]	2.02	7.80	0	0	NP	6.0
NH-8GJ [Km. 441.4 - 442.4]	2.02	7.80	0	0	NP	6.0

Table 4.9 Inventory data of all selected pavement sections of high-speed road corridor network (Time-Series-2)

Section ID	Section Name	Link Name	Speed Flow Type	Traffic Flow Pattern	Climate Zone	Section Length (km)	CW Width (m)	Motorised AADT	AADT Year
NE-1GJ01	NE-1GJ[Km. 14-15]	Ahmedabad - Vadodara	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	9016	2012
NE-1GJ02	NE-1GJ[Km. 32 -31]	Vadodara - Ahmedabad	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	8927	2012
NE-1GJ03	NE-1GJ[Km. 19 - 20]	Ahmedabad - Vadodara	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	9016	2012
NE-1GJ04	NE-1GJ[Km. 15 - 14]	Vadodara - Ahmedabad	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	8927	2012
NH-14GJ05	NH-14GJ[Km. 380.6 - 379.6]	Radhanpur-Deesa	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	3994	2012
NH-2 UP01	NH-2 UP[Km. 607 - 606]	Allahabad - Khaga	Four Lane Road	Inter-city	Sub Humid Subtropical hot	1.0	7.0	5148	2012
NH-2UP02	NH-2UP[Km. 717 - 718]	Allahabad - Varanasi	Four Lane Road	Inter-city	Sub Humid Subtropical hot	1.0	7.0	5274	2012
NH-37AS01	NH-37AS[Km.177.30 - 178.30]	Nagaon - Guwahati	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	5357	2012
NH-4KA01	NH-4KA[Km. 46 - 45]	Tumkur - Bangalore	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	18048	2012
NH-4KA02	NH-4KA[Km. 481.3 - 482.3]	Dharwad - Belgaon	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	5470	2012
NH-4KA03	NH-4KA[Km. 82 - 83]	Tumkur - Sira	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	12315	2012
NH-4MH01	NH-4MH[Km. 539 - 538]	Maharastra Border - Belgaon	Four Lane Road	Inter-city	Semi- Arid –Subtropical hot	1.0	7.0	5406	2012
NH-5AP01	NH-5AP[Km. 691.25 - 690.75]	Vizag- Srikakuram	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	7279	2012
NH-5AP02	NH-5AP[Km. 698.6 - 698.1]	Vizag- Srikakuram	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	7279	2012
NH-5AP03	NH-5 AP[Km. 7.5 - 7.0]	NH-5 - Vizag Port	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	6108	2012
NH-5AP04	NH-5AP[Km. 9.8 - 9.3]	NH-5 - Vizag Port	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	6108	2012
NH-7MH02	NH-7MH[Km. 84.2 - 84.7]	Hyderabad - Nagpur	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	0.5	7.0	4610	2012
NH-7MH03	NH-7MH[Km. 95.5 - 96.0]	Hyderabad - Nagpur	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	0.5	7.0	4610	2012
NH-7AP05	NH-7AP[Km. 41 - 40]	Nagpur - Hyderabad	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	1.0	7.0	6035	2012
NH-7AP06	NH-7AP[Km. 51.4 – 50.4]	Nagpur - Hyderabad	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	1.0	7.0	6035	2012
NH-7KA04	NH-7KA[Km. 518 - 519]	Bagepalli - Bangalore	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	12228	2012
NH-8GJ06	NH-8GJ[Km. 433.7 - 434.7]	Palampur - HimmatNagar	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	8145	2012
NH-8GJ07	NH-8GJ[Km. 441.4 - 442.4]	Palampur - HimmatNagar	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	8145	2012

Note: CW - Carriageway

Table 4.10 Observed condition data on all pavement sections of high-speed road corridor network (Time-Series-2)

Section Name	Condition Year	Roughness IRI (m/km)	Cracking Area (%)	Ravelled Area (%)	Potholes (no./km)	Av. Rut Depth (mm)	Texture Depth (mm)	Skid Resistance (SFC50)	Benkelman Beam Deflection (mm)
NE-1GJ [Km. 14 -15]	2012	2.40	3.5	5	0	2.5	0.50	0.55	0.73
NE-1GJ [Km. 32 – 31]	2012	2.60	10	8	0	4.0	0.80	0.60	0.80
NE-1GJ [Km.19 – 20]	2012	2.60	4	4	0	2.9	0.65	0.55	0.72
NE-1GJ [Km. 15 - 14]	2012	2.71	3	3	0	2.7	0.84	0.60	0.71
NH-14GJ[Km.380.6 - 379.6]	2012	2.80	30	25	3	3.2	0.75	0.60	0.69
NH-2 UP[Km. 607 - 606]	2012	2.60	10	7	2	3.5	0.73	0.72	0.69
NH-2UP[Km. 717 - 718]	2012	3.10	3	3	0	3.0	0.60	0.60	0.71
NH-37AS[Km. 177.30 - 178.30]	2012	2.90	1	0	0	2.5	0.65	0.60	0.83
NH-4KA [Km. 46 - 45]	2012	3.40	10.43	8	0	3.0	0.68	0.55	0.95
NH-4KA [Km. 481.3 - 482.3]	2012	3.80	8	5	0	2.5	0.70	0.60	0.77
NH-4KA [Km. 82 - 83]	2012	4.10	25.45	20	3	4.4	0.75	0.60	0.90
NH-4MH [Km. 539 - 538]	2012	3.10	0	0	0	3.1	1.00	0.65	0.87
NH-5AP [Km. 691.25 - 690.75]	2012	2.70	2.1	0	0	1.7	0.50	0.68	0.78
NH-5AP[Km. 698.6 - 698.1]	2012	2.90	3.34	0	0	3.4	0.55	0.67	0.85
NH-5 AP[Km. 7.5 - 7.0]	2012	3.00	15	10	3	2.7	0.70	0.75	0.80
NH-5AP[Km. 9.8 - 9.3]	2012	3.00	16	12	3	2.5	0.64	0.72	0.99
NH-7MH [Km.84.2 - 84.7]	2012	2.80	0	0	0	2.1	0.60	0.55	0.82
NH-7MH [Km. 95.5 - 96.0]	2012	2.60	0	2	0	2.5	0.70	0.65	0.75
NH-7AP [Km. 41 - 40]	2012	3.10	0	0	0	3.0	0.70	0.63	0.91
NH-7AP [Km. 51.4 -50.4]	2012	2.70	0	0	0	2.8	0.70	0.60	0.75
NH-7KA [Km. 518 - 519]	2012	2.50	2	0	0	2.5	0.65	0.65	0.91
NH-8GJ [Km. 433.7 - 434.7]	2012	2.80	5	2	0	2.6	0.70	0.60	0.86
NH-8GJ [Km. 441.4 - 442.4]	2012	2.70	2	5	0	4.0	0.80	0.68	0.92

Table 4.11 Inventory data of all selected pavement sections of high-speed road corridor network (Time-Series-3)

Section ID	Section Name	Link Name	Speed Flow Type	Traffic Flow Pattern	Climate Zone	Section Length (km)	Carriageway Width (m)	Motorized AADT	AADT Year
NE-1GJ01	NE-1GJ[Km. 14-15]	Ahmedabad - Vadodara	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	9688	2013
NE-1GJ02	NE-1GJ[Km. 32 -31]	Vadodara - Ahmedabad	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	9626	2013
NE-1GJ03	NE-1GJ[Km. 19 - 20]	Ahmedabad - Vadodara	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	9688	2013
NE-1GJ04	NE-1GJ[Km. 15 - 14]	Vadodara - Ahmedabad	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	8.4	9626	2013
NH-14GJ05	NH-14GJ[Km. 380.6 - 379.6]	Radhanpur - Deesa	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	5563	2013
NH-2UP01	NH-2UP[Km. 607 - 606]	Allahabad - Khaga	Four Lane Road	Inter-city	Sub Humid Subtropical hot	1.0	7.0	5836	2013
NH-2UP02	NH-2UP[Km. 717 - 718]	Allahabad - Varanasi	Four Lane Road	Inter-city	Sub Humid Subtropical hot	1.0	7.0	5825	2013
NH-37AS01	NH-37AS[Km. 177.30 - 178.30]	Nagaon - Guwahati	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	6299	2013
NH-4KA01	NH-4KA[Km. 46 - 45]	Tumkur - Bangalore	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	18893	2013
NH-4KA02	NH-4KA[Km. 481.3 - 482.3]	Dharwad - Belgaon	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	5811	2013
NH-4KA03	NH-4KA[Km. 82 - 83]	Tumkur - Sira	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	13238	2013
NH-4MH01	NH-4MH[Km. 539 - 538]	Maharastra Border - Belgaon	Four Lane Road	Inter-city	Semi-Arid–Subtropical hot	1.0	7.0	6248	2013
NH-5AP01	NH-5AP[Km. 691.25 - 690.75]	Vizag - Srikakuram	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	7860	2013
NH-5AP02	NH-5AP[Km. 698.6 - 698.1]	Vizag - Srikakuram	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	7860	2013
NH-5AP03	NH-5AP[Km. 7.5 - 7.0]	NH-5 - Vizag Port	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	6566	2013
NH-5AP04	NH-5AP[Km. 9.8 - 9.3]	NH-5 - Vizag Port	Four Lane Road	Inter-city	Sub Humid Subtropical hot	0.5	7.0	6566	2013
NH-7MH02	NH-7MH[Km. 84.2 - 84.7]	Hyderabad - Nagpur	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	0.5	7.0	5050	2013
NH-7MH03	NH-7MH[Km. 95.5 - 96.0]	Hyderabad - Nagpur	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	0.5	7.0	5050	2013
NH-7AP05	NH-7AP[Km. 41 - 40]	Nagpur - Hyderabad	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	1.0	7.0	6488	2013
NH-7AP06	NH-7AP[Km. 51.4 – 50.4]	Nagpur - Hyderabad	Four Lane Road	Inter-city	Semi- Arid Subtropical hot	1.0	7.0	6488	2013
NH-7KA04	NH-7KA[Km. 518 - 519]	Bagepalli - Bangalore	Four Lane Road	Inter-city	Humid Subtropical cool	1.0	7.0	13145	2013
NH-8GJ06	NH-8GJ[Km. 433.7 - 434.7]	Palampur - Himmat Nagar	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	8859	2013
NH-8GJ07	NH-8GJ[Km. 441.4 - 442.4]	Palampur - Himmat Nagar	Four Lane Road	Inter-city	Semi- Arid -Tropical	1.0	7.0	8859	2013

Table 4.12 Observed condition data on all pavement sections of high-speed road corridor network (Time-Series-3)

Section Name	Condition Year	Roughness IRI (m/km)	Cracking Area (%)	Ravelled Area (%)	Potholes (no./km)	Av. Rut Depth (mm)	Texture Depth (mm)	Skid Resistance (SFC50)	Benkelman Beam Deflection (mm)
NE-1GJ[Km. 14 -15]	2013	2.60	7	6	0	2.8	0.45	0.52	0.81
NE-1GJ[Km. 32 – 31]	2013	3.30	12	11	0	4.3	0.70	0.55	0.85
NE-1GJ[Km.19 – 20]	2013	2.80	6.5	6	0	3.0	0.60	0.50	0.77
NE-1GJ[Km. 15 - 14]	2013	2.85	6.3	5	0	3.0	0.80	0.56	0.75
NH-14GJ[Km. 380.6 - 379.6]	2013	2.95	39	28	5	4.0	0.70	0.53	0.72
NH-2UP[Km. 607 - 606]	2013	3.00	10.5	7.8	1	4.2	0.70	0.70	0.73
NH-2UP[Km. 717 - 718]	2013	3.50	5.4	4.1	0	3.1	0.52	0.55	0.77
NH-37AS[Km.177.30 - 178.30]	2013	3.00	5.1	2	0	3.1	0.56	0.53	0.98
NH-4KA[Km. 46 - 45]	2013	3.50	17	15	1	3.5	0.60	0.50	0.99
NH-4KA[Km. 481.3 - 482.3]	2013	3.90	12	9.5	0	3.0	0.62	0.55	0.87
NH-4KA[Km. 82 - 83]	2013	4.20	30	25	4	5.1	0.60	0.50	0.98
NH-4MH[Km. 539 - 538]	2013	3.30	0	0	0	3.4	0.85	0.54	0.93
NH-5AP[Km. 691.25 - 690.75]	2013	3.00	2.16	2	0	3.2	0.41	0.65	0.86
NH-5AP[Km. 698.6 - 698.1]	2013	3.10	4.8	2	0	4.0	0.50	0.60	0.94
NH-5 AP[Km.7.5 - 7.0]	2013	3.30	16	12	3	3.1	0.60	0.70	0.90
NH-5AP[Km. 9.8 - 9.3]	2013	3.20	18	13	5	3.1	0.61	0.70	1.05
NH-7MH[Km. 84.2 - 84.7]	2013	3.00	10.5	6.2	2	2.3	0.58	0.50	0.90
NH-7MH[Km. 95.5 - 96.0]	2013	3.00	1.1	4.7	0	3.0	0.66	0.60	0.84
NH-7AP[Km. 41 - 40]	2013	3.50	1.5	3.5	0	4.3	0.65	0.61	0.98
NH-7AP[Km. 51.4 -50.4]	2013	2.90	0.52	4	0	5.0	0.68	0.54	0.84
NH-7KA[Km. 518 - 519]	2013	2.80	6	0.8	0	5.1	0.63	0.55	0.97
NH-8GJ[Km. 433.7 - 434.7]	2013	3.10	10	10	2	3.5	0.60	0.55	0.95
NH-8GJ[Km. 441.4 - 442.4]	2013	3.00	7	11	3	5.0	0.70	0.61	0.98

Table 4.13 Motorised vehicles composition of all pavement sections

Pavement Section ID	Motorised Vehicle(MT) percentage in Total AADT (Year 2011)												
	Two Wheeler	Three Wheeler	Car/Jeep (Small)	Car/ Jeep (Big)	Bus	Tractor Trailer	Minibus (LPV)	Light Commercial Vehicle	2-Axle Truck (6-wheel)	3-Axle Truck (10-wheel)	Multi-Axle Truck		
											Single-Axle	Tandem	Tridem
NE-1GJ01	0	0	33.9	33.0	9.0	1.0	1.0	9.0	6.0	4.0	2.0	1.0	0.1
NE-1GJ02	0	0	33.0	35.0	9.0	1.0	0.4	2.0	13.0	4.0	1.3	1.0	0.3
NE-1GJ03	0	0	33.9	33.0	9.0	1.0	1.0	9.0	6.0	4.0	2.0	1.0	0.1
NE-1GJ04	0	0	33.0	35.0	9.0	1.0	0.4	2.0	13.0	4.0	1.3	1.0	0.3
NH-14GJ05	27.1	5.5	27.0	9.8	1.4	2.6	0.3	1.8	3.5	5.0	0.5	5.5	10.0
NH-2 UP01	15.7	8.2	16.9	9.6	3.6	1.0	2.0	5.8	9.6	17.8	2.4	4.3	3.1
NH-2UP02	14.7	2.4	9.8	11.5	3.4	2.5	2.6	6.0	8.3	11.3	10.1	11.2	6.2
NH-37AS01	10.7	6.8	24.2	21.8	5.1	0.1	2.4	8.7	14.4	5.0	0.1	0.5	0.2
NH-4KA01	22.26	4.77	16.64	7.4	7.37	0.37	1.28	11.15	9.56	14.7	4.0	0.41	0.09
NH-4KA02	3.23	3.29	15.56	14.8	8.2	2.26	1.26	9.4	12.52	21.83	6.22	0.59	0.84
NH-4KA03	6.23	0.93	17.56	12.8	8.2	0.26	1.26	11.4	12.52	21.83	6.22	0.59	0.2
NH-4MH01	3.23	3.29	15.56	14.8	8.2	2.26	1.26	9.4	12.52	21.83	6.22	0.59	0.84
NH-5AP01	27.3	23.3	22.0	9.0	5.4	1.9	1.5	1.8	2.5	3.5	0.2	0.8	0.8
NH-5AP02	27.3	23.3	22.0	9.0	5.4	1.9	1.5	1.8	2.5	3.5	0.2	0.8	0.8
NH-5AP03	31.3	11.6	17.4	4.3	0.0	0.0	1.2	4.5	5.8	14.9	0.4	8.0	0.6
NH-5AP04	31.3	11.6	17.4	4.3	0.0	0.0	1.2	4.5	5.8	14.9	0.4	8.0	0.6
NH-7MH02	7.5	0.49	10.0	9.6	4.53	0.22	1.47	6.89	15.0	16.31	15.62	6.3	6.0
NH-7MH03	5.32	0.6	9.3	9.0	4.8	0.24	1.24	6.76	15.0	15.0	12.98	6.98	6.0
NH-7AP05	19.2	1.1	17.8	13.3	4.2	0.8	2.4	4.4	10.2	18.8	1.0	3.2	3.6
NH-7AP06	19.2	1.1	17.8	13.3	4.2	0.8	2.4	4.4	10.2	18.8	1.0	3.2	3.6
NH-7KA04	23.2	1.43	25.31	8.88	7.04	0.84	1.78	13.2	4.52	9.37	2.58	1.06	0.79
NH-8GJ06	26.2	4.9	14.8	18.1	6.1	1.6	0.2	6.8	4.9	8.0	1.5	3.1	3.8
NH-8GJ07	26.2	4.9	14.8	18.1	6.1	1.6	0.2	6.8	4.9	8.0	1.5	3.1	3.8

Table 4.14 Average vehicle damage factor

Section ID	Section Name	Average Vehicle Damage Factor (VDF)
NE-1GJ01	NE-1GJ[Km. 14-15]	2.27
NE-1GJ02	NE-1GJ [Km. 32 -31]	2.13
NE-1GJ03	NE-1GJ [Km. 19 - 20]	2.27
NE-1GJ04	NE-1GJ [Km. 15 - 14]	2.13
NH-14GJ05	NH-14GJ[Km. 380.6 - 379.6]	5.24
NH-2UP01	NH-2UP[Km. 607 - 606]	10.23
NH-2UP02	NH-2UP[Km. 717 - 718]	13.92
NH-37AS01	NH-37AS[Km.177.3 - 178.3]	2.10
NH-4KA01	NH-4KA [Km. 46 - 45]	4.40
NH-4KA02	NH-4KA[Km. 481.3 - 482.3]	2.99
NH-4KA03	NH-4KA [Km. 82 - 83]	3.85
NH-4MH01	NH-4MH [Km. 539 - 538]	3.12
NH-5AP01	NH-5AP[Km.691.25-690.75]	6.23
NH-5AP02	NH-5AP[Km. 698.6 - 698.1]	6.23
NH-5AP03	NH-5 AP[Km. 7.5 - 7.0]	7.44
NH-5AP04	NH-5AP[Km. 9.8 - 9.3]	7.44
NH-7MH02	NH-7MH [Km. 84.2 - 84.7]	4.50
NH-7MH03	NH-7MH [Km. 95.5 - 96.0]	4.50
NH-7AP05	NH-7AP [Km. 41 - 40]	5.11
NH-7AP06	NH-7AP [Km. 51.4 – 50.4]	5.11
NH-7KA04	NH-7KA [Km. 518 - 519]	4.82
NH-8GJ06	NH-8GJ [Km. 433.7 - 434.7]	8.07
NH-8GJ07	NH-8GJ [Km. 441.4 - 442.4]	8.07

4.8 GROUPING OF SECTIONS

The pavement layer compositions of all the sections are given in Table 4.15. The pavement layers provided for NHDP corridors were combinations of granular subbase (GSB) and water bound macadam (WBM) as subbase; cement treated aggregate base (CTAB) and wet mix macadam (WMM) as base; bituminous macadam (BM) and dense bituminous macadam (DBM) with 60/70 penetration grade bitumen as binder course; bituminous concrete (BC) with CRMB/PMB modified bituminous binders as wearing course. These 23 sections were divided into six (6) groups based on similarity in pavement composition and climatic conditions, as shown in Table 4.15. The details of climatic conditions of the sections were shown in Table 4.4.Pavement sections falling under a group can therefore be considered to be homogeneous in terms of their composition and climatic features.

Table 4.15: Details of selected pavement sections

Homogeneous Sections Group	Section Name	Composition of Pavement Sections (Layer thickness in *mm*)	Climate Classification
Group-1	NE-1GJ[Km. 14 -15] NE-1GJ [Km. 15 -14] NE-1GJ[Km. 19 - 20] NE-1GJ[Km. 32 - 31]	Subbase = 300 GSB + 150 WBM Base = 150 CTAB+ 150 WMM Binder Course (60/70 pen) = 150 DBM Wearing Course (CRMB/PMB) = 50 BC	Semi-Arid-Tropical
Group-2	NH-2 UP [Km. 607 - 606] NH-2UP [Km. 717 - 718] NH-5AP [Km. 691.25 - 690.75] NH-5AP [Km. 698.6 - 698.1] NH-5 AP [Km. 7.5 - 7.0] NH-5AP [Km. 9.8 - 9.3]	Subbase = 300 GSB Base = 250 WMM Binder Course (60/70 pen) = 170 DBM Wearing Course (CRMB) = 50 BC	Sub-Humid Sub-Tropical Hot
Group-3	NH-7AP [Km. 41 - 40] NH-7AP [Km. 51.4 – 50.4] NH-7MH [Km. 95.5 - 96.0]	Subbase = 200 GSB Base = 300 WMM Binder Course (60/70 pen) = 140 DBM Wearing Course (CRMB) = 50 BC	Semi-Arid Sub-Tropical Hot
Group-4	NH-14GJ [Km. 380.6 - 379.6] NH-8GJ [Km. 433.7 - 434.7] NH-8GJ [Km. 441.4 - 442.4]	Subbase = 200 GSB Base = 250 WMM Binder Course (60/70 pen) = 160 DBM Wearing Course (CRMB) = 50 BC	Semi-Arid-Tropical
Group-5	NH-4KA [Km. 46 - 45] NH-4KA [Km. 82 - 83] NH-7KA [Km. 518 - 519] NH-4KA [Km. 481.3 - 482.3] NH-37AS [Km. 177.3 – 178.3]	Subbase = 200 GSB Base = 300 WMM Binder Course (60/70 pen) = 120 DBM Wearing Course (CRMB/PMB) = 40 BC	Humid- Sub-Tropical cool
Group-6	NH-4MH [Km. 539 - 538] NH-7MH [Km. 84.2 - 84.7]	Subbase = 200 GSB Base = 275 WMM Binder Course (60/70 pen) = 125 DBM Wearing Course (PMB) = 40 BC	Semi-Arid-Subtropical hot

Note: GSB: Granular Subbase; WBM: Water Bound Macadam; WMM: Wet Mix Macadam; CTAB: Cement Treated Aggregate Base; DBM: Dense Bituminous Macadam; BC: Bituminous Concrete

4.9 SUMMARY

This chapter presented the methodology of data collection for deterioration modelling using HDM-4 for high-speed road corridor network in India. The methodology included identification and selection of pavement test sections on the network, data acquisition, and database management. The high-speed road corridor network included Golden-Quadrilateral (GQ), East-West and North-South (EW-NS) corridors developed under Indian National Highway Development Programme (NHDP). A total of twenty three (23) pavement sections on this highway network were selected for deterioration modelling. All 23 pavement sections were further classified into six (6) homogeneous groups on the basis of crust composition and climatic conditions. The different procedures and equipment used for collection of various kinds of field data for all pavement sections were described in this chapter. The data for vehicle fleet plying on the highway network, maintenance, and rehabilitation activities as obtained from field and relevant government publications were also presented. Details about the six homogeneous groups finalised on the basis of pavement composition and climatic and environmental conditions were also presented. All data were stored in relevant sections of the HDM-4 database for further processing and utilisation.

Chapter 5

CALIBRATION OF HDM-4 DETERIORATION MODELS

5.1 GENERAL

HDM-4 pavement deterioration models attempt to predict the initiation and progression of pavement deterioration manifested as various kinds of distresses such as rutting, cracking, raveling, etc. Since the rate of initiation and propagation of each pavement distress is strongly dependent on local conditions, it is important to calibrate the HDM-4 models. HDM-4 has been designed to be used in a wide range of environments, and therefore, the calibration of HDM-4 models provides the facility to customize system operation to reflect the norms that are customary in the environment under study. The use of appropriate calibration factors in HDM-4 pavement deterioration models facilitate reliable and rational prediction of pavement deterioration for the highway network under study. This, in turn, will help in the better assessment of the maintenance and rehabilitation requirements of pavements.

Calibration of HDM-4 is intended to improve the accuracy of predicted pavement performance. The pavement deterioration models incorporated in HDM-4 were developed from the results of large field experiments conducted in several countries. Consequently, the default (generic) equations in HDM-4 if used without calibration, would predict pavement performance that may not accurately match with that observed on specific road sections. Thus, calibration is necessary to ‘fine-tune’ the regression coefficients to predict more representative outputs in environments, other than the regions in which the models were developed (Bennett and Paterson, 2000). This chapter presents the calibration of HDM-4 pavement deterioration models for the selected 23 high-speed road corridor flexible pavement sections having wearing course constructed with modified binders, which were classified in six homogeneous groups based on climate and pavement composition.

5.2 NEED FOR CALIBRATION

The main objective of an HDM analysis is to model the performance of pavements with time. This entails predicting the deterioration of pavements under time and traffic, the road user effects, and the effects of maintenance on pavement condition and rate of deterioration. As with any model, HDM-4 is intended to be a representation of reality. How well the model predictions reflect reality is dependent upon a combination of several factors listed below (Bennett and Paterson, 2000):

- Validity of underlying HDM relationships
- Accuracy and adequacy of the input data
- Calibration factors used in the analysis

Since the underlying HDM-4 relationships have proven to be robust and applicable in a number of countries, the reliability of most HDM-4 analyses depends on the input data and the calibration factors. The only way of assessing the adequacy of the HDM deterioration models is by comparing the model predictions to known data. For example, one may have data on the current roughness of a number of pavements of known ages. By using the HDM-4 model to predict roughness of the pavement of the same age with same attributes as when they were new, one could assess whether the HDM models are giving appropriate predictions.

5.2.1Steps for Calibration

Calibration of the HDM-4 model involves the following three steps: (i) data input, (ii) calibration factors, and (iii) model coefficients (Bennett and Paterson, 2000).

5.2.1.1 Data input –A good quality input data is indispensable to the desired reliability of the results. The data should be accurate, reliable and appropriate. The accuracy of input data can have a substantial impact on the timing of future interventions, sometimes more important than the deterioration rate. This is because HDM-4 uses incremental models and the existing condition is the start point for modelling. Also, it is important to ensure consistency of data over time and between locations.

5.2.1.2 Calibration factors –The calibration factors are adjusted to enhance how well the forecast and outputs represent the changes in pavement performance over time and under various interventions. The calibration factors need to be adjusted to accurately reflect the rates of deterioration for specific road sections for particular types of pavements. For example, a section of a road in a hilly region may deteriorate at a different rate to a section of road in a plain area, even though the two sections may be nominally homogeneous in other aspects.

5.2.1.3 Model coefficients –Model coefficients are the default values already assigned in HDM-4 with respect to the environmental and climatic conditions. These assigned values are the input steps which are directly related to rate of deterioration for different types of pavement materials.

5.2.2 Levels of Calibration

There are three levels of calibration that can be used with HDM-4 models, which involve low, moderate, and major levels of effort and resources. This concept of increasing effort and increasing resources with levels of calibration is also illustrated through Figure 5.1. There is a direct relationship between the time and effort expended in setting up HDM-4 and reliability and accuracy of its output.

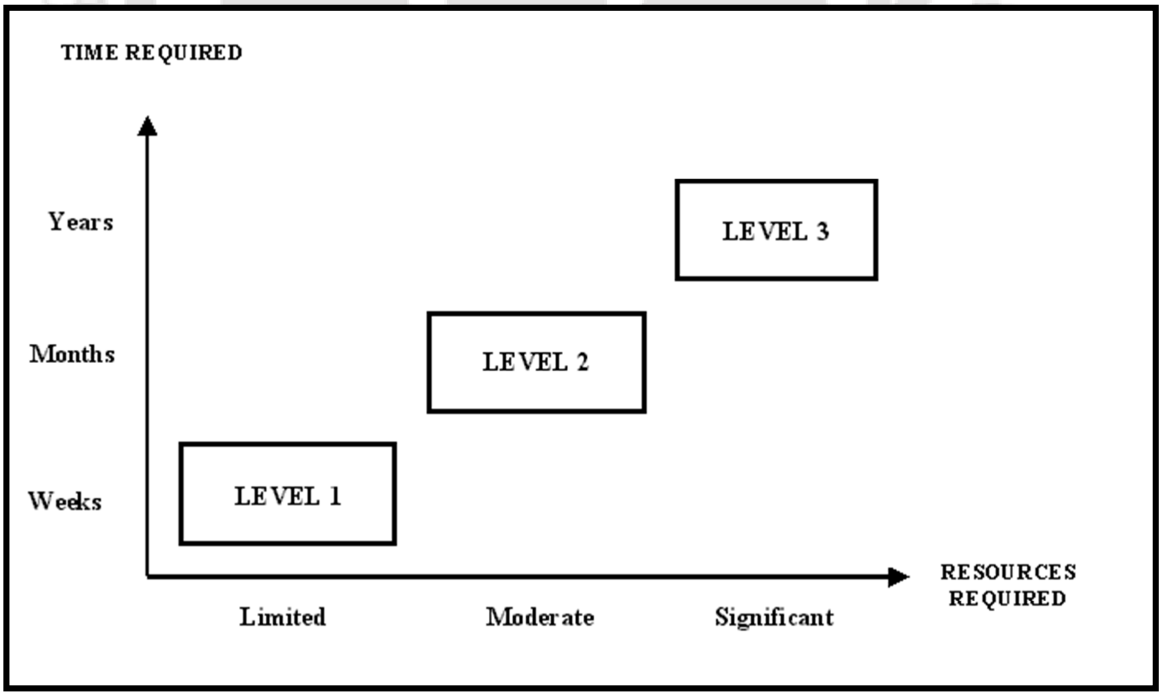


Figure 5.1 Levels of calibration in HDM-4

5.2.2.1 Level 1 –This is a basic application level of calibration under which the values of required basic input parameters are determined, many default values are adopted, and most sensitive parameters are calibrated with best estimates, or minimal field surveys. This level of calibration is largely based on secondary sources such as past studies and other government and industry publications.

5.2.2.2 Level 2–This level of calibration requires measurement of additional input parameters and field surveys to calibrate key predictive relationships to local conditions. This level uses direct measurements of local conditions to verify and adjust the predictive capability of the model. It requires a higher degree of precision and accuracy in time series data collection than the basic application level (Level 1) of calibration.

5.2.2.3 Level 3–Under this level of calibration, major field surveys and controlled experiments are undertaken to enhance/modify the existing predictive relationships or to develop new and locally specific relationships for substitution in the source code of the model. This type of calibration comprises of a highly extensive research, which consists of structured field surveys and experimental studies conducted for several years under local conditions, which lead to alternative relationships. Such work requires a major commitment to good-quality, well-structured field research, and statistical analysis over a period of several years.

5.2.3 Calibration Level of the Present Study

In India, mainly the Level 1 calibrations of HDM-4 have been performed by various research organizations and acdemic institutions. Only a few studies have attempted Level 2 calibration, and furthermore, these studies have mainly considered state highways and other lower category roads with limited road sections.

HDM-4 is a robust and widely accepted pavement management tool and its use has been increasing in developing countries, including India. Further, use of modified binders is becoming more and more popular on high-speed road corridor networks being developed in India. In this study, twenty three (23) in-service flexible pavement road sections having modified bitumen in wearing course were identified from various high-speed road corridors developed under NHDP. These sections were located in different climatic and environmental zones of the country. Complete information about these sections in terms of pavement layer composition, design and traffic data were collected from the respective organizations, extensive field studies, and laboratory examinations. These sections were continuously monitored for 3 years (2011 to 2013) for the time series pavement surface condition data to carry out a Level 2 study.The collected time series data included pavement distress data, traffic and axle load data, pavement crust composition data, pavement material characterisation data, temperature and rainfall data, and maintenance history data. The data were then used as inputs in HDM-4 for calibration of HDM-4 distress models. Calibration was performed separately for each of the six groups to which the 23 pavement sections were divided based on pavement composition and climatic conditions.

5.3 METHODOLOGY ADOPTED FOR CALIBRATION

5.3.1 Determination of Distress Initiation Calibration Factors

For calibration of surface distress initiation factors, the coefficient between the observed year of occurrence of the distress with respect to the year of occurrence as predicted by the un-calibrated models was obtained using Equation 5.1:

$$K_{ci} = \frac{\text{mean OTCI}}{\text{mean PTCI}} \tag{5.1}$$

where, K_{ci}= distress (crack) initiation calibration factor

OTCI = observed time to crack initiation

PTCI = predicted time to crack initiation

Based on this methodology, the initiation calibration factors for all the six groups comprising 23 pavement sections were determined for the following distresses:

- Cracking initiation (K_{cia})
- Ravelling initiation (K_{vi})
- Pothole initiation (K_{pi})

5.3.2 Determination of Distress Progression Calibration Factors

The procedure adopted for determination of calibration factors for distress progression is given as under:

i) HDM-4 was run for each homogeneous group of test sections with the road network and vehicle fleet input data. The range of calibration factors as suggested by HDM-4 varies from 0 to 20 with a default value of 1. As also reviewed from some past studies, the calibration factors generally vary from zero to five in case of developing nations, including India (Jain et al., 2005). Hence, in the first stage, HDM-4 was run for calibration factors varying from 0 to 5 with an increment of 0.10.

ii) Calibration factors were then determined from the results of first run corresponding to minimum values of statistical parameters indicating the error between predicted and observed distresses of 2012 time series data. After getting the calibration factors from first run, the HDM-4 was run for the second stage by taking the calibration factors within the closer range of factors with an increment of 0.01, as determined from first run for further refinement of the calibration factors. The statistical parameters considered for calibration in the study are described further.

Root Mean Square Error (RMSE): RMSE is a frequently used error measure that finds out the difference between values predicted by a model and the values actually observed in the field. It is also called root mean square deviation. The smaller the error, the better is the predicting ability of that model according to the RMSE criterion. The RMSE is calculated as:

$$\mathrm{RMSE} = \sqrt{\frac{\sum_{i=1}^{n}(O_i - P_i)^2}{n}} \tag{5.2}$$

where, O_i = observed value of distress observation *i*

P_i = predicted value of distress observation *i*

n = No. of observations

Mean Absolute Error (MAE): The MAE is another useful measure widely used in model evaluations. It measures the average magnitude of the errors in a set of predictions, without considering their direction. It is the average over the test sample of the absolute differences between prediction and actual observation where all individual differences have equal weight. The smaller the error, the better is the predicting ability of the model. The MAE is defined as:

$$MAE = \frac{1}{n}\sum_{i=1}^{n}\left|O_i - P_i\right| \tag{5.3}$$

RMSE to Observations' Standard Deviation Ratio (RSR): The root mean square error to the observation's standard deviation ratio (RSR) standardizes root mean square error (RMSE) by incorporating both an error index and the standard deviation of the actual or observed data. RSR varies from the optimal value of 0, which indicates zero RMSE or residual variation and therefore perfect model simulation, to a large positive value. Hence, a lower value of RSR, the lower the RMSE, and the better the model simulation performance. The RSR is defined as:

$$\mathrm{RSR} = \frac{\mathrm{RMSE}}{\mathrm{STDEV_{actual}}} = \frac{\sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(O_i - P_i\right)^2}}{\sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(O_i - O_{avg}\right)^2}} \tag{5.4}$$

where, O_{avg}= mean value of distress observations

Nash-Sutcliffe Efficiency (NSE): The Nash-Sutcliffe efficiency (NSE) is a normalized statistic that determines the relative magnitude of the residual variance compared to the measured data variance. NSE ranges from -∞ to 1.0 (1 inclusive), with NSE = 1 being the optimal value. The value close to +1 indicates a close agreement between observed and predicted data. The NSE is defined as:

$$E = 1 - \frac{\sum_{i=1}^{n}\left(O_i - P_i\right)^2}{\sum_{i=1}^{n}\left(O_i - O_{avg}\right)^2} \tag{5.5}$$

Pearson's Correlation Coefficient (R): Pearson's correlation coefficient (R) describes the degree of collinearity between measured and predicted data. The correlation coefficient, which ranges from -1 to 1, is an index of the degree of linear relationship between observed and predicted data. The value of R= 1 or -1 indicates a perfect positive or negative linear relationship, and R = 0 indicates no linear relationship. The correlation coefficient (R) is defined as:

$$R = \frac{\sum_{i=1}^{n}\left(O_i - O_{avg}\right)\left(P_i - P_{avg}\right)}{\sqrt{\sum_{i=1}^{n}\left(O_i - O_{avg}\right)^2 \sum_{i=1}^{n}\left(P_i - P_{avg}\right)^2}} \tag{5.6}$$

where, P_{avg} = mean value of predicted distress

Based on the above methodology, the calibration factors for all the six groups of homogeneous pavement sections were determined for the following distress progression models:

- Cracking Progression (K_{cpa})
- Ravelling Progression (K_{vp})
- Pothole Progression (K_{pp})
- Rut Depth Progression (K_{rst})
- Roughness Progression (K_{gp})
- Texture Depth Progression (K_{td})
- Skid Resistance Progression (K_{sfc})

5.4 RESULTS OF CALIBRATION FACTORS

The HDM-4 calibration factors of each mode of distress were determined for all the twenty-three (23) pavement sections classified under six (6) individual homogeneous groups. The process of obtaining final value of calibration factors for the progression of each distress is illustrated in Tables 5.1 to 5.7 for homogeneous Group-2 having the maximum number of sections (6 sections). Calibration factors for other groups were also determined using the same approach, and the final values are shown in Table 5.8. The best values of all the statistical indicators RMSE, MAE, RSR, NSE, and R are considered to arrive at the final values of calibration factors for all the distress progression models. The values of statistical indicators RMSE, MAE, RSR, NSE, and R obtained for respective calibration factors for different distress progression models for Group-2 homogeneous road sections are also given in Tables 5.1 to 5.7. Final calibration factors and the corresponding values of statistical measures are shown in boldface in Tables 5.1 to 5.7. The calibration factors of progression of each mode of distress determined for respective individual groups Gr.1, Gr.2, Gr.3, Gr.4, Gr.5, and Gr.6 are summarised in Table 5.8.

Table 5.1 Calibration process of HDM-4 cracking progression model for sections ofGroup-2

Calibration Factor	Section-1		Section-2		Section-3		Section-4		Section-5		Section-6		RMSE	MAE	RSR	NSE	R
	NH-2UP [607-606]		NH-2UP [717-718]		NH-5AP [691.25-690.75]		NH-5AP [698.6-698.1]		NH-5AP [7.5-7.0]		NH-5AP [9.8-9.3]						
	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)					
0.30	10.00	9.00	3.00	2.00	2.16	2.50	3.34	1.90	15.00	14.00	16.00	14.00	1.24	1.13	0.2	0.95	0.99
0.40	10.00	10.00	3.00	2.10	2.16	2.50	3.34	1.90	15.00	14.00	16.00	14.00	1.15	0.95	0.18	0.96	0.99
0.50	10.00	10.00	3.00	2.20	2.16	2.50	3.34	1.90	15.00	14.00	16.00	14.00	1.14	0.93	0.18	0.96	0.99
0.55	10.00	10.00	3.00	2.20	2.16	2.50	3.34	1.90	15.00	14.00	16.00	14.00	1.14	0.93	0.18	0.96	0.99
0.60	10.00	10.00	3.00	2.50	2.16	2.70	3.34	1.90	15.00	14.00	16.00	14.00	1.13	0.91	0.18	0.96	0.99
0.61	10.00	10.30	3.00	2.50	2.16	2.70	3.34	2.00	15.00	14.00	16.00	14.00	1.11	0.95	0.18	0.96	0.99
0.62	10.00	10.50	3.00	2.50	2.16	2.70	3.34	2.00	15.00	14.00	16.00	15.00	0.87	0.81	0.14	0.98	0.99
0.63	10.00	10.50	3.00	2.50	2.16	2.70	3.34	2.00	15.00	14.00	16.00	15.00	0.87	0.81	0.14	0.98	0.99
0.64	**10.00**	**10.50**	**3.00**	**2.50**	**2.16**	**3.00**	**3.34**	**2.50**	**15.00**	**15.00**	**16.00**	**16.00**	**0.56**	**0.45**	**0.09**	**0.99**	**1.00**
0.65	10.00	11.00	3.00	2.50	2.16	3.00	3.34	2.50	15.00	15.00	16.00	15.00	0.78	0.7	0.12	0.98	0.99
0.66	10.00	11.00	3.00	2.50	2.16	3.00	3.34	2.50	15.00	15.00	16.00	15.00	0.78	0.7	0.12	0.98	0.99
0.67	10.00	12.00	3.00	2.50	2.16	3.00	3.34	3.00	15.00	15.00	16.00	15.00	1.01	0.78	0.16	0.97	0.98
0.68	10.00	12.00	3.00	2.50	2.16	3.00	3.34	3.00	15.00	15.50	16.00	15.00	1.03	0.86	0.16	0.97	0.99
0.69	10.00	12.00	3.00	2.50	2.16	3.00	3.34	3.00	15.00	16.20	16.00	16.00	1.04	0.81	0.17	0.97	0.99
0.70	10.00	12.50	3.00	3.00	2.16	3.00	3.34	3.50	15.00	17.00	16.00	17.00	1.41	1.08	0.22	0.94	0.99
0.80	10.00	13.00	3.00	4.00	2.16	4.00	3.34	3.50	15.00	17.00	16.00	17.00	1.75	1.5	0.28	0.91	0.99
0.90	10.00	14.00	3.00	5.00	2.16	4.00	3.34	3.50	15.00	18.00	16.00	18.00	2.46	2.17	0.39	0.82	0.99

Table 5.2 Calibration process of HDM-4 ravelling progression model for sections of Group-2

Calibration Factor	Section-1		Section-2		Section-3		Section-4		Section-5		Section-6		RMSE	MAE	RSR	NSE	R
	NH-2UP [607-606]		NH-2UP [717-718]		NH-5AP [691.25-690.75]		NH-5AP [698.6-698.1]		NH-5AP [7.5-7.0]		NH-5AP [9.8-9.3]						
	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)					
0.10	7.00	4.00	3.00	2.00	0.00	0.00	0.00	0.00	10.0	9.0	12.0	10.0	1.58	1.17	0.31	0.89	0.98
0.12	7.00	5.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	9.0	12.0	11.0	1.00	0.67	0.2	0.95	0.99
0.14	7.00	5.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	10.0	12.0	11.0	0.91	0.5	0.18	0.96	0.99
0.16	7.00	5.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	10.0	12.0	11.0	0.91	0.5	0.18	0.96	0.99
0.18	7.00	5.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	10.0	12.0	11.0	0.91	0.5	0.18	0.96	0.99
0.20	7.00	5.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	10.0	12.0	11.0	0.91	0.5	0.18	0.96	0.99
0.21	7.00	6.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	10.0	12.0	11.0	0.58	0.33	0.11	0.98	1.00
0.22	7.00	6.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	10.0	12.0	11.0	0.58	0.33	0.11	0.98	1.00
0.23	**7.00**	**6.00**	**3.00**	**3.00**	**0.00**	**0.00**	**0.00**	**0.00**	**10.0**	**10.0**	**12.0**	**12.0**	**0.41**	**0.17**	**0.08**	**0.99**	**1.00**
0.24	7.00	6.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	11.0	12.0	12.0	0.58	0.33	0.11	0.98	0.99
0.25	7.00	6.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	11.0	12.0	12.0	0.58	0.33	0.11	0.98	0.99
0.26	7.00	6.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	11.0	12.0	12.0	0.58	0.33	0.11	0.98	0.99
0.27	7.00	6.00	3.00	3.00	0.00	0.00	0.00	0.00	10.0	11.0	12.0	12.0	0.58	0.33	0.11	0.98	0.99
0.28	7.00	6.00	3.00	4.00	0.00	0.00	0.00	0.00	10.0	11.0	12.0	13.0	0.82	0.67	0.16	0.97	0.99
0.29	7.00	7.00	3.00	4.00	0.00	0.00	0.00	0.00	10.0	12.0	12.0	13.0	1.00	0.67	0.2	0.95	0.99
0.30	7.00	7.00	3.00	5.00	0.00	0.00	0.00	0.00	10.0	13.0	12.0	14.0	1.68	1.17	0.33	0.87	0.99
0.40	7.00	8.00	3.00	5.00	0.00	0.00	0.00	0.00	10.0	14.0	12.0	15.0	2.24	1.67	0.44	0.77	0.99

Table 5.3 Calibration process of HDM-4 rutting progression model for sections of Group-2

	Section-1		Section-2		Section-3		Section-4		Section-5		Section-6						
	NH-2UP [607-606]		NH-2UP [717-718]		NH-5AP [691.25-690.75]		NH-5AP [698.6-698.1]		NH-5AP [7.5-7.0]		NH-5AP [9.8-9.3]						
Calibration Factor	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	RMSE	MAE	RSR	NSE	R
0.60	3.50	2.50	4.20	2.80	2.70	1.70	3.40	2.70	2.70	1.80	2.50	1.80	1.02	0.98	1.81	-2.92	0.87
0.70	3.50	2.60	4.20	3.00	2.70	1.75	3.40	2.70	2.70	1.80	3.00	1.90	0.93	0.91	1.66	-2.3	0.91
0.71	3.50	2.70	4.20	3.30	2.70	1.80	3.40	2.80	2.70	1.90	3.00	1.90	0.82	0.8	1.47	-1.58	0.94
0.72	3.50	2.90	4.20	4.00	2.70	1.90	3.40	2.80	2.70	2.00	3.00	2.00	0.66	0.6	1.18	-0.66	0.97
0.73	3.50	3.00	4.20	4.60	2.70	1.90	3.40	2.90	2.70	2.10	3.00	2.10	0.61	0.57	1.09	-0.43	0.97
0.74	3.50	3.10	4.20	4.90	2.70	2.00	3.40	3.00	2.70	2.10	3.00	2.10	0.62	0.57	1.11	-0.47	0.97
0.75	3.50	3.20	4.20	5.00	2.70	2.50	3.40	3.00	2.70	2.10	3.00	2.10	0.58	0.48	1.03	-0.27	0.94
0.76	3.50	3.20	4.20	5.10	2.70	2.50	3.40	3.00	2.70	2.10	3.00	2.20	0.58	0.48	1.03	-0.27	0.95
0.77	**3.50**	**3.20**	**4.20**	**5.10**	**2.70**	**2.50**	**3.40**	**3.00**	**2.70**	**2.20**	**3.00**	**2.20**	**0.56**	**0.47**	**1.00**	**-0.2**	**0.95**
0.78	3.50	3.20	4.20	5.20	2.70	2.50	3.40	3.00	2.70	2.20	3.00	2.20	0.59	0.48	1.05	-0.32	0.95
0.79	3.50	3.20	4.20	5.30	2.70	2.50	3.40	3.00	2.70	2.20	3.00	2.20	0.62	0.5	1.10	-0.46	0.95
0.80	3.50	3.30	4.20	5.41	2.70	2.60	3.40	3.00	2.70	2.20	3.00	2.20	0.65	0.52	1.16	-0.6	0.94
0.81	3.50	3.30	4.20	5.41	2.70	2.50	3.40	3.00	2.70	2.20	3.00	2.20	0.65	0.53	1.16	-0.62	0.94
0.82	3.50	3.30	4.20	5.50	2.70	2.60	3.40	3.00	2.70	2.20	3.00	2.20	0.68	0.53	1.21	-0.75	0.94
0.83	3.50	3.30	4.20	5.60	2.70	2.60	3.40	3.00	2.70	2.20	3.00	2.20	0.71	0.55	1.26	-0.92	0.94
0.84	3.50	3.30	4.20	5.60	2.70	2.60	3.40	3.00	2.70	2.20	3.00	2.20	0.71	0.55	1.26	-0.92	0.94
0.85	3.50	3.30	4.20	5.65	2.70	2.70	3.40	3.00	2.70	2.20	3.00	2.20	0.73	0.54	1.29	-1	0.93

Table 5.4 Calibration process of HDM-4 roughness progression model for sections of Group-2

Calibration Factor	Section-1		Section-2		Section-3		Section-4		Section-5		Section-6		RMSE	MAE	RSR	NSE	R
	NH-2UP [607-606]		NH-2UP [717-718]		NH-5AP [691.25-690.75]		NH-5AP [698.6-698.1]		NH-5AP [7.5-7.0]		NH-5AP [9.8-9.3]						
	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)					
0.10	2.60	2.58	3.10	2.99	2.70	2.58	2.90	2.80	3.00	2.99	3.00	2.99	0.08	0.06	0.41	0.8	0.96
0.14	2.60	2.59	3.10	3.01	2.70	2.58	2.90	2.85	3.00	2.99	3.00	3.00	0.06	0.05	0.33	0.87	0.97
0.15	2.60	2.59	3.10	3.04	2.70	2.58	2.90	2.85	3.00	2.99	3.00	3.00	0.06	0.04	0.3	0.89	0.98
0.16	2.60	2.60	3.10	3.06	2.70	2.59	2.90	2.85	3.00	3.00	3.00	3.01	0.05	0.03	0.27	0.91	0.98
0.17	2.60	2.60	3.10	3.08	2.70	2.59	2.90	2.85	3.00	3.00	3.00	3.01	0.05	0.03	0.26	0.92	0.98
0.18	**2.60**	**2.60**	**3.10**	**3.11**	**2.70**	**2.59**	**2.90**	**2.89**	**3.00**	**3.00**	**3.00**	**3.01**	**0.05**	**0.02**	**0.23**	**0.93**	**0.98**
0.19	2.60	2.60	3.10	3.13	2.70	2.59	2.90	2.93	3.00	3.00	3.00	3.02	0.05	0.03	0.25	0.92	0.98
0.20	2.60	2.60	3.10	3.15	2.70	2.60	2.90	2.95	3.00	3.01	3.00	3.02	0.05	0.04	0.26	0.92	0.98
0.21	2.60	2.61	3.10	3.18	2.70	2.60	2.90	2.95	3.00	3.01	3.00	3.02	0.06	0.05	0.29	0.9	0.98
0.22	2.60	2.61	3.10	3.20	2.70	2.60	2.90	2.95	3.00	3.01	3.00	3.03	0.06	0.05	0.32	0.87	0.98
0.23	2.60	2.61	3.10	3.22	2.70	2.60	2.90	2.95	3.00	3.01	3.00	3.03	0.07	0.05	0.35	0.85	0.98
0.24	2.60	2.61	3.10	3.24	2.70	2.61	2.90	2.97	3.00	3.01	3.00	3.03	0.07	0.06	0.39	0.82	0.97
0.25	2.60	2.62	3.10	3.27	2.70	2.61	2.90	2.97	3.00	3.02	3.00	3.04	0.09	0.07	0.44	0.76	0.97
0.30	2.60	2.62	3.10	3.29	2.70	2.61	2.90	2.97	3.00	3.02	3.00	3.04	0.09	0.07	0.48	0.73	0.97

Table 5.5 Calibration process of HDM-4 texture depth progression model for sections of Group-2

Calibration Factor	Section-1		Section-2		Section-3		Section-4		Section-5		Section-6		RMSE	MAE	RSR	NSE	R
	NH-2UP [607-606]		NH-2UP [717-718]		NH-5AP [691.25-690.75]		NH-5AP [698.6-698.1]		NH-5AP [7.5-7.0]		NH-5AP [9.8-9.3]						
	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)					
0.15	0.73	0.79	0.60	0.50	0.50	0.40	0.55	0.60	0.70	0.70	0.64	0.70	0.07	0.06	0.8	0.23	0.89
0.16	0.73	0.78	0.60	0.50	0.50	0.40	0.55	0.60	0.70	0.70	0.64	0.70	0.07	0.06	0.79	0.26	0.89
0.17	0.73	0.76	0.60	0.50	0.50	0.40	0.55	0.60	0.70	0.69	0.64	0.69	0.07	0.06	0.75	0.33	0.88
0.18	0.73	0.75	0.60	0.50	0.50	0.40	0.55	0.58	0.70	0.69	0.64	0.69	0.06	0.05	0.72	0.38	0.90
0.19	0.73	0.74	0.60	0.50	0.50	0.40	0.55	0.58	0.70	0.69	0.64	0.69	0.06	0.05	0.71	0.39	0.91
0.20	0.73	0.73	0.60	0.50	0.50	0.40	0.55	0.55	0.70	0.69	0.64	0.69	0.06	0.04	0.7	0.41	0.92
0.21	**0.73**	**0.71**	**0.60**	**0.50**	**0.50**	**0.40**	**0.55**	**0.55**	**0.70**	**0.68**	**0.64**	**0.65**	**0.06**	**0.04**	**0.67**	**0.46**	**0.93**
0.22	0.73	0.70	0.60	0.50	0.50	0.40	0.55	0.55	0.70	0.68	0.64	0.66	0.06	0.04	0.68	0.44	0.92
0.23	0.73	0.69	0.60	0.50	0.50	0.40	0.55	0.55	0.70	0.68	0.64	0.68	0.06	0.05	0.71	0.39	0.92
0.24	0.73	0.68	0.60	0.50	0.50	0.40	0.55	0.55	0.70	0.68	0.64	0.68	0.06	0.05	0.73	0.37	0.89
0.25	0.73	0.66	0.60	0.50	0.50	0.40	0.55	0.55	0.70	0.67	0.64	0.68	0.07	0.06	0.77	0.29	0.87
0.30	0.73	0.65	0.60	0.50	0.50	0.40	0.55	0.50	0.70	0.67	0.64	0.67	0.07	0.06	0.81	0.2	0.91
0.50	0.73	0.64	0.60	0.50	0.50	0.40	0.55	0.50	0.70	0.67	0.64	0.67	0.07	0.07	0.84	0.16	0.90

Table 5.6 Calibration process of HDM-4 skid resistance progression model for sections of Group-2

	Section-1		Section-2		Section-3		Section-4		Section-5		Section-6						
	NH-2UP [607-606]		NH-2UP [717-718]		NH-5AP [691.25-690.75]		NH-5AP [698.6-698.1]		NH-5AP [7.5-7.0]		NH-5AP [9.8-9.3]						
Calibration Factor	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	RMSE	MAE	RSR	NSE	R
0.50	0.72	0.65	0.60	0.55	0.68	0.60	0.67	0.60	0.75	0.60	0.72	0.77	0.09	0.08	1.61	-2.12	0.43
0.75	0.72	0.66	0.60	0.55	0.68	0.60	0.67	0.60	0.75	0.60	0.72	0.77	0.08	0.08	1.59	-1.61	0.47
0.80	0.72	0.67	0.60	0.55	0.68	0.65	0.67	0.61	0.75	0.60	0.72	0.75	0.08	0.06	1.47	-1.56	0.49
0.90	0.72	0.68	0.60	0.55	0.68	0.67	0.67	0.62	0.75	0.58	0.72	0.75	0.09	0.06	1.46	-2.12	0.55
0.91	0.72	0.69	0.60	0.54	0.68	0.68	0.67	0.63	0.75	0.58	0.72	0.75	0.08	0.06	1.45	-2.03	0.56
0.92	**0.72**	**0.70**	**0.60**	**0.54**	**0.68**	**0.69**	**0.67**	**0.65**	**0.75**	**0.58**	**0.72**	**0.70**	**0.07**	**0.05**	**1.4**	**-1.35**	**0.57**
0.93	0.72	0.65	0.60	0.54	0.68	0.70	0.67	0.66	0.75	0.57	0.72	0.70	0.08	0.06	1.47	-1.61	0.47
0.94	0.72	0.65	0.60	0.54	0.68	0.71	0.67	0.68	0.75	0.57	0.72	0.70	0.08	0.06	1.46	-1.56	0.49
0.95	0.72	0.65	0.60	0.54	0.68	0.72	0.67	0.69	0.75	0.57	0.72	0.65	0.08	0.07	1.42	-1.41	0.46
1.00	0.72	0.65	0.60	0.54	0.68	0.72	0.67	0.72	0.75	0.57	0.72	0.68	0.08	0.07	1.58	-1.99	0.33
1.10	0.72	0.64	0.60	0.54	0.68	0.73	0.67	0.77	0.75	0.55	0.72	0.67	0.08	0.09	1.59	-2.02	0.29
1.20	0.72	0.64	0.60	0.54	0.68	0.73	0.67	0.59	0.75	0.55	0.72	0.65	0.09	0.09	1.69	-2.41	0.24
1.30	0.72	0.64	0.60	0.54	0.68	0.74	0.67	0.59	0.75	0.55	0.72	0.60	0.09	0.1	1.67	-2.33	0.19
1.40	0.72	0.64	0.60	0.54	0.68	0.75	0.67	0.59	0.75	0.50	0.72	0.59	0.1	0.11	1.97	-3.64	0.04
1.50	0.72	0.64	0.60	0.53	0.68	0.75	0.67	0.59	0.75	0.50	0.72	0.59	0.1	0.11	1.95	-3.56	0.21

Table 5.7 Calibration process of HDM-4 pothole progression model for sections of Group-2

Calibration Factor	Section-1		Section-2		Section-3		Section-4		Section-5		Section-6		RMSE	MAE	RSR	NSE	R
	NH-2UP [607-606]		NH-2UP [717-718]		NH-5AP [691.25-690.75]		NH-5AP [698.6-698.1]		NH-5AP [7.5-7.0]		NH-5AP [9.8-9.3]						
	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)	Ob. Value (O)	Pr. Value (P)					
0.010	**2.00**	**2.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**3.00**	**2.00**	**3.00**	**2.00**	**0.58**	**0.33**	**0.38**	**0.82**	**0.97**
0.015	2.00	3.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	2.00	3.00	2.00	0.71	0.50	0.47	0.74	0.87
0.020	2.00	3.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	2.00	3.00	2.00	0.71	0.50	0.47	0.74	0.87
0.030	2.00	3.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	5.00	3.00	3.00	0.91	0.50	0.61	0.56	0.95
0.040	2.00	4.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	5.00	3.00	3.00	1.15	0.67	0.77	0.29	0.93
0.050	2.00	5.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	2.00	3.00	5.00	1.53	1.00	1.01	-0.24	0.81
0.060	2.00	5.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	6.00	3.00	6.00	2.12	1.50	1.41	-1.38	0.99

Table 5.8 Calibration factors obtained for HDM-4 deterioration models for all groups

Model Description	Calibration Factors					
	Group-1	Group-2	Group-3	Group-4	Group-5	Group-6
Cracking Initiation (K_{cia})	1.69	1.50	1.20	1.18	0.80	0.89
Cracking Progression (K_{cpa})	0.47	0.64	0.20	0.70	0.79	0.84
Ravelling Initiation (K_{vi})	2.07	2.00	2.82	1.00	1.71	1.60
Ravelling Progression (K_{vp})	1.03	0.23	0.27	0.35	0.69	0.19
Pothole Initiation (K_{pi})	1.00	1.00	1.20	1.00	1.00	1.00
Pothole Progression (K_{pp})	1.00	0.01	0.67	0.67	0.98	1.00
Rut Depth Progression (K_{rst})	1.61	0.77	0.63	0.97	0.51	0.73
Roughness Progression (K_{gp})	2.65	0.18	1.57	0.62	1.01	1.17
Texture Depth Progression (K_{td})	0.86	0.21	0.10	0.07	1.12	0.20
Skid Resistance Progression (K_{sfc})	1.00	0.92	1.33	2.83	3.40	3.33

5.5 DISCUSSION ON CALIBRATION RESULTS

Based on the calibration factors of inbuilt HDM-4 road deterioration models determined for each group (Table 5.8), the following interpretations can be drawn:

- In the case of four test sections in Group-1, the initiation of pavement surface cracking (K_{cia} =1.69) and ravelling (K_{vi} =2.07) are observed later than HDM-4 models. The crack initiation begins 1.69 times slower, and ravelling initiation 2.07 times slower than the prediction made by HDM-4 models. The pothole initiation (K_{pi}) factor is found to be 1.0; no potholes were observed in these pavement sections during the study. The rate of progression of cracking(K_{cpa} =0.47) and texture depth (K_{td} =0.86) is slower by 53and 14% respectively than those predicted byHDM-4 models (with default calibration factor value of 1.0). The progression of ravelling (K_{vp} =1.03), rut depth (K_{rst} =1.61) and roughness (K_{gp} =2.65) is faster by 3, 61, and 165% respectively. However, the rates of progression of potholes (K_{pp} =1.0) and skid resistance (K_{sfc} =1.0) are found to be the same as the ones predicted by HDM-4 with default value of 1.
- In the case of six test sections in Group-2, the initiation of pavement surface cracking (K_{cia}=1.50) and ravelling (K_{vi} =2.0) are observed later than the prediction made by theHDM-4 models. The crack initiation begins 1.5 times slower and ravelling initiation 2.0times slower than the prediction made by HDM-4 models. The pothole initiation factor (K_{pi}) was found to be 1.0.The rate of progression of cracking (K_{cpa} = 0.64), ravelling (K_{vp}=0.23), roughness (K_{gp}=0.18), rut depth (K_{rst}=0.77),texture depth (K_{td}=0.21) and skid resistance (K_{sfc}=0.92) is slower by 36, 77,82, 23, 79 and 8% respectively than those predicted by HDM-4 models having default values of 1.0.
- In the case of three test sections in Group-3, the initiation of pavement surface cracking (K_{cia}=1.20),ravelling (K_{vi}=2.82) and pothole (K_{pi} =1.20) are observed later that prediction made by the HDM-4 models. The crack initiation begins 1.20 times slower, ravelling initiation 2.82 times slower, and pothole initiation 1.20 times slower than the prediction made by HDM-4 models. The rate of progression of cracking (K_{cpa}=0.20), ravelling (K_{vp}=0.27), rut depth (K_{rst}=0.63), pothole (K_{pp}=0.67) and texture depth (K_{td}=0.10) is slower by 80, 73, 37, 33 and 90% respectively, than those predicted by HDM-4 models. However, the rate of progression of

roughness (K_{gp} =1.57) and skid resistance (K_{sfc}=1.33) is faster by 57 and 33% respectively.

- In the case of three test sections in Group-4, the initiation of pavement surface cracking (K_{cia}=1.18) is observed later than that prediction made by theHDM-4 models. The crack initiation begins 1.18 times slower than the prediction made by HDM-4 models. However, the initiation of pavement surface ravelling(K_{vi}=1.0) and pothole (K_{pi}=1.0) were observed as same as that prediction made by theHDM-4 models. The rate of progression of cracking(K_{cpa} =0.70), ravelling (K_{vp}=0.35), rut depth (K_{rst} =0.97), pothole (K_{pp} =0.67), roughness (K_{gp} =0.62) and texture depth(K_{td} =0.07) is slower by 30, 65,3, 33, 38 and 93% respectively than those predicted by HDM-4 models with default values of calibration factors.
- In the case of five test sections in Group-5, the initiation of pavement surface cracking (K_{cia}=0.80) is observed earlier on the pavement surface, and ravelling(K_{vi}=1.71) observed later than that prediction made by the HDM-4 models. The crack initiation begins 0.80 times faster and the ravelling 1.71 times slower than the prediction made by HDM-4 models. However, the initiation of pothole (K_{pi}=1.0) on the pavement surface were observed as same as that prediction made by the HDM-4 models. The rate of progression of cracking(K_{cpa}=0.79), ravelling (K_{vp}=0.69), rut depth(K_{rst}=0.51) and pothole (K_{pp} =0.98) is slower by 21, 31, 49 and 2% respectively, than those predicted by HDM-4 models. However, the rates of progression of roughness(K_{gp} =1.01),and texture depth (K_{td} = 1.12) are faster by 1% and 12% respectively.
- In the case of two test sections in Group-6, the initiation of pavement surface cracking (K_{cia}= 0.89) is observed earlier, and ravelling (K_{vi} =1.60) observed later than the prediction made by the HDM-4 models. The crack initiation begins 1.12 times faster and ravelling 1.60 times slower than the prediction made by HDM-4 models. The rate of progression of cracking (K_{cpa}=0.84), ravelling (K_{vp}=0.19), rut depth (K_{rst}=0.73) and texture depth (K_{td} =0.20) is slower by 16, 81, 27 and 80% respectively, than those predicted by HDM-4 models. However, the rate of progression of roughness (K_{gp} =1.17) and skid resistance (K_{sfc} =3.33) is faster by 17 and 233% respectively.

- Cracking, ravelling, and potholes initiation was found to be later than that predicted by HDM-4 models, except in case of Groups 5 and 6, where cracking initiation was earlier. Sections in Group 5 and Group 6 had minimum bituminous mix layer thickness as compared to the sections of other groups. Presuming that the pavement design has been performed accurately with respect to traffic load and other design variables, the construction quality also dominates the deterioration process which varies a lot at site during the execution of construction. Progression of cracking, ravelling, and potholes with calibrated HDM-4 models were slower than those with default HDM-4 models.

5.6 SUMMARY

This chapter described the detailed methodology adopted for calibration of HDM-4 deterioration models for the selected 23 test sections of high-speed road corridor network of India. These 23 sections were classified into six homogeneous groups based on similarity in pavement crust composition and climatic conditions. The need of calibration, its different steps and levels, and the various important HDM-4 calibration parameters were discussed. Calibration factors were determined for the pavement deterioration models of different distresses inbuilt in HDM-4 for each of the six groups. Calibration factors for initiation of cracking, raveling, and potholes were determined, which was followed by determination of calibration factors for progression of the following distresses: cracking, ravelling, pothole, rut depth, roughness, texture depth, and skid resistance. To arrive at the final values of the calibration factors, various statistical parameters indicating the measure of error between predicted and observed distressed were used. The statistical parameters employed were: RMSE, MAE, RSR, NSE, and R. Based on the values of obtained for calibration factors; interpretations were drawn with respect to delayed or advanced initiation or progression of pavement distresses compared to HDM-4 models with default values.

Chapter 6

VALIDATION OF HDM-4 DETERIORATION MODELS

6.1 GENERAL

Validation of the calibrated HDM-4 pavement deterioration models is important to check the adequacy of the calibration factors. Validation is needed before the model is put to use for future applications. Therefore, the validity of the calibrated pavement deterioration models was checked to test their efficacy. Validation was performed by comparing the distress predictions made by the calibrated deterioration models with those actually observed on the selected pavement sections.

The time series pavement condition data for all selected 23 test sections of the high-speed road corridor network in India were collected consecutively for the years 2011, 2012, and 2013. The time series condition data collected in the first two years(2011 and 2012)for all the test sections classified into six homogeneous groups were used for calibration of the deterioration models, and the data collected in the year 2013were used for validating these models. The seven deterioration models considered for validation are:

- Cracking progression model
- Ravelling progression model
- Pothole progression model
- Roughness progression model
- Rut depth progression model
- Texture depth progression model
- Skid resistance progression model

6.2 VALIDATION OF DISTRESS MODELS FOR DIFFERENT GROUPS

The observed values of distresses at the end of the year 2013 for pavement sections under different groups are compared with those predicted by HDM-4 model. The comparison is made by using a parameter termed percent variability, defined by Equation 6.1:

$$\text{Percent variability} = \left|\frac{\text{Observed value} - \text{Predicted value}}{\text{Observed value}}\right| \times 100 \qquad (6.1)$$

The observed and predicted distresses are also compared in terms of residual, calculated as per Equation 6.2:

$$\text{Residual} = \text{Observed value} - \text{Predicted value} \qquad (6.2)$$

The residual is a useful parameter to observe the magnitude of difference between the observed and predicted values, especially when percent variability may be difficult to define in the case where the value of an observed distress is zero. Positive residuals indicate under-prediction, and the negative ones indicate over-prediction by the model. The observed and predicted distress values are also plotted together with line of equality (LoE).

6.2.1 Validation for Sections in Group-1

Cracking Progression

The observed values of cracking area at the end of the year 2013 for four pavement sections of Group-1are compared with those predicted by HDM-4 cracking progression model. The observed and predicted values of cracking area for sections of Group1 (abbreviated as Gr-1, *etc.*) at the end of the year 2013 and their variations are given in Table 6.1. These values along with the LoE are plotted in Figure 6.1.

Table 6.1 Variability between observed and predicted cracking, Gr-1

Group-1 Pavement Sections	Cracking Area (%)		Variability (%)	Residual
	Observed	Predicted		
NE-1GJ[Km. 14-15]	7.00	7.50	7.14	-0.5
NE-1GJ[Km. 15-14]	6.30	6.50	3.17	-0.2
NE-1GJ[Km. 19-20]	6.50	7.00	7.69	-0.5
NE-1GJ[Km. 32-31]	12.00	13.00	8.33	-1.0

The percentage variability obtained between the observed and predicted cracking area values ranges between 3.17 to 8.33 percent, which is quite reasonable and indicates that the model predictions lie close to actual observations. From Figure 6.1, the plotted values are quite close to the LoE, indicating a good agreement between the predicted and observed values of the cracks.

Ravelling Progression

The observed values of ravelling area at the end of the year 2013 for the four pavement sections of Group-1 are compared with those predicted by HDM-4 ravelling progression model. The observed and predicted values of ravelled area at the end of the year 2013 and their variations are given in Table 6.2. These values along with the LoE are plotted in Figure 6.2.

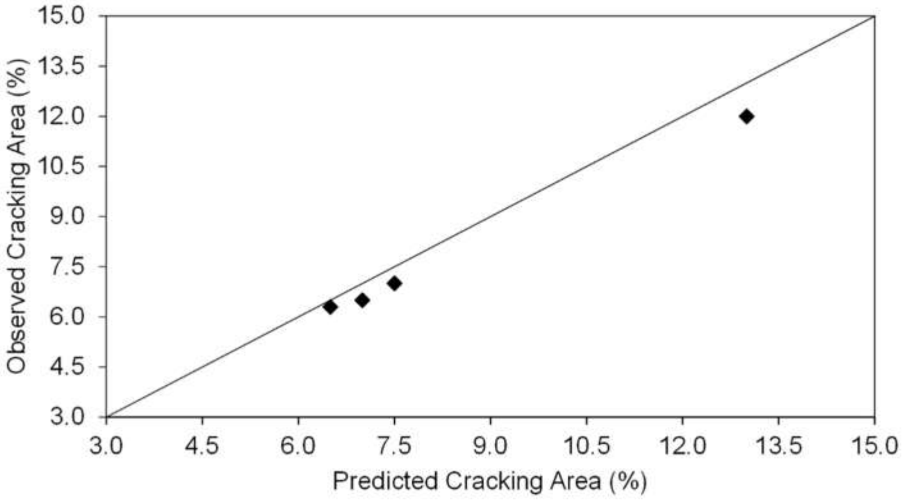


Figure 6.1 Observed v/s predicted cracking (%Area), Gr-1

Table 6.2 Variability between observed and predicted ravelling, Gr-1

Group-1 Pavement Sections	Ravelling Area (%)		Variability (%)	Residual
	Observed	Predicted		
NE-1GJ[Km. 14-15]	6.00	5.80	3.33	0.2
NE-1GJ[Km. 15-14]	5.00	5.10	2.00	-0.1
NE-1GJ[Km. 19-20]	6.00	5.50	8.33	0.5
NE-1GJ[Km. 32-31]	11.00	10.50	4.55	0.5

The percentage variability and residuals obtained between the observed and predicted ravelling area are found to be quite low. As observed in Figure 6.2, the plot between observed and predicted ravelling area is found in close proximity to the LoE, indicating that the calibrated HDM-4 ravelling progression model is adequate for the prediction of ravelling distress.

Roughness Progression

The observed values of roughness at the end of the year 2013 for the same four pavement sections of Group-1 are compared with those predicted by HDM-4 roughness progression model. The observed and predicted values of roughness at the end of the year 2013 and their variations are given in Table 6.3. These values are also plotted against each other, as shown in Figure 6.3.

The percentage variability obtained between the observed and predicted roughness values ranges between 0.0 to1.79 percent, which is low and hence the model predictions are highly acceptable. As observed in Figure 6.3, the plot between observed and predicted roughness is found close to the LoE, indicating the adequacy and validity of the calibrated HDM-4 roughness progression model.

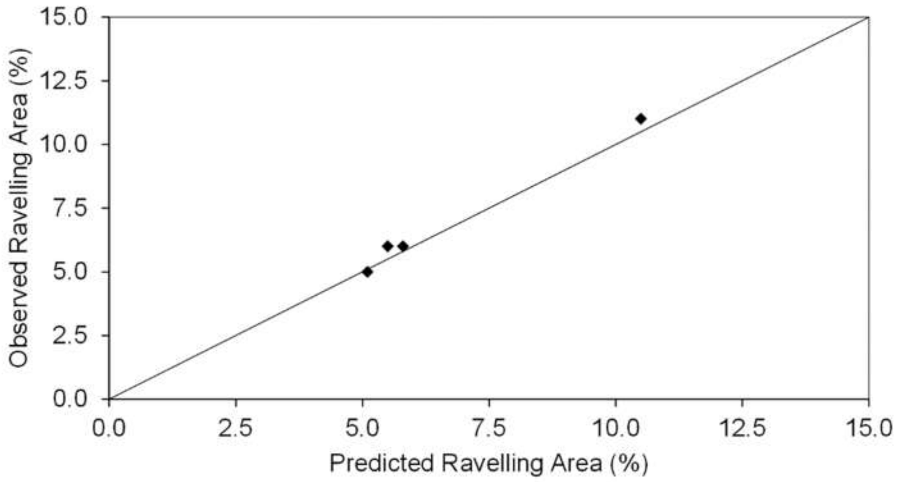


Figure 6.2 Observed v/s predicted ravelling (% Area), Gr-1

Table 6.3 Variability between observed and predicted roughness, Gr-1

Group-1 Pavement Sections	Roughness (IRI m/km)		Variability (%)	Residual
	Observed	Predicted		
NE-1GJ[Km. 14-15]	2.60	2.60	0.00	0.0
NE-1GJ[Km. 15-14]	2.85	2.82	1.05	0.03
NE-1GJ[Km. 19-20]	2.80	2.85	1.79	-0.05
NE-1GJ[Km. 32-31]	3.30	3.25	1.51	0.05

Rut Depth Progression

The observed values of rut depth at the end of the year 2013 for the four pavement sections of Group-1 are compared with those predicted by HDM-4 rut depth progression model. The observed and predicted values of rut depth at the end of the year 2013 and their variations are shown in Table 6.4. These values are plotted against each other, as shown in Figure 6.4.

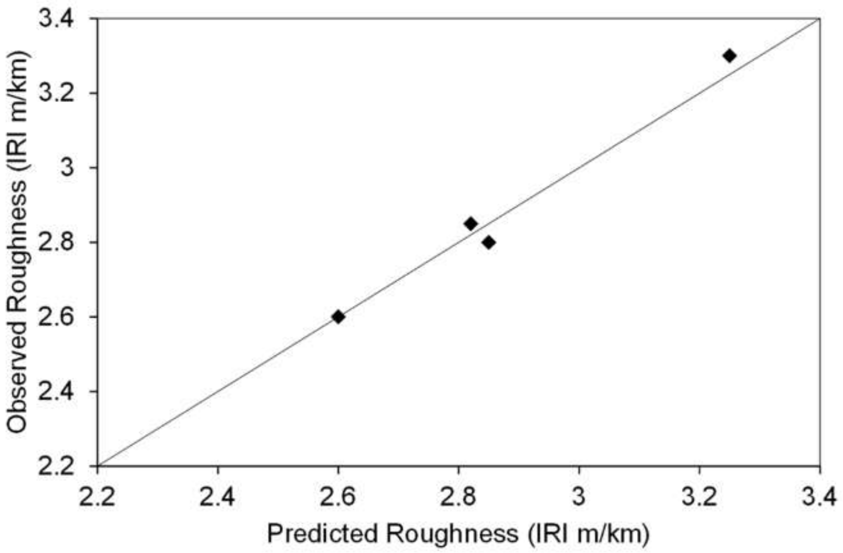


Figure 6.3 Observed v/s predicted roughness (m/km IRI), Gr-1

The percentage variability obtained between the observed and predicted rut-depth values ranges between 6.67 to 7.14 percent, which is quite reasonable considering the inherent variations in climatic and traffic loading conditions. As observed in Figure 6.4, the plot between observed and predicted rut depth is found near to the LoE, indicating the adequacy and validity of the calibrated HDM-4 rut depth progression model.

Table 6.4 Variability between observed and predicted rut depth, Gr-1

Group-1 Pavement Sections	Rut Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NE-1GJ[Km. 14-15]	2.80	2.60	7.14	0.2
NE-1GJ[Km. 15-14]	3.00	3.20	6.67	-0.2
NE-1GJ[Km. 19-20]	3.00	2.80	6.67	0.2
NE-1GJ[Km. 32-31]	4.30	4.00	6.98	0.3

Texture Depth Progression

The observed values of texture depth at the end of the year 2013 for the four pavement sections of Group-1 are compared with those predicted by HDM-4 texture depth progression model. The observed and predicted values of texture depth at the end of the year 2013 and their variations are given in Table 6.5. These values are also plotted against each other, as shown in Figure 6.5.

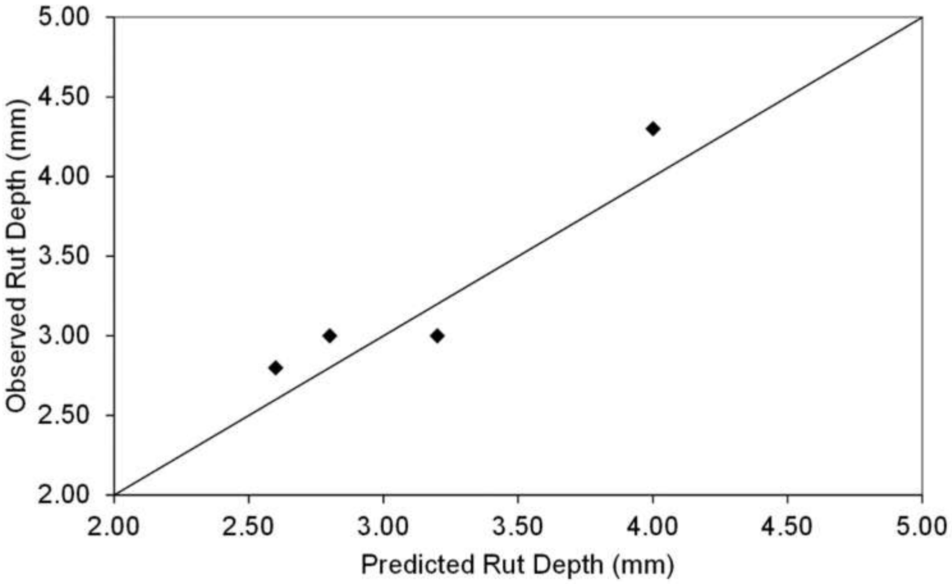


Figure 6.4 Observed v/s predicted rut depth (mm), Gr-1

The percentage variability obtained between the observed and predicted texture depth values ranges between 2.86to 8.33percent, which is quite reasonable. As observed in Figure 6.5, the plot between observed and predicted texture depth is found close to the LoE, indicating the adequacy and validity of the calibrated HDM-4 texture depth progression model.

Table 6.5 Variability between observed and predicted texture depth, Gr-1

Group-1 Pavement Sections	Texture Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NE-1GJ[Km. 14-15]	0.45	0.48	6.67	-0.03
NE-1GJ[Km. 15-14]	0.80	0.75	6.25	0.05
NE-1GJ[Km. 19-20]	0.60	0.65	8.33	-0.05
NE-1GJ[Km. 32-31]	0.70	0.72	2.86	-0.02

Skid Resistance Progression

The observed values of skid resistance at the end of the year 2013 for the four pavement sections of Group-1 are compared with those predicted by HDM-4 skid resistance progression model. The observed and predicted values of skid resistance at the end of the year 2013 and their variations are given in Table 6.6. These values have also been plotted against each other, as shown in Figure 6.6.

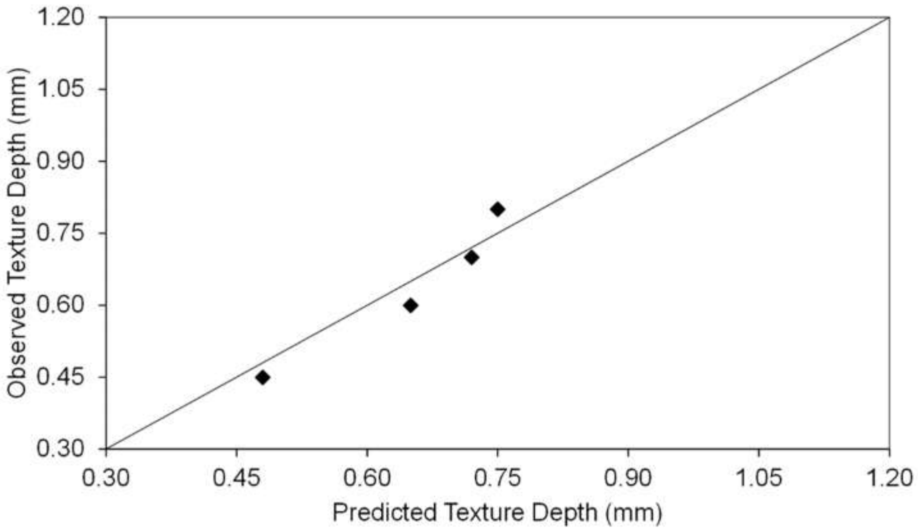


Figure 6.5 Observed v/s predicted texture depth (mm), Gr-1

Table 6.6 Variability between observed and predicted skid resistance, Gr-1

Group-1 Pavement Sections	Skid Resistance (SFC50)		Variability (%)	Residual
	Observed	Predicted		
NE-1GJ[Km. 14-15]	0.52	0.51	1.92	0.01
NE-1GJ[Km. 15-14]	0.56	0.55	1.79	0.01
NE-1GJ[Km. 19-20]	0.50	0.49	2.00	0.01
NE-1GJ[Km. 32-31]	0.55	0.53	3.64	0.02

The percentage variability and the residuals obtained between the observed and predicted skid resistance values are found to be appreciably low, indicating adequate prediction capability of the model. As observed in Figure 6.6, the plot between observed and predicted skid resistance is found in close proximity to the LoE, indicating the adequate validity of the calibrated HDM-4 skid resistance progression model.

6.2.2 Validation for Sections in Group-2

Cracking Progression

The observed values of cracking area at the end of the year 2013 for six pavement sections of Group-2are compared with those predicted by HDM-4 cracking progression model. The observed and predicted values of cracking area at the end of the year 2013 and their variations are given in Table 6.7. These values are also plotted against each other, as shown in Figure 6.7.

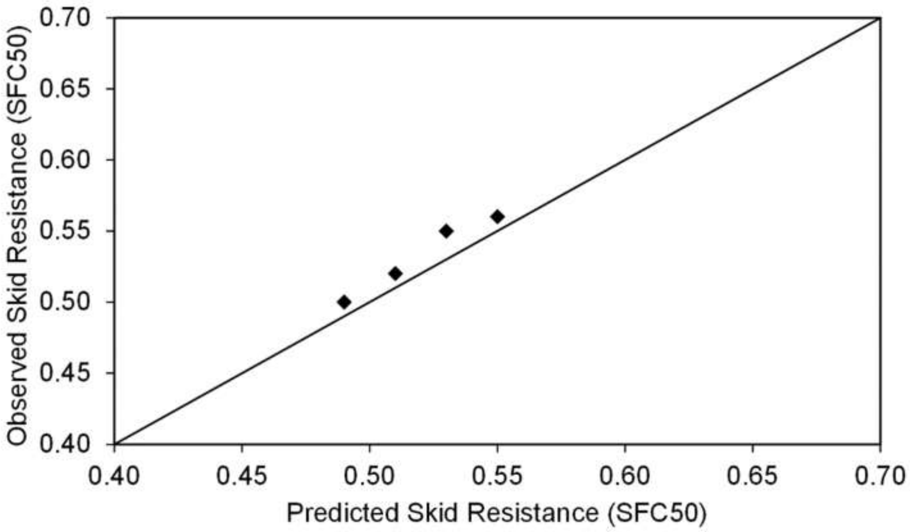


Figure 6.6 Observed v/s predicted skid resistance, Gr-1

Table 6.7 Variability between observed and predicted cracking, Gr-2

Group-2 Pavement Sections	Cracking Area (%)		Variability (%)	Residual
	Observed	Predicted		
NH-2UP[Km. 607-606]	10.5	11.0	4.8	-0.5
NH-2UP[Km. 717-718]	5.4	6.0	11.1	-0.6
NH-5AP[Km. 691.25-690.75]	2.1	2.0	4.8	0.1
NH-5AP[Km. 698.6-698.1]	4.8	5.0	4.2	-0.2
NH-5AP[Km. 7.5-7.0]	16.0	20.0	25.0	-4.0
NH-5AP[Km. 9.8-9.3]	18.0	22.0	22.2	-4.0

The percentage variability obtained between the observed and predicted cracking area values ranges between 4.2 to 25.0 percent, which is reasonable and acceptable considering the inherent variations in climatic and traffic conditions. As observed in Figure 6.7, the plot between observed and predicted cracking area is found close to the LoE.

Ravelling Progression

The observed values of ravelling area at the end of the year 2013 for the six pavement sections of Group-2 are compared with those predicted by HDM-4 ravelling progression model. The observed and predicted values of ravelled area at the end of the year 2013 and their variations are given in Table 6.8. These values are also plotted against each other, as shown in Figure 6.8.

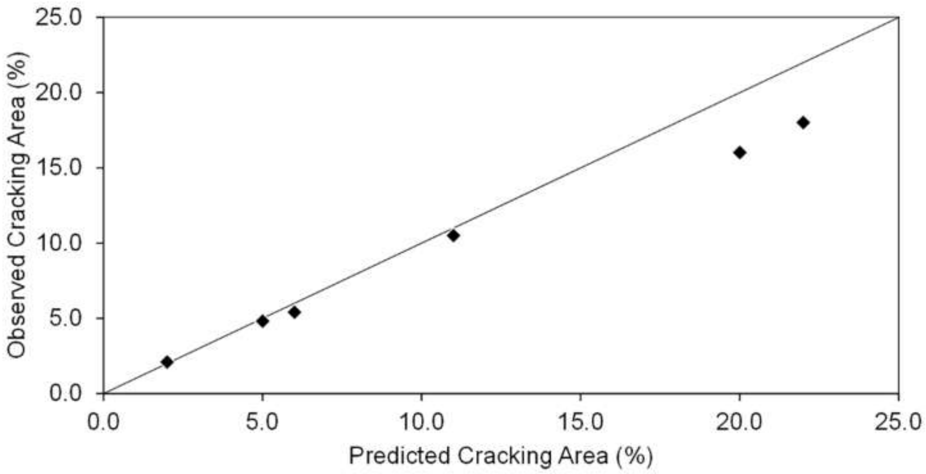


Figure 6.7 Observed v/s predicted cracking (%Area), Gr-2

Table 6.8 Variability between observed and predicted ravelling, Gr-2

Group-2 Pavement Sections	Ravelling Area (%)		Variability (%)	Residual
	Observed	Predicted		
NH-2UP[Km. 607-606]	7.8	8.0	2.6	-0.2
NH-2UP[Km. 717-718]	4.1	4.0	2.4	0.1
NH-5AP[Km. 691.25-690.75]	2.0	2.1	5.0	-0.1
NH-5AP[Km. 698.6-698.1]	2.0	2.5	25.0	-0.5
NH-5AP[Km. 7.5-7.0]	12.0	13.0	8.3	-1.0
NH-5AP[Km. 9.8-9.3]	13.0	16.0	23.1	-3.0

The percentage variability obtained between the observed and predicted ravelling area values ranges between 2.4 to 25.0 percent. As observed in Figure 6.8, the plot between observed and predicted ravelling area is found close to the LoE, indicating the adequacy and validity of the calibrated HDM-4 ravelling progression model.

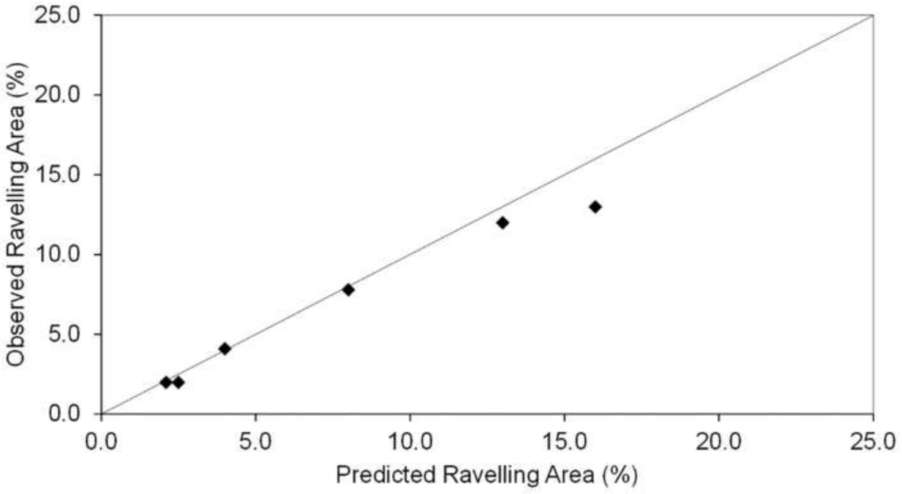


Figure 6.8 Observed v/s predicted ravelling (% Area), Gr-2

Pothole Progression

The observed numbers of standard pothole units at the end of the year 2013 for the same six pavement sections of Group-2 are compared with those predicted by HDM-4 pothole progression model. The observed and predicted values of pothole at the end of the year 2013 and their variations are given in Table 6.9. These values are also plotted against each other, as shown in Figure 6.9.

The percentage variability obtained between the observed and predicted pothole units ranges between 0.0 to40.0 percent. Even though a variability of 30-40% may appear to be large, however, the observed and predicted pothole numbers are quite close to each other as indicated by the residuals. Further, the size of the samples collected for observed and predicted values was only for three years (three observations) and the potholes were found in limited numbers of sections, thus the percentage variability has been found to be slightly on higher side.

Table 6.9 Variability between observed and predicted potholes, Gr-2

Group-2 Pavement Sections	Potholes (no.)		Variability (%)	Residual
	Observed	Predicted		
NH-2UP[Km. 607-606]	1	1	0.0	0
NH-2UP[Km. 717-718]	0	0	0.0	0
NH-5AP[Km. 691.25-690.75]	0	0	0.0	0
NH-5AP[Km. 698.6-698.1]	0	0	0.0	0
NH-5AP[Km. 7.5-7.0]	3.0	4.0	33.3	-1
NH-5AP[Km. 9.8-9.3]	5.0	7.0	40.0	-2

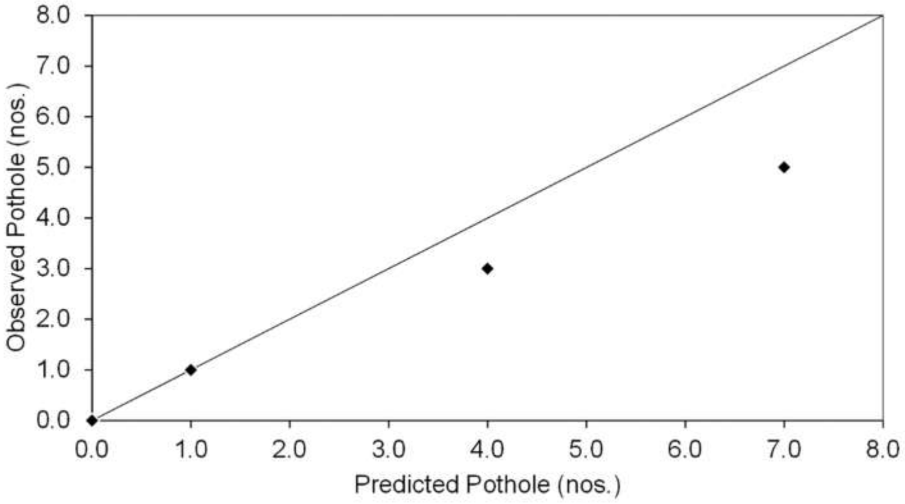


Figure 6.9 Observed v/s predicted potholes, Gr-2

Roughness Progression

The observed values of roughness at the end of the year 2013 for the six pavement sections of Group-2 are compared with those predicted by HDM-4 roughness progression model. The observed and predicted values of roughness at the end of the year 2013 and their variations are given in Table 6.10. These values are also plotted against each other, as shown in Figure 6.10.

The percentage variability obtained between the observed and predicted roughness values ranges between 2.86 to10.0 percent, which is quite acceptable. As observed in Figure 6.10, the plot between observed and predicted roughness is found close to the LoE, indicating adequate validity of the calibrated HDM-4 roughness progression model.

Table 6.10 Variability between observed and predicted roughness, Gr-2

Group-2 Pavement Sections	Roughness (IRI m/km)		Variability (%)	Residual
	Observed	Predicted		
NH-2UP[Km. 607-606]	3.0	2.7	10.0	0.3
NH-2UP[Km. 717-718]	3.5	3.4	2.86	0.1
NH-5AP[Km. 691.25-690.75]	3.0	2.9	3.33	0.1
NH-5AP[Km. 698.6-698.1]	3.1	3.0	3.23	0.1
NH-5AP[Km. 7.5-7.0]	3.3	3.1	6.06	0.2
NH-5AP[Km. 9.8-9.3]	3.2	3.1	3.13	0.1

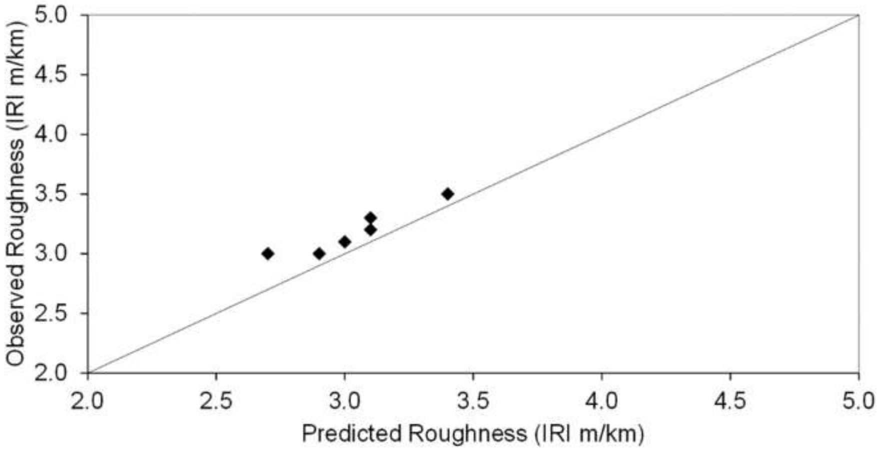


Figure 6.10 Observed v/s predicted roughness (m/km IRI), Gr-2

Rut Depth Progression

The observed values of rut depth at the end of the year 2013 for the six pavement sections of Group-2 are compared with those predicted by HDM-4 rut depth progression model. The observed and predicted values of rut depth at the end of the year 2013 and their variations are given in Table 6.11. These values are also plotted against each other, as shown in Figure 6.11.

The percentage variability obtained between the observed and predicted rut-depth values ranges between 3.2 to 12.9 percent, which is quite reasonable. As observed in Figure 6.11, the plot between observed and predicted rut depth is found close to the LoE, indicating the adequacy and validity of the calibrated HDM-4 rut depth progression model.

Table 6.11 Variability between observed and predicted rut depth, Gr-2

Group-2 Pavement Sections	Rut Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-2UP[Km. 607-606]	4.2	4.5	7.1	-0.3
NH-2UP[Km. 717-718]	3.1	3.2	3.2	-0.1
NH-5AP[Km. 691.25-690.75]	3.2	3.6	12.5	-0.4
NH-5AP[Km. 698.6-698.1]	4.0	4.2	5.0	-0.2
NH-5AP[Km. 7.5-7.0]	3.1	2.7	12.9	0.4
NH-5AP[Km. 9.8-9.3]	3.1	3.5	12.9	-0.4

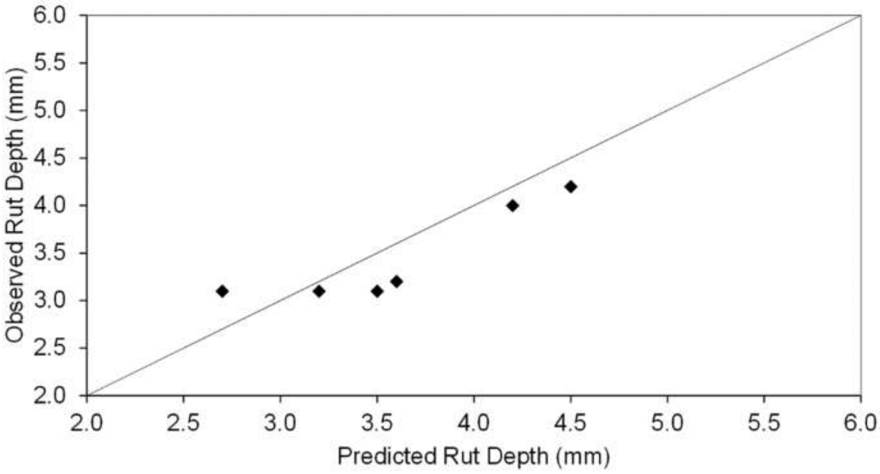


Figure 6.11 Observed v/s predicted rut depth (mm), Gr-2

Texture Depth Progression

The observed values of texture depth at the end of the year 2013 for the six pavement sections of Group-2 are compared with those predicted by HDM-4 texture depth progression model. The observed and predicted values of texture depth at the end of the year 2013 and their variations are given in Table 6.12. These values are also plotted against each other, as shown in Figure 6.12.

Table 6.12 Variability between observed and predicted texture depth, Gr-2

Group-2 Pavement Sections	Texture Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-2UP[Km. 607-606]	0.70	0.79	12.9	-0.09
NH-2UP[Km. 717-718]	0.52	0.50	3.8	0.02
NH-5AP[Km. 691.25-690.75]	0.41	0.40	2.4	0.01
NH-5AP[Km. 698.6-698.1]	0.50	0.60	20.0	-0.10
NH-5AP[Km. 7.5-7.0]	0.60	0.70	16.7	-0.10
NH-5AP[Km. 9.8-9.3]	0.61	0.70	14.8	-0.09

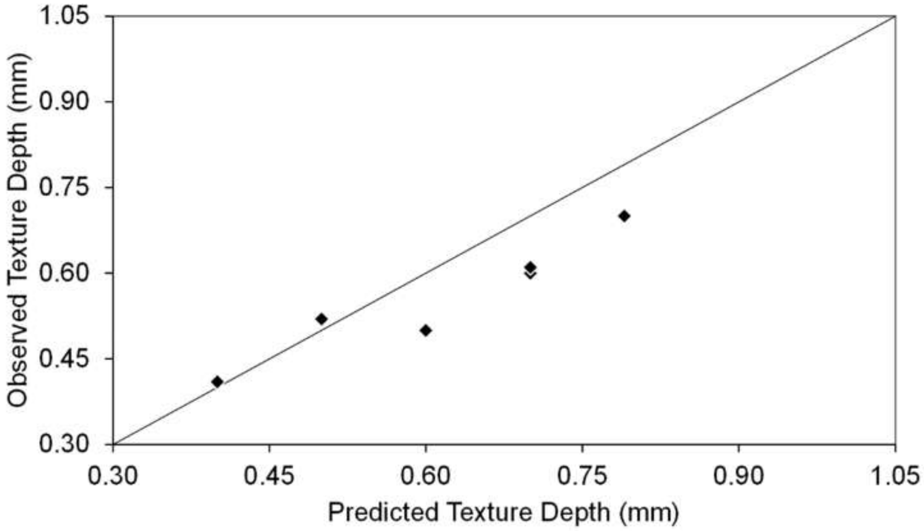


Figure 6.12 Observed v/s predicted texture depth (mm), Gr-2

The percentage variability obtained between the observed and predicted texture depth values ranges between 2.4to 20.0percent, which is quite reasonable.

Skid Resistance Progression

The observed values of skid resistance at the end of the year 2013 for the same six pavement sections of Group-2 are compared with those predicted by HDM-4 skid resistance progression model. The observed and predicted values of skid resistance at the end of the year 2013 and their variations are given in Table 6.13. These values are also plotted against each other, as shown in Figure 6.13.

Table 6.13 Variability between observed & predicted skid resistance, Gr-2

Group-2 Pavement Sections	Skid Resistance (SFC50) Observed	 Predicted	Variability (%)	Residual
NH-2UP[Km. 607-606]	0.70	0.65	7.14	0.05
NH-2UP[Km. 717-718]	0.55	0.50	9.09	0.05
NH-5AP[Km. 691.25-690.75]	0.65	0.59	9.23	0.06
NH-5AP[Km. 698.6-698.1]	0.60	0.55	8.33	0.05
NH-5AP[Km. 7.5-7.0]	0.70	0.67	4.29	0.03
NH-5AP[Km. 9.8-9.3]	0.70	0.64	8.57	0.06

The percentage variability obtained between the observed and predicted skid resistance values ranges between 4.29 to9.23 percent, which is quite reasonable. As observed in Figure 6.13, the plot between observed and predicted skid resistance is found near to the LoE indicating the adequacy and validity of the calibrated HDM-4 skid resistance progression model.

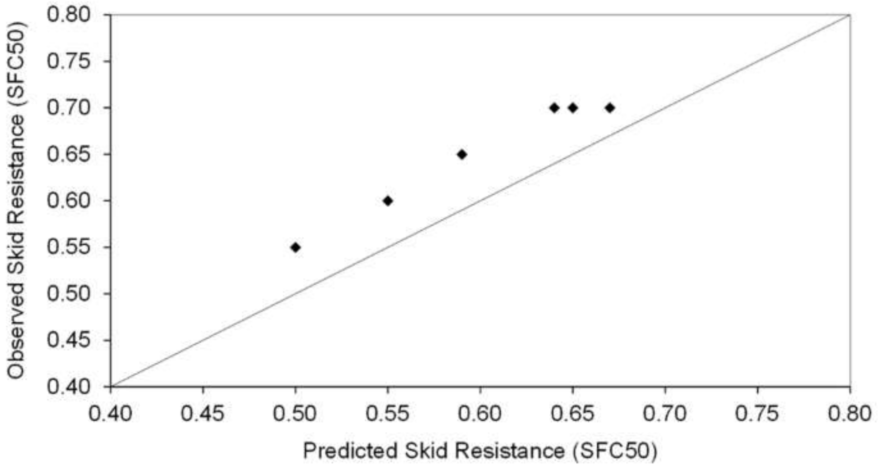


Figure 6.13 Observed v/s predicted skid resistance, Gr-2

6.2.3 Validation for Sections in Group-3

Cracking Progression

The observed values of cracking area at the end of the year 2013 for three pavement sections of Group-3 are compared with those predicted by HDM-4 cracking progression model. The observed and predicted values of cracking area at the end of the year 2013 and their variations are given in Table 6.14. These values are also plotted against each other, as shown in Figure 6.14.

Table 6.14 Variability between observed and predicted cracking, Gr-3

Group-3 Pavement Sections	Cracking Area (%)		Variability (%)	Residual
	Observed	Predicted		
NH-7AP[Km. 41-40]	1.50	1.50	0.00	0.00
NH-7AP[Km. 51.4-50.4]	0.52	0.50	3.85	0.02
NH-7MH[Km. 95.5-96.0]	1.10	1.10	0.00	0.00

The percentage variability obtained between the observed and predicted cracking area values ranges between 0.0 to 3.85 percent. Figure 6.14 indicates that the adequacy and validity of the calibrated HDM-4 cracking progression model for the present study.

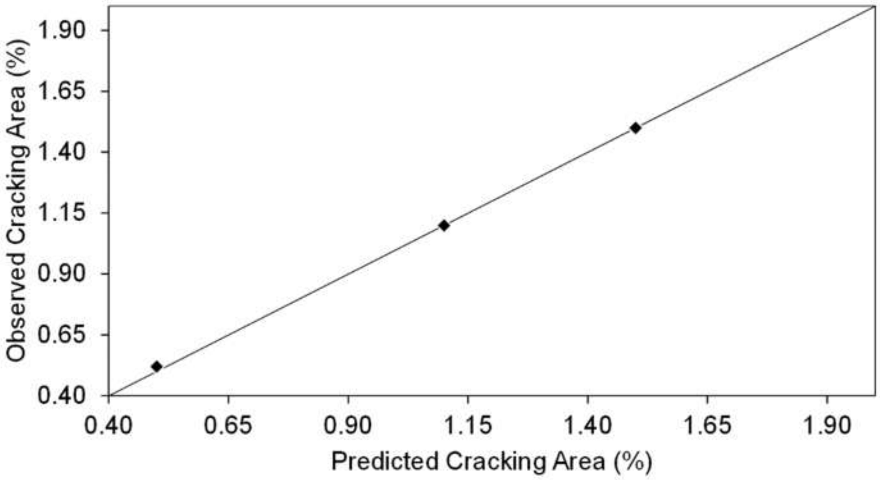


Figure 6.14 Observed v/s predicted cracking (%Area), Gr-3

Ravelling Progression

The observed values of ravelling area at the end of the year 2013 for the three pavement sections of Group-3 are compared with those predicted by HDM-4 ravelling progression model. The observed and predicted values of ravelled area at the end of the year 2013 and their variations are given in Table 6.15. These values are also plotted against each other, as shown in Figure 6.15.

Table 6.15 Variability between observed and predicted ravelling, Gr-3

Group-3 Pavement Sections	Ravelling Area (%)		Variability (%)	Residual
	Observed	Predicted		
NH-7AP[Km. 41-40]	3.50	3.00	14.29	0.5
NH-7AP[Km. 51.4-50.4]	4.00	3.50	12.50	0.5
NH-7MH[Km. 95.5-96.0]	4.70	5.00	6.38	-0.3

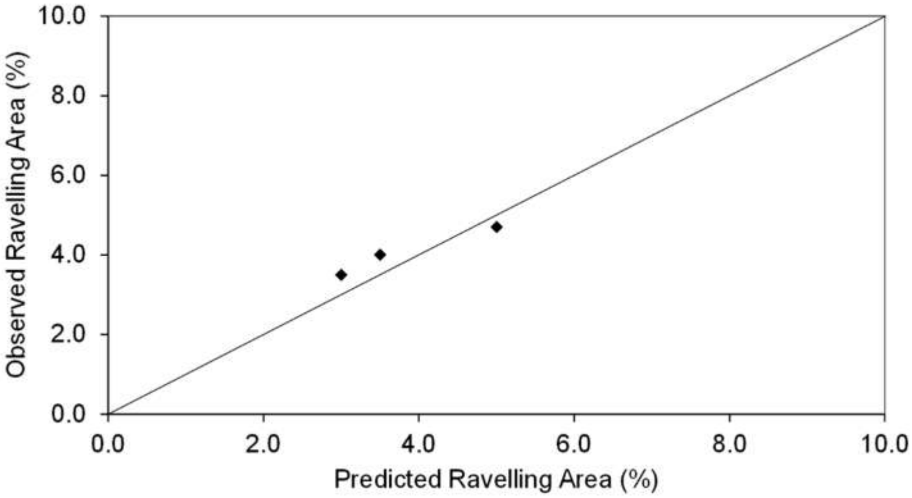


Figure 6.15 Observed v/s predicted ravelling (% Area), Gr-3

The percentage variability obtained between the observed and predicted ravelling area values ranges between 6.38 to 14.29 percent, which is quite reasonable. As observed in Figure 6.15, the plot between observed and predicted ravelling area is found close to the LoE, indicating the adequacy and validity of the calibrated HDM-4 ravelling progression model.

Roughness Progression

The observed values of roughness at the end of the year 2013 for the three pavement sections of Group-3 are compared with those predicted by HDM-4 roughness progression model. The observed and predicted values of roughness at the end of the year 2013 and their variations are given in Table 6.16. These values are also plotted against each other, as shown in Figure 6.16.

Table 6.16 Variability between observed and predicted roughness, Gr-3

Group-3 Pavement Sections	Roughness (IRI m/km)		Variability (%)	Residual
	Observed	Predicted		
NH-7AP[Km. 41-40]	3.50	3.40	2.86	0.1
NH-7AP[Km. 51.4-50.4]	2.90	3.00	3.45	-0.1
NH-7MH[Km. 95.5-96.0]	3.00	2.90	3.33	0.1

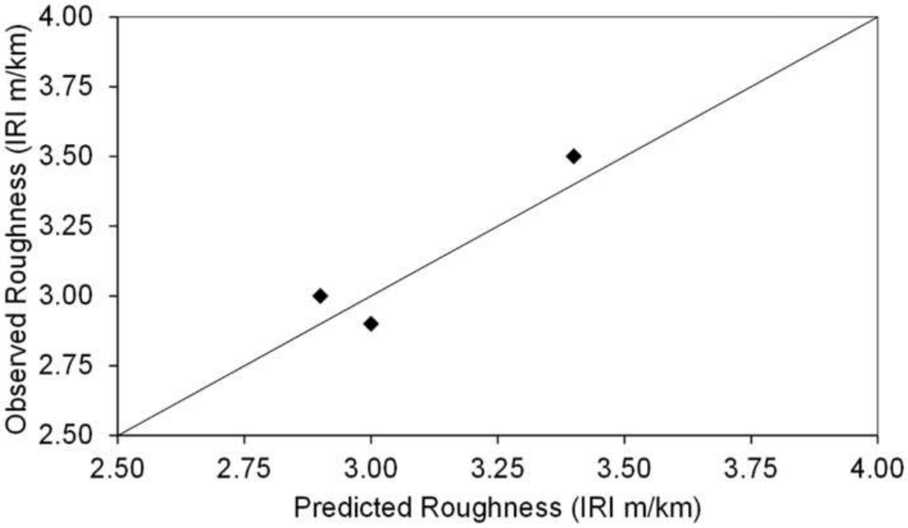


Figure 6.16 Observed v/s predicted roughness (m/km IRI), Gr-3

The percentage variability obtained between the observed and predicted roughness values ranges between 2.86 to 3.44 percent, which is quite low and hence indicates good prediction capability of the model. As observed in Figure 6.16, the plot between observed and predicted roughness is also found in close proximity to the LoE.

Rut Depth Progression

The observed values of rut depth at the end of the year 2013 for the three pavement sections of Group-3 are compared with those predicted by HDM-4 rut depth progression model. The observed and predicted values of rut depth at the end of the year 2013 and their variations are given in Table 6.17. These values are also plotted against each other, as shown in Figure 6.17.

Table 6.17 Variability between observed and predicted rut depth, Gr-3

Group-3 Pavement Sections	Rut Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-7AP[Km. 41-40]	4.30	4.50	4.65	-0.2
NH-7AP[Km. 51.4-50.4]	5.00	5.20	4.00	-0.2
NH-7MH[Km. 95.5-96.0]	3.00	3.10	3.33	-0.1

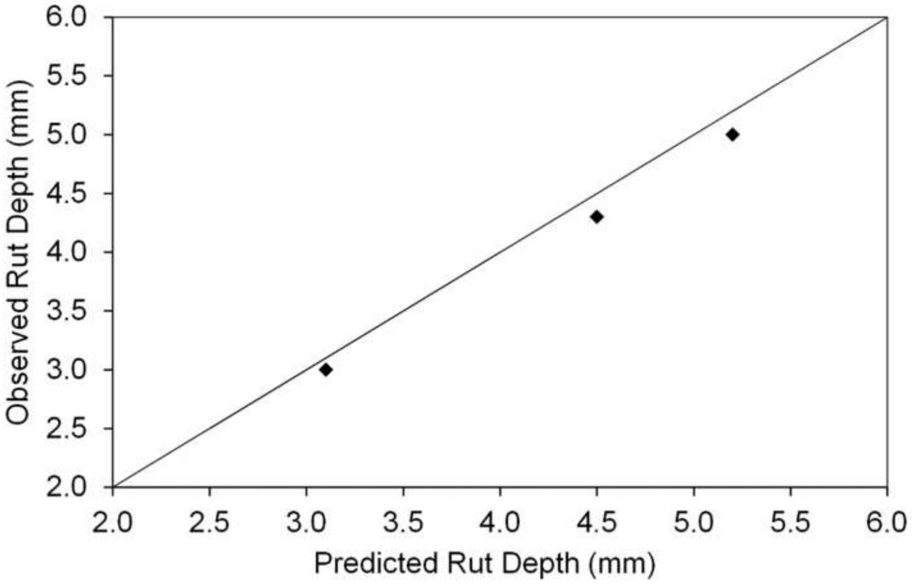


Figure 6.17 Observed v/s predicted rut depth (mm), Gr-3

The percentage variability obtained between the observed and predicted rut-depth values ranges between 3.33 to 4.65 percent. As observed in Figure 6.17, the plot between observed and predicted rut depth is found near to the LoE, indicating the adequacy and validity of the calibrated HDM-4 rut depth progression model.

Texture Depth Progression

The observed values of texture depth at the end of the year 2013 for the same three pavement sections of Group-3 are compared with those predicted by HDM-4 texture depth progression model. The observed and predicted values of texture depth at the end of the year 2013 and their variations are given in Table 6.18. These values are also plotted against each other, as shown in Figure 6.18.

Table 6.18 Variability between observed and predicted texture depth, Gr-3

Group-3 Pavement Sections	Texture Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-7AP[Km. 41-40]	0.65	0.66	1.54	-0.01
NH-7AP[Km. 51.4-50.4]	0.68	0.69	1.47	-0.01
NH-7MH[Km. 95.5-96.0]	0.66	0.67	1.52	-0.01

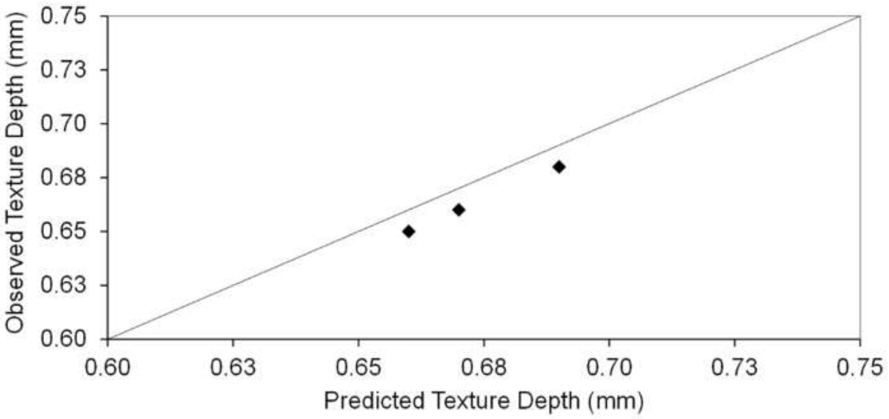


Figure 6.18 Observed v/s predicted texture depth (mm), Gr-3

The percentage variability obtained between the observed and predicted texture depth values is about 1.5 percent, which is quite low and hence indicates good prediction capability of the model. As observed in Figure 6.18, the plot between observed and predicted texture depth is found in close proximity to the LoE indicating proper validity of the calibrated HDM-4 texture depth progression model.

Skid Resistance Progression

The observed values of skid resistance at the end of the year 2013 for the three pavement sections of Group-3 are compared with those predicted by HDM-4 skid resistance progression model. The observed and predicted values of skid resistance at the end of the year 2013 and their variations are given in Table 6.19. These values are also plotted against each other, as shown in Figure 6.19.

Table 6.19 Variability between observed & predicted skid resistance, Gr-3

Group-3 Pavement Sections	Skid Resistance (SFC50)		Variability (%)	Residual
	Observed	Predicted		
NH-7AP[Km. 41-40]	0.61	0.59	3.28	0.02
NH-7AP[Km. 51.4-50.4]	0.54	0.53	1.85	0.01
NH-7MH[Km. 95.5-96.0]	0.60	0.60	0.00	0.00

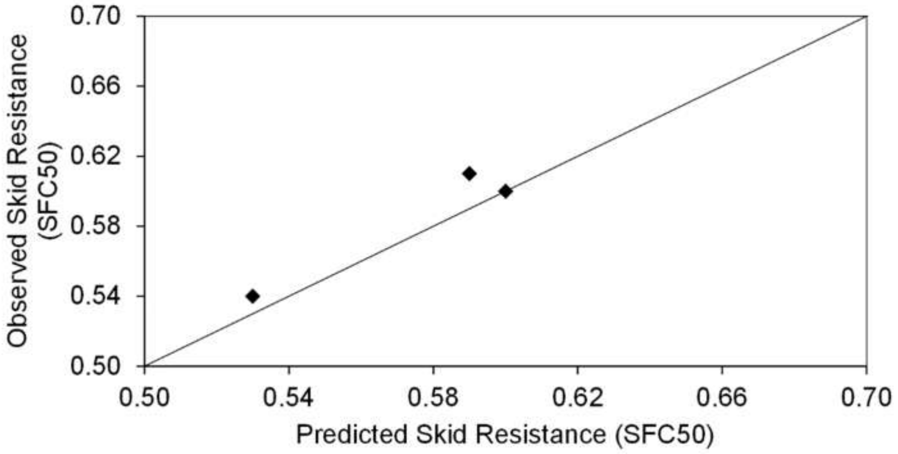


Figure 6.19 Observed v/s predicted skid resistance, Gr-3

The percentage variability obtained between the observed and predicted skid resistance values ranges between 0.0to3.28percent, which is quite reasonable. As observed in Figure 6.19, the plot between observed and predicted skid resistance is also found near to the LoE.

6.2.4 Validation for Sections in Group-4

Cracking Progression

The observed values of cracking area at the end of the year 2013 for three pavement sections of Group-4are compared with those predicted by HDM-4 cracking progression model. The observed and predicted values of cracking area at the end of the year 2013 and their variations are given in Table 6.20. These values are also plotted against each other, as shown in Figure 6.20.

Table 6.20 Variability between observed and predicted cracking, Gr-4

Group-4 Pavement Sections	Cracking Area (%)		Variability (%)	Residual
	Observed	Predicted		
NH-14GJ[Km. 380.6-379.6]	39.00	38.00	2.56	1.0
NH-8GJ[Km. 433.7-434.7]	10.00	11.00	10.00	-1.0
NH-8GJ[Km. 441.4-442.4]	7.00	7.00	0.00	0.0

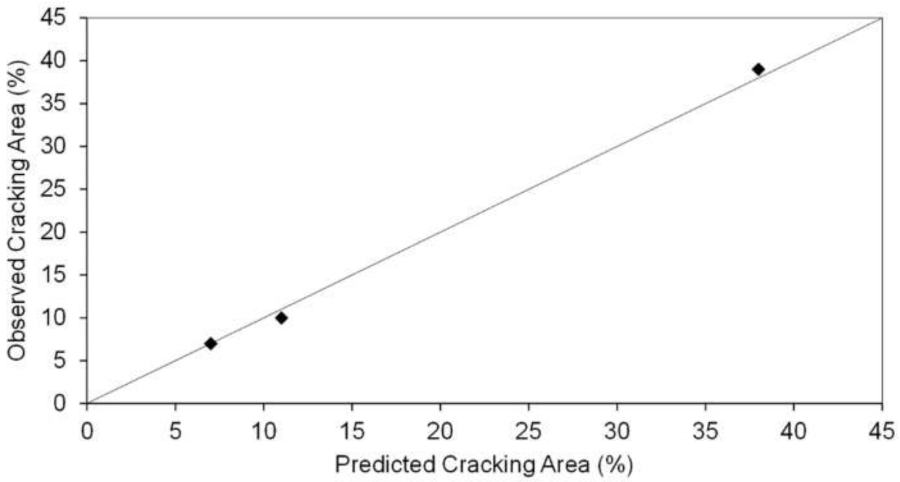


Figure 6.20 Observed v/s predicted cracking (%Area), Gr-4

The percentage variability obtained between the observed and predicted cracking area values ranges between 0.0 to 10.0 percent, which is quite reasonable. As observed in Figure 6.20, the plot between observed and predicted cracking area is found quite close to the LoE, indicating a good validity of the calibrated HDM-4 cracking progression model for the present study.

Ravelling Progression

The observed values of ravelling area at the end of the year 2013 for the three pavement sections of Group-4are compared with those predicted by HDM-4 ravelling progression model. The observed and predicted values of ravelled area at the end of the year 2013 and their variations are given in Table 6.21. These values are also plotted against each other, as shown in Figure 6.21.

Table 6.21 Variability between observed and predicted ravelling, Gr-4

Group-4 Pavement Sections	Ravelling Area (%) Observed	Ravelling Area (%) Predicted	Variability (%)	Residual
NH-14GJ[Km. 380.6-379.6]	28.0	26.0	7.14	2.0
NH-8GJ[Km. 433.7-434.7]	10.0	9.0	10.00	1.0
NH-8GJ[Km. 441.4-442.4]	11.0	13.0	18.18	-2.0

The percentage variability obtained between the observed and predicted ravelling area values ranges between 7.14 to 18.18 percent. As observed in Figure 6.21, the plot between observed and predicted ravelling area is found in close proximity to the LoE.

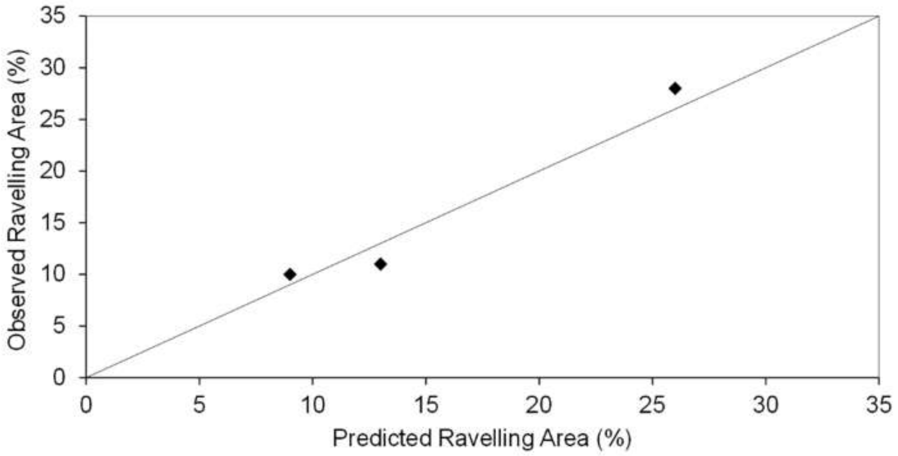


Figure 6.21 Observed v/s predicted ravelling (% Area), Gr-4

Roughness Progression

The observed values of roughness at the end of the year 2013 for the three pavement sections of Group-4 are compared with those predicted by HDM-4 roughness progression model. The observed and predicted values of roughness at the end of the year 2013 and their variations are given in Table 6.22. These values are also plotted against each other, as shown in Figure 6.22.

Table 6.22 Variability between observed and predicted roughness, Gr-4

Group-4 Pavement Sections	Roughness (IRI m/km)		Variability (%)	Residual
	Observed	Predicted		
NH-14GJ[Km. 380.6-379.6]	2.95	2.90	1.69	0.05
NH-8GJ[Km. 433.7-434.7]	3.10	3.21	3.55	-0.11
NH-8GJ[Km. 441.4-442.4]	3.00	2.93	2.33	0.07

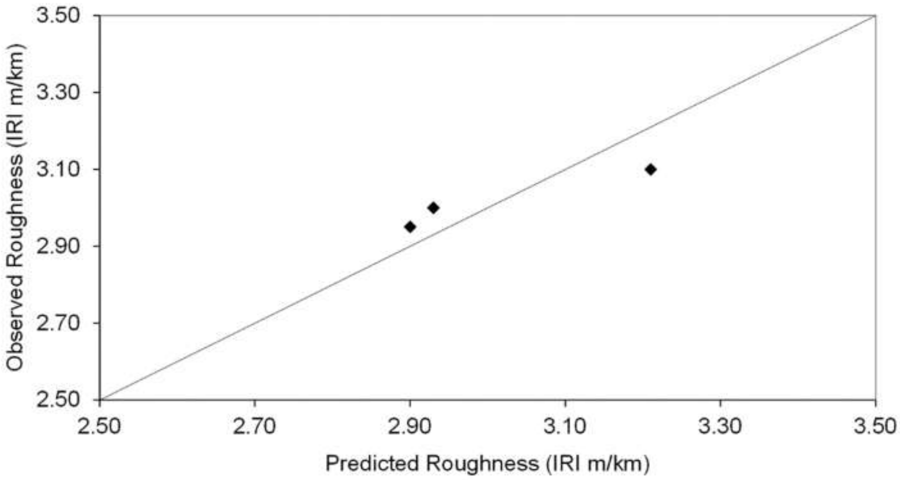


Figure 6.22 Observed v/s predicted roughness (m/km), Gr-4

The percentage variability obtained between the observed and predicted roughness values ranges between 1.69to 3.55percent, which is quite reasonable. As observed in Figure 6.22, the plot between observed and predicted roughness shows small deviations from the LoE, which is quite acceptable.

Rut Depth Progression

The observed values of rut depth at the end of the year 2013 for the three pavement sections of Group-4 are compared with those predicted by HDM-4 rut depth progression model. The observed and predicted values of rut depth at the end of the year 2013 and their variations are given in Table 6.23. These values are also plotted against each other, as shown in Figure 6.23.

Table 6.23 Variability between observed and predicted rut depth, Gr-4

Group-4 Pavement Sections	Rut Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-14GJ[Km. 380.6-379.6]	4.00	3.90	2.50	0.1
NH-8GJ[Km. 433.7-434.7]	3.50	3.20	8.57	0.3
NH-8GJ[Km. 441.4-442.4]	5.00	5.10	2.00	-0.1

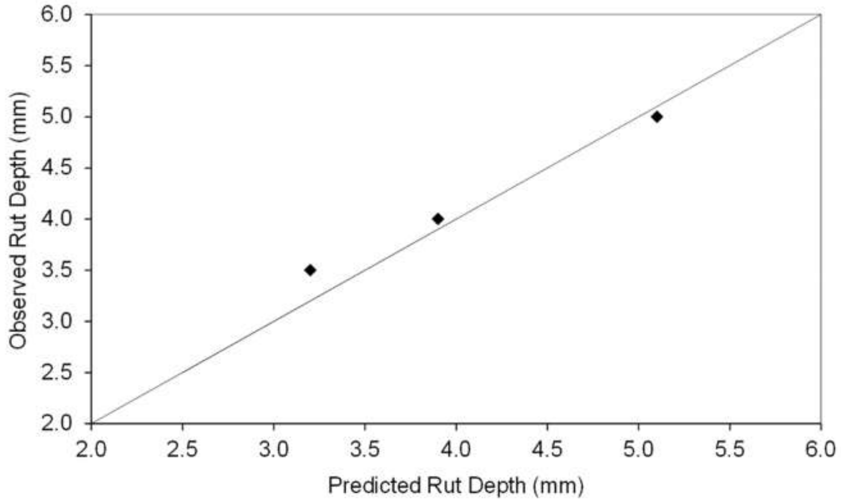


Figure 6.23 Observed v/s predicted rut depth (mm), Gr-4

The percentage variability obtained between the observed and predicted rut depth values ranges between 2.00to 8.57 percent, which is quite reasonable. As observed in Figure 6.23, the plot between observed and predicted rut depth shows very less deviations from the LoE, indicating the adequacy and validity of the calibrated HDM-4rut depth progression model for the present study.

Texture Depth Progression

The observed values of texture depth at the end of the year 2013 for the three pavement sections of Group-4 are compared with those predicted by HDM-4 texture depth progression model. The observed and predicted values of texture depth at the end of the year 2013 and their variations are given in Table 6.24. These values are also plotted against each other, as shown in Figure 6.24.

Table 6.24 Variability between observed and predicted texture depth, Gr-4

Group-4 Pavement Sections	Texture Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-14GJ[Km. 380.6-379.6]	0.70	0.72	2.86	-0.02
NH-8GJ[Km. 433.7-434.7]	0.60	0.56	6.67	0.04
NH-8GJ[Km. 441.4-442.4]	0.70	0.69	1.43	0.01

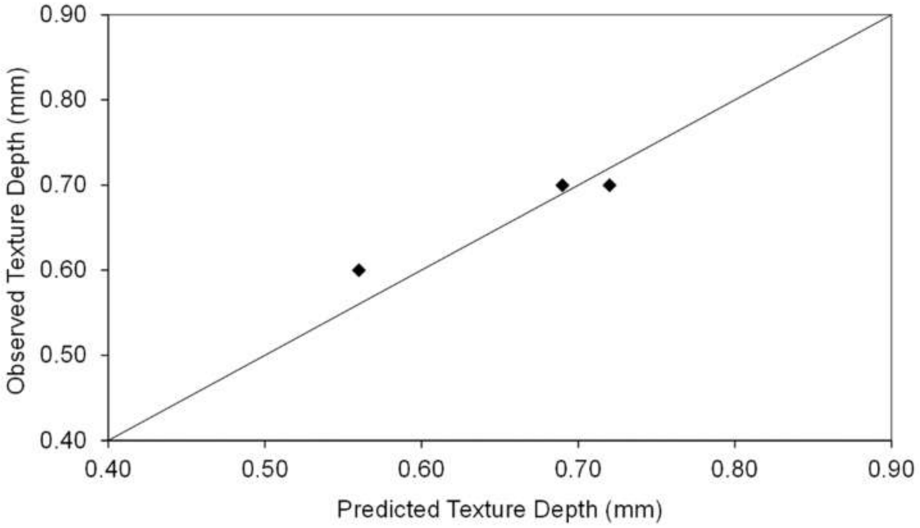


Figure 6.24 Observed v/s predicted texture depth (mm), Gr-4

The percentage variability obtained between the observed and predicted texture depth values ranges between 1.43to 6.67percent, which is quite reasonable. As observed in Figure 6.24, the plot between observed and predicted texture depth is found in close proximity to the LoE.

Skid Resistance Progression

The observed values of skid resistance at the end of the year 2013 for the three pavement sections of Group-4 are compared with those predicted by HDM-4 skid resistance progression model. The observed and predicted values of skid resistance values at the end of the year 2013 and their variations are given in

Table 6.25. These values are also plotted against each other, as shown in Figure 6.25.

Table 6.25 Variability between observed & predicted skid resistance, Gr-4

Group-4 Pavement Sections	Skid resistance (SFC) Observed	Skid resistance (SFC) Predicted	Variability (%)	Residual
NH-14GJ[Km. 380.6-379.6]	0.53	0.51	3.77	0.02
NH-8GJ[Km. 433.7-434.7]	0.55	0.54	1.82	0.01
NH-8GJ[Km. 441.4-442.4]	0.61	0.62	1.64	-0.01

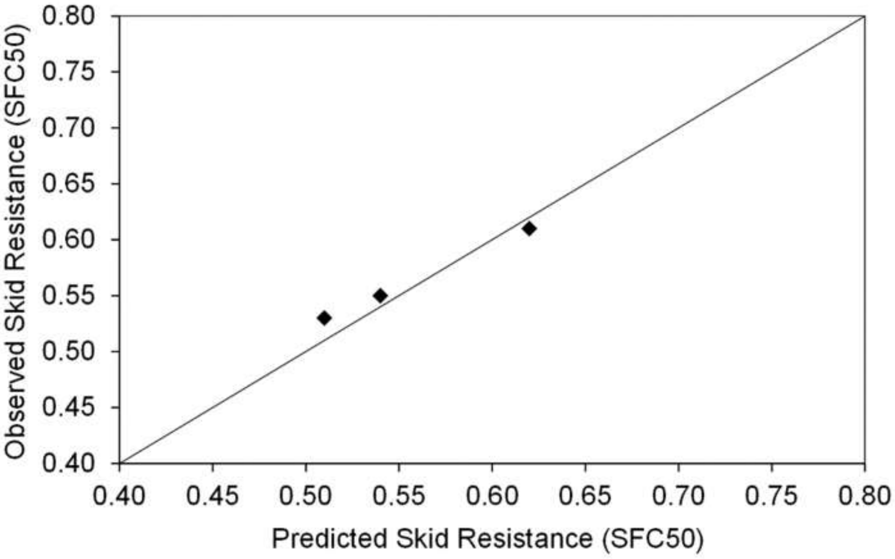


Figure 6.25 Observed v/s predicted skid resistance, Gr-4

The percentage variability obtained between the observed and predicted skid resistance values ranges between 1.82to 3.77percent, which is quite reasonable. As observed in Figure 6.25, the plot between observed and predicted skid resistance shows small deviation from the LoE, indicating the validity of the calibrated HDM-4 skid resistance progression model.

6.2.5 Validation for Sections in Group-5

Cracking Progression

The observed values of cracking area at the end of the year 2013 for five pavement sections of Group-5 are compared with those predicted by HDM-4 cracking progression model. The observed and predicted values of cracking area at the end of the year 2013 and their variations are given in Table 6.26. These values are also plotted against each other, as shown in Figure 6.26.

Table 6.26 Variability between observed and predicted cracking, Gr-5

Group-5 Pavement Sections	Cracking Area (%)		Variability (%)	Residual
	Observed	Predicted		
NH-4KA[Km.46-45]	17.00	18.00	5.88	-1.0
NH-4KA[Km. 82-83]	30.00	32.00	6.67	-2.0
NH-7KA[Km. 518-519]	6.00	4.00	33.3	2.0
NH-4KA[Km. 481.3- 482.3]	12.00	13.00	8.33	-1.0
NH-37AS[Km.177.3-178.3]	5.10	5.00	1.96	0.1

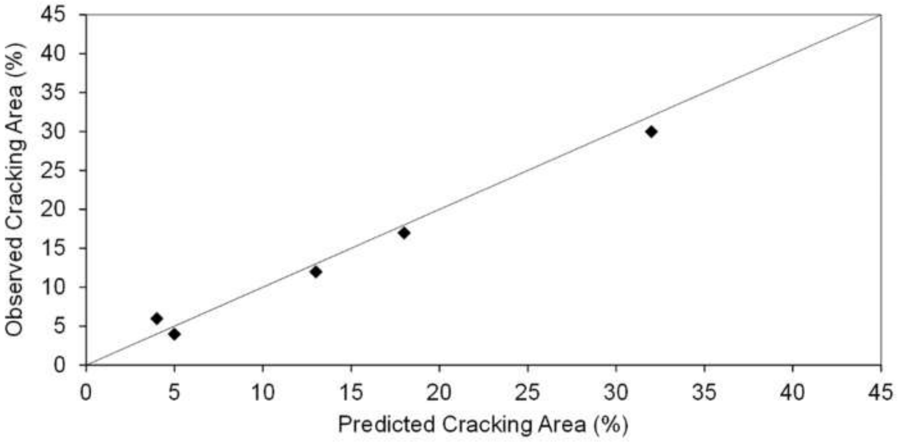


Figure 6.26 Observed v/s predicted cracking (%Area), Gr-5

The percentage variability obtained between the observed and predicted cracking area values ranges between 1.96 to 33.3 percent, which is slightly high probably due to wide variations in climate and traffic loading conditions to which the pavements are subjected. As observed in Figure 6.26, the plot between observed and predicted cracking area shows that the values are close to the LoE, indicating the adequacy and validity of the calibrated HDM-4 cracking progression model.

Ravelling Progression

The observed values of ravelling area at the end of the year 2013 for the five pavement sections of Group-5 are compared with those predicted by HDM-4 ravelling progression model. The observed and predicted values of ravelled area at the end of the year 2013 and their variations are given in Table 6.27. These values are also plotted against each other, as shown in Figure 6.27.

The percentage variability obtained between the observed and predicted ravelling area values ranges between 0.0 to 8.0 percent, which is quite reasonable considering the wide variations in climate and traffic that the pavements are subjected to. As observed in Figure 6.27, the plot between observed and predicted ravelling area shows that the points are quite near to the LoE, indicating the validity of the calibrated HDM-4 ravelling progression model.

Table 6.27 Variability between observed and predicted ravelling, Gr-5

Group-5 Pavement Sections	Ravelling Area (%) Observed	Ravelling Area (%) Predicted	Variability (%)	Residual
NH-4KA[Km.46-45]	15.00	15.50	3.33	-0.5
NH-4KA[Km. 82-83]	25.00	27.00	8.00	-2.0
NH-7KA[Km. 518-519]	0.80	0.80	0.00	0.0
NH-4KA[Km. 481.3- 482.3]	9.50	9.00	5.26	0.5
NH-37AS[Km.177.3-178.3]	2.00	2.00	0.00	0.0

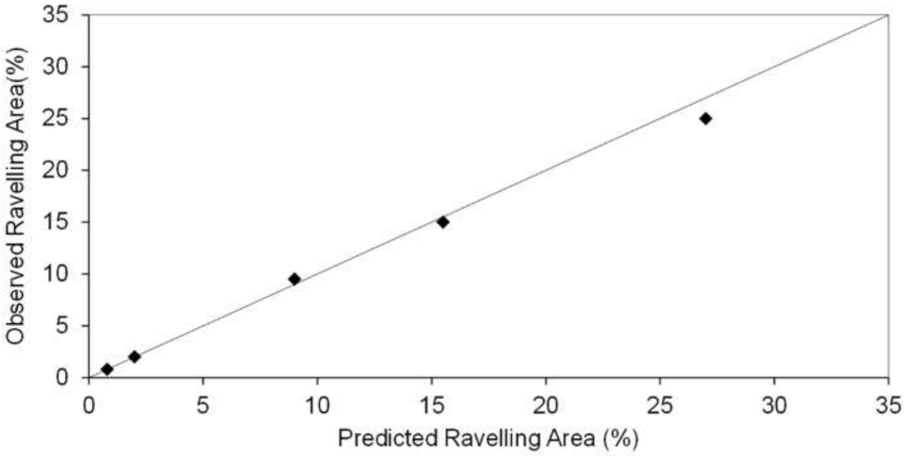


Figure 6.27 Observed v/s predicted ravelling (% Area), Gr-5

Pothole Progression

The observed numbers of standard pothole units at the end of the year 2013 for the five pavement sections of Group-5are compared with those predicted by HDM-4 pothole progression model. The observed and predicted values of pothole at the end of the year 2013 and their variations are given in Table 6.28. These values are also plotted against each other, as shown in Figure 6.28.

Table 6.28 Variability between observed and predicted pothole, Gr-5

Group-5 Pavement Sections	Pothole (nos.) Observed	Pothole (nos.) Predicted	Variability (%)	Residual
NH-4KA[Km.46-45]	1	1	0.0	0
NH-4KA[Km. 82-83]	4	4	0.0	0
NH-7KA[Km. 518-519]	0	0	0.0	0
NH-4KA[Km. 481.3- 482.3]	0	0	0.0	0
NH-37AS[Km.177.3-178.3]	0	0	0.0	0

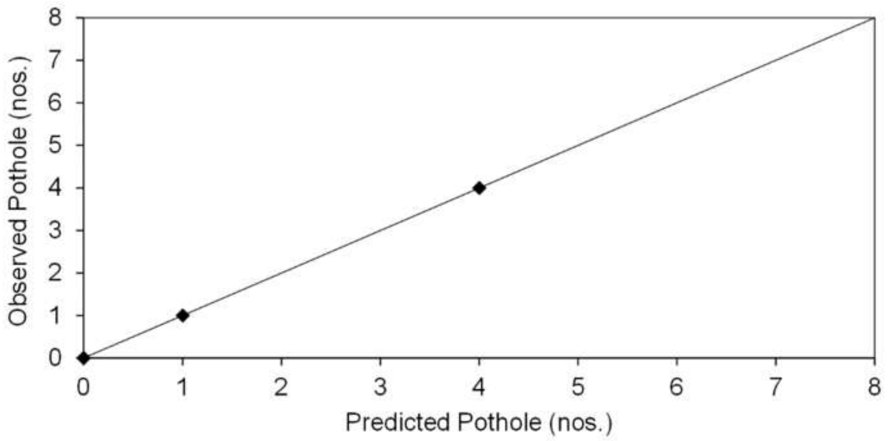


Figure 6.28 Observed v/s predicted pothole (nos.), Gr-5

No variability is observed between the observed and predicted pothole values. As observed in Figure 6.28, the plot between observed and predicted pothole shows that the values fall on the LoE, indicating the adequate validity of the calibrated HDM-4 pothole progression model for the present study.

Roughness Progression

The observed values of roughness at the end of the year 2013 for the five pavement sections of Group-5 are been compared with those predicted by HDM-4 roughness progression model. The observed and predicted values of roughness at the end of the year 2013 and their variations are given in Table 6.29. These values are also been plotted against each other, as shown in Figure 6.29.

Table 6.29 Variability between observed and predicted roughness, Gr-5

Group-5 Pavement Sections	Roughness (IRI m/km)		Variability (%)	Residual
	Observed	Predicted		
NH-4KA[Km.46-45]	3.50	3.66	4.37	-0.16
NH-4KA[Km. 82-83]	4.20	4.31	2.55	-0.11
NH-7KA[Km. 518-519]	2.80	2.61	7.28	0.19
NH-4KA[Km. 481.3- 482.3]	3.90	3.91	0.26	-0.01
NH-37AS[Km.177.3-178.3]	3.00	3.12	3.85	-0.12

The percentage variability obtained between the observed and predicted roughness values ranges between 0.26 to 7.28 percent, which is quite reasonable. As observed in Figure 6.29, the plot between observed and predicted roughness shows that the points lie in close proximity to the LoE, indicating the adequacy and validity of the calibrated HDM-4 roughness progression model.

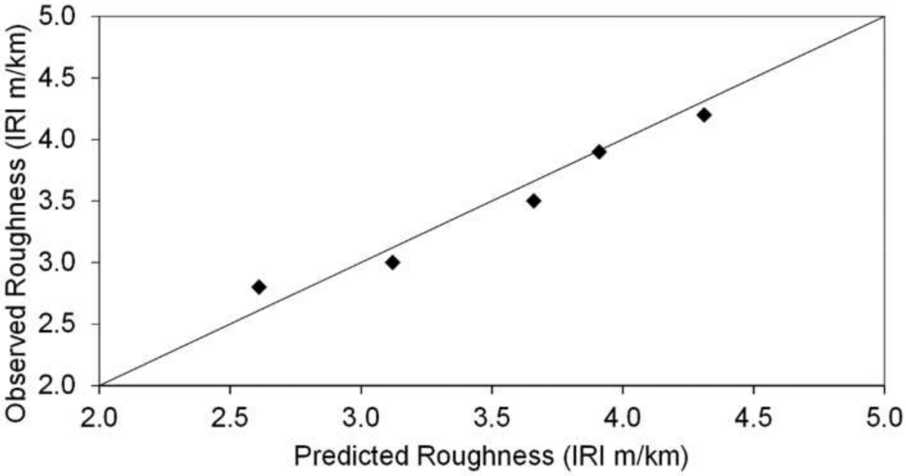


Figure 6.29 Observed v/s predicted roughness (m/km), Gr-5

Rut Depth Progression

The observed values of rut depth at the end of the year 2013 for the five pavement sections of Group-5 are compared with those predicted by HDM-4 rut depth progression model. The observed and predicted values of rut depth at the end of the year 2013 and their variations are given in Table 6.30. These values are also plotted against each other, as shown in Figure 6.30.

Table 6.30 Variability between observed and predicted rut depth, Gr-5

Group-5 Pavement Sections	Rut Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-4KA[Km.46-45]	3.50	3.40	2.86	0.1
NH-4KA[Km. 82-83]	5.10	5.20	1.96	-0.1
NH-7KA[Km. 518-519]	5.10	5.00	1.96	0.1
NH-4KA[Km. 481.3- 482.3]	3.00	2.80	6.67	0.2
NH-37AS[Km.177.3-178.3]	3.10	3.20	3.22	-0.1

The percentage variability obtained between the observed and predicted rut depth values ranges between 1.96 to 6.67 percent, which is quite reasonable. As observed in Figure 6.30, the plot between observed and predicted rut depth shows the points fall near to the LoE, indicating the adequacy and validity of the calibrated HDM-4 rut depth progression model for the present study.

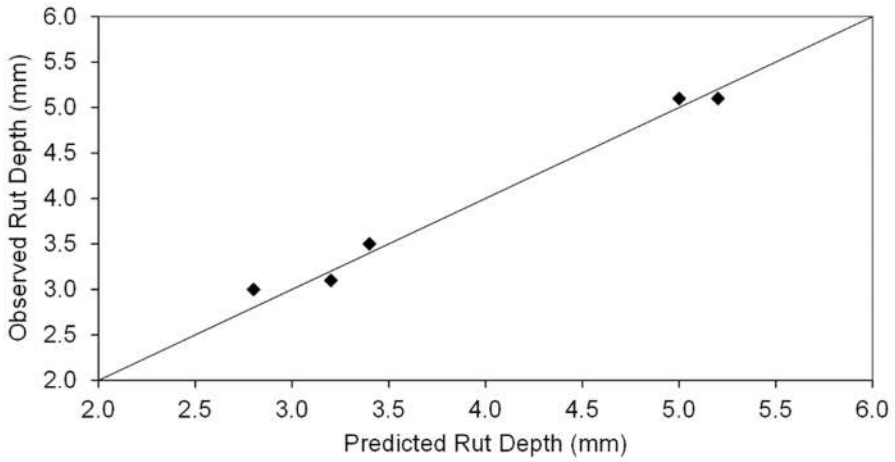


Figure 6.30 Observed v/s predicted rut depth (mm), Gr-5

Texture Depth Progression

The observed values of texture depth at the end of the year 2013 for the five pavement sections of Group-5 are compared with those predicted by HDM-4 texture depth progression model. The observed and predicted values of texture depth at the end of the year 2013 and their variations are given in Table 6.31. These values are also plotted against each other, as shown in Figure 6.31.

Table 6.31 Variability between observed and predicted texture depth, Gr-5

Group-5 Pavement Sections	Texture Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-4KA[Km.46-45]	0.60	0.58	3.33	0.02
NH-4KA[Km. 82-83]	0.61	0.62	1.64	-0.01
NH-7KA[Km. 518-519]	0.63	0.60	4.76	0.03
NH-4KA[Km. 481.3- 482.3]	0.62	0.59	4.84	0.03
NH-37AS[Km.177.3-178.3]	0.56	0.57	1.78	-0.01

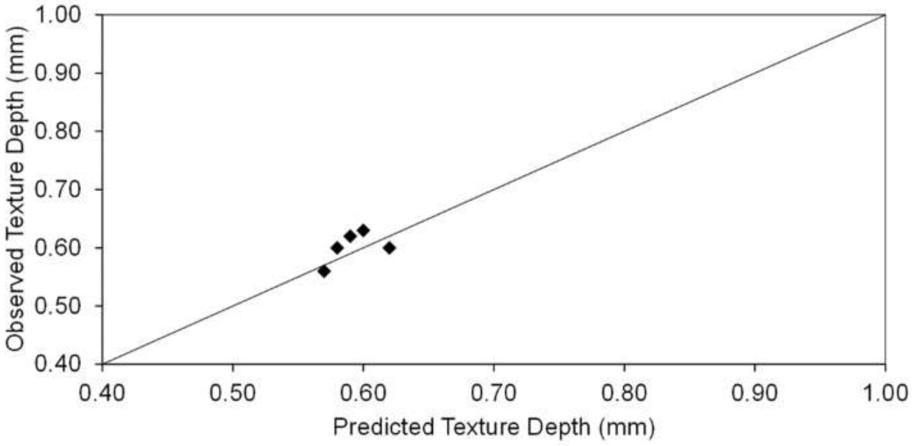


Figure 6.31 Observed v/s predicted texture depth (mm), Gr-5

The percentage variability obtained between the observed and predicted texture depth values ranges between 1.64 to 4.84percent, which is acceptable. As observed in Figure 6.31, the plot between observed and predicted texture depth indicates that the values show low deviation from the LoE.

Skid Resistance Progression

The observed values of skid resistance at the end of the year 2013 for the five pavement sections of Group-5 are compared with those predicted by HDM-4 skid resistance progression model. The observed and predicted values of skid resistance at the end of the year 2013 and their variations are given in Table 6.32. These values are also plotted against each other, as shown in Figure 6.32.

Table 6.32 Variability between observed & predicted skid resistance, Gr-5

Group-5 Pavement Sections	Skid Resistance (SFC50)		Variability (%)	Residual
	Observed	Predicted		
NH-4KA[Km.46-45]	0.50	0.50	0.00	0.00
NH-4KA[Km. 82-83]	0.50	0.49	2.00	0.01
NH-7KA[Km. 518-519]	0.55	0.56	1.82	-0.01
NH-4KA[Km. 481.3- 482.3]	0.55	0.53	3.64	0.02
NH-37AS[Km.177.3-178.3]	0.53	0.55	3.77	-0.02

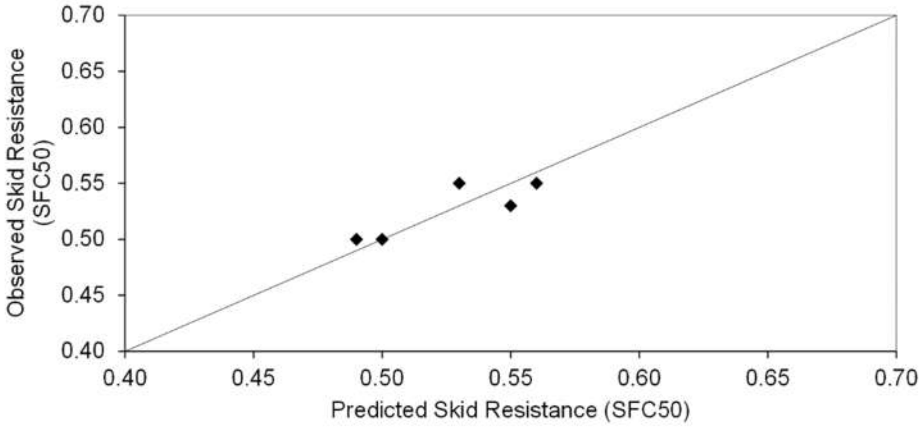


Figure 6.32 Observed v/s predicted skid resistance, Gr-5

The percentage variability obtained between the observed and predicted skid resistance values ranges between 0.0 to 3.77 percent, which is acceptable. As observed in Figure 6.32, the plot between observed and predicted skid resistance shows that the points fall close to the LoE, indicating the adequacy and validity of the calibrated HDM-4 skid resistance progression model for the present study.

6.2.6 Validation for Sections in Group-6

Cracking Progression

The observed values of cracking area at the end of the year 2013 for two pavement sections of Group-6are compared with those predicted by HDM-4 cracking progression model. The observed and predicted values of cracking area at the end of the year 2013 and their variations are given in Table 6.33. These values are also plotted against each other, as shown in Figure 6.33.

Table 6.33 Variability between observed and predicted cracking, Gr-6

Group-6 Pavement Sections	Cracking Area (%)		Variability (%)	Residual
	Observed	Predicted		
NH-4MH(539-538)	0.0	1.0	0.0	-1.0
NH-7MH(84.2-84.7)	10.5	10.0	4.76	0.5

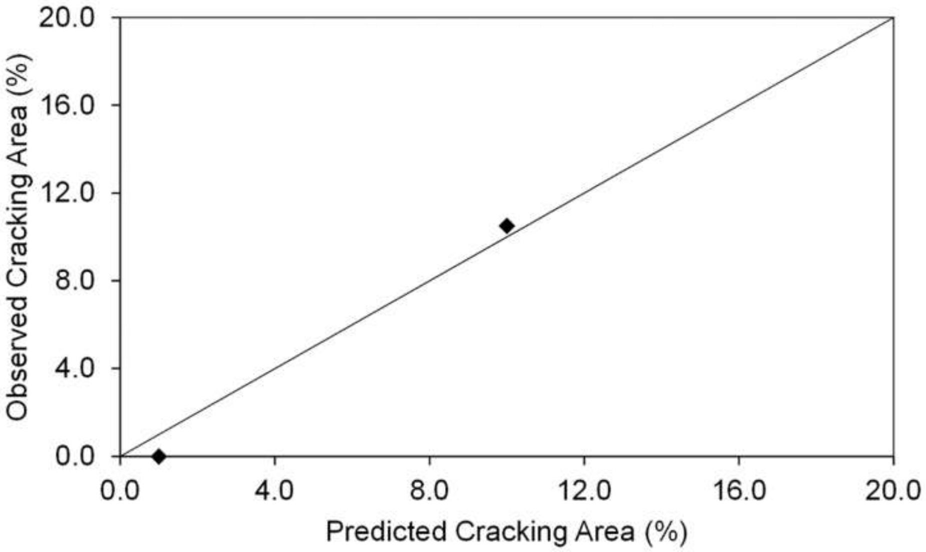


Figure 6.33 Observed v/s predicted cracking (% Area), Gr-6

The percentage variability obtained between the observed and predicted cracking area values ranges between 0.0 to 4.76 percent, which is quite reasonable wide variations in climate and traffic loading conditions to which the pavements are subjected. As observed in Figure 6.33, the plot between observed and predicted cracking area shows that the values are close to those.

Ravelling Progression

The observed values of ravelling area at the end of the year 2013 for the two pavement sections of Group-6are compared with those predicted by HDM-4 ravelling progression model. The observed and predicted values of ravelled area at the end of the year 2013 and their variations are given in Table 6.34. These values are also plotted against each other, as shown in Figure 6.34.

Table 6.34 Variability between observed and predicted ravelling, Gr-6

Group-6 Pavement Sections	Ravelling Area (%)		Variability (%)	Residual
	Observed	Predicted		
NH-4MH(539-538)	0.0	1.0	0.0	-1.0
NH-7MH(84.2-84.7)	6.2	6.0	3.22	0.2

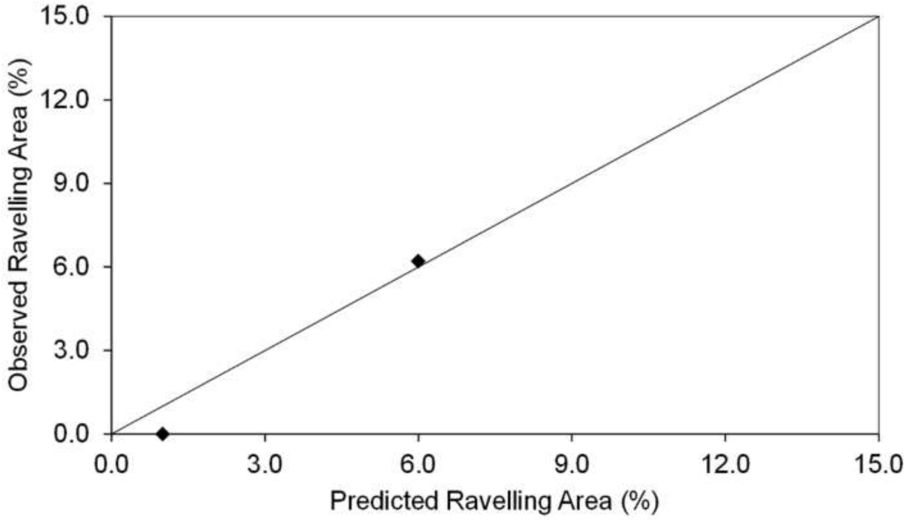


Figure 6.34 Observed v/s predicted ravelling (% Area), Gr-6

The percentage variability obtained between the observed and predicted ravelling area values ranges between 0.0 to 3.22 percent, which is quite reasonable. As observed in Figure 6.34, the plot between observed and predicted ravelling area shows that the points are quite near to the LoE, indicating the adequacy and validity of the calibrated HDM-4 ravelling progression model.

Pothole Progression

The observed numbers of standard pothole units at the end of the year 2013 for the two pavement sections of Group-6are compared with those predicted by HDM-4 pothole progression model. The observed and predicted values of pothole at the end of the year 2013 and their variations are given in Table 6.35. These values are also plotted against each other, as shown in Figure 6.35.

Table 6.35 Variability between observed and predicted potholes, Gr-6

Group-6 Pavement Sections	Pothole (nos.)		Variability (%)	Residual
	Observed	Predicted		
NH-4MH(539-538)	0	0	0.0	0
NH-7MH(84.2-84.7)	2	2	0.0	0

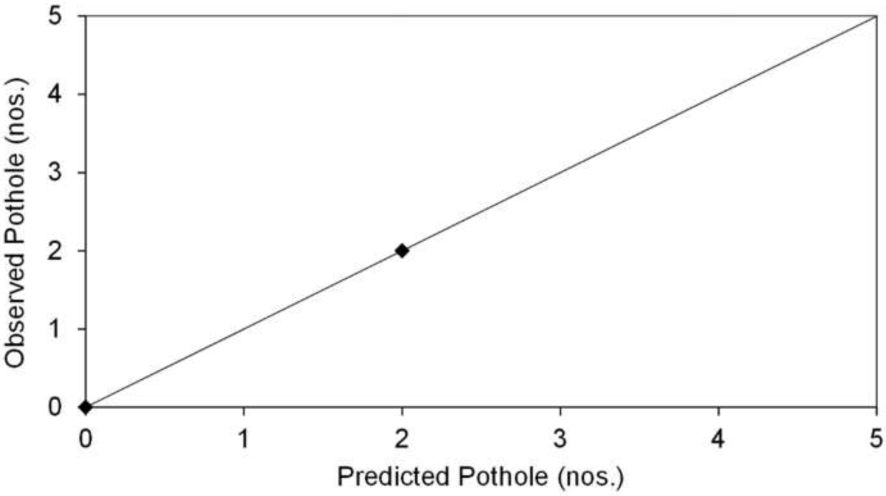


Figure 6.35 Observed v/s predicted pothole, Gr-6

There is no variability between the observed and predicted pothole values. As observed in Figure 6.35, the plot between observed and predicted pothole shows that the values fall near the LoE, indicating the adequacy and validity of the calibrated HDM-4 pothole progression model for the present study.

Roughness Progression

The observed values of roughness at the end of the year 2013 for the two pavement sections of Group-6are compared with those predicted by HDM-4 roughness progression model. The observed and predicted values of roughness at the end of the year 2013 and their variations are given in Table 6.36. These values are also been plotted against each other, as shown in Figure 6.36.

Table 6.36 Variability between observed and predicted roughness, Gr-6

Group-6 Pavement Sections	Roughness (IRI m/km)		Variability (%)	Residual
	Observed	Predicted		
NH-4MH(539-538)	3.3	3.2	3.03	0.1
NH-7MH(84.2-84.7)	3.0	3.1	3.33	-0.1

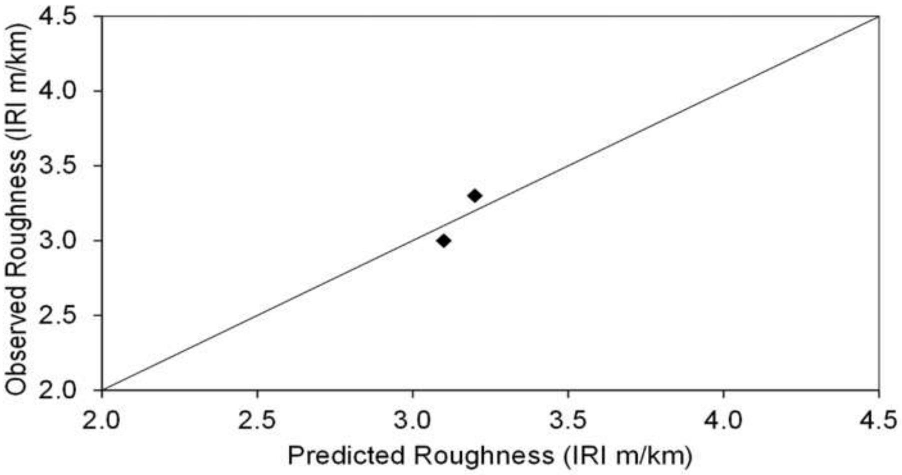


Figure 6.36 Observed v/s predicted roughness (m/km), Gr-6

The percentage variability obtained between the observed and predicted roughness is about 3 percent, which is quite low and hence acceptable. As observed in Figure 6.36, the plot between observed and predicted roughness shows that the points lie in close proximity to the LoE, indicating the adequacy and validity of the calibrated HDM-4 roughness progression model for the present study.

Rut Depth Progression

The observed values of rut depth at the end of the year 2013 for the two pavement sections of Group-6are compared with those predicted by HDM-4 rut depth progression model. The observed and predicted values of rut depth at the end of the year 2013 and their variations are given in Table 6.37. These values are also plotted against each other, as shown in Figure 6.37.

The percentage variability obtained between the observed and predicted rut depth values ranges between 4.35 to 5.88percent, which is quite reasonable. As observed in Figure 6.37, the plot between observed and predicted rut depth shows the points fall near to the LoE, indicating the adequacy and validity of the calibrated HDM-4 rut depth progression model for the present study.

Table 6.37 Variability between observed and predicted rut depth, Gr-6

Group-6 Pavement Sections	Rut Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-4MH(539-538)	3.4	3.2	5.88	0.2
NH-7MH(84.2-84.7)	2.3	2.2	4.35	0.1

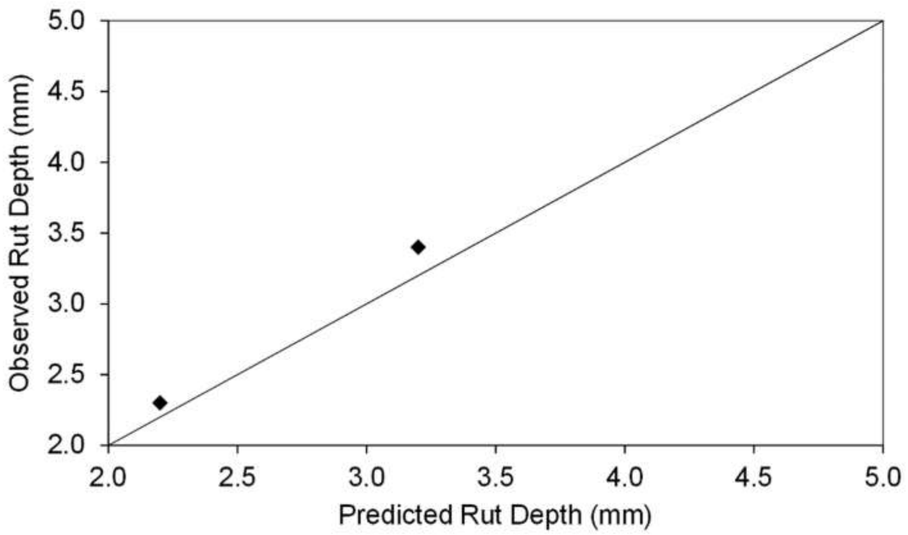


Figure 6.37 Observed v/s predicted rut depth (mm), Gr-6

Texture Depth Progression

The observed values of texture depth at the end of the year 2013 for the two pavement sections of Group-6are compared with those predicted by HDM-4 texture depth progression model. The observed and predicted values of texture depth at the end of the year 2013 and their variations are given in Table 6.38. These values are also plotted against each other, as shown in Figure 6.38.The percentage variability obtained between the observed and predicted texture depth values ranges between 5.88 to 12.07 percent, which is acceptable.

Table 6.38 Variability between observed and predicted texture depth, Gr-6

Group-6 Pavement Sections	Texture Depth (mm)		Variability (%)	Residual
	Observed	Predicted		
NH-4MH(539-538)	0.85	0.9	5.88	-0.05
NH-7MH(84.2-84.7)	0.58	0.65	12.07	-0.07

Skid Resistance Progression

The observed values of skid resistance at the end of the year 2013 for the two pavement sections of Group-6are compared with those predicted by HDM-4 skid resistance progression model. The observed and predicted values of skid resistance at the end of the year 2013 and their variations are given in Table 6.39. These values are also plotted against each other, as shown in Figure 6.39.

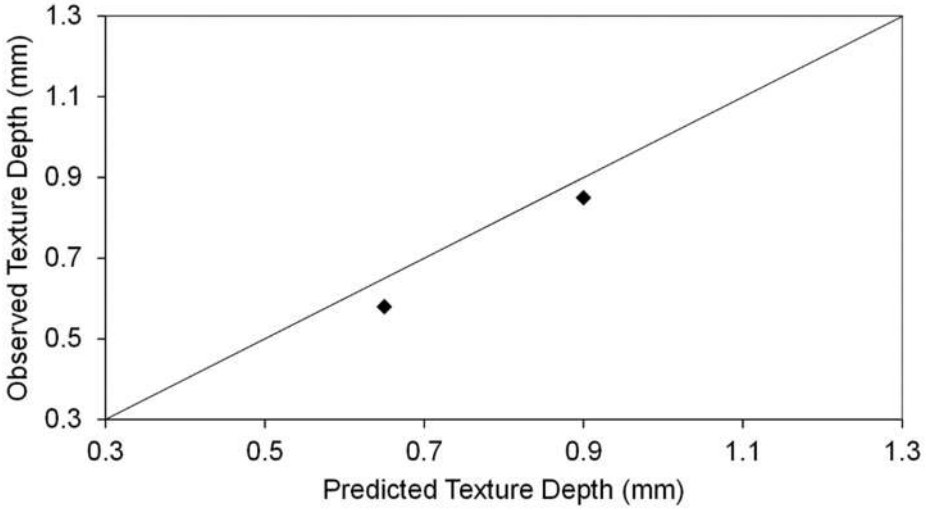


Figure 6.38 Observed v/s Predicted Texture Depth (mm), Gr-6

Table 6.39 Variability between observed & predicted skid resistance, Gr-6

Group-6 Pavement Sections	Skid Resistance (SFC50)		Variability (%)	Residual
	Observed	Predicted		
NH-4MH(539-538)	0.54	0.55	1.85	-0.01
NH-7MH(84.2-84.7)	0.50	0.45	10.0	0.05

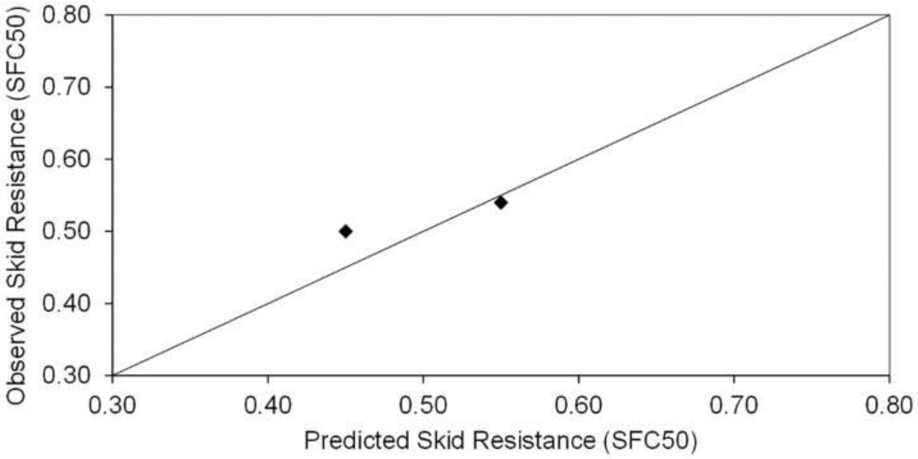


Figure 6.39 Observed v/s predicted skid resistance, Gr-6

The percentage variability obtained between the observed and predicted skid resistance values ranges between 1.85to 10.0 percent, which is acceptable. As observed in Figure 6.39, the plot between observed and predicted skid resistance shows that the points fall close to the LoE, indicating the adequacy and validity of the calibrated HDM-4 skid resistance progression model.

6.3 CHI-SQUARE TEST

The chi-square test was performed out to find out the significance of the difference between the observed and predicted distress values, in respect of various deterioration models. In probability theory and statistics, the chi-squared distribution (or, theX^2-distribution) with *k* degrees of freedom is the distribution of a sum of the squares of *k* independent standard normal random variables.

For the X^2 test, if $X^2_{calculated} < X^2_{critical}$ then the null hypothesis is accepted and vice-versa. X^2 is calculated as per Equation 6.2:

$$\chi^2 = \sum\left(\frac{(O_d - P_d)^2}{P_d}\right) \qquad (6.2)$$

where, O_d = observed distress, P_d = theoretical(predicted) distress.

The values of '$X^2_{calculated}$' determined for all the models were compared with tabulated '$X^2_{critical}$'for a level of significance of 5% and degree of freedom (N-1).The detailed analyses for all the observed and predicted distress parameters of the homogeneous pavement sections of individual groups were done and only the results of the are given in Table 6.40. From the results, it is observed that $X^2_{calculated} < X^2_{critical}$ for all deterioration models. Hence, the difference between the observed and predicted distress values is not statistically significant.

6.4 DISCUSSION ON VALIDATION OF MODELS

The calibrated HDM-4 pavement deterioration models were validated by comparing the values of distresses predicted by the HDM-4 model with those actually observed in the field. Further, a statistical analysis using the chi-square test was conducted to statistically determine the similarity of observed and predicted values of various distresses modelled in the study. The percent variability and residuals between observed and predicted values of distresses were obtained for all the pavement sections of all the six groups.

For Group-1 homogeneous pavement sections, a variation of 3.17 to 8.33% for cracking area, and 2.0 to 8.33% for ravelling area was obtained. No pothole was observed in these pavement sections during the study. However, the variability obtained for roughness was quite low in the range of 0.0 to1.79%. The variability obtained for rut-depth, texture depth and skid resistance ranged from 6.67 to 7.14%, 2.86 to 8.33%, and 1.79 to 3.64% respectively.

For Group-2 homogeneous pavement sections, a variation of 4.2 to 25.0% for cracking area, 2.4 to 25.0% for ravelling, and 0.0 to 40.0% for pothole was observed. However, the variability obtained for roughness ranged from 2.86 to10.0%. The variability values obtained for rut-depth, texture depth and skid resistance were quite low and ranged from 3.2 to 12.9%, 2.40 to 20.0%, and 4.29 to 9.23%, respectively.

For Group-3, a variability of 0.0 to 3.85% for cracking area, and 6.38 to 14.29% for ravelling area was observed. The variability obtained for roughness is ranged from 2.86 to 3.45%. The variability obtained for rut-depth, texture depth and skid resistance ranged from 3.33 to 4.65%, 1.47 to 1.54%, and 0.0 to 3.28% respectively. It can be clearly seen that the percent variability values were quite low.

For Group-4 homogeneous pavement sections, a variation of 0.0 to 10.0% for cracking area, and 7.14 to 18.18% for ravelling area was obtained. However, the variability obtained for roughness is ranged from 1.69 to 3.55%. The variability obtained for rut-depth, texture depth and skid resistance ranged from 2.00 to 8.57%, 1.43 to 6.67%, and 1.82 to 3.77% respectively.

For Group-5 pavement sections, a variation of 1.96 to 33.3% for cracking area, 0.0 to 8.0% for ravelling area, and 0.0% for potholes was obtained. However, the variability obtained for roughness is ranged from 0.26 to 7.28%. The variability obtained for rut-depth, texture depth and skid resistance ranged from 1.96to 6.67%, 1.78 to 4.84%, and 0.0 to 3.77% respectively. The above variations were reasonable and acceptable for a complex phenomenon such as the behaviour of pavements under varied conditions of traffic loading, climate, and other conditions.

For Group-6 pavement sections, variability values ranging from 0.0 to 4.76% for cracking area, and 0.0 to 3.22% for ravelling area were obtained, and but no variability was observed for potholes. However, the variability obtained for roughness ranged from 3.03 to 3.33%. The variability obtained for rut-depth, texture depth and skid resistance ranged from 4.35to 5.88%, 5.88 to 12.07%, and 1.85 to 10.0% respectively.

The observed and predicted values of all the distresses were also plotted in respect to the line of equality (LoE). It was found that the observed and predicted distresses were close to the LoE for the pavement sections of all the groups, indicating the adequacy and validity of all calibrated HDM-4 distress progression models. The analysis results also showed good agreement between observed and predicted distress values.

Chi-square test was performed to find difference between observed and predicted distress values for all groups and for all the deterioration models. The calculated '$X^2_{calculated}$' values for all models were compared with the tabulated '$X^2_{critical}$' values at 5% level of significance. It was concluded that there was no statistically significant difference between the observed and predicted distress values. Based on the results and analyses, the efficacy of the calibrated HDM-4 deterioration models for the high-speed road corridor network was established. Outcomes of the analyses strongly suggest that the developed models can be used for prediction of distresses and the development of maintenance management strategies for the selected high-speed road corridor network.

Table 6.40 Results of Chi-Square test

Model	Group-1		Group-2		Group-3		Group-4		Group-5		Group-6	
	X^2_{Cal}	DOF	X^2_{Cal}	DOF	X^2_{Cal}	DOF	X^2_{Cal}	DOF	X^2_{Cal}	DOF	X^2_{Cal}	DOF
Crack progression	0.152	3	1.623	5	0.001	2	0.117	2	1.259	4	1.025	1
Ravelling progression	0.078	3	0.752	5	0.173	2	0.573	2	0.192	4	1.007	1
Pothole progression	–*	3	0.821	5	–*	2	–*	2	0.000	4	0.000	1
Rut depth progression	0.065	3	0.182	5	0.020	2	0.033	2	0.024	4	0.017	1
Roughness progression	0.002	3	0.059	5	0.010	2	0.006	2	0.028	4	0.006	1
Texture depth progression	0.010	3	0.054	5	0.000	2	0.004	2	0.005	4	0.010	1
Skid resistance progression	0.001	3	0.026	5	0.001	2	0.001	2	0.002	4	0.006	1
$X^2_{Critical}$& DOF (From chi-square table)	7.815	3	11.070	5	5.991	2	5.991	2	9.488	4	3.841	1

Note: DOF – Degree(s) of freedom; * : No observations of potholes

6.5 SUMMARY

This chapter presented the validation of calibrated HDM-4 deterioration models for the selected pavement test sections on high-speed road corridor network in India. Validation was performed by a comparison of the distresses predicted using the HDM-4 models and those actually observed in the field. The prediction made by the pavement deterioration models incorporated in HDM-4 system for cracking, ravelling, pothole, rut depth, roughness, texture depth, and skid resistance were compared with those actually observed in the field. The observed and predicted values for all the distresses were also plotted with respect to the line of equality. This chapter also discussed the chi-square test conducted for validation, and the significance of the results obtained for validation of the calibrated HDM-4 pavement deterioration models. All deterioration models showed a good agreement between observed and predicted distress values. The results of the chi-square test also indicated that the difference between the observed and predicted distress values were not statistically significant at 5% level of significance. Hence, the models were successfully validated. In most of the cases, the variability between the observed and predicted values was found to be low (about 5–10%). Though some higher variations in the range of 20–40% were found between observed and predicted distresses in a few cases, these are justified due to large data collection from field, variation in the construction quality, and collection of data from secondary sources. The outcomes of the study indicated that the HDM-4 deterioration models developed in the study can be adopted and used for prediction of distresses for similar pavement sections for Indian conditions for development of maintenance strategies.

Chapter 7

SUMMARY AND CONCLUSIONS

7.1 SUMMARY

In order to boost the economic development of the country through improvement in the road infrastructure, Government of India has embarked upon a massive National Highway Development Program (NHDP) in the year 1999 which encompasses development of largest number of highway projects in seven phases, ever undertaken in the past.Since 1999, under the NHDP about 27,062 km length of National Highways has been completed from 2 lanes to 4/6 lanes, and presently 10,269 km length is under implementation. The first phase of NHDP included the development of Golden Quadrilateral (GQ), *i.e.* National Highways and Expressways connecting four metropolitan cities – Delhi, Mumbai, Chennai and Kolkata of India having an aggregate length of 5846 km. The second phase included the development of North–South and East–West corridors connecting Srinagar to Kanyakumari, and Silchar to Porbandar, respectively, comprising 7142 km length of National Highways. It also included road connectivity to major ports of the country by linking them with key national highways, comprising a length of 435 km. Road network developed under NHDP is considered as high-speed road corridor of India.

Meanwhile, unprecedented growth of road traffic, high variations in temperature, and the need for long lasting pavements have increased the use of modified bituminous binders, especially in wearing courses of flexible pavements—the most widely used pavement type in India. Crumb rubber-modified bitumen (CRMB) and polymer-modified bitumen (PMB) are the commonly used modified binders in India, and have been reported to yield better pavement performance than that obtained with conventional (unmodified) binders.

In light of the fact that the high-speed road corridors are a vital asset for socio-economic development of the country, adoption of a scientific approach for their maintenance is imperative. Pavements deteriorate with time due to the combined influence of traffic loads and environment, and the knowledge of these factors is essential in order to make predictions for future performance. Highway Development and Management (HDM-4) is one of the most useful and internationally recognised tools available for pavement performance and management analysis. The use of HDM-4 has been advocated by the government agencies in many developing countries, including India. The HDM-4 pavement deterioration models predict initiation and progression of various distresses under the combined influence of traffic, time, climate, and pavement structure and composition.Since the rate of initiation and propagation of each pavement distress is strongly dependent on local conditions, it is important to calibrate the HDM-4 models before their implementation. Calibration of these models requires an extensive time series data collection and its analysis for the existing local conditions.

At present, there are no well-calibrated and validatedHDM-4 pavement deterioration models available for the development of pavement maintenance and management strategy, especially for the high-speed road corridors of India constructed under the NHDP with modified binders in the wearing course. Against this backdrop, the present study attempted calibration and validation of HDM-4 pavement deterioration models for high-speed road corridors developed under the NHDP in India having modified bituminous binders in the wearing course. A total of 23 in-service flexible pavement road sections were identified from various sections of NHDP network. These sections were spread throughout the country and covered different climatic and environmental zones. Extensive field studies were carried to gather time series data consecutively for three years for all the 23 sections. The collected time series data included pavement distress data, traffic and axle load data, pavement crust composition data, pavement material characterisation data, temperature and rainfall data, and construction and maintenance history data.These 23 sections were divided into six groups based on similarity in crust thickness, pavement layer composition, and climatic conditions.

The acquired time series data were used as inputs in HDM-4 for calibration and validation of HDM-4 distress models.Using time series data of first two years, calibration factors were obtained for various deterioration models, including cracking, ravelling, potholing, rut depth, roughness, texture depth, and skid resistance. To arrive at the final values of the calibration factors, various statistical parameters indicating the measure of error between predicted and observed distressed were used. Validation of the calibrated pavement deterioration models was performed to test their efficacy. Statistical analysis using the chi-square test was used to perform validation by statistical comparison of the distress predictions made by the calibrated deterioration models with those actually observed on the selected pavement sections from the time series data for the third year.

7.2 CONCLUSIONS

The major conclusions of the present study are summarized as follows:

- The selected 23 test sections in this study covered almost entire country from east to west and north to south including the variations in climatic and environmental conditions, traffic loading, and the prevailing pavement layer compositions. These sections were classified under six groups based on similarity in pavement crust composition and climatic conditions.
- On the basis of time series data of first two years, final values of calibration factors were determined using the statistical parameters: root mean square error (RMSE), mean absolute error (MAE), RMSE to observations' standard deviation ratio (RSR), Nash-Sutcliffe efficiency (NSE), and correlation coefficient (R)**.**The calibration factors determined for the six groups for initiation and progression of various distresses considered in the study are presented in Table 7.1. The calibration factors for different pavement distresses were quite realistic and logical for flexible pavement sections of high-speed road corridor with modified binders in wearing course. It was also noted that calibration factors for deterioration models of distress initiation and progression found for Indian high-speed corridors with modified binders significantly varied from the default values of HDM-4.
- The HDM-4 pavement deterioration models for flexible pavements were calibrated based on the available time series data collected from the selected 23 pavement test sections. These models can be put to use for predicting the road conditions for modified bituminous road surfacing for planning future maintenance strategies.

Table 7.1 Calibration factors obtained from the present study

Distress model	Calibration factors					
	Gr-1	Gr-2	Gr-3	Gr-4	Gr-5	Gr-6
Cracking initiation	1.69	1.50	1.20	1.18	0.80	0.89
Cracking progression	0.47	0.64	0.20	0.70	0.79	0.84
Ravelling initiation	2.07	2.00	2.82	1.00	1.71	1.60
Ravelling progression	1.03	0.23	0.27	0.35	0.69	0.19
Pothole initiation	1.00	1.00	1.20	1.00	1.00	1.00
Pothole progression	1.00	0.01	0.67	0.67	0.98	1.00
Rut depth progression	1.61	0.77	0.63	0.97	0.51	0.73
Roughness progression	2.65	0.18	1.57	0.62	1.01	1.17
Texture depth progression	0.86	0.21	0.10	0.07	1.12	0.20
Skid resistance progression	1.00	0.92	1.33	2.83	3.40	3.33

Note: “Gr”: Group

- The percentage variability obtained between observed and predicted distresses during the validation of calibrated HDM-4 distress models was quite low in general. However, relative higher variability values found in few cases are fairly acceptable considering the inherent stochasticity and variability related to climate and traffic loading which affects the pavement deterioration phenomenon.
- Based on statistical analysis using chi-square test, no significant differences were found between the observed distresses and the ones predicted using HDM-4 models. The calibrated HDM-4 deterioration models were found to be suitable for prediction of distresses and the development of maintenance management strategies for high-speed national highway sections in India with modified binders in wearing courses.

The calibrated HDM-4 deterioration models developed in this study will be useful to the highway agencies in planning pavement maintenance strategies in a scientific manner, and to ensure rational utilization of limited maintenance funds. The models can also be used by various technical and administrative officials of the highway agency in making decisions at project level.

7.3 RECOMMENDATIONS FOR FUTURE WORKS

- HDM-4 deterioration models in this study can be used for preparing the maintenance and management programs for the high-speed road corridors and national highways of India.
- The scope of present study may be extended for the deterioration modelling for the high speed road corridor network having rigid pavements.
- Once the HDM-4 deterioration models for high speed road corridor network is implemented, this would serve as a window to National Highways Authority of India (NHAI), particularly for National Highways under NHDP. A similar kind of modelling may be done for other categories of roads such as multi-lane expressways using the inputs of this study.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LIST OF PUBLICATIONS

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1. **Deori, S.,** Choudhary, R., Tiwari, D., and Gangopadhyay, S. (2016). “Calibration of HDM-4 Models for Indian conditions of flexible pavement having modified bitumen in wearing course” *International Journal of Pavement Engineering*, Taylor and Francis, DOI: 10.1080/10298436.2016.1208196.

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1. **Deori. S.,** Choudhary, R., Tiwari, D., and Kumar, A. (2017). “Deterioration Modelling of Flexible Pavements using HDM-4”. National Conference on Roads and Transport (NCORT-2017), Indian Institute of Technology Roorkee, October 14 -15, Roorkee India.
2. **Deori, S.,** Choudhary, R., Tiwari, D., and Gangopadhyay, S. (2015). “Evaluation of Flexible Pavement Performance using Laser based Multifunction Automated Road Survey System”. 14th Annual International Conference on Asphalt, Pavement Engineering and Infrastructure, Liverpool John Moores University, February 11-12, Liverpool, UK.
3. **Deori, S.,** Choudhary, R., Tiwari, D., and Gangopadhyay, S. (2015). Field Performance Study on Modified Bituminous Road sections of National Highway Corridors”. 3rd Conference of Transportation Research Group of India (CTRG-2015), December 17-20, Kolkata, India.
4. **Deori, S.,** Choudhary, R., Tiwari, D., and Gangopadhyay, S. (2014). “HDM-4 Pavement Deterioration Modelling for Ahmedabad-Vadodara Expressway”. International Conference on Sustainable Civil Infrastructure (ICSCI-2014), Indian Institute of Technology (IIT) Hyderabad, October 17-18, Hyderabad, India.
5. **Deori, S.,** Choudhary, R., Tiwari, D., and Sitaramanjaneyulu, K. (2012). “Deterioration Modelling of Flexible Pavements with Modified Bitumen”. 3rd International Conference on Construction in Developing Countries **(**ICCIDC–III), July 4-6, Bangkok, Thailand.
6. **Deori, S.,** Choudhary, R., Tiwari, D., and Sharma, B.M. (2011). “A Critical Review of Flexible Pavement Performance Prediction Models”. International Conference on Advances in Materials and Techniques for Infrastructure Development (AMTID-2011), National Institute of Technology Calicut, September 28-29, Calicut, India.