

# **Comparison of Predictive Pavement Management Models (HDM-III, HDM-4, NZ dTIMS) for New Zealand Conditions**

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# Contents

<b>Acknowledgments</b> .....	4
<b>Abbreviations &amp; Acronyms</b> .....	8
<b>Executive Summary</b> .....	9
<b>Abstract</b> .....	14
<b>1. Introduction</b> .....	15
1.1 Background .....	15
1.2 Objectives of the Study .....	15
1.3 Scope of the Study .....	16
1.4 Structure of the Report .....	16
<b>2. Overview of NZ dTIMS &amp; HDM-4 Systems</b> .....	17
2.1 Introduction .....	17
2.2 NZ dTIMS Setup .....	17
2.3 HDM-4 System .....	18
2.4 HDM-4 Interface Requirement .....	19
2.5 HDM-4 Information Management System (HIMS) .....	19
2.6 HDM-4 Interface .....	20
<b>3. Study Approach &amp; Methodology</b> .....	23
3.1 Introduction .....	23
3.2 Desk Study Comparison (Stage 1) .....	23
3.3 Input Data Issues .....	23
3.4 Database Preparation .....	23
3.5 Parametric Study (Stage 2) .....	24
3.6 Initial calibration of HDM-4 Models (Stage 3) .....	24
3.7 Comparisons with Real Network Data (Stage 4) .....	25
<b>4. Desk Study Comparison of RD Models</b> .....	26
4.1 Introduction .....	26
4.2 Pavement Defects Modelled .....	26
4.3 Traffic Characteristics .....	27
4.4 Pavement Characteristics .....	27
4.5 Drainage Factor .....	29
4.6 Construction Quality .....	30
4.7 Cracking .....	30
4.8 Ravelling .....	35
4.9 Potholing .....	38
4.10 Rutting .....	40
4.11 Roughness .....	41
4.12 Edge Break .....	42
4.13 Pavement Texture .....	43
4.14 Conclusions .....	43
<b>5. Desk Study Comparison of WE Models</b> .....	45
5.1 Introduction .....	45
5.2 Maintenance Works & Resets .....	45
5.3 Routine Maintenance .....	46
5.4 Periodic Maintenance .....	49
5.5 Conclusions .....	53

<b>6.</b>	<b>Data Requirements for HDM-4 Models</b>	54
6.1	Introduction	54
6.2	Pavement Strength	54
6.3	Drainage and Environmental Factors	55
6.4	Traffic Characteristics	55
6.5	Construction Quality	55
6.6	Pavement Condition	56
6.7	Pavement History	57
6.8	Calibration Factors	57
6.9	Conclusions	58
<b>7.</b>	<b>HDM-4 Model Calibration Issues</b>	59
7.1	Introduction	59
7.2	An Overview of HDM Model Calibration	59
7.3	Calibration Factors	61
7.4	Additional Calibration Factors	62
7.5	Sensitivity of Calibration Factors	64
7.6	Calibration Data Issues	64
7.7	Calibration Methods	65
7.8	Conclusions	67
<b>8.</b>	<b>Assembly of Project Databases</b>	68
8.1	Introduction	68
8.2	Data Acquisition and Processing	68
8.3	Preparation of Synthetic Datasets for Parametric Study	69
8.4	Preparation of Synthetic Dataset for Sensitivity Analysis	70
8.5	Preparation of Synthetic Dataset for Initial Calibration	71
8.6	Preparation of Dataset for Model Outputs Comparison	72
<b>9.</b>	<b>Comparison of HDM Models Using Synthetic Dataset</b>	73
9.1	Introduction	73
9.2	Dataset Preparation and Model Setup	73
9.3	Drainage Factor	73
9.4	Structural Number	74
9.5	Cracking	75
9.6	Ravelling	78
9.7	Potholes	80
9.8	Rutting	81
9.9	Roughness	82
9.10	Comparison of WE Models	83
9.11	Conclusions	87
<b>10.</b>	<b>Initial Calibration of HDM-4 Models</b>	88
10.1	Introduction	88
10.2	Sensitivity Analysis of Calibration Coefficients	88
10.3	Initial Calibration	90
10.4	Customisation of HDM-4 Parameters to New Zealand Conditions	100
10.5	Conclusions	107

<b>11. Comparison of Models using Real Network Data</b> .....	110
11.1 Introduction .....	110
11.2 Preparation for Analysis .....	110
11.3 Comparison of Models for Do-Nothing Case .....	110
11.4 Comparison of Models for Routine Maintenance .....	111
11.5 Conclusions .....	113
<b>12. Conclusions</b> .....	114
<b>13. Recommendations</b> .....	117
13.1 Introduction .....	117
13.2 Potential Enhancements to NZ dTIMS Setup .....	117
13.3 Future Research Requirements .....	118
<b>14. Bibliography</b> .....	119
<b>APPENDICES</b>	
A. NZ dTIMS Pavement Deterioration Models .....	123
B. HDM-4 Pavement Deterioration Models .....	127
C. HDM-4 Interface .....	146
D. Drainage Factor (DF) Values .....	150

## Abbreviations & Acronyms

AADT	Annual Average Daily Traffic volume	LTPPM	Long-Term Pavement Performance Maintenance
AC	Asphalt concrete	MS	Microsoft
ACC	Auckland City Council	MMP	Mean Monthly Precipitation
ACA, ACT, ACW, ACX, ARV APOT, AVEB, RDM	- cracking, ravelling, pothole, edge break, rut depth parameters are listed in Tables	NA	Not Applicable
AGE	Age for preventive treatment	NDLI	ND Lea International, Canada
ALGENZ	Association of Local Government Engineers of NZ	NIWA	National Institute of Water & Atmospheric Research Ltd, NZ
AMGB	Asphalt Concrete Granular Base	NMT	Non-Motorised Transport
ARRB	Australian Road Research Board, NSW	NZ	New Zealand
BB	Benkelman Beam	Pavement	classifications listed in Table 4.2
DD	Drainage Deficiency	PEM	Project Evaluation Manual
DF	Drainage Factor	PIARC	Permanent International Association of Road Congresses
Calibration factors (Kf, Kddf, etc.)	Table 7.1	RAMM	Road Asset Maintenance & Management
Calibration coefficients (Kf, Kddf, etc.)	Table 12.1	RCA	Road Controlling Authority
CCT	Coefficient of Thermal Cracking	RD	Road deterioration
CDS	Construction Defects of Surfacing	RDM	Mean Rut Depth (mm)
CDB	Construction Defects of Base	RDS	Standard Deviation of Rut Depth
COMP	Compaction indicator	RIMS	Road Information Management System
CQ	Construction Quality indicator	ROMDAS	Road Measurement Data Acquisition System
CRT	Cracking Retardation factor	RRF	Ravelling Retardation factor
dTIMS	Deighton's Total Infrastructure Management System	RUE	Road User Effects
NZ dTIMS	NZ version of dTIMS	SCF	Skid resistance (sideways-force coefficient)
F	Occurrence distribution factor	SCRIM	Sideways-force Coefficient Routine Investigation Machine
FM	Freedom to Manoeuvre	SNC	modified Structural Number
FWD	Falling Weight Deflectometer	SNP	adjusted Structural Number
GEIPOT	Empresa Brasileira de Planejamento de Transportes, Ministry of Transport, Brasilia	ST	Surface Treatment
H	Thickness (mm)	STGB	Surface Treatment Granular Base
HCC	Hamilton City Council	TCP	Traditional Cetrius Paribus method
HDM-III	Highway Design & Maintenance Standards model, version III	TD	Texture Depth
HDM-4	Highway Development & Management model, version 4	TLF	Time Lapse Factor
HIMS	HDM-4 Information Management System	TRL	Transport Research Laboratory (>1992)
HSD	High Speed Data	TRRL	Transport & Road Research Laboratory, Crowthorne, UK (before 1992)
HTC	Highway & Traffic Consultants, Infrastructure Management Ltd, NZ	UK	United Kingdom
ICA	Roughness All Cracking;	US; USA	United States (of America)
ICW	Roughness Wide Cracking	Veh/day	vehicles per day
IRI	International Roughness Index (m/cm)	WE	Works (Maintenance) Effects
ISO HDM	International Organisation for Standardisation, Highway Development Model	YAX	Number of axles in million vehicles per year (Traffic parameter)

# Executive Summary

## Objective

The main objective of this project was to identify the enhancements carried out to the HDM-4 road deterioration (RD<sup>1</sup>) and works effects (WE) models compared with the earlier HDM-III models used in the NZ dTIMS Setup. Also the objective was to recommend the suitability of the HDM-4 RDWE models for inclusion in the NZ dTIMS Setup. It was also required to determine the future research requirements for the optimum utilisation of the HDM-4 models for predictive pavement modelling in New Zealand.

## Scope of Study

The study, carried out in 2000-01, was mainly focused on the comparison of RD and WE models available in the NZ dTIMS Setup and the HDM-4 system, Version 1.1. It was limited to paved bituminous roads, as these comprise most of the sealed roads in New Zealand.

## Methodology

To fulfil the objectives of the study, both a desk study and an experimental analysis with some field verification and data collection were carried out.

### • Desk Study Comparison of HDM Models (Stage 1)

In the desk study of the models, the differences between the RD and WE models in the NZ dTIMS Setup and the HDM-4 system were determined. For this purpose, reference was made to available literature on HDM models (HDM-III technical reports, HDM-4 documentation series, NZ dTIMS reports, ISOHDM study reports, ARRB reports on HDM-4 software testing, etc.). The additional data requirements for HDM-4 models in comparison to the NZ dTIMS Setup were analysed. Availability of these additional data in the RAMM database was also verified, and a summary of the additional data requirements for the HDM-4 models was prepared.

The latest version of HDM-4 software (Version 1.0) that was available at the start of the study could not be used as the reporting function had some bugs. Hence, an interface, HDM-4 Information Management System (HIMS), was enhanced to program HDM-4 RDWE models for this study.

Historical road inventory and condition data available in the RAMM database were taken as the primary source for the initial calibration and other tasks to complete the study. The datasets from the RAMM database were carefully chosen so that roads from different traffic, physical and environmental conditions were included. RAMM databases from five different Road Controlling Authorities (RCAs) representing different geographical and traffic conditions were acquired for the project. The necessary datasets were compiled as follows:

- Synthetic dataset for the parametric study of the HDM-4 models;
- Synthetic dataset for the sensitivity analysis of the calibration coefficients;
- Real network dataset for the initial calibration of RD and WE models; and
- Real network dataset for comparison of the predictions of HDM-4 and the NZ dTIMS Setup.

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<sup>1</sup> Abbreviations and acronyms are listed on p.8.

- **Comparison of HDM Models using Synthetic Dataset (Stage 2)**

The modifications to RD and WE models carried out in HDM-4, identified in the desk study as having potential use in New Zealand conditions, were further researched in the parametric study. A synthetic dataset was used in the parametric study to quantify the effects of various parameters (within the maximum and minimum range applicable to New Zealand conditions).

- **Initial Calibration of HDM Models (Stage 3)**

As the calibration of the HDM models generally involved many resources, the most sensitive parameters had to be identified so that they could be prioritised for calibration. Therefore, sensitivity analysis of the calibration factors was carried out using the traditional *ceteris paribus* (TCP) method. Impact elasticity of various calibration factors was determined through HDM-4 interface with a routine maintenance option. Next, the sensitivity analysis was carried out with a synthetic dataset prepared for this task. The sensitivity of the various calibration parameters was studied separately for asphalt concrete and surface treatment pavements for 5, 10, 15 and 20 years. The maximum (of the 4 values) of the impact elasticities was taken as representative of the given calibration coefficient.

The methodology to calibrate the additional calibration factors included in the HDM-4 system was neither available in the HDM-4 documentation series, nor could be found in any other published literature. The HDM-4 calibration guide (released in 2001) included the calibration issues relating to the HDM-III calibration factors. Hence, three preliminary calibration methods were developed for this study.

Initial calibration of the HDM-4 models was carried out using snap-shot and time-series methods to compare the outputs of the HDM-4 and NZ dTIMS models with the real network data. Some of the additional models and simplified models introduced in the HDM-4 system were customised to New Zealand conditions to verify the suitability of these models for New Zealand.

- **Comparison of HDM Models using Real Network Data (Stage 4)**

The model outputs were compared using the real network dataset to determine the similarity in the model predictions of the NZ dTIMS Setup and the HDM-4 system. The other objective was to make sure that the models and the interrelation between the models are properly simulated in the HDM-4 interface built into HIMS so that, if required, it can be incorporated later in the NZ dTIMS Setup.

## **Outcomes**

- With the enhancements carried out to RD and WE models, the HDM-4 model is more suitable than HDM-III for detailed analysis at network level as well as for project level analysis, provided that data of required quality is available.
- A comprehensive pavement classification philosophy has been introduced in HDM-4 models which enables the appropriate pavement type to be chosen for analysis. To implement this, some changes in the NZ dTIMS Setup will be required if the HDM-4 approach for pavement classification is adopted.
- In the HDM-4 system, pavement strength (SNP) is estimated by considering the effects of seasonal changes and drainage condition. The effects of the drainage condition are quantified in HDM-4 by introducing the drainage factor (DF). Research work is needed to establish the effects of drainage on pavement strength.

- Continuous variables (CDS and CDB) are introduced in HDM-4 models to express construction defects, in place of a flag value (CQ) presently used in the NZ dTIMS Setup. These variables were very sensitive to the surface distress prediction and can be used to reflect quality of construction. Research study is needed to outline procedure to evaluate these indicators.
  - Transverse thermal cracking (which is an additional cracking modelled in HDM-4) is considered not to be applicable to New Zealand conditions. However the reflection cracking model is proposed for inclusion in HDM-4, but a study is required to verify the suitability of this model to New Zealand conditions.
  - In HDM-4, the traffic parameter (YAX) was introduced in the ravelling progression model to consider the effect of traffic volume on ravelling progression, but a study is required to verify the suitability of this model (once it is available) to New Zealand conditions.
  - Pothole component of roughness has been extensively modified in HDM-4 by introducing time lapse in patching of potholes and freedom to manoeuvre factors. However, the effect of this change appeared to be minimal for New Zealand conditions considering the current routine maintenance practice of 100% patching of potholes as soon as they appear.
  - Rutting caused by plastic deformation can be estimated with the enhanced HDM-4 rutting model. However, detailed study on calibration of this model is required. Another component of rutting, i.e. the surface wear component, was found to be not relevant for New Zealand conditions.
  - In New Zealand, edge break is observed on some narrow roads, particularly in rural areas. Thus the edge break model should be considered for inclusion in the NZ dTIMS Setup. Further study on the applicability of the edge break model to New Zealand conditions is needed together with development of an additional shoulder deterioration model.
  - The calibration coefficients listed in the following table were used for the comparison of models using real network data in this study. These calibration factors were based on the data considered for the present study and can generally be used elsewhere in New Zealand. However, regional calibration is recommended for best results.
  - Comparison of the model predictions with the real network dataset has shown good correlation between the HDM-4 system and the NZ dTIMS Setup.
  - Routine maintenance activities have extensively been modified in the HDM-4 system. The effects of drainage maintenance, patching of wide structural cracking, and time lapse in pothole patching appeared to be significant in the predicted deterioration.
- To obtain quantitative comparisons, model predictions of NZ dTIMS Setup and HDM-4 models were compared using a common input dataset.
- The reset models for pavement strength and roughness have been modified in HDM-4 for resurfacing and overlay. The customisation carried out in this study revealed that these new models could be calibrated for the New Zealand condition with the data that is presently available. The default values provided in the HDM-4 for these models can be considered at the national level and can easily be customised to local conditions at regional level.

### Initial Calibration Coefficients recommended for New Zealand conditions

Calibration Factor	RD Model	Values
Kf	Wet/dry season SNP ratio	1
Kddf	Drainage factor	1
Kcia	All structural cracking initiation	1
Kcpa	All structural cracking progression	1
Kciw	Wide structural cracking initiation	1
Kcpw	Wide structural cracking progression	1
Kcit	Transverse thermal cracking initiation	0
Kcpt	Transverse thermal cracking progression	0
Kvi	Ravelling initiation	1
Kvp	Ravelling progression	0
Kpi	Pothole initiation	1
Kpp	Pothole progression	1
Keb	Edge break	1
Krid	Initial densification of rutting	0.5
Krst	Structural deformation of rutting	0.7
Krpd	Plastic deformation of rutting	0.7
Krsw	Surface wear of rutting	0
Kgm	Environmental coefficient of roughness	0.56
Ksnpk	Adjusted structural number for roughness	1
Kgs	Structural component of roughness	1.7
Kgr	Rutting component of roughness	2.4
Kgp	Patching component of roughness	0.03
Ktd	Texture depth progression	1
Ksfc	Skid resistance	1
Ksfcs	Speed effects of skid resistance	1

#### Potential Enhancement of the NZ dTIMS Setup

Two options for the enhancement of the NZ dTIMS Setup are possible:

- Full implementation of HDM-4 RDWE models for bituminous pavements; and
- Partial implementation of HDM-4 RDWE models.

The modelling approach in HDM-4 has been changed and hence, for its full implementation, the basic setup of NZ dTIMS needs to be changed. Such modifications are desirable during the switch over of the software platform from dTIMS 6.1 to dTIMS CT (the new international versions of dTIMS).

Partial enhancement of the NZ dTIMS Setup can be done by incorporating some of the additional models or by replacing available models with HDM-4 models. Models that could be considered for this implementation are as follows:

- The concept of adjusted structural number in HDM-4 can be introduced into the NZ dTIMS Setup, and this will include the effects of seasonal changes and drainage condition on pavement performance.



- The drainage factor can be included in the NZ dTIMS Setup to quantify the annual drainage condition. This factor would allow the various RCAs to recognise annual drainage maintenance requirements..
- CQ used in the NZ dTIMS Setup can be replaced by the continuous variables CDS and CDB that were introduced in the HDM-4 system. This would require incorporation of some of the HDM-4 models in the NZ dTIMS Setup.
- Modified reset relationships for pavement strength, rutting and roughness for periodic maintenance treatments can be included in the works effects modelling of the NZ dTIMS Setup.

### **Future Research Requirements**

The further research required for the optimum use of the HDM-4 models have been identified, together with their priorities, as follows:

- Research is needed to establish the effect of drainage on the pavement strength. It should also include the effect of seasonal variation on pavement strength (high priority).
- Research to define the procedure for defining the values of continuous construction quality indicators (CDS and CDB) for different surface materials and construction quality (high priority).
- Study on the validity and calibration of reflection cracking model to New Zealand pavements (medium priority).
- Study on the rutting related to plastic deformation and structural deformation in New Zealand pavements (medium priority).
- Study on the HDM-4 ravelling initiation and progression models, and their applicability to New Zealand conditions (low priority).
- Study on the applicability of the edge break model to New Zealand conditions by developing a shoulder deterioration model (high priority).

## Abstract

This report describes the results of a project, carried out in 2000-01, to investigate the enhancements carried out to road deterioration (RD) and works effects (WE) models included in the predictive pavement modelling system, HDM-4, with respect to the HDM-III models used in the NZ dTIMS Setup used now in New Zealand.

The main objective was to recommend the suitability of the HDM-4 RDWE models for the inclusion in the NZ dTIMS Setup. The study also recommended future research requirements for the optimum utilisation of the HDM-4 models for use in New Zealand.

The study was carried out in four stages i.e. desk study comparison of models, comparison of models using a synthetic dataset, initial calibration of models, and comparison of calibrated models using a real network dataset.

RAMM databases for five different road-controlling authorities (RCAs) were compiled for the required analyses. Synthetic datasets and real network datasets were prepared considering the values relevant to New Zealand conditions. Sensitivity analysis was carried out for calibration factors to identify the most sensitive calibration factors.

Special consideration was given to prepare the dataset for initial calibration, with detailed field verification and some additional data collection undertaken. Initial calibration of the HDM-4 models used snap-shot and time-series methods. The outputs from both NZ dTIMS and HDM-4 were compared using the real network data, and the HDM-4 models were found to include some of the features that are suitable for New Zealand conditions. HDM-4 models were found to be more flexible than the HDM-III models used in the current NZ dTIMS Setup. The potential enhancement of the current NZ dTIMS Setup and further research needs were prepared.

## **1. Introduction**

### **1.1 Background**

A large portion of the road controlling authorities (RCAs<sup>1</sup>) in New Zealand are now using the NZ dTIMS Setup for predicting the future maintenance needs of their road networks, at early stages before pavement deterioration becomes a major and costly problem. This recently (1998) implemented predictive modelling system simulates the future pavement deterioration based on the existing condition. In the process it defines various maintenance strategies (sequence of treatments for a given analysis period) based on the intervention criteria, and recommends a strategy for each section through optimisation of network condition or total transportation costs. Hence, road deterioration (RD) and works effects (WE) models form the basis of the predictive modelling system, and it is essential that both RD and WE models used in the NZ dTIMS Setup are based on the latest developments and are well verified. The dTIMS is an open environment software platform developed by Deightons Associates, Canada, for road maintenance management. A New Zealand version, NZ dTIMS, is built on the dTIM platform with HDM-III and some other locally developed pavement deterioration models suitable for New Zealand conditions.

Currently (2001), the NZ dTIMS Setup is based on the earlier HDM-III road deterioration and maintenance effects (RDWE) models for predicting the road condition. Since its release in 1987, HDM-III models have been used world-wide in numerous roading projects covering a range of climates and conditions. Their basic structure and the pavement performance predictions derived from the models have been widely confirmed by use. However, during implementation of the HDM-III system, limitations of the models in terms of scope have been identified. Subsequently, the ISO HDM study (International Organisation for Standardisation Highway Development Model) was carried out. Based on this study the HDM-4 software was released by PIARC (Permanent International Association of Road Congresses) in mid 2000.

The RD and WE models used in the HDM-4 system consist of some enhancements to the HDM-III models. It was essential to verify the suitability of these enhancements to New Zealand pavement conditions.

### **1.2 Objectives of the Study**

The main objective of this project, carried out in 2000-2001, was to determine the improvements carried out in the HDM-4 models with respect to HDM-III models now being used in the NZ dTIMS Setup. The objectives of the study are:

- To compare the differences between the pavement deterioration models included in the HDM-4 system and those from the NZ dTIMS Setup;

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<sup>1</sup> Abbreviations and acronyms are listed on p.8.

- To carry out initial calibration, and compare the pavement deterioration predicted by HDM-4 models with that from the NZ dTIMS Setup;
- To recommend the suitability of different models available in HDM-4 for implementation in the NZ dTIMS Setup;
- To recommend the future research requirements for the possible utilisation of the HDM-4 models for predictive pavement modelling in New Zealand.

### **1.3 Scope of the Study**

The study was mainly focused on the comparison of RD and WE models available in the NZ dTIMS Setup and the HDM-4 system. This study was based on HDM-4 Version 1.1, the latest version of HDM-4 available during the study period. Although the HDM-4 Version 1.2 was released in May 2001, it could not be used since the major portion of this study had already been completed.

The study was limited to paved bituminous roads as these comprise most of the sealed roads in New Zealand. The conclusions and recommendations were drawn mainly based on the desk study and analysis of the data that are available in RAMM. Only some additional data were collected to complement the available data.

### **1.4 Structure of the Report**

The report has been divided into 1. the main report and 2. appendices. The main report focuses on the study methodology, data requirements, data preparation, initial calibration, comparison of models, conclusions and recommendations of the study, while the appendices provide the supporting documentation and analyses used or generated as part of the study. The main report consists of 13 chapters as outlined below.

Chapter 1 discusses the objectives and scope of the study, followed by an overview of the NZ dTIMS Setup and HDM-4 system in Chapter 2. The basic approach and methodology followed in the study is discussed in Chapter 3.

The desk study comparison of the RD and WE models is discussed in Chapters 4 and 5. Additional data input requirements together with the availability of these data in the RAMM database are analysed in Chapter 6. Chapter 7 discusses the HDM-4 model calibration issues, and assembly of the project databases is given in Chapter 8.

The comparison of the RD and WE models using a synthetic dataset is given in Chapter 9. Initial calibration of the models is presented in Chapter 10, and comparison of the HDM-4 system and NZ dTIMS Setup using real network data in Chapter 11.

The conclusions of the study are given in Chapter 12. Recommendations on potential enhancements of the NZ dTIMS Setup and the future research requirements are given in Chapter 13.

## **2. Overview of NZ dTIMS & HDM-4 Systems**

### **2.1 Introduction**

The NZ dTIMS Setup and HDM-4 systems are both predictive pavement management system packages used for planning and programming of future maintenance needs. The specific features of these systems, their current status of development, and the difference between the two systems are briefly described.

### **2.2 NZ dTIMS Setup**

#### **2.2.1 Background**

The NZ dTIMS Project started in late 1998 under the stewardship of the RIMS (Road Information Management System) Group. The RIMS Group is comprised of members from Transfund New Zealand, Transit New Zealand, and members representing Local Government. It operates under the Association of Local Government Engineers of New Zealand (ALGENZ).

A pragmatic approach was followed in developing the NZ dTIMS Setup. It was decided to develop the preliminary system based on the available information, and to improve the system with ongoing operational research and experience in using the system. A flexible software platform based on Deightons Total Infrastructure Management System (dTIMS), developed by Deightons Associates, Canada, was used as a platform to develop the NZ dTIMS Setup which was based on:

- HDM-III RDWE models for pavement performance prediction;
- Intervention criteria for maintenance works based on local practice; and
- Optimisation based on Transfund's (1999) Project Evaluation Manual (PEM).

Supporting dTIMS interface program software was developed to convert the RAMM (Road Asset Maintenance & Management) data to HDM (Highway Development Management) format for preparing the dTIMS input file. Similarly a 'Pavement strength' program was used for calculating the required strength parameters for the dTIMS analysis.

#### **2.2.2 RD and WE Models in NZ dTIMS Setup**

The pavement deterioration models currently used in the NZ dTIMS Setup basically consist of HDM-III models together with some locally developed models. These include:

- Cracking initiation and progression;
- Potholing initiation and progression;
- Rutting;
- Roughness;
- Texture depth;
- Skid resistance; and
- Gravel road models.

It should be appreciated that ravelling (scabbing) progression is turned off in the current NZ dTIMS Setup, as in its present form it was unsuitable for New Zealand conditions (HTC 1999). A brief note on the RD models used in the NZ dTIMS Setup is given in Appendix A.

## **2.3 HDM-4 System**

### **2.3.1 HDM-4 Study**

The ISO HDM study was conducted to collate the results of research carried out throughout the world after the release of the HDM-III models, and to come up with specifications for the improved models (NDLI 1995). Based on the recommendations of this ISOHDM study, HDM-4 models have been released in 2001. Most of the HDM-III Pavement Deterioration and Works Effects (NZ) models have been improved and some new models were added in HDM-4.

The additional features available in HDM-4 in comparison with the HDM-III system include (Morosiuk et al. 2000):

- A greater range of physical environments (climatic zones including cold/freezing climates);
- Rigid and block pavements, and a wider range of flexible pavements;
- Deterioration and maintenance of side-drains and their effects on pavement strength;
- Texture depth and skid resistance models;
- Edge break, particularly on narrow roads;
- A broader range of routine maintenance operations and their effects; and
- A broader range of improvement and new construction works.

### **2.3.2 HDM-4 Software**

The HDM-4 software was developed, and is based on the findings of the ISOHDM study. The following components are available in the HDM-4 software:

- Road deterioration modelling;
- Road works effects modelling;
- Road user effects modelling;
- Social and environmental effects modelling; and
- Economic analysis.

The HDM-4 version 1.0 was released in mid 2000. The updated version 1.1 was released in February 2001, and version 1.2 in May 2001. In each new version, the errors on the coding have been rectified and some modifications have been incorporated.

The HDM-4 version 1.1 was used in our study, as version 1.2 became available only towards the end of the study period.

### **2.3.3 HDM-4 Models**

It is necessary to distinguish between HDM-4 software and HDM-4 models. HDM-4 models are the principles and expressions describing the modelling procedure, whereas HDM software includes these models programmed in an application for running analyses. The main objective of this project is to study the appropriateness of the HDM-4 models with regards to New Zealand conditions, not of the software itself. Models documented in the draft HDM-4 series document (Morosiuk et al. 2000) were mainly used in the study. The revised draft (Morosiuk & Riley 2001) was only briefly reviewed because it was not available until late in this project.

A brief description on HDM-4 RD models is given in Appendix B.

## **2.4 HDM-4 Interface Requirement**

As stated earlier, the primary objective of the study was to verify and to determine the applicability of the HDM-4 models. The proposal suggested that the HDM-4 software would be used for the comparison between the HDM-4 and the NZ dTIMS Setup. However, the HDM-4 software could not be used as the reporting function of HDM-4 software version 1.0 (which was the only version available at the start of the project) contained some errors.

Hence the decision was to build an independent platform to make it possible to:

- Complete the project on time as there was no surety that the updated software version would have these bugs fixed;
- Change any expressions or parameters generally required for this kind of study;
- Compare various additional outputs (generally not provided by HDM-4 software but required for the comparison);
- Verify the reasonableness of the available HDM-4 models in the literature and the HDM-4 software; and
- Determine the complexity in incorporating the models into dTIMS CT, which is expected to be released in early January 2002.

It was decided to programme the HDM-4 models in the HDM-4 Information Management System (HIMS) as it is of similar architecture as the dTIMS CT and has facilities for HDM-4 data processing.

## **2.5 HDM-4 Information Management System (HIMS)**

HIMS was originally developed by HTC to prepare the input data for HDM-4 models and for calibration of the models. It is a modern asset database management system with the capability of:

- Preparing input data for HDM-4 software;
- Automatic sectioning and data processing including dynamic segmentation;
- Setting up and running the incremental models;
- Treatment selection intelligence; and

- Analysing the outputs of HDM-4 analysis.

Most of the functions required to programme the models were already available, but some modifications were needed for this project.

Various models and expressions can be programmed in HIMS. The following parameters are available for this purpose:

- **Table (Field)** – stores data for calculations;
- **Constants** – contains data of various constants used in formula;
- **Lookup table** – contains tables for various coefficients for different conditions;
- **Lookup expression** – required for converting lookup table into formula;
- **Formula** – consists of expressions defined for calculations;
- **Filter** – filters out a particular formula for specified criteria;
- **Variables** – combines the filter and formula;
- **Treatments** – defines various treatments;
- **Interventions** – defines intervention criteria through filters;
- **Resets** – contains resets to be applied after treatment;
- **Model** – defines the calculation properties (i.e. initiation, incremental or calculation);
- **Utilities** – used to run the analysis; and
- **Reports** – used to generate reports for the analysis.

HIMS functions need to be carried out in a sequence, as shown in Figure 2.1.

## 2.6 HDM-4 Interface

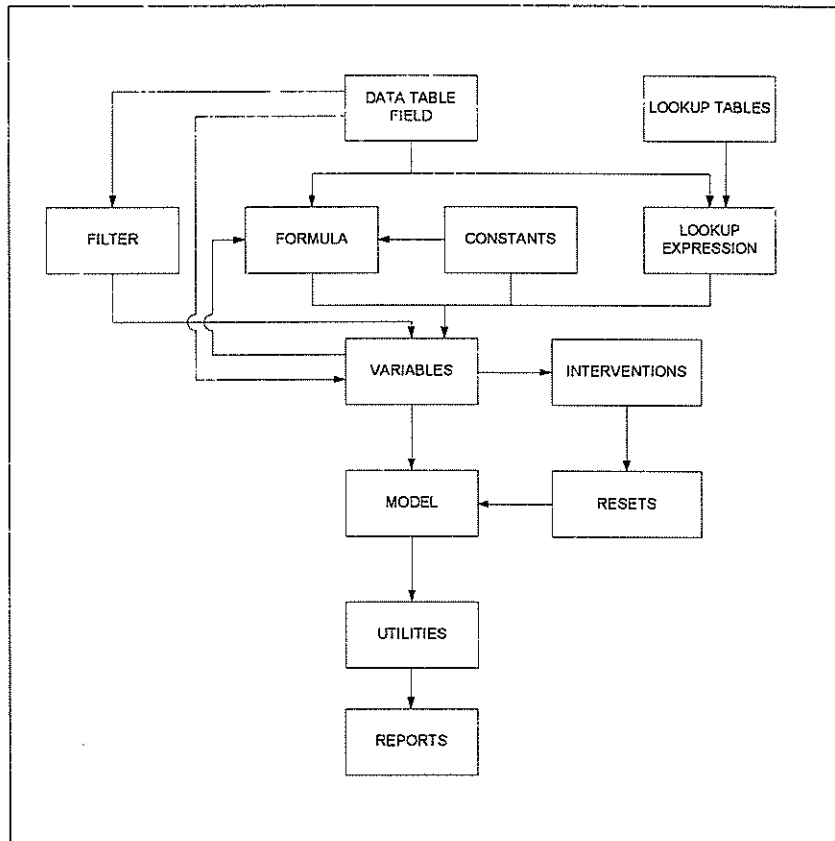
Building the HDM-4 Interface was not an easy task. It should be appreciated that the incorporation of HDM-4 logic and replication of RD and WE models in the HDM-4 Interface required a lot of effort. The sequence followed to build the HDM-4 Interface into HIMS is described below (Figure 2.1).

An integrated data input table was prepared by combining the HDM-4 network and traffic tables. This table contained all the input fields that are required for HDM-4 RD and WE models.

The model coefficients were included in HIMS in the form of ‘Look up tables’ and ‘Constants’. Model coefficients that change value frequently, depending upon the given scenario, were programmed under ‘Look up tables’. ‘Look up expressions’ were used to define the conditions to make sure that the right model coefficients were applied for a given expression. The coefficients that did not change their value frequently were defined as ‘Constants’.



Figure 2.1 Sequence of HIMS functions.



Model relationships were programmed through ‘Formula’. Only those expressions required for the present study were replicated in the HDM-4 Interface. ‘Filter’ was included to apply the model expressions for the applicable conditions. ‘Variables’ were defined with the combination of ‘Formula’ and ‘Filter’. Variables allow the value for different parameters to be calculated for each year of the analysis period.

‘Interventions’ criteria were set up to apply a treatment at a specific condition. Triggers are comprised of a group of Filters. ‘Resets’ were defined through ‘Formula’ to reset the pavement distresses after the application of a treatment.

At the end of the process the model was defined by combining all the variables in the predefined sequence to make sure that a given parameter is calculated before the other parameter. The HIMS Screen with some of the HDM-4 models is shown in Figure 2.2.

After incorporating the HDM-4 RD and WE models with the HDM-4 Interface, the output generated by the HDM-4 interface was thoroughly compared with HDM-4 software output to ascertain that the logic and models were incorporated properly and represented the HDM-4 system.

An example of the setup document prepared while incorporating HDM-4 models in HIMS is given in Appendix C. The prepared setup can be readily used with some modifications to the NZ dTIMS Setup.

Figure 2.2 The HIMS screen showing some HDM-4 models.

The screenshot displays the HDM-4 Information Management System interface. The main window is titled "HDM-4 Information Management System" and shows a file path: "C:\Files\hdm-4\Final\Works Effects\Roadsurface\Case1\Hims\Final\_Roadline\Hims\_Res\_case1.mdb". The interface is divided into several panes:

- Left Pane:** A navigation tree with categories like "Road Referencing", "Tables", "Library", "Road", "HDM-4", "Queries", "Lookup Tables", "Constants", "Filters", "Formulas", "Lookup Expressions", "Variables", "Treatments", "Interventions", "Models", "Reports", "Data Conversion", "Utilities", "Source Data", and "Templates".
- Top Center Pane:** "Formulas under HDM-4: Library". It contains a table with columns: Item Name, Description, Memo, and Def.
 

Item Name	Description	Memo	Def
EH_PACAw1	Area of Previous Cr...	[VAR][ACA]	
EH_PACAw2	Area of Previous Cr...	[[EXP][EH_w] * [[HDM_INTF][PCRA]]] + [[1...	
EH_PACWw1	Area of Previous Wt...	[VAR][ACW]	
EH_PACWw2	Area of Previous Wt...	[[EXP][EH_w] * [[HDM_INTF][PCRW]]] + [[1...	
EH_SHPdAw	Dry season SHP at...	max(1.5, [[VAR][SNPck]] + 0.0394 * 0.19 * 2...	
EH_SHPdWw	Dry Season SHP at...	max([[HDM_INTF][HSOLD][EXP]]ER_HSD...	
EH_w	Factor used to calc...	max([[HDM_INTF][PCRA]]] + [[1...	
EO_PACAw1	Area of Previous Cr...	[VAR][ACA]	
EO_PACAw2	Area of Previous Cr...	[[EXP][EO_w] * [[HDM_INTF][PCRA]]] + [[1...	
EO_PACWw1	Area of Previous Cr...	[VAR][ACW]	
EO_PACWw2	Area of Previous Cr...	[[EXP][EO_w] * [[HDM_INTF][PCRW]]] + [[1...	
EO_RDmAw	Mean Rut Depth a...	0.15 * [VAR][RDM]	
EO_RfAw	RDugness after 0...	2 + 0.01 * max([[EXP][ER_Rfobw]] - 2), 0] m...	
EO_SHPdAw	Dry Season SHP at...	max(1.5, [[VAR][SNPck]] + 0.0394 * 0.4 * 25...	
EO_w	Factor used to calc...	min(0.7 + 0.1 * (25), 1)	
EP_A_c1	for All Structural Cr...	max ([[2 * [50 * [EXP][KP_PAS_e1]]] - [VAR]...	
EP_A_Y	for All Crack Proge...	[[EXP][KP_PAS_e0]] * [EXP][KP_PAS_e1] * ...	
- Top Right Pane:** "HDM-4 Data". A table with columns: HIMS LRTID, Road ID, Road Name, Item ID.
 

HIMS LRTID	Road ID	Road Name	Item ID
1	100	Test Road	1
2	100	Test Road	2
3	100	Test Road	3
4	100	Test Road	4
5	100	Test Road	5
6	100	Test Road	6
7	100	Test Road	7
8	100	Test Road	8
9	100	Test Road	9
10	100	Test Road	10
11	100	Test Road	11
12	100	Test Road	12
13	100	Test Road	13
- Bottom Left Pane:** "Variables under HDM-4: Library". A table with columns: Item Name, Description, Memo.
 

Item Name	Description	Memo
ACA	Total Area of All Cracking for th...	
ACAa	Area of Cracking at the start of th...	
ACRA	Total Area of Cracking for the a...	
ACRAa	Total area of Cracking at the st...	
ACT	Area of Transverse Thermal Cra...	
ACTa	Area of Transverse Thermal Cra...	
ADW	Area of Wide Structural Crackin...	
ADWec	temporary value of ADW to che...	
ADWpe	Cumulative area of Wide Crack...	
ADWu	Area of Untreated Wide Cracking	
ADXa	Area of Indexed Cracking at the...	
ADT1	ADT of car	
ADT2	ADT of LCV	
ADT3	ADT of MCV	
ADT4	ADT of HCV1	
ADT5	ADT of HCV2	
ADT6	ADT of bus	
AGE1	Years from last preventive treat...	
AGE2	Years from last surfacing	
- Bottom Right Pane:** "Filters under HDM-4: Library". A table with columns: Item Name, Description, Memo, Defined.
 

Item Name	Description	Memo	Defined
FH_PACAw1	Area of Previous Cracking at...	[VAR][ACA] > [[HDM_INTF][PCRA]]	
FH_PACAw2	Area of Previous Cracking at...	[VAR][ACA] < [[HDM_INTF][PCRA]]	
FH_PACWw1	Area of Previous Wide Crack...	[VAR][ACW] > [[HDM_INTF][PCRW]]	
FH_PACWw2	Area of Previous Wide Crack...	[VAR][ACW] < [[HDM_INTF][PCRW]]	
FD_PACAw1	Area of Previous Cracking at...	[VAR][ACA] > [[HDM_INTF][PCRA]]	
FD_PACAw2	Area of Previous Cracking at...	[VAR][ACA] < [[HDM_INTF][PCRA]]	
FD_PACWw1	Area of Previous Wide Crack...	[VAR][ACW] > [[HDM_INTF][PCRW]]	
FD_PACWw2	Area of Previous Wide Crack...	[VAR][ACW] < [[HDM_INTF][PCRW]]	
FP_ACTb	Area of Transverse Thermal...	[VAR][ACTa] - [VAR][dACT]] > 0	
FP_ADWb1	ACRAa=0 for ADW	[VAR][ACRAa] = 0	
FP_ADWb2	ACRAa=0 for ADW	[VAR][ACRAa] > 0	
FP_ADWbamp	Temporary ADW value for A...	[VAR][ADWa] <= 1 And [VAR][ACAa] > 11	
FP_dACA1	Y<0, Increment in ASC	[VAR][AL_Y] < 0	
FP_dACA2	Y>0, Increment in ASC	[VAR][AL_Y] > 0	
FP_dACA3	ACAa<=50, ACAa+dACA>50...	[VAR][ACAa] <= 50 And [VAR][ACAa] >...	
FP_dACRARV1	Increase in Area of Cracking...	[VAR][ARVa] > 0	
FP_dADW1	Y<0, Increment in WSC	[VAR][Wide_Y] < 0	
FP_dADW2	Y>0, Increment in WSC	[VAR][Wide_Y] > 0	
FP_dADW3	ADWu<=50, ADWu+dADW>50...	[VAR][ADWu] <= 50 And [VAR][ADWu] >...	

### **3. Study Approach & Methodology**

#### **3.1 Introduction**

To fulfil the objectives of the study, both a desk study and an experimental analysis with some field verification were carried out. This chapter briefly describes the methodology used in the study.

#### **3.2 Desk Study Comparison (Stage 1)**

The primary objective of the desk study comparison was to determine the difference between the RD and WE models available in the NZ dTIMS Setup and the HDM-4 system. For this purpose, available literature on HDM models (HDM-III technical reports, HDM-4 documentation series, NZ dTIMS reports, ISO HDM study reports, ARRB (Australian Road Research Board) reports on HDM-4 software testing, etc.) were referred to. The fundamental differences between the models included in the NZ dTIMS Setup and the HDM-4 system, and their applicability to New Zealand conditions, were identified and documented during the process. The experimental analyses required to quantify the effects of various model parameters available in the HDM-4 models were also identified in the desk study.

#### **3.3 Input Data Issues**

Analysis for additional data requirements for HDM-4 models in comparison to the NZ dTIMS Setup was carried out. Availability of these additional data in the RAMM database was also verified, and a summary of the additional data requirements for the HDM-4 models was prepared.

#### **3.4 Database Preparation**

A number of datasets had to be prepared for different types of analyses carried out in the project. Historical road inventory and condition data available in the RAMM database were taken as the primary data source for the study. The datasets from the RAMM database were carefully chosen so that roads from different traffic, physical and environmental conditions, with different levels of data collection, were included. Data for the preliminary analysis was taken from the following RCAs.

- Auckland City Council;
- Napier Transit Network;
- Northland Transit Network;
- Southland District Council; and
- North Shore City Council.

HIMS was used to manipulate, store and analyse the data that were required for the project. In the preparation of the real network database, for comparison and calibration, the sections were carefully selected to meet the requirements of the experimental design. More attention was given to the calibration database preparation. For example, the field verification and collection of additional data were carried out to obtain the updated data for the calibration purpose. Preparation of the synthetic datasets was based on the analysis of the RAMM database. The possible range of values for different parameters relevant to New Zealand were defined and used to prepare synthetic datasets. Chapter 8 describes the database assembly process followed during the project.

### **3.5 Parametric Study (Stage 2)**

The modifications to RD and WE models carried out in HDM-4, identified in the desk study as having potential use in New Zealand conditions, were further studied in the parametric study. Synthetic datasets were used in the parametric study to quantify the effects of various parameters (within minimum and maximum range). A detailed description on the preparation of the synthetic data is given in Chapter 8.

The HDM-4 software could not be used as it did not fulfil the requirements of the study (Chapter 2 has more details). HDM-4 models were programmed in the HDM-4 HIMS for this study, and the NZ dTIMS Setup was used to generate outputs for NZ dTIMS models. MS Excel was used for the preparation of graphs, etc.

### **3.6 Initial calibration of HDM-4 Models (Stage 3)**

#### **3.6.1 Sensitivity Analysis of Calibration Factors**

The number of calibration factors in the HDM-4 model, for paved bituminous roads, was about 23. As the calibration of each factor generally is time consuming and involves a lot of resources, it was essential to identify the most sensitive parameters so that they could be prioritised for calibration. Therefore, sensitivity analysis of the calibration factors was carried out using the traditional *ceteris paribus* (TCP) method. Impact elasticities of various calibration factors were defined.

The sensitivity analysis of the HDM-4 models used in the NZ dTIMS (Pradhan 2001) showed that the sensitivity of different parameters depends on the analysis period, traffic volume, and condition of the road, etc. Hence, the following two different traffic levels were used:

- High (10,000 vpd (vehicles per day)); and
- Low (200 vpd).

The sensitivity of the different calibration parameters was studied separately for asphalt concrete and surface-treated pavements of ages 5, 10, 15 and 20 years. The maximum (of the 4 values) was taken as representative for the given calibration coefficient.

### **3.6.2 Defining Model Calibration Coefficients**

Various methods are available for calibrating HDM-III models. The same methodology was recommended to calibrate the coefficients in HDM-4 models where the expressions are similar (Bennett & Paterson 2001). It was decided to use the Level 2 calibration of the HDM models, and special attention was given to the preparation of data for the calibration, as detailed in Chapter 8. Two methods were used, the 'snap shot' (based on condition of road at a certain point of time) and 'time series' (based on historical data of performance of road).

In the snap-shot method, data was segregated based on the current characteristics like traffic, surface and pavement age. After segregating the sections into different age groups, these sections were amalgamated as one representative section for a particular traffic range. The data were plotted for each distress against age, i.e. AGE2 and AGE3 as applicable, to produce distress progression curves.

In the time-series method, the calibration coefficients were calculated based on the observed and predicted values. Historical data available in the RAMM database were used for this task. Routine maintenance was considered to be applied, as no data for the sterilised sections (i.e. road sections continuously monitored, and no maintenance work allowed, for research purposes) were available.

### **3.6.3 Calibration of Additional Calibration Coefficients**

The methodology to calibrate the additional calibration factors included in the HDM-4 system was neither available in the HDM-4 documentation series, nor could be found in any other published literature. The HDM-4 calibration guide (Bennett & Paterson 2001) included the calibration issues relating to the HDM-III calibration factors.

Data requirements for the initial calibration of additional calibration factors included in the HDM-4 models were explored and recommendations on the further research works were made.

## **3.7 Comparisons with Real Network Data (Stage 4)**

The objective of comparing the model prediction with the real network data was to verify the similarity in the model predictions of the NZ dTIMS Setup and the HDM-4 system. The other objective was to make sure that the models and the interrelation between the models are properly simulated in the HDM-4 interface so that it can be transferred to NZ dTIMS CT later.

About 200 sections from each of the three road networks (Transit New Zealand, District Council, City Council) representing different network conditions were randomly selected from RAMM treatment length tables. The NZ dTIMS input file was prepared using the dTIMS Interface program, and an HDM-4 input file was prepared using HIMS. The model predictions were compared and reported.

## 4. Desk Study Comparison of RD Models

### 4.1 Introduction

The Highway Design and Maintenance Standards (HDM-III) Model, developed in the 1980s by the World Bank, has been widely used for more than two decades now. The models have been instrumental in justifying the maintenance budgets to maintain and improve the road standards in many developing countries. In recent years many industrialised countries have also begun to apply the model. This has resulted in a need for additional capabilities to be included such as traffic congestion, wider range of pavements, different types of vehicles operating under different environmental conditions, etc. Some modifications and enhancements have been made to the existing HDM-III models and some additional models have been incorporated in an HDM-4 system to enhance the predictive capability of the HDM models.

This chapter briefly discusses the difference between the HDM-III RD and WE models used in the NZ dTIMS Setup and the HDM-4 system. Potential use of these enhancements in the context of New Zealand conditions is also reviewed.

### 4.2 Pavement Defects Modelled

HDM-4 RD and WE models, as stated earlier, have been built upon the HDM-III models. Some additional models for various pavement defects were also incorporated in HDM-4. Table 4.1 presents an overview of the pavement defects modelled in the three different systems. For the bituminous pavements, edge break is the only additional defect modelled in HDM-4 that is not modelled in the NZ dTIMS Setup.

**Table 4.1 Pavement defects modelled in the 3 RD systems used in New Zealand.**

Pavement Type	Distress Type	HDM-III	NZ dTIMS	HDM-4
Bituminous	Cracking	•	•	•
	Ravelling	•	•	•
	Potholing	•	•	•
	Edge break			•
	Rut depth	•	•	•
	Roughness	•	•	•
	Texture depth		•	•
	Skid resistance		•	•
Concrete	Cracking			•
	Joint spalling			•
	Faulting			•
	Failures			•
	Serviceability rating			•
	Roughness			•
Block (Proposed to include in HDM-4 later)	Rutting			•
	Surface texture			•
	Roughness			•
Unsealed	Gravel loss	•	•	•
	Roughness	•	•	•

Sources: Morosuik et al. (2000); HTC (2000)

### 4.3 Traffic Characteristics

Traffic characteristics in HDM models are described as:

- Annual average daily traffic volume;
- Mode-wise distribution,
- Axle load factors; and
- Growth rates.

The NZ dTIMS Setup considers almost all types of vehicles that use New Zealand roads, and no major change has been made to the existing traffic characteristics used by the RDWE models. Non-motorised traffic (NMT) in HDM-4 models mainly influences the road user effects (RUE) models and, hence, is not considered in this study. Bicycles are the only NMT mode plying on New Zealand roads and would not contribute to pavement deterioration.

### 4.4 Pavement Characteristics

#### 4.4.1 Pavement Type

A comprehensive pavement classification has been developed to cater for the expanded scope of the RD and WE models in HDM-4. The term 'pavement type' is included which combines the surface type and base type (i.e. basecourse, etc.). Table 4.2 shows the comparison between the pavement classifications in HDM-III and HDM-4 systems for bituminous surfaces.

**Table 4.2 Pavement classification for bituminous surfaces.**

Surface Type		Base Type		HDM-4 Pavement Type
HDM-III	HDM-4	HDM-III	HDM-4	
Surface treatment (ST)	ST	Granular	Granular (GB)	STGB
Slurry on surface treatment (SSST)		Cement stabilised	Stabilised (SB)	STSB
Reseal on surface treatment (RSST)		Bituminous	Asphalt (AB)	STAB
Reseal on AC (RSAC)			Asphalt pavement (AP)	STAP
			Rigid (concrete) base (RB)	STRB
Asphalt concrete (AC)	AC	Granular	Granular (GB)	ACGB
Asphalt overlay on AC (OVAC)		Cement stabilised	Stabilised (SB)	ACSB
Open graded cold mix on surface treatment (OCMS)		Bituminous	Asphalt (AB)	ACAB
			Asphalt pavement (AP)	ACAP
			Rigid (concrete) base (RB)	ACRB

Note: STRB (surface treatment rigid base) and ACRB (asphalt concrete rigid base) are proposed to be included at a later stage.

The concrete pavements modelled in HDM-4 were not considered, as they are generally not constructed for roads in New Zealand. Bituminous surfacing on rigid (concrete base) and blocked pavements are not yet modelled in HDM-4.

Unsealed roads in HDM-4 have been divided into three categories based on types of surfacing:

- Gravel (GR);
- Earth (EA); and
- Sand (SA).

Unsealed roads have not been considered in this study as a separate research work is being considered by Transfund on Unsealed road modelling.

Although most of the coefficients for parameters for different types of surfaces and basecourses remain the same, the difference in approach of modelling between HDM-III and HDM-4 will require some changes in the logic of preparation of filters in the NZ dTIMS Setup. Logic followed for HDM-4 was successfully replicated in the HDM-4 interface built in to HIMS.

#### **4.4.2 Structural Number**

HDM-4 used the concept of adjusted structural number (SNP) instead of modified structural number (SNC), which was used in the HDM-III and NZ dTIMS Setup. The new SNP ensures that the over-prediction of the pavement strength found in the SNC calculation is taken into account (Rolt & Parkman 2000). This new SNP calculation procedure is adopted in the pavement strength program used by the NZ dTIMS Setup.

SNP in HDM-4 includes the effect of climatic conditions through seasonal variation of SNP and drainage effects. The average annual strength of the pavement is estimated from:

- Strength of the pavement during the dry season and during the wet season; and
- Duration of each season.

An empirical equation is also provided in HDM-4 to calculate the annual average SNP, in case the wet/dry season SNP ratio is not available. Appendix B contains the model form for SNP.

A major enhancement to the HDM-4 models is that the SNP is adjusted every year based on the surface distress(es) and drainage factor. The HDM-III models also stipulated the use of annual average modified structural number, but only under conditions where the material properties change significantly with seasons (Watanatada et al. 1987).

#### **4.4.3 Pavement Age**

The three variables defining the age of the pavement that were used in HDM-III generally relate to the age of the surface. A fourth variable AGE4 has been included in HDM-4 to express the base construction age. This is used in the modelling of initial densification of rutting in the HDM-4 rutting model. The definition of the other three age parameters (AGE1, AGE2 and AGE3) used in HDM-III remains the same in HDM-4, and the factors used to define the pavement age characteristics in different systems are given in Table 4.3.



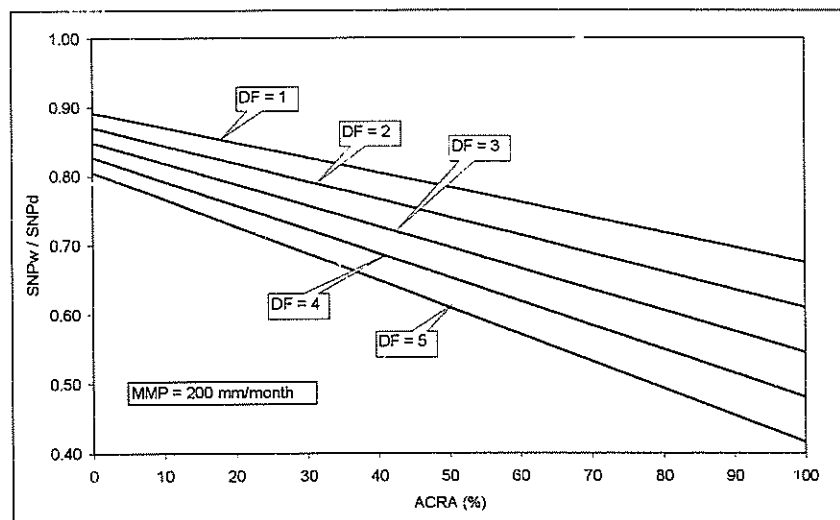
Table 4.3 Pavement age factors used in the three systems.

Pavement Age Factor	HDM-III	NZ dTIMS	HDM-4
Preventive treatment age (AGE1)	•	•	•
Surfacing age (AGE2)	•	•	•
Construction age (AGE3)	•	•	•
Base construction age (AGE4)			•

#### 4.5 Drainage Factor

The surface drainage condition and drainage channels have a significant impact on the pavement strength and pavement deterioration. The proposed drainage factor takes into effect the type of drain and condition of the drain. The value of the drainage factor can range between 1 (excellent) and 5 (very poor) depending on the type and condition of the drain. (See Appendix D for values for the drainage factor.) Figure 4.1 shows the significant impact of drainage factor on wet/dry season SNP ratio.

Figure 4.1 Impact of the drainage factor on SNP.



Source: Morosuik et al. (2000)

The HDM-III model also recognised the significance of poor drainage on pavement strength. It was also appreciated that the road segments, having identical mean values of pavement strength indicators, often deteriorated at different rates because of the difference in structural properties, drainage conditions and construction quality. However, these effects were not quantified in HDM-III and, hence, some of these effects were included in the form of 'occurrence distribution factor' (F) in HDM-III cracking and ravelling initiation models (Watanatada et al. 1987).

Detailed research is recommended for New Zealand conditions to quantify the effect of drainage condition on pavement strength and subsequently on pavement performance. It is especially important in the case of New Zealand conditions as most of the roads are constructed as thin flexible pavements, and incursion of water into the pavement can have severe impact on pavement deterioration.

## 4.6 Construction Quality

Initiation of some of the distresses may be attributed to the material handling and construction quality rather than structural failure of the pavement (Morosiuk et al. 2000). In HDM-III two construction quality indicators were used, a surfacing construction quality indicator (CQ) and a compaction indicator (COMP) (Watanatada et al. 1987). CQ had two values: 0 (no defects) and 1 (with surface defects) which was used in cracking and ravelling models. COMP was used in the rutting model in HDM-III.

CQ was replaced in HDM-4 and two continuous variables have been used to express the construction defects for the surfacing (CDS) and base preparation (CDB). Table 4.4 shows the construction quality indicators used for bituminous pavements and their applicable values in different systems.

**Table 4.4 Construction quality indicators used in the three systems.**

Construction Quality Indicator	NZ dTIMS/HDM-III		HDM-4	
	Indicator	Value	Indicator	Value
Bituminous surface construction defects indicator	CQ	0 or 1	CDS	0.5 to 1.5
Base construction defects indicator	CQ	0 or 1	CDB	0 to 1.5
Base, sub-base and subgrade layers compaction	COMP	Up to 100%	COMP	Up to 100%

CDS and CDB are continuous variables. CDS can have any value between 0.5 and 1.5 and CDB between 0 and 1.5 (Morosiuk et al. 2000). This is quite a significant modification to the HDM models and the impacts of the changes were tested during the parametric study (Chapter 9).

The continuous construction defects indicators allow the incorporation of wide variations in the construction and maintenance quality experienced in New Zealand. It is recommended to use these factors instead of CQ in the NZ dTIMS Setup.

## 4.7 Cracking

### 4.7.1 Background

Cracking is the most important distress seen on bituminous pavements. It is readily identifiable and universally acknowledged as a sign of pavement deterioration. The analysis of cracking on bituminous pavements is extremely complex as it may have more than one cause, while other distresses such as rutting, ravelling and potholing generally have a single definition and are fairly and easily identified and quantified. When a fracture appears on the pavement surface, it is identified as a crack within the context of distress identification. By the time cracking becomes evident, significant damage to the pavement has already occurred.

The different categories of types of cracking based on their causative factors are given in Table 4.5.

**Table 4.5 Categories of cracking types and causes.**

Type of Cracking	Cause
Structural cracking	<ul style="list-style-type: none"> <li>• associated with load and indicates the structural deficiency of the pavement</li> <li>• could be due to fatigue, as in crocodile or alligator cracking</li> <li>• could be due to shear failure as longitudinal wheelpath cracking in thin surfacing</li> </ul>
Longitudinal cracking	<ul style="list-style-type: none"> <li>• longitudinal wheelpath cracking due to shear failure, particularly in thin surfacing pavements</li> <li>• longitudinal cracking due to reflection from an underlying crack</li> <li>• longitudinal cracking due to construction defects</li> </ul>
Transverse cracking	<ul style="list-style-type: none"> <li>• transverse cracking due to reflection</li> <li>• transverse cracking due to thermal cracking (caused by large changes in daily temperatures)</li> <li>• transverse cracking due to subsurface failure</li> <li>• transverse cracking due to construction defects</li> </ul>
Reflection cracking	<ul style="list-style-type: none"> <li>• cracking due to the reflection of underlying cracks</li> </ul>
Thermal cracking	<ul style="list-style-type: none"> <li>• primarily a function of material and environmental factors, and initiated by large variations in daily temperatures</li> </ul>

#### 4.7.2 Cracking Types Considered in NZ dTIMS and HDM-4

The NZ dTIMS cracking models are based on the HDM-III cracking models that were developed mainly on a Brazilian study. This study included only crocodile and irregular cracking which was predominant under Brazilian situations. Other types of cracking like longitudinal, transverse and map cracking, were not considered (Watanatada et al. 1987).

During the Brazil study, in order to develop the model, cracking was classified by severity as shown in Table 4.6.

**Table 4.6 Severity of cracking based on Brazilian study.**

Severity Class	Definition
1	Hairline cracks, width 1mm or less
2	Crack widths 1 to 3mm
3	Crack width greater than 3mm without spalling
4	Spalled cracks, i.e. fragments of the surfacing adjacent to crack were lost

To reduce the number of classes, HDM-III models are based on two severity levels:

- All cracking: narrow and wide cracks (Classes 2, 3 and 4); and
- Wide cracking: wide cracks (Class 4).

A single model is practically impossible to use for cracking prediction when so many different types of cracking exist because of so many causes. In HDM-4 cracking is modelled as a function of the factors that are known to contribute to their development and progression (Morosuk et al. 2000).

In HDM-4, structural cracking is effectively load- and age-associated and is modelled as ‘all’ and ‘wide’ cracking based on HDM-III relationships. Transverse thermal cracking has been introduced as a new type of distress. Reflection cracking originally proposed by NDLI (1995) is yet to be implemented in HDM-4. In both HDM-III and HDM-4 cracking is modelled in two phases: initiation and progression.

Table 4.7 shows the different types of cracking modelled in HDM-III, the NZ dTIMS Setup and HDM-4.

**Table 4.7 Cracking types modelled in HDM-III, NZ dTIMS and HDM-4.**

Cracking Type	HDM-III	NZ dTIMS	HDM-4
All structural cracking initiation	•	•	•
All structural cracking progression	•	•	•
Wide structural cracking initiation	•	•	•
Wide structural cracking progression	•	•	•
Transverse thermal cracking initiation			•
Transverse thermal cracking progression			•
Reflection cracking initiation *			•
Reflection cracking progression *			•

\* Proposed to be implemented in HDM-4.

The ensuing sections describe the different cracking models included in the NZ dTIMS Setup and the HDM-4 system.

#### 4.7.3 Initiation of All Structural Cracking

The basic structure of the All Structural Cracking Initiation model of HDM-III has not been changed in HDM-4. The surface construction quality indicator, CQ, has been replaced by a continuous variable, labelled CDS, that can have any value between 0.5 and 1.5. This makes the model more flexible and provides an opportunity to calibrate to different environments.

Table 4.8 summarises the independent variables on which all structural cracking initiation is dependent in different systems.

**Table 4.8 Independent variables for All Structural Cracking Initiation model.**

Independent variable	HDM-III	NZ dTIMS	HDM-4	Comments
Drainage condition			•	
Pavement strength	•	•	•	
Traffic loading	•	•	•	
Environment	•	•	•	
Pavement history	•	•	•	
Elapsed time				
Construction quality	•	•	•	CDS in HDM-4; CQ in NZ dTIMS and HDM-III

The other major difference in the HDM-4 and HDM-III Crack Initiation model is the estimation of SNP. In HDM-4, SNP is estimated each year considering the surface distresses and drainage condition.

#### 4.7.4 All Structural Cracking Progression

Both traffic- and time-based models were initially developed for HDM-III (Watanatada et al. 1987). Although the traffic-based model was considered to be “generally superior” it was found to be inapplicable with the given data. Hence, time-based models were included in HDM-III and followed by the NZ dTIMS Setup. The basic structure of the cracking progression model also has not been changed in HDM-4, thus adhering to the time-based model. CQ has been replaced by CDS in HDM-4.

Some change may be made in the Cracking Progression model as proposed by Morosiuk & Riley (2001) to replace the existing time-based Cracking Progression model with the improved traffic-based model.

The independent variables that affect All Structural Crack Progression are given in Table 4.9.

**Table 4.9 Independent variables for the All Structural Crack Progression models.**

Independent variable	HDM-III	NZ dTIMS	HDM-4
Drainage condition			
Pavement strength			
Traffic loading			
Environment			
Pavement history			
Elapsed time	•	•	•
Construction quality	•	•	•

#### 4.7.5 Wide Structural Cracking

The modelling of wide structural cracking is considered in two phases, as initiation and progression in HDM-4, HDM-III and NZ dTIMS Setup. The basic model form is similar in all the systems. However, CDS has been used in HDM-4 in place of CQ to express the surface construction quality of the bituminous pavements.

Initiation of wide structural cracking is entirely dependent on the initiation of all structural cracking. The independent variables for wide structural cracking are also similar and influence it indirectly through all structural cracking.

In HDM-III and NZ dTIMS Setup, the calibration factors (deterioration factor) ‘Kcia’ and ‘Kcpa’ were used in All Structural Cracking and Wide Structural Cracking models. New calibration factors ‘Kciw’ and ‘Kcpw’ have been introduced in HDM-4 in the Wide Structural Cracking model.

The All Structural Cracking and Wide Structural Cracking models in the present NZ dTIMS Setup can be replaced with HDM-4 models that incorporate the effects of drainage conditions and adjusted structural number. Besides, the continuous construction quality indicator allows incorporation of wide ranges of maintenance practices.

#### 4.7.6 Reflection Cracking

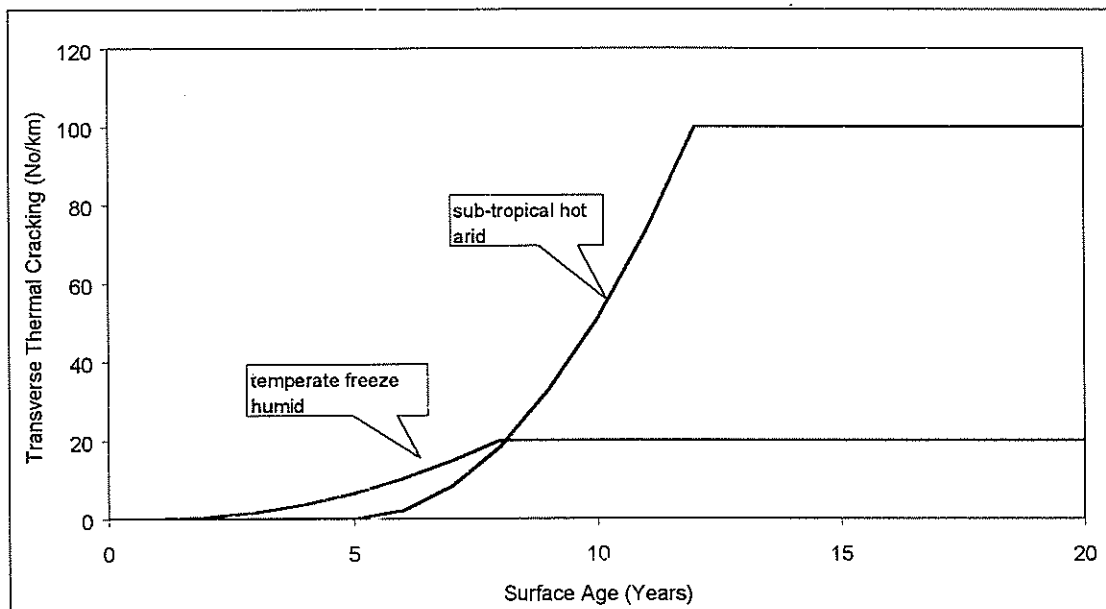
Reflection Cracking Initiation and Progression models have been proposed to be included in HDM-4. The model was based on the comprehensive study carried out in Malaysia (Rolt 2000). Both the initiation and progression of Reflection Cracking models in HDM-4 are functions of traffic, thickness of overlay, and amount of deflection before the overlay. It considers previous wide structural cracking as the minimum amount of cracking once the reflection crack is initiated (Morosiuk & Riley 2001).

These models could not be incorporated in the HDM-4 interface as the documentation of the models was not available until the end of the project. However most of the urban roads with the AC overlay in New Zealand have shown, by the appearance of the reflective cracking, that there is a potential use of this model in this country. Hence, a further study of reflective cracking is recommended.

#### 4.7.7 Transverse Thermal Cracking

Transverse thermal cracking has been introduced as a new type of pavement distress in HDM-4. It is generally considered to initiate where there is a large variation in daily temperatures. It is modelled as the number of cracks per km (Riley 1997).

Figure 4.2 Transverse Thermal Cracking model for two climate types.



Source: Morosiuk et al. 2000

The transverse thermal cracking has also been modelled in two phases, initiation and progression, like other cracking types. The model is illustrated for two climates in Figure 4.2.

A coefficient of thermal cracking (CCT) has been used as a variable to predict time to initiation of thermal cracks for the different climate zones (Morosiuk et al. 2000). The CCT value has been chosen so that transverse thermal cracking is initiated only in sub-tropical hot (arid and semi-arid) and temperate freezing climate zones (i.e. CCT  $\neq$  100). However, a facility in the HDM-4 software is available that can change these default values to enable transverse thermal cracking to be initiated in other climate zones. The maximum number of thermal cracks allowed on a particular surface has to be specified while modelling.

As the climate in New Zealand is moderate, no cracking due to thermal changes generally appears in bituminous pavements. Hence, transverse thermal cracking is not considered for further analysis in this study for New Zealand conditions.

#### 4.7.8 Total Area of Cracking

The total area of cracking is defined as the sum of the area of all structural and thermal cracking in HDM-4, while in the HDM-III and NZ dTIMS systems the area of all structural cracking itself is the total area of cracking. This definition is shown in the form of an equation in Table 4.10.

**Table 4.10 Total area of cracking for the three modelling systems.**

Parameter	HDM-III	NZ dTIMS	HDM-4
Total area of cracking	ACA	ACA	ACA + ACT

The inclusion of reflection cracking makes the total cracking as the sum of all structural cracking, transverse thermal cracking and reflection cracking.

In HDM-III and HDM-4 the damaged and undamaged area has been taken as 100%. This implies that the area of total damaged surface shall not exceed 100% at any given time. In NZ dTIMS, cracking and ravelling together have been taken as the maximum of 100% which implies the total damaged area, including the area of potholing at any given point, may exceed 100% (HTC 2000). This approach has been adopted to keep the system simple, and with the assumption that the pavement is not allowed to deteriorate to such a condition.

#### 4.8 Ravelling

Ravelling is the loss of surface aggregate particles from the bitumen-aggregate matrix. The occurrence of ravelling depends upon the construction methods, specifications adopted, materials used, and local practice. Ravelling is a common distress in poorly constructed, thin, bituminous layers, such as surface treatments, but is rarely seen in high quality, hot-mix asphalt.

Ravelling is typically limited to the pavement surface, and contributes to a reduction in the functional rather than in the structural performance of a pavement. However, in severe cases, ravelling of a thin surface treatment may contribute to potholing, which does affect the structural performance of the pavement.

Ravelling is modelled, like cracking, in two phases, initiation and progression.

The ravelling model progression is turned off in the present NZ dTIMS setup by taking '0' value for the ravelling progression factor (HTC 2000).

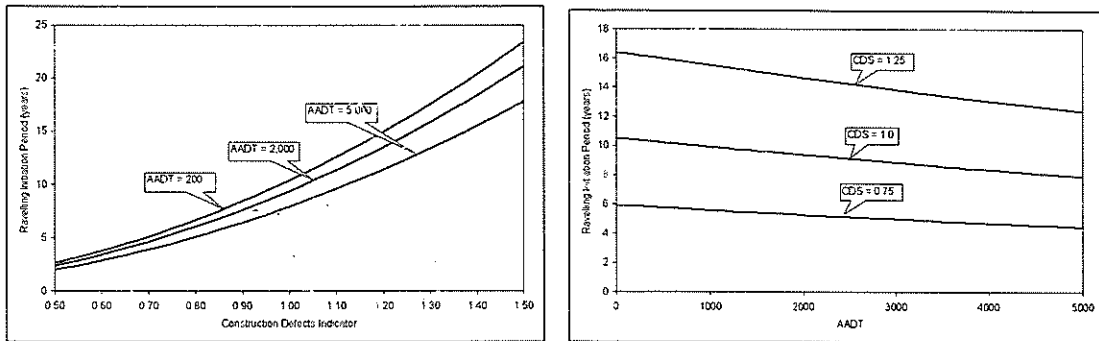
The differences between the models used in HDM-III, NZ dTIMS and HDM-4 are given in the following sections.

#### 4.8.1 Ravelling Initiation

The model for predicting the initiation of ravelling in HDM-4 is based on that in the HDM-III model. The only difference is that the construction defects indicator for bituminous surfacing (CDS) is used instead of the original construction quality variable CQ.

The traffic factor and construction defects indicator were plotted separately against ravelling initiation time (Morosiuk et al. 2000) and this showed that CDS has a greater influence than the traffic factor on ravelling initiation time. Figure 4.3 shows the graphs plotted for ravelling initiation time.

Figure 4.3 Impact of Traffic and CDS on Ravelling Initiation Time model.



The independent variables that affect ravelling initiation time are given in Table 4.11.

Table 4.11 Independent variables for Ravelling Initiation model.

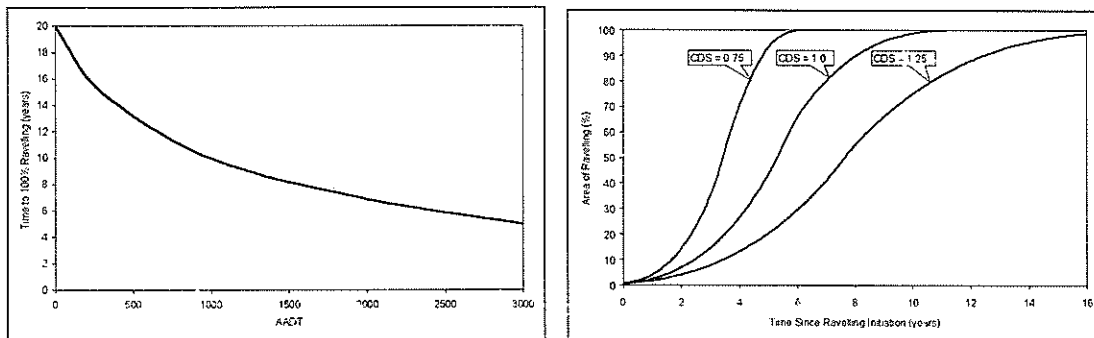
Independent variable	HDM-III	NZ dTIMS	HDM-4
Drainage condition			
Pavement strength			
Traffic loading	•	•	•
Environment			
Pavement history			
Elapsed time			
Construction quality	•	•	•



### 4.8.2 Ravelling Progression

A time-based model had been considered in HDM-III and NZ dTIMS systems. A traffic variable (YAX) has been added in the HDM-4 ravelling progression model as proposed by Riley (1999) to indicate the difference in the rates of ravelling progression on low volume roads and high volume roads. The variable CDS has also been included in the ravelling progression model that makes the model more flexible (Morosiuk et al. 2000). The effects of YAX after ravelling initiation and the influence of CDS on ravelling progression are shown in Figure 4.4.

Figure 4.4 Effects of YAX and CDS on Ravelling Progression model.



Source: Riley (1999)

In NZ dTIMS, a ravelling progression factor was introduced and assigned a value of '0' to turn off the ravelling progression, while in HDM-III the ravelling initiation factor 'Kvi' was used in a reciprocal form (HTC 2000). In HDM-4, ravelling progression factor 'Kvp' has been introduced to make the model more flexible in order to customise to different environments.

The independent variables included in the ravelling progression model in the three different systems are given in Table 4.12.

Table 4.12 Independent variables for Ravelling Progression model.

Independent variable	HDM-III	NZ dTIMS	HDM-4
Drainage condition			
Pavement strength			
Traffic loading			•
Environment			
Pavement history			
Elapsed time	•	•	•
Construction quality			•

Construction quality indicator was not included in the ravelling progression model in HDM-III and NZ dTIMS. However, it does indirectly influence ravelling progression through ravelling initiation.

With the inclusion of the traffic variable in ravelling progression, and replacing the construction quality indicator by a continuous variable, ravelling models have

become more flexible and applicable to New Zealand pavements in HDM-4 system. Research conducted by Cenek et al. (1998) suggested applying the HDM-4 ravelling initiation model to New Zealand conditions. Under this scenario, a good step would be to re-evaluate the ravelling mechanism if it can be applied to New Zealand conditions through a specialised study.

## **4.9 Potholing**

Potholes are the most visible and severe forms of pavement distress. Paterson (1987) defined it as “a cavity in the road surface which is 150mm or more in diameter and 25mm or more in depth”, or alternatively, loss of material from a pavement that penetrates through the asphalt layers into the unbound layers.

In the Brazilian study (GEIPOT 1982) potholing was recorded as volume per unit length. This was converted to % area by applying a standard depth of 80mm in the HDM-III model. The use of volume as a unit of measurement was highly correlated with simulations of roughness effects. In HDM-4, the potholing is expressed in terms of ‘pothole units’, i.e. number of potholes of standard size, for ease and accuracy of recording under field conditions. Each pothole unit has a surface area of  $0.1\text{m}^2$ , i.e. approximately 300mm in diameter and therefore can be adequately estimated by reference to a person’s foot (Riley 1999). For estimating maintenance requirements in HDM-4, the depth of a ‘pothole unit’ has been assumed to be 100mm.

As with other distress modes (cracking, ravelling), the model first defines an initiation period followed by the annual occurrence of new potholes. It also models the enlargement of existing potholes if no patching is carried out. HDM-4 potholing initiation and progression models have been changed significantly from HDM-III, and the following sections describe this in detail.

### **4.9.1 Pothole Initiation**

The basic philosophy to predict pothole initiation from either wide cracking or ravelling was kept, unchanged, in HDM-4. In all the systems (NZ dTIMS, HDM-III and HDM-4), initiation of potholes related to cracking only arises once the total Area of Wide Cracking (ACW) exceeds 20%. Ravelling-initiated potholes arise when the Ravelled Area (ARV) exceeds 30%.

However, the potholing initiation model used in HDM-III and NZ dTIMS has been replaced by new modified relationships provided by Riley (1996). The potholing models in HDM-4 introduced the construction defects indicator for base (CDB). Potholing is expressed in terms of the number of ‘pothole units’ of area of  $0.1\text{m}^2$ . Environmental factors have also been included in the HDM-4 pothole initiation model in the form of rainfall, and the model is given in Appendix B.

The independent variables those used to model pothole initiation are given in Table 4.13.

**Table 4.13 Independent variables for Pothole Initiation model.**

Independent variable	HDM-III	NZ dTIMS	HDM-4
Drainage condition			
Pavement strength			
Traffic loading	•	•	•
Environment			•
Pavement history *	•	•	•
Elapsed time			
Construction quality			•

\* Pavement history included thickness of the bituminous surfacing.

Two new calibration factors, Environment and Construction Quality, for pothole initiation has been introduced in the HDM-4 system, which were not used in the HDM-III and NZ dTIMS systems.

#### 4.9.2 Pothole Progression

Pothole progression arises from cracking, ravelling and the enlargement of existing potholes. The progression of potholes is affected by the time lapse between the occurrence of potholes and their patching. For example, a response time of say 2 weeks between the occurrence of potholes and patching will result in fewer potholes occurring during the course of a year, than if the frequency of patching potholes was say 3 months. Early patching does result in fewer potholes occurring and it prevents the enlargement of the potholes that are patched. This appeared more relevant for the New Zealand condition where potholes are generally fixed within a certain response time.

To incorporate this effect in the Potholing Progression model, a time lapse factor (TLF) has been introduced in HDM-4 as an indicator of the response time to patching potholes (Odoki 1997, Riley 1997). TLF is defined as a function of the average time between the occurrence of potholes and patching them. The model is given in Appendix B.

The independent variables that influence pothole progression in the different systems are given in Table 4.14.

**Table 4.14 Independent variables for Pothole Progression model.**

Independent variable	HDM-III	NZ dTIMS	HDM-4
Drainage condition			
Pavement strength	•	•	
Traffic loading	•	•	•
Environment	•	•	•
Pavement history *	•	•	•
Elapsed time			
Construction quality	•	•	•
Local maintenance practice			•

\* Pavement history included thickness of the bituminous surfacing.

With the inclusion of local maintenance practices in HDM-4, the use of the HDM-4 Pothole model would be more appropriate for New Zealand conditions, in place of the HDM-III. However, some kind of groundwork is required, like quantification of CDB and TLF factors, before attempting to modify the present NZ dTIMS Setup.

#### 4.10 Rutting

Some of the limitations of the HDM-III rutting model have been addressed in the HDM-4 model. In particular these are (Morosiuk et al. 2000):

- Separate relationships for various phases of the progression of structural deformation;
- A new relationship for modelling the plastic deformation of pavements;
- A new relationship for modelling the seasonal surface wear which occurs in countries where vehicles use snow chains or studded tyres on roads covered with snow and ice; and
- Standardised rut depth predictions using those measured under a 2.0m straightedge.

The HDM-4 rut depth model is based on four components of rutting while the NZ dTIMS and HDM-III systems include only two components. Table 4.15 shows the rutting components included in the different systems.

**Table 4.15 Rutting components included in the three systems.**

<b>Rutting component</b>	<b>HDM-III</b>	<b>NZ dTIMS</b>	<b>HDM-4</b>
Initial densification	•	•	•
Structural deformation	•	•	•
Plastic deformation			•
Surface wear			•

The HDM-III and HDM-4 systems use a similar model for the initial densification component of rutting. The structural deformation component of rutting in HDM-4 is based on the simplified version (linear form) of the rutting progression model available in HDM-III (Morosiuk & Riley 2001). The additional model on plastic deformation model included in HDM-4 is a function of:

- Surface construction defects indicator;
- Speed of the vehicles;
- Traffic loading; and
- Thickness of bituminous layers.

Surface wear component is dependent on the number of passes of studded tyres, the average speed of the traffic, and the use of salt on roads. The detailed model forms for these components are given in Appendix B.

All these four components have been included in HDM-4 with an exclusive calibration factor for each one of them, to customise to the local conditions, while only two calibration factors were used in the NZ dTIMS and HDM-III systems.

In HDM-4, standard deviation of rutting is calculated from mean rut depth as proposed by NDLI (1995), instead of a relationship involving traffic and strength components as used in the NZ dTIMS and HDM-III systems. The model included in HDM-4 is given in Appendix B.

The independent variables included in the rutting model in different systems are given in Table 4.16.

**Table 4.16 Independent variables for Rutting model.**

Independent variable	HDM-III	NZ dTIMS	HDM-4
Drainage condition			
Pavement strength	•	•	•
Traffic loading	•	•	•
Environment	•	•	•
Pavement history *	•	•	•
Elapsed time			
Construction quality	•	•	•
Local maintenance practice			

\* Pavement history included thickness of the bituminous surfacing.

Separate relationships for different components facilitate the modelling of rut depth more reasonably. With the inclusion of separate calibration factors for each of the components, the rutting model is more flexible to customise to the local conditions. In New Zealand, multi-layers of chipseals create a poly-structured pavement that are prone to plastic deformation. The low use of studded tyres and salt on the roads in this country hardly has an effect on the surface wear component of rutting. However, to evaluate the contribution of different components to mean rut depth, a specialised study is recommended.

#### 4.11 Roughness

The roughness model in HDM-4 is based on the HDM-III model and has the same five components of roughness, as follows:

- structural component;
- cracking component;
- rutting component;
- pothole component; and
- environmental component.

The total roughness at any given time is the sum of these five components. However, the potholing component in HDM-4 has been extensively modified from that of the HDM-III and NZ dTIMS systems. The effect of potholes on a vehicle is complex, being a function of the occurrence and size of potholes and the freedom to

manoeuvre the vehicle to take avoiding action. If all potholes were in the wheelpaths and the vehicle had no freedom to manoeuvre (either because the road width is the same as vehicle width or because of traffic congestion), the vehicle would hit 100% of the potholes. At the other extreme, with a few isolated potholes on a two-lane road with no other traffic, the vehicle would probably avoid most if not all potholes.

To incorporate this concept in HDM-4, a variable known as 'freedom to manoeuvre' (FM) is introduced in the potholing component of roughness. The model included in the HDM-4 system is presented in Appendix B.

All the distresses like cracking, ravelling, potholing and rutting would contribute to the roughness progression, and hence the independent variables for the roughness model have not been analysed.

In the NZ dTIMS Setup, the maximum value of roughness was kept at '20' (HTC 2000), while HDM-4 limits the roughness to '16' (Morosiuk & Riley 2001).

The existing NZ dTIMS pothole component of roughness can be replaced by the HDM-4 model component. As the kind of maintenance regime existing in New Zealand does not allow potholes to exist, the change in pothole prediction due to the new model will be minimal. However, it will help to decide the optimal maintenance regime to be followed.

#### **4.12 Edge Break**

Edge break can be defined as the loss of surface, and possibly base materials, from the edge of the pavement, and commonly arises on narrow roads with unsealed shoulders. Ideally, three parameters are needed to define edge break:

- length of the occurrence;
- width of the lost material; and
- depth of the lost material.

Edge break was not modelled in the HDM-III and NZ dTIMS systems. Hoban (1987) provided an approach to modelling edge break with a very little research data, that included traffic volume and road geometry (pavement width). Hoff & Overgaard (1995) showed that edge step seemed to be well correlated with the volume of edge break. Although it was not possible to quantify rainfall effects, it is considered that this parameter should be included in the model to allow for later calibration by users. The model is presented in Appendix B. The independent variables included in the edge break model are given in Table 4.17.

In New Zealand, edge break is observed on a number of narrow roads in rural areas. Although vehicular traffic is restricted going to the edge by clearly marking the edge of the pavements, sometimes and especially at curves vehicles do travel near the edges.

Table 4.17 Independent variables for Edge Break model.

Independent variable	HDM-III	NZ dTIMS	HDM-4
Drainage condition			
Pavement strength			
Traffic loading			•
Environment			•
Pavement history			
Elapsed time			
Construction quality			
Local maintenance practice			
Road geometry			•

### 4.13 Pavement Texture

Perhaps the most important single variable that determines the magnitude of longitudinal and lateral forces at the tyre-road interface is pavement texture. A road surface exhibits two types of texture, classified as macrotexture and microtexture.

In general, microtexture determines the maximum skid resistance afforded by a dry pavement, while macrotexture determines the drainage ability and therefore how effective the microtexture will be when the pavement is wet. Most skidding-related accidents occur on wet pavements. Thus the change in macrotexture due to wear and compaction resulting from traffic has important safety as well as economic consequences because rolling resistance is a function of pavement texture.

Texture depth and skid resistance (SFC) were not modelled in HDM-III. The texture depth and SFC models available in the NZ dTIMS Setup are the improved version of models used in HDM-4. Separate research studies are being carried out on these models and hence no further work on them was done under this project.

### 4.14 Conclusions

- The scope of the HDM-4 models has been increased by extending the models over a wider range of pavements, to include rigid and block pavements as well as bituminous and unsealed roads. HDM-III and NZ dTIMS are applicable only to bituminous pavements and unsealed roads at the moment.
- NZ dTIMS Setup considered almost all vehicle types that use New Zealand roads. The only mode in the NMT category plying on New Zealand roads is the bicycle.
- A comprehensive pavement classification philosophy has been introduced in the HDM-4 models, which enables the selection of the appropriate pavement type.
- HDM-4 models the pavement strength based on the annual average structural number adjusted for seasonal changes. Pavement strength can affect the pavement performance significantly.
- The effects of the drainage condition are quantified in HDM-4 through a drainage factor.

- Continuous variables have been introduced in HDM-4 models to express the construction quality instead of a flag value as used in the NZ dTIMS and HDM-III. These variables make the HDM-4 models more flexible to customise to the local conditions.
- The All Structural Cracking and Wide Structural Cracking models included in the HDM-4 models incorporated the effects of drainage condition and adjusted structural number. As the climate in New Zealand is moderate, no cracking related to thermal changes generally appear in bituminous pavements. Hence, transverse thermal cracking was not considered to be significant to New Zealand conditions.
- With the inclusion of the traffic variable in ravelling progression and replacing the construction quality indicator (CQ) by a continuous variable (CDS), ravelling models in HDM-4 have become more flexible and applicable.
- With the inclusion of local maintenance practices in the HDM-4 in terms of a TLF (Time lapse factor), the use of the HDM-4 pothole model would be more appropriate for New Zealand conditions in place of the HDM-III model.
- The rutting model in HDM-4 has been extensively modified by including plastic deformation and surface wear components. Separate calibration factors for each of these components in HDM-4 makes it more flexible to customise to the local conditions.
- NZ dTIMS roughness model appeared to be modelling roughness reasonably well with the given conditions in New Zealand.
- In New Zealand, edge break is observed on some roads, particularly on rural roads. The edge break model included in the HDM-4 system has been adopted.
- Texture depth and skid resistance models are included in both the NZ dTIMS Setup and HDM-4 system. No further study was carried out on these models.
- Parametric study was considered necessary and was carried out by creating a synthetic data set (Chapter 9) to quantify the effects of various parameters in the HDM-4 models.



## 5. Desk Study Comparison of WE Models

### 5.1 Introduction

Pavements deteriorate under given traffic and environmental conditions. To ensure safe and comfortable travel, roads need to be maintained at regular intervals, and also to ensure that pavement standards are maintained and improved where applicable. Pavement defects are generally reset (i.e. quantity of defects reduce once they are fixed), after maintenance has been carried out, to values that relate to the type and degree of maintenance works carried out. The different types of maintenance works modelled in the NZ dTIMS Setup and HDM-4, and their effects on pavement defects, are briefly discussed in this chapter, with emphasis given to new maintenance works that were introduced in the HDM-4 model.

### 5.2 Maintenance Works & Resets

Maintenance works will have immediate and/or long-term effects on pavement performance depending on the type of work carried out. The maintenance works in the HDM-4 system are generally broken down into the following four categories:

- Routine maintenance;
- Periodic maintenance;
- Improvement works; and
- Construction works.

Improvement and new construction works are not modelled in the HDM-III or in the NZ dTIMS Setup (Table 5.1).

**Table 5.1** Types of maintenance works modelled in the three systems.

Maintenance type	HDM-III	NZ dTIMS	HDM-4
Routine maintenance	•	•	•
Periodic maintenance	•	•	•
Improvement works			•
New construction works			•

For the HDM-III, HDM-4 and NZ dTIMS Setup, the following parameters are reset after the application of a treatment:

- Pavement strength;
- Pavement condition; and
- Pavement history

Different types of treatments affect the road characteristics differently, and the effects also depend on the condition of the pavement when the treatment is applied. The reset values for various parameters can be based on:

- Absolute values;
- Percent of the previous value;
- Ratio of the previous value; or
- Expression.

### 5.3 Routine Maintenance

Routine maintenance works comprise those works that may need to be undertaken every year or at intervals during the course of a year. These works may be cyclic or reactive works (Robinson et al. 1998).

- Cyclic or scheduled – dependent on environmental effects such as vegetation, control and cleaning of drainage systems, etc.
- Reactive – responding to minor defects caused by traffic and environmental conditions such as crack sealing and pothole patching, etc.

Table 5.2 presents an overview of the types of routine maintenance activities included in different systems.

**Table 5.2 Types of routine maintenance works modelled in the three systems.**

Routine maintenance activity	HDM-III <sup>1</sup>	NZ dTIMS	HDM-4
Drainage maintenance and rehabilitation			•
Sealing of all structural cracking		•	•
Sealing/filling of wide structural cracking		•	•
Sealing of transverse thermal cracking			•
Surface patching of ravelled areas	•		•
Patching of potholes	•	•	•
Deep patching of wide structural cracking	•	•	•
Repair of pavement edge break			•
Shoving repairs		•	

<sup>1</sup> Total area of patching can only be specified in HDM-III.

In HDM-III, patching is applied for potholes, wide cracking and raveling on a priority basis, and is limited to the specified area of total patching. In contrast, the HDM-4 system user is allowed to specify an area of maintenance for each individual maintenance activity. Some new types of routine maintenance activities have been introduced in HDM-4. The NZ dTIMS Setup incorporates most of the routine maintenance activities now (2001) practised in New Zealand. The limiting area of these maintenance activities cannot be specified separately by the user in the NZ dTIMS Setup.

#### 5.3.1 Drainage Maintenance and Rehabilitation

In HDM-4, drainage factor (DF) represents the drainage condition. The value of DF varies between 1 (excellent) and 5 (very poor), and reduces each year if no maintenance is carried out (Morosiuk et al. 2000). Drainage maintenance operations can be of two types:

- Recurrent operations which counteracts annual deterioration; and
- Rehabilitation of drainage system that enhances the drainage condition.

The immediate effect of these operations is to reset the DF value, and an empirical relationship is available to reset this.

DF (Drainage Factor) is a new parameter introduced in HDM-4 for estimating the adjusted structural strength each year of an analysis period. It was found to be one of the major influencing factors for estimating pavement strength (annual adjusted structural number). However the effect on different pavement distresses was found not to exceed 10% (Chapter 9). The default values used in HDM-4 can be used as a starting point and these default values are given in Appendix D.

### **5.3.2 Sealing of Wide Structural Cracking**

Crack sealing is normally a recurrent operation that may be applied at regular intervals during the course of a year.

The effects of wide crack sealing on pavement performance are as follows.

- Potholes do not develop from wide cracks that have been sealed; and
- Water ingress is inhibited by crack sealing, and consequently affects SNP.

These benefits of crack sealing are augmented if the drainage is maintained in good condition.

The percentage of the wide structural cracks to be sealed, and the expected seal life (crack seal life, in years) are specified by the user in HDM-4. Hence the reset for wide cracking is a function of the seal life. Seal life itself is a function of the crack sealing method, materials used, climate, and other factors that may be specific to a particular road network.

Crack sealing in the NZ dTIMS Setup is a user-defined activity. But the reset of the cracking is directly based on the area of sealed cracking, and no consideration to the crack seal life is given.

RCAs in New Zealand do not generally wait until a considerable amount of wide structural cracking has developed. Instead crack sealing or patching is undertaken at regular intervals to prevent further deterioration. Crack sealing is generally done using polymer-modified bitumen with longer life than the bitumen that is used for surfacing. Under these circumstances, inclusion of the crack seal life in resetting the wide structural cracking after crack sealing may be ineffective for New Zealand conditions.

### **5.3.3 Sealing of Transverse Thermal Cracking**

Sealing of transverse thermal cracking in HDM-4 is a user-specified operation similar to that for wide structural crack sealing. The quantity of sealing is expressed in terms of linear meters, and the life of the seal is also specified by the user, which is used in resetting the thermal cracking area. However, transverse thermal cracks are not generally applicable to New Zealand conditions. (Chapter 4 has more details.)

### **5.3.4 Surface Patching of Ravelled Areas**

Surface patching of ravelled areas is carried out to replace lost surface material. As well this prevents the formation of potholes caused by ravelling.

Presently ravelling is not applied in the NZ dTIMS Setup. Ravelling (also known as scabbing) in New Zealand is generally considered to be related more to construction defects rather than to mechanical failure of the pavement. Construction defects can be arrested by maintaining good construction quality. Hence at this time, no detailed investigations were made into this treatment.

### **5.3.5 Patching of Potholes**

In HDM-4, all maintenance works are modelled and applied at the end of the analysis year except potholing, which can be specified at intervals within a year. For example, pothole patching can be modelled for intervals that range from 2 weeks to 1 year. In the case of HDM-III and the NZ dTIMS Setup, pothole patching is assumed to be an annual operation carried out at the end of the analysis year.

The effects of pothole patching on pavement performance are as follows.

- Reduces the further progression of potholes (enlargement of patched potholes does not occur); and
- Reduces the roughness (pothole component of roughness).

Sometimes poor workmanship of patching may increase the roughness because patches may protrude. This is defined as the 'residual roughness due to patching/residual patch roughness'. In HDM-4 and NZ dTIMS Setup, the effect of the residual roughness is related to the area of patching. However in HDM-III, the residual patch roughness is limited to 10% of the roughness due to potholing (a function of the area of potholes). The same principle is followed in the NZ dTIMS Setup.

In the case of HDM-4, relating residual patch roughness to the overall roughness was not appropriate given the nature of the new potholing roughness model. This new model assumes avoidance of potholes by some road users (Morosiuk et al. 2000). Pothole patching carried out in that year is dependent on the frequency of patching, annual increment in potholing, traffic volume, carriageway width, and likely avoidance of potholes (freedom to manoeuvre), etc.

In New Zealand, potholes are generally patched within a month (and mostly within a week) from their initiation. Hence the situation relating to the avoidance manoeuvre of some drivers, as simulated by the HDM-4 model, may not be effective for the New Zealand condition. Hence, no alteration of the existing roughness reset expression in the NZ dTIMS Setup for pothole patching is envisaged in our study.

### **5.3.6 Deep Patching of Wide Structural Cracking**

Deep patching of wide structural cracking is modelled in HDM-4 as a permanent effect, unlike crack sealing which has a specified life. In HDM-4 the area of wide cracking is reset by the user-specified percentage of crack patching.

The immediate effect of deep patching is a reduction in roughness. Other effects on future deterioration are as follows:

- Structural strength of the cracked area is restored;
- Area of cracking that allows ingress of water is reduced; and
- Patched cracks do not transform into potholes.

This operation is generally performed over an area where the wide structural cracking is predominant and the sealing is not effective.

In the NZ dTIMS Setup, sealing of wide surface cracks is triggered under routine maintenance before they can expand over a wider area. No patching of the wide structural cracking is envisaged under routine maintenance. Detailed discussions with maintenance practitioners indicated that some areas of wide structural cracking on chipsealed surfaces are patched with digout (deep patching) method. Hence the same principle, as used in HDM-4, has been incorporated in the NZ dTIMS Setup.

### 5.3.7 Repair of Pavement Edge Break

Edge break is an additional parameter modelled in the HDM-4 system. It is envisaged that this model will be incorporated in the NZ dTIMS Setup as well.

The immediate effect of repairing an edge break is to reduce the area of the edge break and subsequently the total damaged area (cumulative volume of the distress) by the percentage of the area fixed. The longer term effect is to reduce the effective roughness experienced by vehicles during partial shoulder use on narrow pavements (Morosiuk & Riley 2001). No reset for roughness is applicable as it is considered that the edge break will not expand into the wheelpath.

## 5.4 Periodic Maintenance

Periodic maintenance includes those activities that are scheduled to be undertaken after several years. They are usually classified as preventive, resurfacing, overlay and reconstruction (Robinson et al. 1998). Table 5.3 shows the types of periodic maintenance activities included in the NZ dTIMS Setup, HDM-III and HDM-4 systems.

**Table 5.3** Types of periodic maintenance modelled in the three systems.

Periodic Maintenance Type	HDM-III	NZ dTIMS	HDM-4
Preventive treatments	•		•
Resurfacing	•	•	•
Overlay	•	•	•
Reconstruction	•	•	•

The different types of periodic treatments are discussed below. Resurfacing and Overlay have been combined in this chapter, as most of the resets are similar for these two treatments.

### 5.4.1 Preventive Treatments

Preventive treatments are not widespread in practice but have been applied in some countries over long periods. Their purpose is to extend the life of the bituminous surface by retarding the effects of weathering and ageing before significant distresses have occurred. In general, preventive treatments are only expected to be effective and economic on relatively low volume roads where ageing effects dominate the trafficking effects, with say less than 1000 vpd (Vehicles per day) (Watanatada et al. 1987). The various types of preventive treatments modelled in the three systems are given in Table 5.4.

**Table 5.4** Types of preventive treatments modelled in the three systems.

Type of treatment	HDM-III	NZ dTIMS	HDM-4
Fog seal	•		•
Rejuvenation	•		•
Slurry seal	•		

The effects of preventive treatments have been simulated through:

- Cracking retardation factor (CRT); and
- Ravelling retardation factor (RRF).

The reset of these factors in the HDM-4 model was adopted from the HDM-III reset relationships. AGE1 (preventive treatment age) will be reset to '0' in HDM-III and HDM-4 models. Preventive treatments are not in widespread use in New Zealand and, hence, are not considered for inclusion in the NZ dTIMS Setup.

### 5.4.2 Resurfacing and Overlay

Resurfacing is applied to relatively low levels of distresses like cracking and ravelling, etc. Resurfacing includes surface treatments and thin AC overlays for pavements in New Zealand. Alternatively overlay is applied to high levels of distresses and deformation or rutting, etc., because they enhance the strength of the pavement. Overlays on New Zealand pavements include thick AC overlays, and mill and replace. Table 5.5 shows the resets applied after resurfacing and overlay treatments.

Most of the reset values used in the NZ dTIMS Setup and HDM-4 are similar for the models available in both systems. Some difference was found in the reset value of the previous cracking. Both HDM-III and HDM-4 consider the previous cracking value before resurfacing to be weighted, based on the ratio of the new and old surfacing (including stabilised base if available). At the time of the development of the NZ dTIMS Setup, the decision was to use a simplified option taking just the area of cracking before resurfacing. The reason was that, as local maintenance regimes in New Zealand practice regular sealing, increased cracking does not occur to a large extent. Hence, the existing reset in the NZ dTIMS Setup for previous cracking shall be continued.

**Table 5.5 Resets to be applied after Resurfacing and Overlay treatments.**

Parameter	NZ dTIMS	HDM-4	Remarks
Preventive treatment age (AGE1)	0	0	
Surfacing age (AGE2)	0	0	
Construction age (AGE3)	0	0	
Pavement strength (SNP)	Expression	Expression	Same principles used except mill and replace treatment
Total cracking (ACA)	0	0	
Ravelling (ARV)	0	0	
Potholing (APOT)	0	0	
Patching (APH)	0	0	
Mean rut depth (RDM)	Expression	Expression	Not applicable for resurfacing; same expression used for overlay
Roughness (IRI)	Expression	Simplified Expression	
Edge break (EB)	NA	0	Not modelled in the NZ dTIMS Setup
Texture depth (TD)	Expression	User-specified	Expression in NZ dTIMS user-specified based on sealing chip size
Side force coefficient (SFC)	User-specified	User-specified	
Shoving (ASH)	0	NA	Not considered in HDM-4
Flushing (AFL)	0	NA	Not considered in HDM-4
Previous all cracking (PCA)	ACA before resurfacing	Expression	Expression used by HDM-III and HDM-4 considers previous cracking value before resurfacing or overlay
Previous wide cracking (PCW)	ACW before resurfacing	Expression	

In the case of mill and replace treatment, no reset value for the structural number is applied in the NZ dTIMS Setup. However the reset value of SNP in the HDM-4 system is based on the thickness of the surfacing, base milled, and the AC overlay added:

$$SNP_{daw} = SNP_{dbw} - SN_{mill} + SN_{sw}$$

where:  $SN_{mill}$  strength contribution of layers removed by milling  
 $SN_{sw}$  strength contribution of new surfacing layer  
 $SNP_{daw}$  adjusted structural number for dry season after works  
 $SNP_{dbw}$  adjusted structural number for dry season before works  
 $SN_{mill} = 0.0394 \{ \min (MILLD, HSNEW_{bw}) a_n + \min [\min (MILLD - HSNEW_{bw}, 0), HSOLD_{bw}] a_0 + \min (MILLD - HSNEW_{bw} - HSOLD_{bw}, 0) a_b \}$

$$SN_{sw} = 0.0394 HSNEW_{aw} a_{sw}$$

where:  $MILLD$  specified depth of milling  
 $a_n$  strength coefficient of new surfacing layers before works  
 $a_0$  strength coefficient of old surfacing layers before works

a	strength coefficient of old surfacing layers before works
$a_{sw}$	specified strength coefficient of new surfacing layer
$HSNEW_{aw}$	thickness (mm) of new surfacing layers after works
$HSNEW_w$	thickness (mm) of new surfacing layers before works
$HSOLD_w$	thickness (mm) of old surfacing layers before works

Thus it is recommended to use the HDM-4 principle in the NZ dTIMS Setup.

The model form used for resetting the roughness value for the AC overlay has been simplified in HDM-4 as the HDM-III model seemed to be unnecessarily complex (Morosuik et al. 2000). The NZ dTIMS Setup is using the HDM-III reset model. As the simpler model form will be easier to calibrate, it was decided to verify the following HDM-4 reset model for the New Zealand condition:

$$IRI_a = a_0 + a_1 \max (IRI_b - a_0, 0) * \max (a_2 - H, 0)$$

where:	$a_0$	general standard of workmanship (default 2)
	$a_1$	sensitivity of IRI reduction to overlay thickness(default 0.01)
	$a_2$	maximum thickness of the overlay (default 80mm)
	H	thickness (mm) of the overlay
	IRI	International roughness index: $IRI_a$ after works; $IRI_b$ before works

### 5.4.3 Reconstruction

Reconstruction is the operation which replaces or reconstructs one or more pavement layers. Reconstruction is applied to severe distresses in the pavement or severe deformation. The following generic treatments available in the NZ dTIMS Setup can be considered as reconstruction operations:

- Reconstruction AC (asphalt concrete);
- Reconstruction ST (surface treatment);
- Thin and thick granular overlay;
- Rip and remake AC;
- Rip and remake ST.

All surface defects including rutting are reset to '0' in both HDM-4 and the NZ dTIMS Setup in this case. Roughness values are generally not dependent on old values, and are reset based on the local maintenance practice and construction quality. By default the reset value for roughness in the NZ dTIMS Setup is taken as 2m/km for AC surfacing and 2.5m/km for ST surfacing. Some study is required to verify the appropriateness of these values.

Considerable effect on pavement strength will result after reconstruction of the pavement. In HDM-4 the SNP will be user-defined, based on the strength of any added pavement layers. This principle is applied in the NZ dTIMS Setup also.



## **5.5 Conclusions**

- Two additional maintenance treatments - improvement works and new constructions - are available in the HDM-4 system. The works effect (WE) of these categories were not considered in the NZ dTIMS Setup, which was mainly developed as a predictive modelling for pavement management system, and does not consider improvements and new construction activities.
- Routine maintenance operations have extensively been modified in the HDM-4 system. The effects of drainage maintenance, patching of wide structural cracking, and time lapse in pothole patching appeared to be relevant for New Zealand conditions. Hence, the effects of these maintenance activities have been considered for further analysis in this study.
- The method used by HDM-4 for the adjusted structural number (SNP) calculation for mill and replace treatment was found to be reasonable and can be used for the NZ dTIMS Setup.
- The simplified model used in HDM-4 for resetting of roughness after overlay will be easier to calibrate, and hence will be considered for verification for New Zealand conditions.

## 6. Data Requirements for HDM-4 Models

### 6.1 Introduction

Most of the HDM-4 models are enhancements of the HDM-III models. Hence, a large portion of the input data for NZ dTIMS (HDM-III) and HDM-4 models is similar. However, some additional data were required for new parameters and models that have been added in the HDM-4 system.

Road condition and inventory data in New Zealand are kept in the RAMM database. As some data stored in RAMM are different from the format used by dTIMS, a specific procedure was developed, in which a software application, dTIMS Interface Program, was prepared to convert the RAMM data into HDM model format, in order to prepare the dTIMS data input file (HTC 2000). Hence, the additional data requirements for HDM-4 models are discussed in this chapter. Included also is a review of the RAMM database on the availability of these data.

### 6.2 Pavement Strength

The input data required for estimating pavement strength in HDM-4 are given in Table 6.1.

**Table 6.1 Input data required to estimate HDM-4 Pavement Strength from RAMM.**

HDM-4 Requirement	Included in NZ dTIMS	Data used	RAMM table
Dry season SNP	•	<ul style="list-style-type: none"> <li>• Falling weight Deflectometer (FWD)</li> <li>• Benkelman Beam (BB)</li> <li>• Layer properties</li> </ul>	<ul style="list-style-type: none"> <li>• FWD data table</li> <li>• BB data table</li> <li>• Pavement structure table</li> </ul>
Length of the dry season			
Wet/dry season SNP ratio			

The dTIMS Pavement Strength Program can be used to calculate the adjusted structural number (SNP) from FWD data, BB deflection data and layer properties data. The reading date when a survey was done is also kept in RAMM that allows the user to find out whether the data relates to a dry or wet season.

The length of a dry season for various regions in New Zealand can be estimated based on National Institute of Water and Atmosphere Research Ltd. (NIWA) data. Properly planned surveys with the collection of the FWD data in dry and wet seasons on a few controlled sections could be used to define the representative wet/dry season SNP ratio for a given road network.

### 6.3 Drainage and Environmental Factors

The input data required for estimation of drainage and environmental parameters in HDM-4 are summarised in Table 6.2.

**Table 6.2 Input parameters for drainage and environmental factors.**

HDM-4 requirement	Included in NZ dTIMS	RAMM Table	Remarks
Type of drainage channel		Water Channel	
Drainage condition		Rating	
Mean monthly rainfall (MMP)	•		Regional MMP available
Moisture index	•		

Type of drainage channel is available in the RAMM database system, for which drainage condition is visually rated annually or every alternative year. The estimated remaining life of the drain is also available in RAMM.

### 6.4 Traffic Characteristics

The traffic data available in RAMM was suitable for HDM-4 RD and WE models, and no additional data needed to be collected. The input data required to estimate the traffic parameters in HDM-4 are given in Table 6.3.

**Table 6.3 Input parameters for traffic characteristics required for HDM-4 models.**

HDM-4 requirement	Included in NZ dTIMS	RAMM Table	Remarks
Traffic Volume	•	Traffic	Traffic volume is mostly estimated, not counted
Mode-wise distribution	•	Loading	Estimated mode-wise distribution
Axle load factors	•	Loading	Based on PEM
Traffic growth rates	•	Treatment Length	Recently included in RAMM but not generally populated*. NZ dTIMS used default value specified by PEM

\*Populated = data field is available but no data have been entered

### 6.5 Construction Quality

The required data for construction quality in HDM-4 are given in Table 6.4.

The quality of maintenance work was found to vary considerably in New Zealand, and to date, no information on construction quality is kept in the RAMM database. A recommendation is to keep the data on the surface defect indicator (CDS) and base preparation defects indicator (CDB) in the RAMM database. A detailed procedure on defining these parameters for New Zealand conditions also needs to be developed, though initially the default values for CDS and CDB available in HDM-4 can be used for network level monitoring.

**Table 6.4 Input parameters for construction quality required for HDM-4 models.**

HDM-4 requirement	Included in NZ dTIMS*	RAMM Table	Remarks
Surface defects indicator, CDS	•		NZ dTIMS used '0' by default
Base preparation defects indicator, CDB	•		NZ dTIMS used '0' by default.
Relative compaction of the base, COMP	•		

CQ is used in NZ dTIMS, while HDM-4 uses CDS and CDB to express construction quality indicator.

## 6.6 Pavement Condition

Most of the condition parameters are found to be similar for HDM-4 and the NZ dTIMS Setup except for 'edge break'. Table 6.5 shows the HDM-4 data requirements, together their availability in the RAMM database.

**Table 6.5 Input parameters for pavement condition required for HDM-4 models.**

HDM-4 requirement	Included in NZ dTIMS	RAMM Table	Remarks
All structural cracking	•	Rating	
Wide structural cracking	•		
Transverse thermal cracking		Rating	
Ravelling	•	Rating	
Potholing	•	Rating	
Mean rut depth	•	HSD rutting/Rating	
Roughness	•	HSD roughness/Roughness	
Edge break		Rating	
Texture depth	•	HSD texture	
Side friction coefficient	•	HSD SCRIM	
Previous all cracking	•	Treatment length	
Previous wide cracking	•	Treatment length	

A review of RAMM databases showed that all the condition data required for HDM-4 RD and WE models are available in RAMM database for New Zealand state highways and arterial networks. In the RAMM databases of local RCAs, data on rutting, texture depth and SFC were not available. Some data are collected in different format than that required by HDM models in the RAMM rating system, although a procedure for converting these data to HDM format are available and well documented (HTC 2000).

## 6.7 Pavement History

The input data required to estimate the pavement history parameters are given in Table 6.6.

**Table 6.6 Input parameters for pavement history required for HDM-4 models.**

HDM-4 requirement	Included in NZ dTIMS	RAMM Table	Remarks
Thickness of new surfacing layer (HNEW)	•	<ul style="list-style-type: none"> <li>• Top surface</li> <li>• Treatment length</li> </ul>	Available only for asphalt concrete surfacing
Thickness of old bituminous layers (HOLD)	•	<ul style="list-style-type: none"> <li>• Carriageway surface</li> </ul>	Need to summarise; available only for AC surfacing
Preventive treatment age (AGE1)			Not used in NZ
Surfacing age (AGE2)	•	<ul style="list-style-type: none"> <li>• Top surface</li> <li>• Treatment length</li> </ul>	
Construction age (AGE3)	•	<ul style="list-style-type: none"> <li>• Pavement structure</li> <li>• Treatment length</li> </ul>	
Base construction age (AGE4)			Can be estimated on historic data
Roughness after reconstruction	•	<ul style="list-style-type: none"> <li>• HSD roughness</li> <li>• Roughness</li> </ul>	Can be estimated on historic data
Time lapse in patching potholes			Depends on maintenance practice for a given RCA

Most of the pavement history data required for HDM-4 are similar to the NZ dTIMS Setup. These data are available in RAMM or generated on the standard procedure developed for the NZ dTIMS Setup, based on the data available in RAMM (HTC 2000).

Only preventive treatment age (AGE1), base construction age (AGE4), and time lapse in pothole patching (TLF) were not included in the NZ dTIMS Setup. AGE1 is not applicable in New Zealand as preventive maintenance is not widespread. AGE4 can be estimated based on historical information. The general routine maintenance practice in New Zealand warrants 100% patching of potholes. The response periods may vary slightly among different RCAs but do not generally exceed one month. Hence, most additional historic data required for HDM-4 are easily obtainable.

## 6.8 Calibration Factors

Some additional calibration factors have been included in the HDM-4 system compared with those from the NZ dTIMS Setup. Calibration issues are discussed in Chapter 7.

## 6.9 Conclusions

- Most of the data required for HDM-4 RD and WE modelling are similar to those required for the NZ dTIMS Setup. These data are either available in the RAMM database or generated, based on the well-defined standard procedures for converting RAMM data to HDM format.
- A few additional data were required for HDM-4 models and these are given in Table 6.7. Most of these data are either available in different format or can easily be obtained.

**Table 6.7 Additional data required for HDM-4 Models.**

<b>Data Type</b>	<b>Parameter</b>	<b>Remarks</b>
Pavement Strength	<ul style="list-style-type: none"> <li>• Length of the dry season</li> <li>• Wet/dry season SNP ratio</li> </ul>	
Drainage and Environmental factors	<ul style="list-style-type: none"> <li>• Type of drainage channels</li> <li>• Drainage condition</li> </ul>	Data available in RAMM
Pavement Condition	<ul style="list-style-type: none"> <li>• Edge break</li> </ul>	Data available in RAMM
Construction Quality	<ul style="list-style-type: none"> <li>• Surface construction indicator</li> <li>• Base construction indicator</li> </ul>	Procedures need to be developed to define these parameters
Pavement history	<ul style="list-style-type: none"> <li>• Time lapse in pothole patching</li> </ul>	Can be based on the response time in local maintenance practice

## 7. HDM-4 Model Calibration Issues

### 7.1 Introduction

To apply pavement deterioration models properly, they need to be customised to local conditions. Although the HDM models were developed from a good dataset using sound principles, as evidenced by their successful application in over 100 countries, model calibration was found to be essential to ensure that the results are reasonable for local and regional conditions.

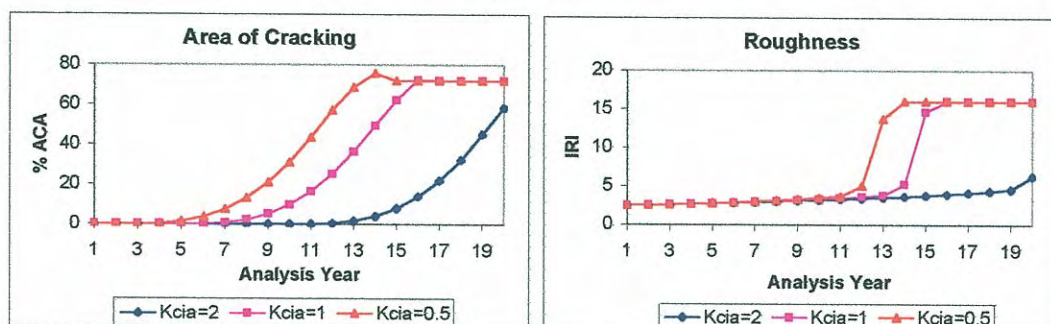
This chapter starts with a brief background on HDM model calibration. Thereafter, a detailed discussion is presented on HDM model calibration factors, additional calibration factors incorporated in HDM-4, and the data required for their estimation. Also included is a note on the past experience on calibration input data and available calibration methods.

### 7.2 An Overview of HDM Model Calibration

#### 7.2.1 Need for HDM Model Calibration

Calibration is required for HDM-4 models to incorporate the local construction techniques and environmental conditions. For this, certain coefficients are purposely embedded in the HDM models that are known as calibration factors (deterioration factors). Figure 7.1 illustrates the change in area of all cracking and roughness with change in crack initiation ( $K_{cia}$ ) and crack progression ( $K_{cpa}$ ) coefficients. The values of these factors are changed for different environmental and physical characteristics.

**Figure 7.1** Impact of values of crack initiation calibration factor ( $K_{cia}$ ) in pavement deterioration, for area of cracking (ACA) and roughness progression (IRI).



#### 7.2.2 Calibration Levels

Bennett & Paterson (2000) indicated that the HDM model calibration can be divided into the following three levels based on level of effort and time required:

Level 1 **Basic Application** – determines the values of required basic input parameters, adopts many default values, and calibrates the most sensitive parameters with best estimates, desk studies or minimal field surveys.

Level 2 **Calibration** – requires measurement of additional input parameters and moderate field surveys to calibrate key predictive relationships to local conditions. This level may entail slight modification of the model source code.

Level 3 **Adaptation** – undertakes major field surveys and controlled experiments to enhance the existing predictive relationships or to develop new and locally specific relationships for substitution in the source code of the model.

Level 1 calibration is largely based on secondary sources, i.e. it is a desk study.

Level 2 calibration uses direct measurements of local conditions to verify and adjust the predictive capability of the model. It requires a higher degree of data collection and precision than for a Level 1 calibration, and extends the scope.

Level 3 consists of structured field surveys and experimental studies conducted 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.

In this project, for the purpose of verification of the suitability of the HDM-4 models to New Zealand conditions, Level 2 Calibration was carried out based on the available condition and historical data with some additional data collection and verification.

### 7.2.3 Data Sources

The three methods generally used to analyse pavement condition data for the purpose of pavement deterioration model calibration are as follows (HTC 2000):

- **Snap-shot or Slice-in-Time** analysis where a set of current data is compared to construction and/or surfacing ages;
- **Historic Time-Series** analysis where data over a period of years for the same road section is analysed to determine incremental changes; and
- **Controlled studies** where calibration sites are defined and accurately monitored for several years into the future.

Level 1 or 2 calibration would normally use the first two methods. The third method is preferred for conducting a Level 3 calibration.

The snap-shot method is the least demanding in terms of data, but normally the least accurate way of calibration. The data may be taken from routine surveys or special surveys of a sample of road sections made for calibration purposes. Accurate pavement history is essential to have any chance of success for this type of calibration.

Where continuous datasets spanning 5-6 years or more are available, the historic time-series method provides a more accurate way of HDM model calibration than the slice-in-time method.



The only way of completely calibrating the HDM pavement deterioration model is by conducting a study into the rate of pavement deterioration with precise data collection equipment. Hodges et al. (1975), GEIPOT (1982), and Paterson (1987) included various methodologies on how to conduct these surveys. Long-term pavement performance maintenance (LTPPM) sites were established in Australia to study the influence of various maintenance treatments. Transit New Zealand has also identified about 60 calibration sites throughout New Zealand and is planning to consider these as pavement deterioration monitoring sites.

### 7.3 Calibration Factors

HDM-4 models are developed in such a way that they can be applied to a wide range of environments. To facilitate local adaptation, 23 calibration factors were incorporated in HDM-4 RD models for bituminous pavements. HDM-III models already have 9 of them. NZ dTIMS Setup, being based on HDM-III models, incorporated all the calibration factors used by HDM-III models and, in addition, it included calibration factors for ravelling progression (Kvp), texture depth (Ktd) and skid resistance (Ksfc). The calibration factors included in different systems are given in Table 7.1.

**Table 7.1 HDM model calibration factors.**

Calibration Factor	RD Model	HDM-III	NZ dTIMS	HDM-4
Kf	Wet/dry season SNP ratio			•
Kddf	Drainage factor			•
Kcia	All structural cracking initiation	•	•	•
Kcpa	All structural cracking progression	•	•	•
Kciw	Wide structural cracking initiation *	•	•	•
Kcpw	Wide structural cracking progression *	•	•	•
Kcit	Transverse thermal cracking initiation			•
Kcpt	Transverse thermal cracking progression			•
Kvi	Ravelling initiation	•	•	•
Kvp	Ravelling progression		•	•
Kpi	Pothole initiation	•	•	•
Kpp	Pothole progression	•	•	•
Keb	Edge break			•
Krid	Initial densification of rutting	•	•	•
Krst	Structural deformation of rutting	•	•	•
Krpd	Plastic deformation of rutting			•
Krsw	Surface wear of rutting			•
Kgm	Environmental coefficient of roughness	•	•	•
Ksnpk	Adjusted structural number for roughness			•
Kgp	Roughness progression	•	•	•
Ktd	Texture depth progression		•	•
Ksfc	Skid resistance		•	•
Ksfcs	Speed effects of skid resistance			•

\* All structural cracking calibration factors are used in NZ dTIMS and HDM-4.

The significance of additional calibration factors incorporated in HDM-4 is briefly discussed in the ensuing sections.

## 7.4 Additional Calibration Factors

### 7.4.1 Drainage Factor (Kddf)

The condition of the drains will deteriorate unless they are maintained properly. The annual incremental change in the drainage factor is calculated every year and is used in the prediction of adjusted structural number and subsequently other distresses. The calibration coefficient 'Kddf' is used to calibrate the rate of deterioration of the drainage factor, and the data required for calibration of drainage factor are:

- Life of the drain; and
- Limiting values for drain/drainage condition.

Limiting values (maximum and minimum) for drain and drainage conditions and estimated life of the drain have to be determined first. Then every year the drainage condition has to be rated depending on the prevailing condition. At least 5-6 years of data needs to be available before attempting an initial calibration for drainage factor. The RAMM condition rating of the drain only defines good or poor condition of drain and hence cannot be used for the calibration exercise. The sensitivity analysis (Chapter 10) showed that 'Kddf' is relatively not so sensitive and hence a default value of '1' can be used for general purposes.

### 7.4.2 Wet/Dry Season SNP Ratio (Kf)

In HDM-4 RD models, annual average structural number is used instead of seasonal structural number. Wet/dry season SNP is calculated based on the following relationship:

$$f = K_f \left[ 1 - \frac{(1 - \exp(-a_1 \text{MMP}))}{a_1} (1 + a_2 \text{DF}_a) (1 + a_3 \text{ACRA}_a + a_4 \text{APOT}_a) \right]$$

where:

f	=	SNP <sub>w</sub> / SNP <sub>d</sub> ratio
MMP	=	mean monthly precipitation (mm/month)
DF <sub>a</sub>	=	drainage factor at start of analysis year
ACRA <sub>a</sub>	=	total area of cracking at start of analysis year (%)
APOT <sub>a</sub>	=	area of potholing at the start of the analysis year (%)
K <sub>f</sub>	=	calibration factor for wet/dry season SNP ratio

Data required for the calibration purpose included:

- Structural number at different seasons;
- Cracking;
- Potholes; and
- Drainage factor.

The structural number at different seasons is not available. Hence, the testing of the model with the real condition was not possible. As the drainage condition was found to affect the pavement deterioration in New Zealand it is recommended to carry out further study in this regard.

### **7.4.3 Transverse Thermal Cracking Factors (Kcit, Kcpt)**

Transverse thermal cracking has been modelled in HDM-4. The 'Kcit' allowed for calibration of the initiation period of the thermal cracking, whereas 'Kcpt' was incorporated in order to calibrate for the progression of the thermal cracking. Calibration of these models will become more important in harsh climates, in which large variations in the daily temperatures occur throughout a year. Such climates are generally not experienced in New Zealand and, hence, these factors are not considered to be relevant to New Zealand conditions.

### **7.4.4 Rutting Factors (Krpd, Krsw)**

Rutting caused by plastic deformation and surface wear have been added to the HDM-4 rut depth model. Calibration coefficient for plastic deformation of the rutting (Krpd) allowed calibrating plastic deformation component of the rutting based on material properties and construction quality. The default value for 'Krpd' for surface-treated pavements was taken as '0' in the HDM-4 system (Morosiuk et al. 2000). However in New Zealand, with the application of multi-layers of chipseal surfacing, pavements are behaving like poly-structured bituminous pavements and show rutting in the form of plastic deformation. Further research is recommended in this regard.

The surface wear model is applied to environments where vehicles use studded tyres during freezing periods, which results in wear of the surface. However, since studded tyres are generally not used in New Zealand, the surface wear component of rutting is considered not applicable to New Zealand conditions.

### **7.4.5 Edge Break Factor (Keb)**

The edge break model is one of the new models introduced in the HDM-4 system. From discussions with RCAs, edge break is observed particularly on narrow rural roads, and hence the model is applicable to New Zealand conditions. In HDM-4 edge break is adjusted before cracking and ravelling at the end of the analysis year, and thus, adjusted areas of cracking and ravelling are dependent on edge break predictions made each year. Edge break calibration coefficient (Keb) allowed calibrating the edge break model for adaptation to local conditions.

### **7.4.6 Surface Texture Coefficients (Ktd, Ksfc)**

Texture depth and skid resistance models are additional models available in the HDM-4 system. Calibration factors for texture depth (Ktd) and skid resistance (Ksfc) are included in the HDM-4 system to calibrate these models to local conditions. However, similar models have already been included in the NZ dTIMS Setup.

### **7.4.7 Adjusted Structural Number for Roughness (Ksnpk)**

'Ksnpk' has been incorporated in HDM-4 to estimate the annual reduction in pavement strength due to cracking, that is used in the structural component of roughness. Many road agencies use roughness as the trigger to apply treatments considering the fact that other distresses contribute to the roughness progression. To calibrate the roughness model to the local conditions, it is necessary to determine the applicable value for 'Ksnpk'.

Data required are:

- SNP at different time intervals;
- Area of cracking;
- Surface layer thickness (old & new);
- Area of previous cracking.

SNP at different intervals was not available for calibration and hence at this stage the default value of '1' is used.

## **7.5 Sensitivity of Calibration Factors**

### **7.5.1 Background**

Not all the calibration coefficients used in the NZ dTIMS Setup were found very sensitive to the output of the analysis (Pradhan 2001). Calibration of various coefficients requires extensive data collection efforts and is a costly affair. Hence it is preferable to carry out sensitivity analysis of calibration coefficients and calibrate only those coefficients which are most sensitive.

### **7.5.2 Impact Elasticity**

Bennett & Paterson (2000) quantified the sensitivity of parameters by impact elasticity, which is simply a ratio of the percentage change in a specific output (parameter) to the percentage change of the input parameter, holding all other parameters constant at a mean value.

For example, if a 10% increase in traffic loading causes a 5% increase in roughness after 15 years, the impact elasticity term of traffic loading for that roughness is 0.5. If there was a 5% decrease, the value would be -0.5.

The higher the elasticity, the more sensitive is the model prediction. The data items with high and moderate impacts (S-I and S-II) should receive the most attention. The low to negligible impact (S-III and S-IV) items should receive attention only if time or resources permit. One usually assumes the default values for S-III and S-IV items since these generally give adequate results.

A preliminary analysis has been carried out to determine sensitivity of the HDM-4 models to the calibration factors. The traditional *ceteris paribus* (TCP) method was adopted, which is based on changing a single factor while holding all other factors constant. The sensitivity analysis carried out in this study for HDM-4 calibration factors is discussed in Chapter 10.

## **7.6 Calibration Data Issues**

Quality of the input data has the capability of changing the outcome of the pavement predictive modelling upside down and particularly in calibration studies. The past calibration studies in New Zealand have clearly brought out the quality and limitations of the data available in the country. Some of the key issues raised in the

past (Cenek et al. 1998) regarding the data availability for calibration studies are summarised below:

- Pavement strength is one of the critical inputs to the calibration of models. RAMM database stores the strength data in the form of Benkelman Beam (BB) Deflections and Falling Weight Deflections (FWD). However, as these data are generally not available for most parts of the network in the country, this imposes a major constraint in selection of the calibration sections.
- Traffic volume data is extremely variable in nature, and in most cases, it is estimated instead of counted. Traffic volume is one of the key input parameters that goes into deterioration models (Pradhan 2001), and ranges in New Zealand from very low (100 to 200 vpd) to very high (50 000 vpd). Hence, the actual traffic flows occurring on the network are needed in order to obtain reasonable results from the predictive modelling exercise.
- The existing pavement condition significantly influences the prediction of future pavement deterioration. The RAMM database provides the condition rating on a sample length of 50m for every 500m length, and is done in the wheelpath area only, but HDM models require distresses over the entire surface area.
- The quality of the construction should not be ignored in the predictive modelling exercise and particularly at the calibration stage. Unfortunately, the RAMM database makes no mention of construction quality, which makes it difficult to incorporate construction defects, if any, into the HDM models.

A recent Transfund research study on data quality (Bennett 2001) highlighted, in a comprehensive manner, the shortcomings of the data.

At the same time reasonable calibrations were carried out using the available time series HSD data (Hallet & Tapper 2000). Similarly major consultants in New Zealand have carried out calibration exercises for 'Performance Specified Maintenance Contract'. Unfortunately, these data are not made available to the public domain.

All the issues relating to the deficiency in the available data were considered during the preparation of the calibration database. Field visits to verify the appropriateness of the available data and collection of the additional data were undertaken to make sure that a reliable database could be compiled for calibration exercise. (Chapter 8 has more details.)

## **7.7 Calibration Methods**

No published documentation was available for calibrating the additional calibration coefficients. Hence, the following three potential methods have been initially explored for calibration.

### **Method 1**

- Select the values of CDS and CDB based on local construction practices and site condition as shown in Table 7.2.

**Table 7.2 Default CDS and CDB values.**

Indicator	Top Surface / Criteria	Value
CDS	Brittle - binder content less than required/ optimal design	0.5
	Normal - optimal binder content	1.0
	Soft - binder content is more than required/ optimal design	1.5
CDB	Poor gradation of material only	0.5
	Poor aggregate shape only	0.5
	Poor compaction only	0.5
	Combination of any two of the above	1.0
	All the above	1.5
	None of the above	0.0

Source: Morosiuk et al. (2000)

- Analyse the data for predicted and observed distresses. Compare the values.
- Estimate the calibration factors. Two methods are available for the purpose:
  - Compare the observed values against predicted values. Plot a scatter diagram and conduct some statistical analysis. Adjust the calibration coefficients to get the best fit between observed and predicted values.
  - Calculate the error values. Adjust the calibration coefficients to minimise the error function.

**Method 2**

- Adopt the national calibration coefficients evaluated for the HDM-III model for available parameters (Cenek et al. 1998) and default values for remaining coefficients.
- Analyse the data for predicted and observed distresses. Compare the values.
- Compare the observed values against predicted values. Plot a scatter diagram and conduct some statistical analysis. Adjust the CDS and CDB, and if required other calibration coefficients, to get the best fit between observed and predicted values.

**Method 3**

- Adopt the national calibration coefficients evaluated for the HDM-III model for available parameters (Cenek et al. 1998) and default values for remaining coefficients.
- Select values for CDS and CDB from Table 7.2.
- Compare the observed values against predicted values. Plot a scatter diagram and conduct some statistical analysis.
- Adjust the model coefficients, such as  $a_0$ ,  $a_1$  etc., to get the best fit between the observed and predicted values.

Method 1 was considered for the initial calibration of HDM-4 models in this study.

## **7.8 Conclusions**

- HDM-4 models require customisation of the models to the local conditions. For this purpose 23 calibration factors were included in for bituminous paved roads.
- No definite calibration methods are available in the HDM-4 document series for calibration factors that have been added in HDM-4. It is proposed that some principles applied for HDM-III models can be extended for the calibration of HDM-4 models.
- The experience of calibrating HDM models to New Zealand conditions has proved cumbersome, because the data available in RAMM are not accurate enough for a detailed calibration.
- Three different methods for initial calibration were identified.

## **8. Assembly of Project Databases**

### **8.1 Introduction**

The project required the assembly of a number of databases for different analyses to be carried out. These included:

- Synthetic dataset for the parametric study of the HDM-4 models;
- Synthetic dataset for the sensitivity analysis of the calibration coefficients;
- Real network dataset for the calibration of RD and WE models; and
- Real network dataset for comparison of model outputs of HDM-4 and NZ dTIMS Setup.

As the main objective of the project was to verify the suitability of the HDM-4 models to New Zealand conditions, it also had to ensure that all the possible conditions in New Zealand were well represented. Hence, RAMM databases of the following RCAs, representing a range of geographical and traffic conditions, were considered for the project, were obtained:

- Napier State Highway;
- Northland State Highway;
- Auckland City Council;
- North Shore City Council; and
- Southland District Council.

The analysis of the RAMM database and assembly of the databases were slightly different depending on the purpose for which they were used.

### **8.2 Data Acquisition and Processing**

#### **8.2.1 RAMM Tables Used**

The following tables in ASCII format from the RAMM system were taken:

- Roadname table
- Carriageway table
- Carriageway surface table
- Top surface table
- Pavement layer table
- Pavement structure table
- Rating table
- Roughness table
- HSD Roughness table
- HSD Rut table
- FWD Data table
- Traffic table
- Loading table



- Surface water channel table
- Treatment length table

The HDM-4 Information Management System (HIMS) was used to process the data and prepare the database in the required format. Databases prepared for different analyses required different precision with different objectives and, hence, are discussed separately in the sections below.

### 8.2.2 Analysis of RAMM Database

A preliminary analysis of the above mentioned RAMM tables was carried out to identify the availability of the required data for the study. It included:

- Minimum and Maximum range of various HDM model input parameters applicable for New Zealand conditions;
- Availability of additional data required for HDM-4 RD and WE models in the RAMM database;
- Selection of sections with good history for the preparation of the real network dataset with detailed information required for the model calibration; and
- Preparation of the real network dataset with 600 sections to compare the predictions of the NZ dTIMS Setup and HDM-4 model.

### 8.3 Preparation of Synthetic Datasets for Parametric Study

To compare the NZ dTIMS Setup and HDM-4 model, a synthetic dataset was prepared considering the ranges for each parameter that are applicable to New Zealand conditions. The synthetic dataset consisted of a total of 96 sections made out of the matrix shown in Table 8.1.

**Table 8.1 Synthetic data matrix used for preparation of synthetic dataset.**

Pavement Type	Traffic Volume (AADT)	Pavement Strength (SNP)	Pavement Condition	Pavement History
Asphalt concrete (AMGB)	Low (200)	Weak (2.0)	Good	New pavement
Surface treatment (STGB)	Medium (2000)	Average (3.5)	Poor	Resealed pavement
	High (10,000)	Strong (6.0)		
	Very high (20,000)			

In preparation of the synthetic dataset, reasonable and applicable values for New Zealand conditions were considered. For good pavement condition, initial values for surface distresses have been assigned while for poor pavement conditions a mix of values have been used.

A three-letter code was assigned for each combination, e.g. LWG, which means low traffic, weak pavement and good condition of AC pavement type. Suffix '1' was used to differentiate ST pavements from AC pavements. For example 'LWG' means low-weak-good pavement for AC pavement while 'LWG1' indicates low-weak-good ST pavement. To investigate the difference between the deterioration of a resealed

pavement and an original construction (new pavement), separate scenarios were considered for 'new pavement' and 'resealed pavement'.

The preliminary analysis was carried out for all 96 combinations. To draw meaningful conclusions, detailed comparisons were made for 16 combinations. The 8 combinations shown in Table 8.2 were analysed for 'new' pavement as well as for 'resealed' pavement for both AC and ST surfacing.

**Table 8.2 Synthetic dataset descriptions for preparing synthetic data.**

Dataset	Description
LWG	Low traffic Weak pavement with Good pavement condition
LWP	Low traffic Weak pavement with Poor pavement condition
LSG	Low traffic Strong pavement with Good pavement condition
LSP	Low traffic Strong pavement with Poor pavement condition
HWG	High traffic Weak pavement with Good pavement condition
HWP	High traffic Weak pavement with Poor pavement condition
HSG	High traffic Strong pavement with Good pavement condition
HSP	High traffic Strong pavement with Poor pavement condition

The dataset considered for detailed comparison reasonably represented the real conditions like low traffic volume on weak pavement, and high traffic volume on strong pavement, etc. The extreme cases such as low traffic on strong pavement and high traffic on weak pavement were also included in the dataset in order to compare the pavement deterioration between different possible scenarios.

#### 8.4 Preparation of Synthetic Dataset for Sensitivity Analysis

As described in Chapter 7.5.2, the traditional ceteris paribus (TCP) method, based on 'changing a single factor while keeping all others constant' was used for the sensitivity analysis. Five different levels (within the allowable range) for 23 calibration factors were considered while preparing the dataset. Four different scenarios were considered to accommodate the effects of traffic, pavement strength and pavement surface type, and are shown in Table 8.3.

**Table 8.3 Scenarios for preparing synthetic data for sensitivity analysis.**

Scenario	Description	Traffic, AADT	Strength (SNP)
ACL	AC surface with Low traffic	200	2.0
ACH	AC surface with High traffic	10,000	3.5
STL	ST surface with Low traffic	200	2.0
STH	ST surface with High traffic	10,000	3.5

A customised software program using Visual Basic was developed for preparing the dataset for the analysis. The dataset containing 460 sections (23 calibration factors, 5 levels, 4 scenarios) was used to run the HDM-4 analysis.

### **8.5 Preparation of Synthetic Dataset for Initial Calibration**

Special considerations were given while preparing the data for calibration purposes. The data were prepared in a number of stages:

- Prepare the database with good quality time series data;
- Prepare the database without maintenance treatment;
- Prepare the database with 500 sections for snap-shot calibration method; and
- Prepare the database with 50 sections for time-series calibration method.

While processing the RAMM data for preparing the database with time-series data, the following steps were taken to make sure that the available data are the best possible quality:

- Because of doubts on the validity of very old data, only data from 1990 were included in the analysis.
- Data were organised into different year/lane, based on lanes rather than entire roads for more accurate results.
- Data were summarised in 100m sections for all roads. This was to counteract the problem of data in RAMM being not well referenced, making data from different years not directly comparable, and also to compensate for the localised defects.
- In the case of the network where 10% sampling is used, only those sections were considered in which the inspection section (of a rating section) falls.
- If a treatment date was before the survey date, the treatment was considered to have been done in that year, because the survey would reflect the changed pavement condition. If a treatment was done after the survey, then the treatment was assigned to the next year.

The database was further filtered by considering the following issues:

- Validation routine was run to make sure that data are within the allowable range and those not confirming to the requirement were eliminated;
- Sections with all the data required for calibration were considered; and
- Sections with the maintenance treatments were eliminated in the case of time-series method.

For the snap-shot calibration method, 500 sections were selected from different datasets. The sections were selected so that they represented a range of surfacing age, pavement strength, traffic volume, and other parameters.

For the time-series method of calibration, 50 sections were randomly selected, out of 100 different sections were originally identified. Field verification of the calibration sections were carried out to verify the accuracy of the data available in RAMM and the reliability of the RAMM data conversion to the HDM data format. After field verifications two of the sections were found to have been already resealed and, hence, did not reflect the condition included in RAMM. One of them was found to have localised defects relating to poor subsoil drainage, resulting in a number of

failures of the pavement. The rating condition in most of the cases showed some variation but within the margin of error, except one where data did not match with the RAMM data at all. Conversion coefficients used to change the RAMM data format to HDM format gave reasonable results.

Additional data were also collected using the ROMDAS system for the works effects (WE) calibration of the HDM models.

## **8.6 Preparation of Dataset for Model Outputs Comparison**

As stated earlier, Transit network, District Council and City Council data were considered to identify the variation in the model outputs for different conditions. The following datasets were chosen for comparison of models:

- Northland Transit Network;
- Southland District Council; and
- Auckland City Council.

Each dataset was thoroughly scanned and 200 treatment length sections from each dataset were selected so that they represented the typical values. The total number of sections considered for analysis were 600 in number.

Data available in the treatment length table were used for this purpose. The dTIMS interface program was used to prepare the NZ dTIMS data input file. Similarly HIMS software was used to prepare the HDM-4 data input file.

## **9. Comparison of HDM Models Using Synthetic Dataset**

### **9.1 Introduction**

The desk study carried out in Chapter 4 had identified the difference between the RD models included in the NZ dTIMS Setup and HDM-4 system, and the potential application of the enhancements carried out in HDM-4. To obtain quantitative comparisons, model predictions of NZ dTIMS Setup and HDM-4 models were compared using a common input dataset.

The parametric comparison carried out to quantify the enhancements is discussed in this chapter, but only for those enhancements relevant to New Zealand conditions.

### **9.2 Dataset Preparation and Model Setup**

A detailed description on the dataset preparation is given in Section 8.3.

As stated earlier, the NZ dTIMS Setup is primarily based on HDM-III models. Hence, comparison was done only for the NZ dTIMS Setup and HDM-4 models. HDM-III models are referred only when there is a difference between NZ dTIMS and HDM-III models. The HDM-4 interface built in for HIMS was used instead of HDM-4 software, as stated in Section 2.4 of this report.

The NZ dTIMS Setup and HDM-4 system use a number of calibration factors in RD and WE models. To nullify the effects of calibration factors in this parametric study, unit values were assigned to all the calibration factors. The ensuing sections describe the comparisons between the NZ dTIMS Setup and HDM-4 model outputs in detail by each parameter.

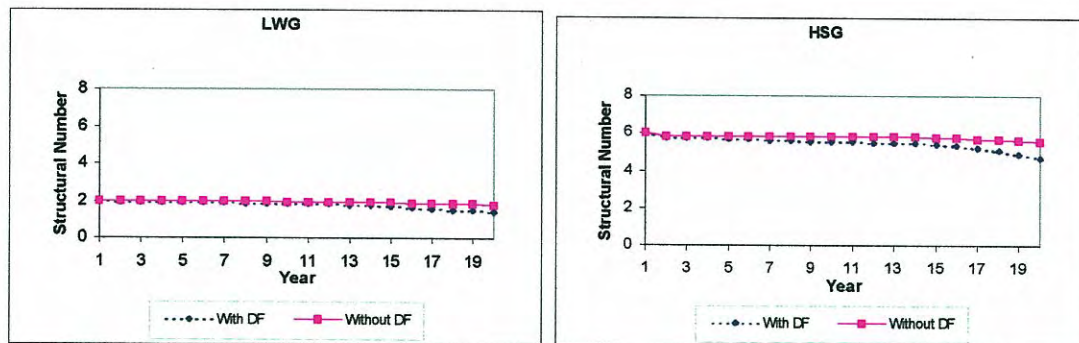
### **9.3 Drainage Factor**

The Drainage Factor (DF) was introduced in HDM-4 models to take into effect the drainage condition. DF can have any value between 1 (good) and 5 (very poor), depending on the type of drains and drainage condition (Morosiuk et al. 2000). DF was not included in the NZ dTIMS and HDM-III models, and hence, the HDM-4 model has only been used to highlight the effects of DF on pavement performance. The outputs of the HDM-4 model with and without DF were compared, taking the initial and maximum values for it as '2' and '5' respectively for comparison purposes (Appendix D gives applicable values for DF).

The desk study comparison of the models showed that the DF is used to estimate the annual adjusted structural number. Pavement strength is one of the key input parameters in pavement predictive modelling (Pradhan 2001), as it can influence initiation of all structural cracking, mean rut depth and roughness progression. Hence, analysis of the impact of DF on these parameters was carried out.

Figure 9.1 shows the variation in pavement strength (structural number) with and without drainage factor scenarios.

**Figure 9.1 Influence of drainage factor (DF) on pavement strength (Structural Number) over years 1 to 19.**



The pavement strength was reducing with the deterioration in drainage condition, and the reduction was as high as 20% for high traffic on the strong pavement (HSG) scenario. Pavement strength reduced even without any changes in drainage factor due to the cracking and potholing developed during the course of the time. The effect of drainage factor is cumulative, in that cracking and potholing causes reduction of SNP if drainage condition is not maintained. The effect of drainage factor on pavement strength increased with the increase in traffic volume.

Drainage factor had very little effect on cracking initiation period (through pavement strength). However, area of cracking was not influenced by drainage factor, because a time-based model for crack progression is used when no change in the year of crack initiation was observed.

There was some impact on rut depth and roughness values due to the drainage factor, because HDM-4 models use an adjusted structural number to predict mean rut depth and roughness.

The effect of the drainage factor on different modelling parameters for two common (LWG and HSG) scenarios in terms of percent variation is given in Table 9.1.

**Table 9.1 Effect of drainage factor (DF, % variation).**

Distress parameter	LWG	HSG
Initiation of all structural cracking	5	5
Mean rut depth	10	4
Roughness	0.5	3

#### 9.4 Structural Number

The desk study showed that adjusted structural number (SNP) used in HDM-4 takes into account the drainage condition and surface distress. SNP is included in the modelling of different distress parameters, i.e. cracking, rutting and roughness, etc. (Table 9.2). On the other hand the SNC that is used in the NZ dTIMS Setup does not



account for drainage condition. Besides, in HDM-III and in the NZ dTIMS Setup, SNC is adjusted based on surface condition only for the roughness modelling.

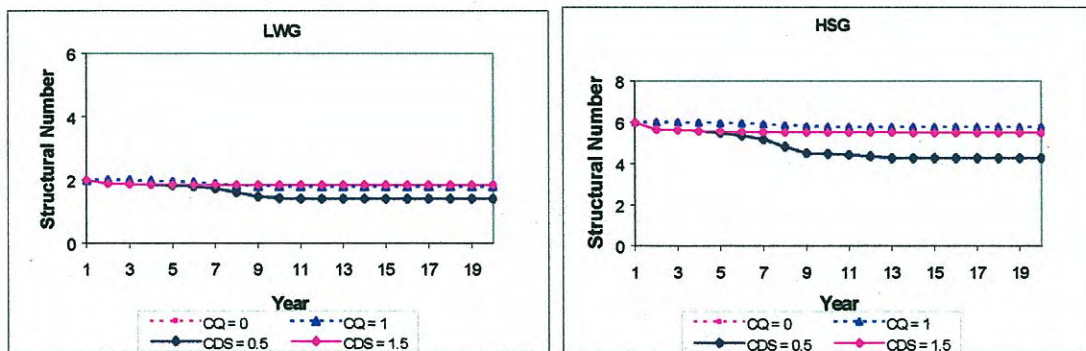
**Table 9.2 Use of annual adjusted structural number in the 3 models.**

Model	HDM-III *	NZ dTIMS *	HDM-4
Cracking			•
Ravelling			
Potholing			
Edgebreak			•
Rutting			•
Roughness	•	•	•

\* Modified structural number adjusted for potholing and cracking is used.

CQ (construction quality) was found to have no impact on pavement strength in the NZ dTIMS Setup, while CDS (construction defects of surfacing) had significant influence on pavement strength in HDM-4. Figure 9.2 shows the influence of construction quality on pavement strength (structural number) in the HDM-4 and NZ dTIMS models. Lower CDS values resulted in a decrease in SNP value, but this could be related to the rapid cracking progression and pothole progression.

**Figure 9.2 Impact of construction quality indicator on structural number for 2 types of pavement (LWG - low traffic, weak, good; HSG - high traffic, strong, good).**



## 9.5 Cracking

### 9.5.1 Cracking Initiation

Comparison was made between the outputs of crack initiation models used in the NZ dTIMS Setup and HDM-4 system. As stated earlier, the main difference between the NZ dTIMS and HDM-4 crack initiation models is the use of a continuous construction quality indicator for surfacing (CDS) in HDM-4 instead of a flag construction quality indicator (CQ).

The models predicted similar cracking initiation period when default values ('0' for CQ in the NZ dTIMS and '1' for CDS in HDM-4) were used. Changing the flag value (CQ) from '0' to '1' in the NZ dTIMS Setup had little impact on cracking

initiation time except in the worst possible scenario (heavy traffic on weak pavement), which was up to 40%. CDS in HDM-4 has an impact to the extent of 90% in the case of AMGB pavements and ranges from 65% to 85% for STGB pavements when its value changed from 0.5 to 1.5.

Tables 9.3 and 9.4 present the percent of variation in crack initiation periods for all structural cracking (ICA) and wide structural cracking (ICW) respectively.

**Table 9.3 Impact of construction quality on all structural cracking (ACA) for 2 pavement types (AMGB, STGB).**

Scenario	Variation in Crack Initiation Period (ICA) in %			
	AMGB		STGB	
	NZ dTIMS	HDM-4	NZ dTIMS	HDM-4
Low traffic on weak pavement	0.9	89	1	88
High traffic on weak pavement	35	89	40	88
Low traffic on strong pavement	0.1	89	0.1	88
High traffic on strong pavement	4.7	89	5.7	88

**Table 9.4 Impact of construction quality on wide structural cracking (ACW) for 2 pavement types (AMGB, STGB).**

Scenario	Variation in Crack Initiation Period (ICW) in %			
	AMGB		STGB	
	NZ dTIMS	HDM-4	NZ dTIMS	HDM-4
Low traffic on weak pavement	0.6	75	0.9	82
High traffic on weak pavement	22	65	32	65
Low traffic on strong pavement	0.1	80	0.1	82
High traffic on strong pavement	3.7	80	4.7	82

CQ is changed from '0' to '1' in NZ dTIMS

CDS is changed from '0.5' to '1.5' in HDM-4

The comparison showed that, with the introduction of CDS, HDM-4 cracking initiation models have become more flexible and thus enable customising for a variety of conditions.

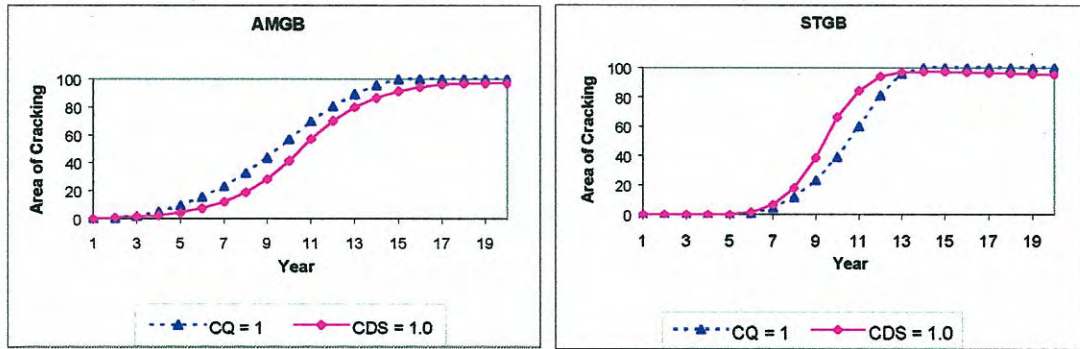
### 9.5.2 Cracking Progression

As stated earlier, NZ dTIMS and HDM-4 crack progression models considered time-based models. The area of all structural cracking as predicted by NZ dTIMS and HDM-4 crack progression models differed slightly for different default values for construction quality indicator.

The rate of progression of all structural cracking for asphalt concrete (AMGB) was found to be slightly higher in the NZ dTIMS Setup than HDM-4, while for STGB pavements it was less. Detailed comparison of the NZ dTIMS crack progression model with the actual HDM-III model revealed that the NZ dTIMS Setup is using simplified models. The deviation in area of cracking predicted by the NZ dTIMS Setup and HDM-4 system could be attributed to the use of these simplified models in the NZ dTIMS Setup. A typical case (low volume on weak pavement) is illustrated in Figure 9.3.

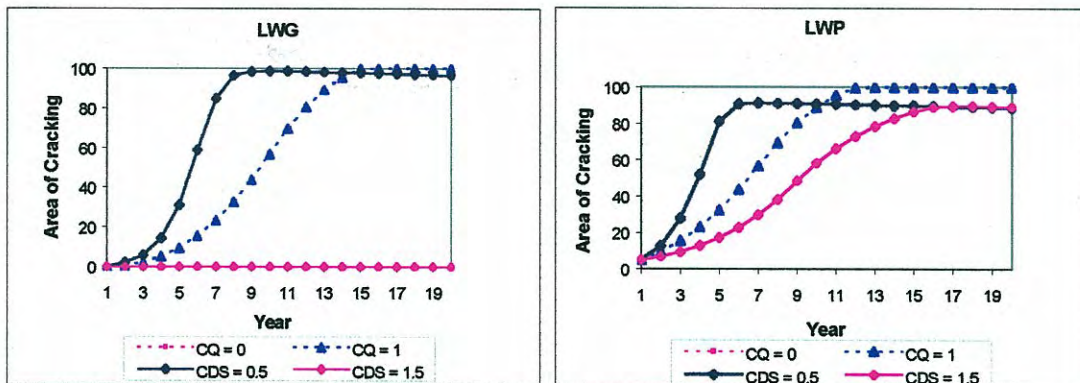


Figure 9.3 Variation in prediction of area of cracking (%) in 2 types of low volume, weak pavements (AMGB, STGB).



CQ used in the NZ dTIMS Setup did not influence cracking progression. In HDM-4, the rate of progression of cracking decreased as the value of CDS increased from 0.5 to 1.5. As a result, lower values for CDS predicted higher areas of cracking and vice-versa. The variation in the area of cracking was as high as 100% for different values of CDS. Figure 9.4 shows predicted area of cracking when CQ and CDS are different.

Figure 9.4 Area of cracking when CQ and CDS is changed for 2 pavement types (LWG, LWP).



In the case of LWP, the maximum area of cracking (ACA) is always less than 100% in HDM-4, because the limiting condition applied in HDM-4 models of the total damaged area is 100%. Calculated ACA is adjusted based on other surface distresses predicted (potholes and edge break).

Figure 9.4 shows that, with the introduction of CDS, the rate of cracking progression differs a lot. Hence, a detail research study is needed to calibrate the CDS values for different construction quality levels in New Zealand conditions.

## 9.6 Ravelling

### 9.6.1 Ravelling Initiation

The HDM-III ravelling initiation model is limited to surface treatment, slurry seal, and open-graded cold mix asphalt pavements. A fixed value of 100 years was taken for AC pavements to make sure that ravelling will not be initiated at all. The NZ dTIMS Setup adopted the HDM-III ravelling initiation model and considered a value of '50' for AC pavements. HDM-4 predicted the ravelling initiation period for AC pavements through an empirical relationship instead of using a constant value. But this equation incorporated the limitation of the data on which these relationships have been built upon and thus predicted a ravelling initiation period as high as 150 to 200 years for AC.

In the case of surface treatment pavements, the NZ dTIMS Setup and HDM-4 predicted similar initiation times. Ravelling initiation period for surface treatments was as low as 1.5 years for an extreme case.

### 9.6.2 Ravelling Progression

Ravelling progression model is turned off in the NZ dTIMS Setup by considering ravelling progression factor as '0'. HDM-III ravelling model adopted in the NZ dTIMS Setup was not predicting reasonable ravelling (scabbing) for New Zealand conditions (Cenek et al. 1998). However, for comparison purposes, ravelling progression factor was kept at '1' in both the NZ dTIMS Setup and HDM-4 system.

The rate of ravelling progression was slightly higher in the NZ dTIMS Setup than in the HDM-4 system. This could be attributed to the use of simplified models in the NZ dTIMS Setup.

No change in rate of ravelling progression was observed while changing the value of the CQ from 0 to 1 in the NZ dTIMS Setup. On the other hand significant change in ravelling progression was found in HDM-4 when the CDS was changed from 0.5 to 1.5. A typical case is illustrated in Figure 9.5 that shows the variation in area of ravelling for different values of construction quality indicator in NZ dTIMS and HDM-4 models. The prediction of NZ dTIMS model for both CQ equals to '1' and '0' overlap in the graph.

The area of ravelling was found to be decreasing after a period, both in NZ dTIMS and HDM-4 because the limiting condition applied to the total damaged area in these models does not exceed 100%.

In the NZ dTIMS Setup, traffic volume had no effect on the rate of ravelling progression as it was based on a time-based model. However, HDM-4 ravelling progression model incorporated the traffic parameter, which does have significant influence on ravelling progression. Figure 9.6 shows the impact of traffic on ravelling progression in NZ dTIMS and HDM-4 models.



Figure 9.5 Influence of CQ and CDS on ravelling progression for 2 surface treatment pavements (HSG1 good, HSP1 poor).

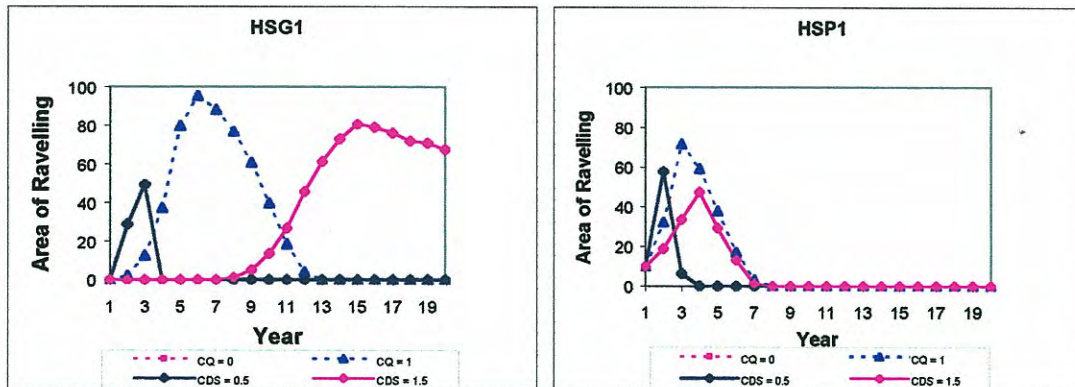
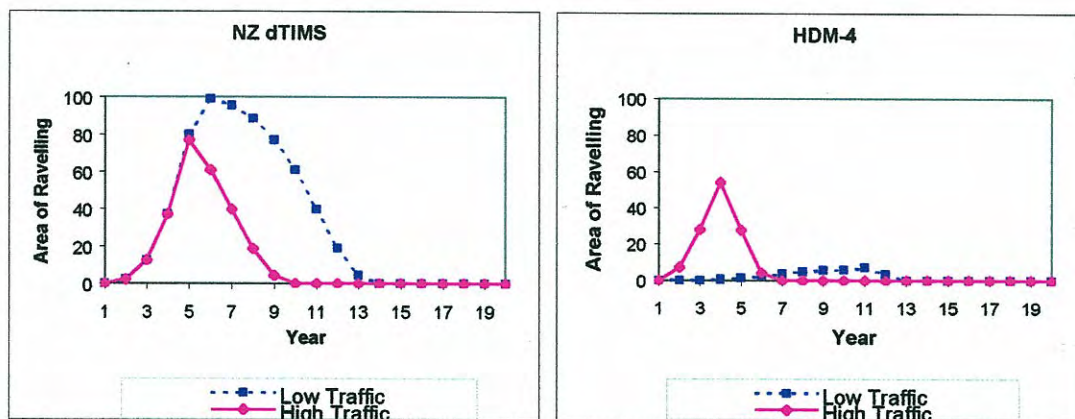


Figure 9.6 Influence of traffic on ravelling progression for NZ dTIMS and HDM-4 models.



In Figure 9.6 the NZ dTIMS Setup curve also showed some variation in ravelling progression beyond analysis-year 5. Detailed investigation into the output showed that the difference in areas of ravelling for low traffic and high traffic volume scenarios is related to the limiting condition for total area of surface distresses, which is 100%. For example, cracking was found to be progressing faster in the section with high traffic. Otherwise CQ and traffic value do not have any impact on the ravelling progression in the NZ dTIMS Setup.

CDS in HDM-4 had greater influence on the rate of progression of ravelling. The prediction of ravelling progression is enhanced by using the traffic parameter. For example, very little progression in ravelling is observed for the low traffic road. Hence, with these enhancements, the HDM-4 ravelling model can be considered to be applicable for New Zealand conditions, but detailed research work is recommended before implementing the model.



## 9.7 Potholes

### 9.7.1 Pothole Initiation

In the NZ dTIMS Setup, pothole initiation model is a function of traffic flow and thickness of the asphaltic layers. In the case of HDM-4, the CDB indicator for basecourse and rainfall are included in the expression.

The pothole initiation period predicted by HDM-4 model is higher than that predicted by the NZ dTIMS Setup. The variation increased with the increase in the traffic volume. CQ did not have any influence on the initiation of potholes in the NZ dTIMS Setup as expected. On the other hand, significant variation was observed in the initiation of potholing in the case of HDM-4 for different values of CDB (Table 9.5). No impact of the pavement strength on the pothole initiation was observed.

**Table 9.5 Variation in pothole initiation in HDM-4 model for 4 pavement types (LWG, HWG, LSG, HSG).**

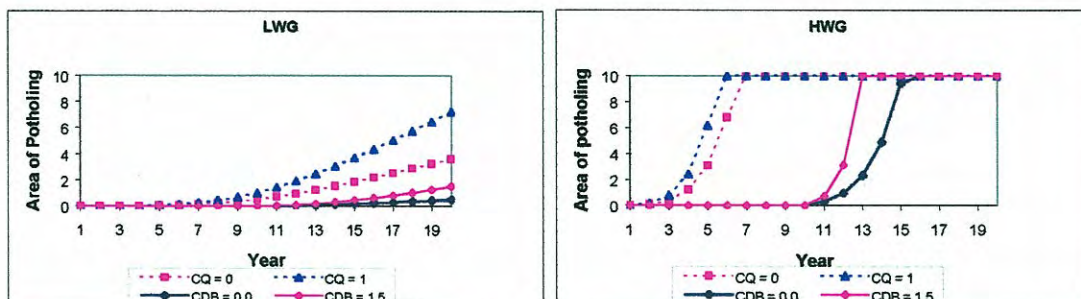
Scenario	Pothole initiation period in years for CDB		% variation in pothole initiation
	0	1.5	
LWG	5.5	2.2	60.0
HWG	2.4	0.9	62.5
LSG	5.5	2.2	60.0
HSG	2.4	0.9	62.5

### 9.7.2 Pothole Progression

The experience gained from using HDM-III models under different physical and environmental conditions showed that the pothole progression modelled in HDM-III over-estimated the area of potholing in some cases. This is particularly true where potholes are patched as soon as they appear, or when signs of potholes appear on the surface, as occurs in New Zealand. A time lapse factor (Odoki 1997, Riley 1997) was introduced in the HDM-4 pothole progression model to account for the response time to patching the potholes.

The rate of pothole progression was higher in the NZ dTIMS Setup than in HDM-4 model for all scenarios. Figure 9.7 shows the variation in areas of potholes as modelled by the NZ dTIMS Setup and HDM-4 system.

**Figure 9.7 Variation in area of potholes for 2 pavement types (LWG, HWG).**



Increase in CDB value increased the area of potholes in HDM-4 in the case of the weak pavements (LWG, HWG) (Table 9.6).

**Table 9.6 Variation in area of potholes in HDM-4 model for weak and strong pavement types (LWG, HWG, LSG, HSG).**

Scenario	Area of potholes in percentage for CDB		% variation in Pothole Progression
	0	1.5	
LWG	5.9	9.4	60
HWG	9.4	16.0	70
LSG	4.1	4.1	0
HSG	4.3	4.3	0

The roughness predicted also changed under this scenario due to changes in the pothole component of roughness. For some scenarios, roughness was found to range up to 60% for different values of CDB.

Considering the routine maintenance regime practised in New Zealand, the HDM-4 pothole progression appeared to be more relevant for New Zealand conditions. The field verification showed that few potholes developed in the pavement and they were generally due to localised failure.

## 9.8 Rutting

Unlike other distresses such as cracking, ravelling and potholing, rutting is modelled in one phase as rutting progression. The initial densification of rutting was modelled in the first year after base construction/reconstruction in the NZ dTIMS Setup and HDM-4 system. However the HDM-4 initial densification model predicted higher rutting than NZ dTIMS. In the NZ dTIMS Setup, calibration factor for initial densification factor is kept at '0.5' while in HDM-4 the default value for this factor is '1'. Thus the deviation in initial densification of rutting predicted by NZ dTIMS Setup and HDM-4 system could be attributed to the different values used for initial densification calibration factor in different systems. Table 9.7 shows the mean rutting values measured in mm as depth, predicted for AMGB pavements (original construction) in the first year after base construction for both NZ dTIMS and HDM-4 models.

**Table 9.7 First year mean rutting value (mm) after base construction using the 2 models for 4 pavement types (LWG, HWG, LSG, HSG).**

Scenario	NZ dTIMS	HDM-4
Low traffic on weak pavement (LWG)	1.5	3.0
High traffic on weak pavement (HWG)	2.9	5.8
Low traffic on strong pavement (LSG)	0.5	1.1
High traffic on strong pavement (HSG)	0.8	1.9

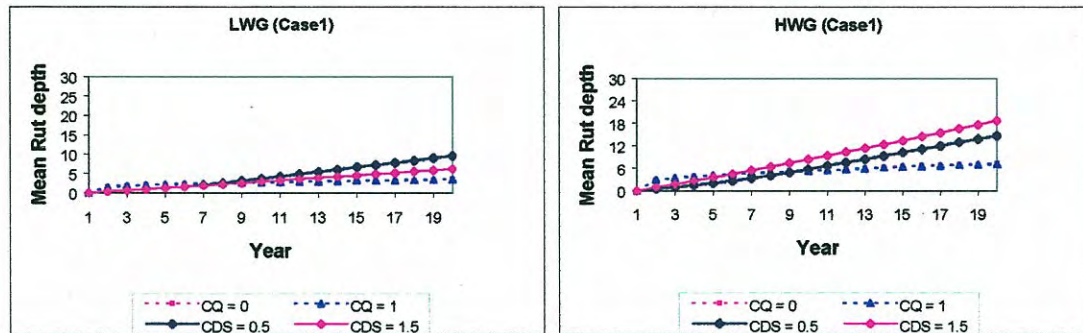
Two new components, plastic deformation and surface wear of rutting, have been included in the HDM-4 rutting model, but the surface wear component of rutting is



not applicable to New Zealand conditions as studded tyres are generally not used. In the New Zealand case, structural deformation and plastic deformation contributed to the further progression of rutting (from the second year onwards).

CQ had almost no effect on rutting progression except for high traffic volumes in the case of NZ dTIMS. In HDM-4, the CDS has considerable effect on the rutting progression. Figure 9.8 presents the variation in rutting for different CDS values for NZ dTIMS and HDM-4 models.

Figure 9.8 Variation in rutting in NZ dTIMS and HDM-4 Models for 2 pavement types (LWG, HWG).



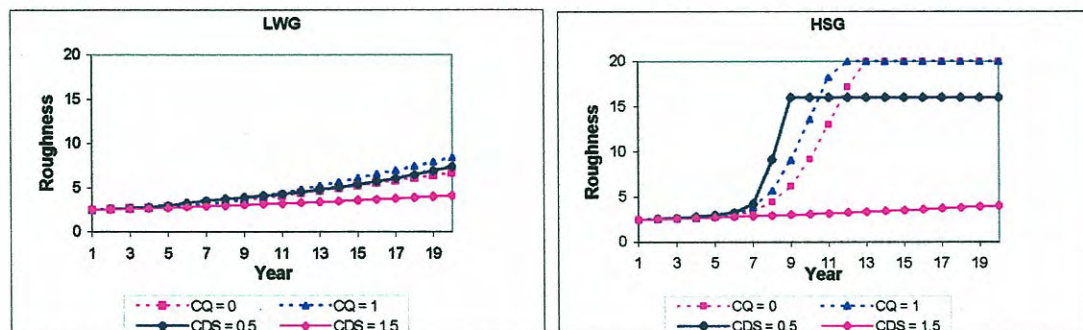
## 9.9 Roughness

The roughness progression model in HDM-4 had the same five components as the HDM-III and NZ dTIMS roughness models, as explained in Chapter 4. The potholing component of roughness in HDM-4 was modified extensively taking into account the effects of time to patch the potholes and freedom to manoeuvre to avoid potholes.

The NZ dTIMS Setup generally predicted higher roughness values than the HDM-4 model. The lower values of roughness in the HDM-4 model could be attributed to the modifications made to the potholing component of roughness, and to other models in the HDM-4 system.

The limiting value for roughness is kept at '20' in the NZ dTIMS Setup and '16' in HDM-4 system. Figure 9.9 shows the influence of construction quality indicator on roughness progression.

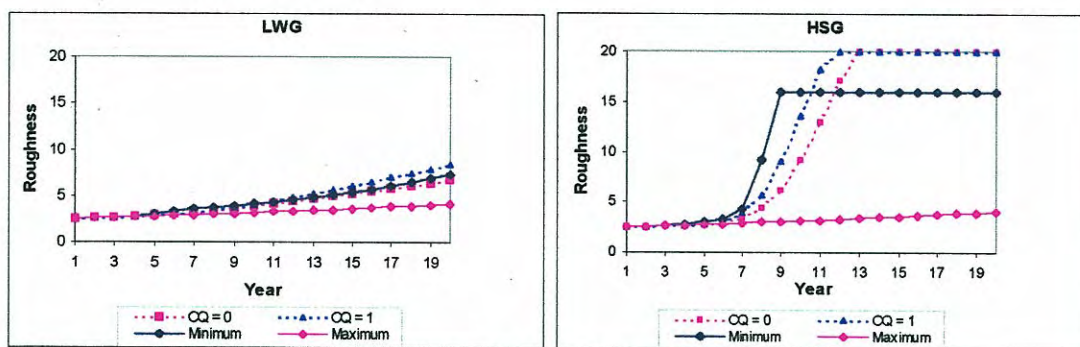
Figure 9.9 Variation in roughness for 2 pavement types influenced by CQ and CDS.





Different rates of roughness progression were achieved in HDM-4 by changing the values of CDS and CDB at the same time. The limiting values for CDS and CDB in the HDM-4 system generated different roughness progressions. Figure 9.10 shows the impact of CDS, CDB and drainage factor on roughness progression.

**Figure 9.10 Impact of CDS, CDB and DF on roughness for 2 pavement types (LWG, HSG).**



CDS = 0.5 and CDB = 0 for minimum case. Effect of drainage is considered in this case.  
 CDS = 1.5 and CDB = 1.5 for maximum case. Drainage factor was kept constant at 2.5 in this case.

From Figure 9.10, the CQ indicator clearly influenced roughness progression in both NZ dTIMS and HDM-4 models. However, the variation is small in the NZ dTIMS Setup, while HDM-4 can predict a range of roughness progression applicable to different conditions. With the introduction of CDS, CDB, DF and other enhancements, HDM-4 roughness model has become more flexible and can be customised to a variety of conditions.

## 9.10 Comparison of WE Models

### 9.10.1 Introduction

Maintenance works are applied to road sections to keep the standard of the network at the desired level.

Modifications and enhancements carried out to the HDM-4 WE models for resetting values are explained earlier in Chapter 5 of this report. In the life cycle analysis, various parameters are reset after the treatment, if that considerably affects the further performance of the pavement. Hence, a parametric study of the WE models was carried out.

### 9.10.2 Treatments Selected for Comparison

Although several new models have been included in HDM-4, only maintenance types relevant to New Zealand conditions were selected for comparison at this stage. Table 9.8 explains the types of treatments selected for comparison and the effects of these maintenance operations.

**Table 9.8 Treatments selected for comparison.**

Maintenance type	HDM-4 Treatment	NZ dTIMS treatment code
Routine maintenance	<ul style="list-style-type: none"> <li>• pothole patching (90%)</li> <li>• drainage maintenance (75%)</li> <li>• deep patching of wide structural cracking (80%)</li> <li>• repair of pavement edge break (90%)</li> </ul>	<ul style="list-style-type: none"> <li>• pothole patching (90%)</li> <li>• crack sealing of wide structural cracking (80%)</li> </ul>
Resurfacing (periodic maintenance)	<ul style="list-style-type: none"> <li>• 8mm thick ST</li> <li>• 25mm thick AC</li> </ul>	<ul style="list-style-type: none"> <li>• 8mm thick ST (RSL)</li> <li>• 25mm thick AC (MOA)</li> </ul>
Rehabilitation (periodic maintenance)	<ul style="list-style-type: none"> <li>• 150mm thick granular material + 8mm ST</li> <li>• 150mm thick granular material + 50mm AC</li> </ul>	<ul style="list-style-type: none"> <li>• 150mm thick granular material + 8mm ST (SOL)</li> <li>• 150mm thick granular material + 50mm AC (SRA)</li> </ul>

The trigger limits were kept similar for the NZ dTIMS Setup and HDM-4 system so that a comparison can be made at the same datum. These trigger limits do not exactly coincide with the NZ dTIMS Setup but represent the average condition of New Zealand. The trigger limits selected for the study are given in Table 9.9.

**Table 9.9 Trigger limits for comparison study of the 2 models.**

Maintenance type/ Treatment	Trigger Limit	Remarks
Routine maintenance	True	Applied when no other treatment is applied
Resurfacing <ul style="list-style-type: none"> <li>• 8mm ST (RSL)</li> <li>• 25mm AC (MOA)</li> </ul>	(ACA $\geq$ 5 or ARV $\geq$ 30) and Surf_Type=ST (ACA $\geq$ 5) and Surf_Type=AC	
Rehabilitation <ul style="list-style-type: none"> <li>• 150mm thick granular material + 8mm ST (SOL)</li> <li>• 150mm thick granular material + 50mm AC (SOA)</li> </ul>	<ul style="list-style-type: none"> <li>• (ACA<math>\geq</math>30 or IRI<math>\geq</math>6 or RDM<math>\geq</math>20) and Surf_Type=ST</li> <li>• (ACA<math>\geq</math>30 or IRI<math>\geq</math>6 or RDM<math>\geq</math>20) and Surf_Type=ST</li> </ul>	

ACA – area of all cracking (%); ARV – area of ravelling (%);  
RDM – mean rut depth (mm); IRI – roughness (m/cm)

Output parameters selected for comparison included those parameters which reflect the maintenance treatments applied, and these are given in Table 9.10.

**Table 9.10 Comparison of output factors (IRI, RDM) used in the 2 models.**

Treatment	Output parameter	Remarks
Routine maintenance	IRI	
Periodic maintenance <ul style="list-style-type: none"> <li>• Resurfacing</li> <li>• Rehabilitation</li> </ul>	RDM and IRI	Selected for both resurfacing and rehabilitation treatments

### 9.10.3 Routine Maintenance

Similar types of maintenance activities were considered where possible. The area of potholes for patching was specified to 90% (i.e. 90% of potholes developed are repaired) in HDM-4 so that it is similar to the NZ dTIMS Setup (default setup).



Similarly 80% area of existing wide cracking is repaired in both systems. Drainage condition and edge break reset can only be done with HDM-4 models and they are taken as 75% maintenance for drainage maintenance, and 90% maintenance for edge break.

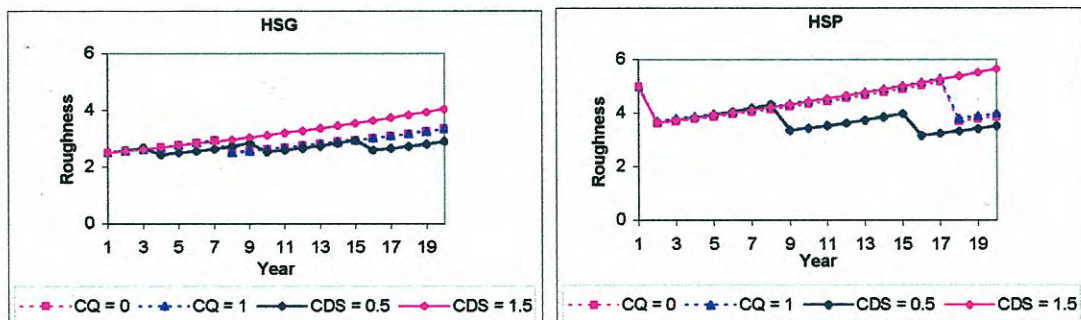
A detailed comparison was made for roughness progression using the synthetic dataset for AC and ST pavements. The HDM-4 roughness progression was slightly lower than that predicted by the NZ dTIMS Setup, and this could be attributed to the introduction of a time-lapse factor and freedom to manoeuvre, and other modifications made to HDM-4 models.

#### 9.10.4 Resurfacing and Overlay

Empirical relationships were provided for roughness reset after maintenance for ST and AC pavements in both the NZ dTIMS Setup and HDM-4 system. HDM-4 roughness progression was lower than the NZ dTIMS Setup using its default construction quality indicator. Different roughness progression curves were predicted by the HDM-4 system for different values of CDS using the same trigger limit and reset equations. This is attributed to the value of CDS that takes into effect of the construction defects, if any.

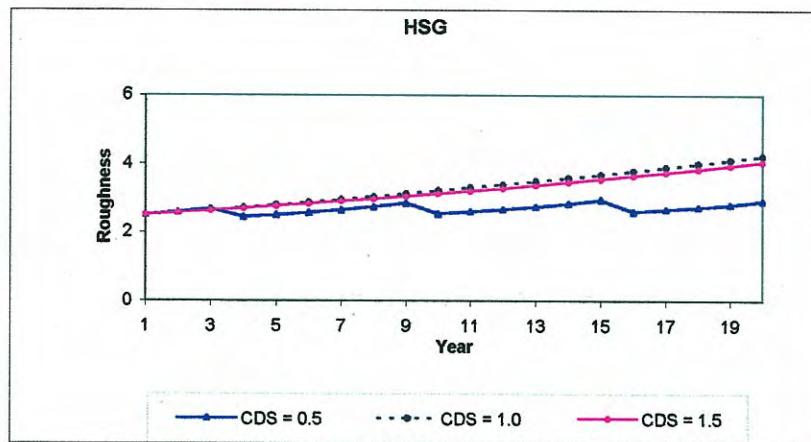
Figure 9.11 presents a typical case which illustrates the different roughness curves predicted by the HDM-4 system for different CDS values. The NZ dTIMS Setup predictions are also included in the figure, in order to show the difference in prediction by the two modelling systems (i.e. NZ dTIMS Setup and HDM-4 system).

**Figure 9.11** Variation in roughness with resurfacing treatment, predicted by the 2 models for 2 pavement types (HSG, HSP).



The number of applications of overlays (resurfacings) also varied with CDS value. A lower number of overlays means less agency cost to be incurred for the same trigger limits. Hence, CDS values have to be chosen carefully so that they represent the field conditions, otherwise the modelling of pavement deterioration and strategies (treatment at a specified time) predicted might not represent real conditions. Then the output produced may over-estimate or under-estimate, not only pavement performance, but the funding requirements as well. Figure 9.12 shows a typical example illustrating different strategies for different CDS values.



**Figure 9.12** Different strategies for applying CDS values to an HSG pavement.

The variation in construction quality value in the NZ dTIMS Setup did not have any impact on strategies predicted. However in HDM-4, three overlays were required to maintain the road at similar standards for a value of '0.5' for CDS, while no overlays were required when CDS equalled to '1' and '1.5'. The test road section synthetic data was maintained within the allowable maintenance standards in all three scenarios.

### 9.10.5 Reconstruction

Two different reconstruction treatments were selected as follows for comparison purposes:

- 150mm granular material and 8mm surface treatment for ST pavements.
- 150mm granular material and 50mm asphalt concrete for AC pavements.

The trigger limit was kept the same for both the NZ dTIMS Setup and HDM-4 system. Roughness (IRI) value after rehabilitation was taken as 2m/km for AC and 2.5m/km for ST pavements. These were the default values in the NZ dTIMS Setup and, hence, were adopted for HDM-4.

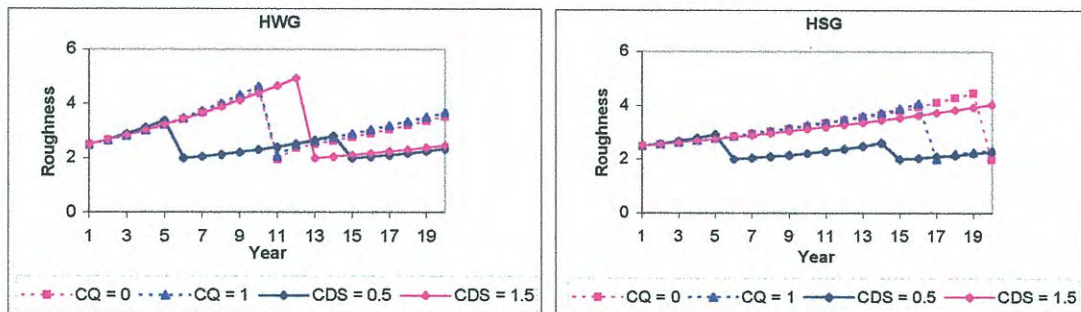
Rut depth and roughness were selected for comparison. The observations made for rehabilitation treatment are as follows:

- Rut depth progression was higher in the case of the HDM-4 system due to the inclusion of plastic deformation of rutting component.
- Roughness progression was lower in the HDM-4 system compared to the NZ dTIMS Setup.
- Different values for CDS produced different strategies for the same set of triggers and reset equations.

Figure 9.13 shows the roughness progression for HDM-4 and the NZ dTIMS Setup for different construction quality values.



**Figure 9.13** Variation of roughness with rehabilitation treatment predicted by the 2 models for 2 pavement types (HWG, HSG).



### 9.11 Conclusions

- The drainage factor was introduced in the HDM-4 system to consider the effects of drainage condition. The pavement strength reduced with the deterioration in drainage condition. The reduction was found as high as 20% for a given scenario. Also the effect of drainage factor is cumulative on reduction of SNP, if drainage condition is not maintained.
- CQ did not have any impact on pavement strength in the NZ dTIMS Setup, but CDS influenced pavement strength significantly in HDM-4.
- The new continuous construction quality indicator (introduced in the HDM-4 system) significantly influenced the initiation and progression of distresses, and also in the strategies, while the effect of CQ in the NZ dTIMS Setup was negligible.
- The progression of the distresses in the HDM-4 system was lower compared to that predicted in the NZ dTIMS Setup with its particular default values for construction quality indicator.
- HDM-4 ravelling model has become more flexible and applicable with the introduction of a traffic factor and construction quality. Further study is required to determine the suitability of this model to New Zealand conditions.
- With the introduction of the base construction quality indicator (CDB) in the pothole initiation model, HDM-4 pothole initiation model is flexible to be customised to different conditions. The pothole progression was lower in the HDM-4 system.
- The rutting progression was higher in the HDM-4 system due to inclusion of additional rutting components (plastic deformation and surface wear) in the HDM-4 rutting model.
- Roughness progression was lower in the HDM-4 system. Pothole component of roughness in HDM-4 has been extensively modified by introducing the time lapse factor and freedom to manoeuvre parameter.
- Maintenance works have been modified in HDM-4. Some new user-specified routine maintenance treatments were included in the HDM-4 system, and provide a platform for comprehensive modelling of routine maintenance treatments.

## **10. Initial Calibration of HDM-4 Models**

### **10.1 Introduction**

Calibration of the HDM-4 models to local conditions is necessary as the main objective of the initial calibration (discussed in Chapter 7) was to verify the applicability of the HDM-4 predictions to New Zealand conditions. Before carrying out the initial calibration, sensitivity analysis was carried out on the 23 or so calibration factors that are included in the HDM-4 system for paved bituminous roads. This analysis determined which were the most sensitive calibration factors to use in New Zealand conditions, and also enabled a preliminary calibration to be made.

### **10.2 Sensitivity Analysis of Calibration Coefficients**

#### **10.2.1 Data Preparation**

Data preparation for the sensitivity analysis of the calibration coefficients is explained in Section 8.4.

#### **10.2.2 HDM-4 Model Setup**

Sensitivity analysis was carried out using the HDM-4 interface. The preliminary analysis carried out with the HDM-4 models, without considering any maintenance activities, distorted the output because the various distress parameters deteriorated quite rapidly without any maintenance. For example pothole initiation, pothole progression, and edge break were sensitive to cracking.

Hence, sensitivity analysis of the calibration coefficients was carried out applying the routine maintenance practice used in New Zealand as follows:

- Patching of the all potholes; and
- Dig out (Deep Patching) of all wide structural cracking.

The analysis was carried out to determine the sensitivity of each calibration coefficient in response to predominant distresses, e.g. area of all structural cracking (ACA), area of ravelling (ARV), area of potholes (APOT), edge break (AVEB), mean rut depth (RDM), and roughness (IRI). The impact elasticity was determined for years 5, 10, 15 and 20, and the maximum of these four observations was taken as representative for that particular parameter.

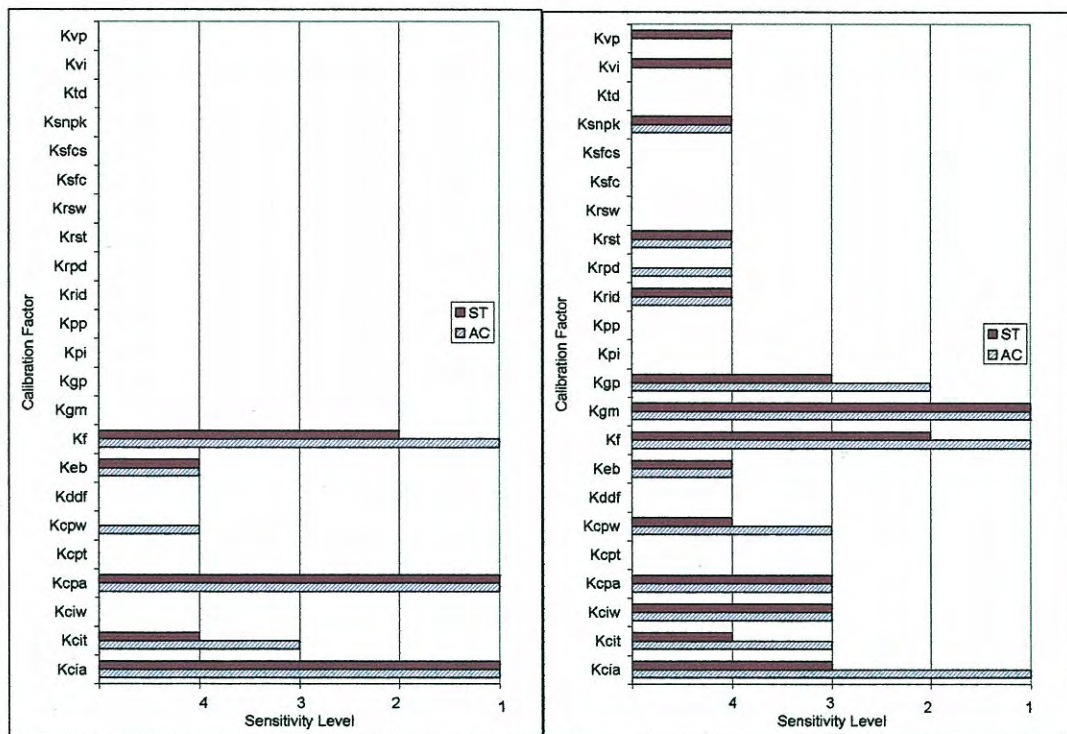
#### **10.2.3 Sensitivity Analysis Results**

The output of the HDM-4 analysis was further analysed using MS Excel to obtain the sensitivity of the calibration factors. Sensitivity of each calibration coefficient in response to the predicted deterioration parameters was calculated in terms of impact elasticity (see Chapter 7 for details).



The sensitivity of the calibration factors in response to the area of all structural cracking ACA (Figure 10.1) showed that only 6 factors were sensitive. Only crack initiation (Kcia) and progression (Kcpa) coefficients were found to be very sensitive. Wet/dry season SNP ratio calibration coefficient (Kf) was found Level 1 sensitive to AC-surfaced pavement, but level 2 sensitive to the surface-treated (ST) pavement.

**Figure 10.1 Sensitivity of calibration factors to area of all structural cracking (left) and to roughness (right) for 2 pavement types (ST, AC).**



Note:

- Level 1 Impact Elasticity greater than 0.5
- Level 2 Impact Elasticity greater than 0.2 and less than 0.5
- Level 3 Impact Elasticity greater than 0.05 and less than 0.2
- Level 4 Impact Elasticity less than 0.05

Lower value for sensitivity level indicates higher sensitivity

About 15 (out of 23) coefficients were found to be sensitive for roughness (Figure 10.1). Among them only 4 (Kcia, Kf, Kgm, and Kgp) are quite sensitive (to Levels 1 and 2). Of importance is that the wet/dry season SNP ratio calibration coefficient (Kf) was Level 1 sensitive to Roughness. Calibration coefficients with higher sensitivity were given higher priority in allocating resources and time in calibration task, whereas less importance was given to those calibration factors with sensitivity levels 3 and 4

## 10.3 Initial Calibration

### 10.3.1 Calibration Method

One of the major weaknesses in the HDM-4 documentation is that no indication is given on how to calibrate for construction quality indicators (CDS and CDB), and no published literature is available on this.

The value of the CDS has to be chosen carefully so that it does not distort another model (say ravelling) while calibrating for one (say cracking) model. Hence, at this stage of developing HDM-4 for New Zealand conditions CDS and CDB were kept at default value for the preliminary calibration purpose. The preliminary calibration was carried out using 'Method 1' as explained in Section 7.7. The following assumptions were taken:

- The values of the construction quality indicators were fixed at default values (CDS = 1, CDB = 0).
- Calibration of the coefficients is based on the HDM-4 logic for calculation of various distresses.
- HDM-4 interface with the routine maintenance option was used to generate pavement performance (output parameters). It was assumed that routine maintenance is applied every year. Drainage maintenance, pothole patching, deep patching of wide structural cracking, and edge break maintenance were included in the annual routine maintenance operations.
- Observed values were taken either from the RAMM database or collected for the purpose, as explained in Section 8.2.

### 10.3.2 Drainage Factor Coefficient

Condition rating for the drainage is carried out every year in New Zealand using the RAMM rating system, for the deficiency of the drainage only, and these data could not be used for calibration purposes. As the calibration coefficient for the drainage factor ( $K_{ddf}$ ) was of low sensitivity, the default of '1' was used in this study.

### 10.3.3 Wet/Dry Season SNP Ratio

Wet/dry season SNP ratio was one of the calibration coefficients most sensitive to the distress prediction.

No published work could be found on the study of defining wet/dry season SNP ratio in New Zealand and, where FWD or any other measurements are carried out in the dry and wet seasons, no other relevant road section data were available. Hence, calibrating for the wet/dry season SNP ratio coefficient ( $K_f$ ) was not possible because of the absence of the observed value. As the nature of the present study did not allow for carrying out specific data collection for the calibration, the default value of '1' was used for the comparison analysis. However, considering the sensitivity of this coefficient, a specific study is recommended.

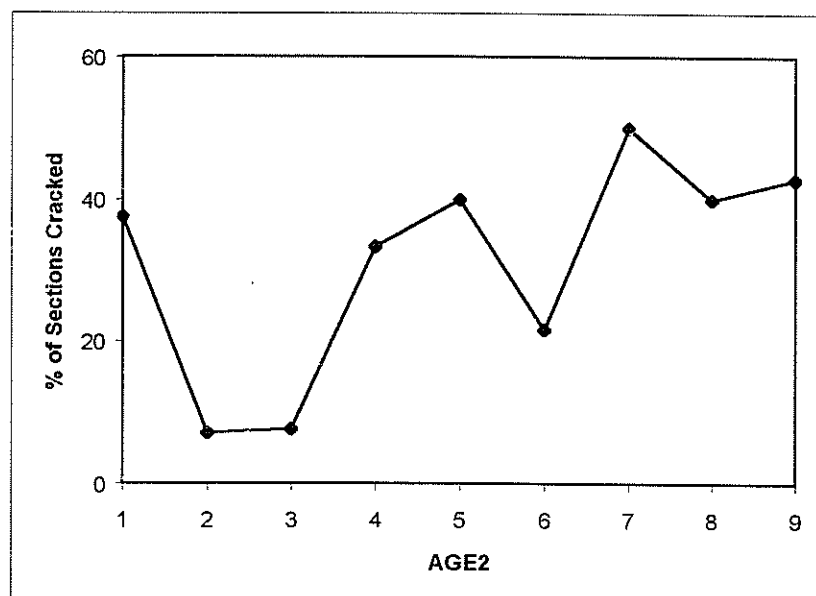
### 10.3.4 Cracking Calibration Coefficients

Crack initiation coefficient is one of the highly sensitive parameters in the HDM model prediction and, hence, effort was put into studying and defining the crack initiation coefficient. Both snap-shot and time-series methods (see Chapter 3) were used for the calibration of the crack initiation coefficient. As it is very difficult to estimate crack initiation period from the condition database, the method based on the availability of cracked sections was used (Riley 1998).

In the snap-shot method, 500 sections of the calibration database were segregated by different traffic ranges based on RAMM pavement use criteria. The percentage of the number of cracked sections was plotted against AGE2 for each traffic range. Extreme values (outliers) were excluded from the analysis.

An example of the graphs for the traffic range 4 000 to 10 000 vehicles per day (veh/day) for the ST pavement is given in Figure 10.2. The first year observation was considered as an outlier (on the assumption that the survey preceded resurfacing), and years 2 and 3 were considered as local defects rather than structural cracking. Year 4 can be considered as the crack initiation period for this traffic range, i.e. the year from which the number of cracked sections increased.

**Figure 10.2** % sections in which crack initiation occurred , over a 9-year period using the snap-shot method.

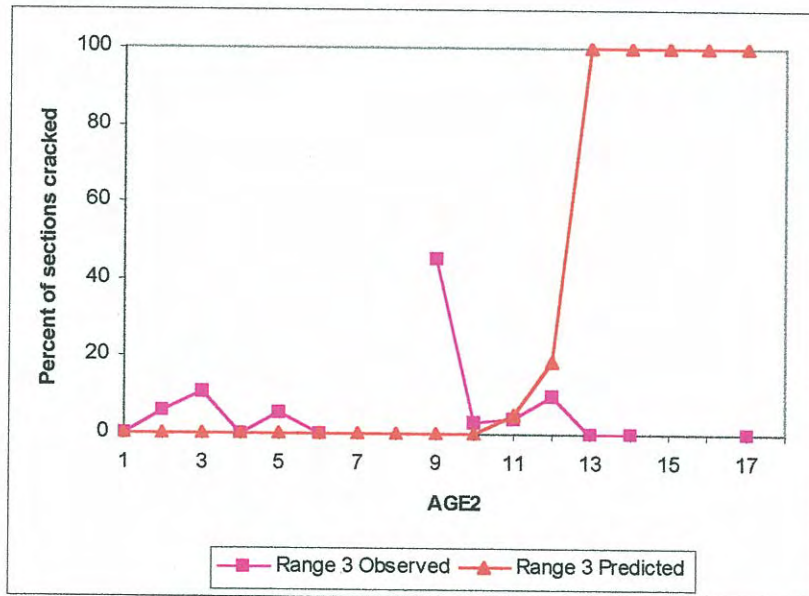


The snap-shot method gave an indication of what the calibration factor could be but, to refine the factor, the calibration was carried out through the time-series method.

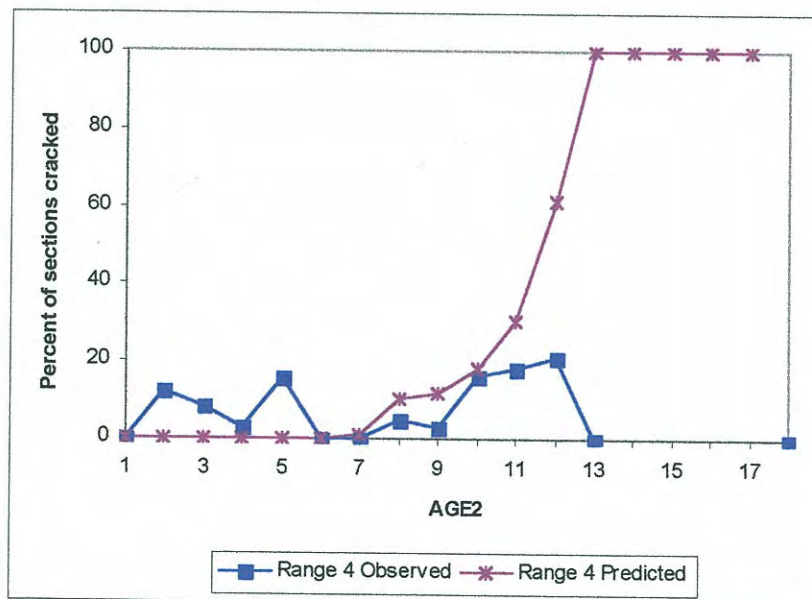
For the time-series method, sections with 'routine maintenance only' were considered for the analysis. The sections were segregated first into original construction and resealed, and then into different RAMM traffic ranges. Plots were drawn for percent of sections cracked against surfacing age (AGE2) for different traffic ranges (Figures 10.3, 10.4).



**Figure 10.3** % cracked sections, observed and predicted, versus AGE2 for low traffic range of 500 – 2000 veh/day.



**Figure 10.4** % cracked sections, observed and predicted, versus AGE2 for high traffic range of 2000 – 4000 veh/day.

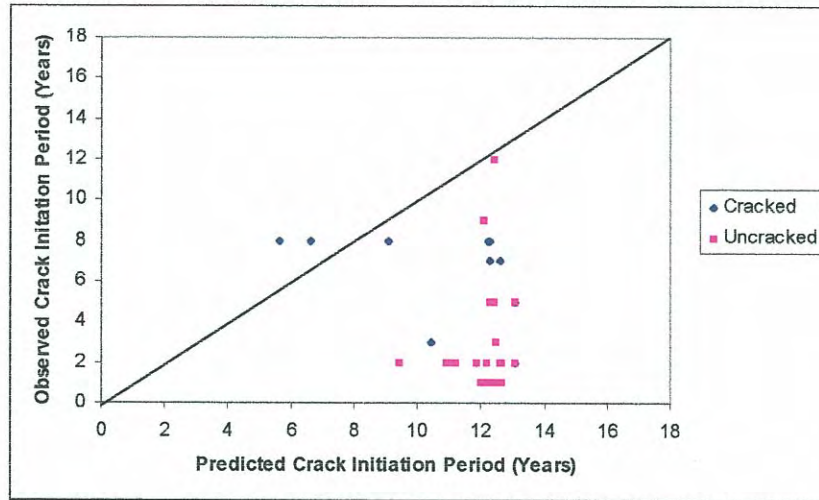


The above figures suggested that the crack initiation period decreased as the traffic volume increased, in both the predicted as well as observed scenarios. The predicted values with Kcia of '1' coincided reasonably with the observed values between pavements of ages 5 and 13 for the two traffic ranges. The cracked sections at early ages were further investigated, and found that the area of cracking that occurs then was negligible and therefore was not considered.

Figure 10.5 shows the comparison between the predicted and observed crack initiation periods for the considered dataset over 18 years.



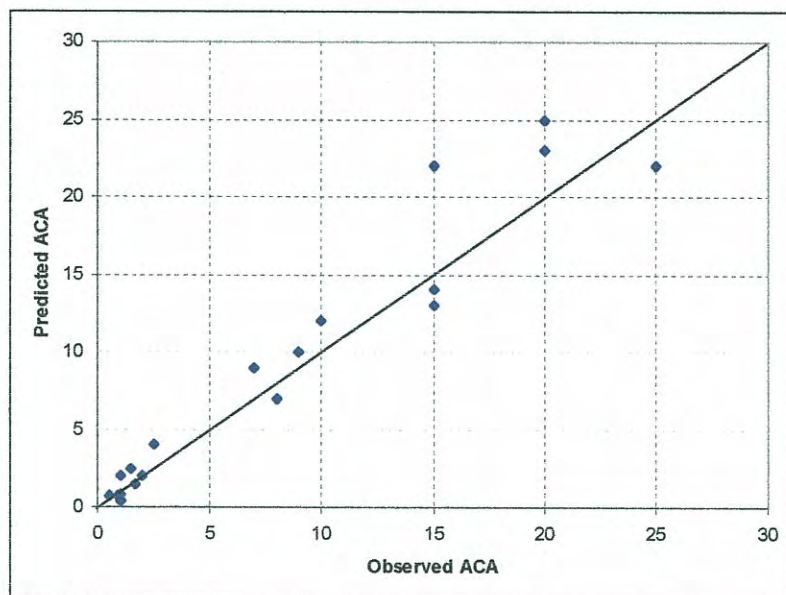
**Figure 10.5 Predicted v observed crack initiation period of 18 years.**



Many sections are uncracked at ages before the predicted crack initiation period (i.e. younger than 4 years), which is reasonable. Hence, considering the small amounts of cracking at early age and the coincidence of predicted and observed values occurring between 5 and 13 years, it was considered that  $K_{cia}$  can be taken as '1'.

Time-series method was used to calibrate for the crack progression factor. Out of the 50 sections selected for analysis, only 5 sections showed some trend in the area of cracking. All other sections showed either some kind of scatter in their values or the area of cracking was very low (less than 10%), and could not be used for calibration purposes. This low value could be related to the local maintenance practice to patch the surface when the cracking appears on the pavement surface. The area of cracking for these 5 sections was predicted after the initiation of the cracking. The observed and predicted values for different ages for the sections were compared (Figure 10.6).

**Figure 10.6 Predicted v observed area of all cracking (ACA).**





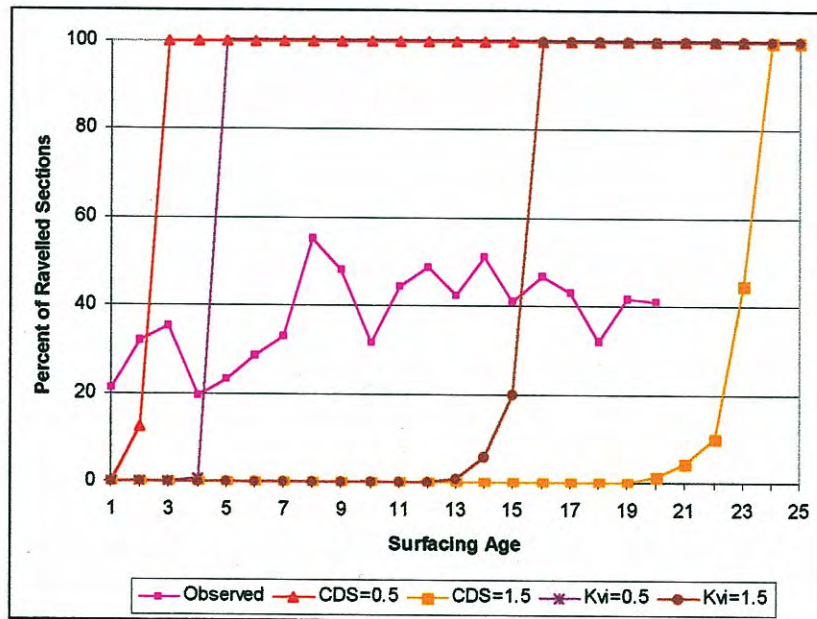
The predictions were found to be reasonable using the default value of '1' for crack progression factor (Kcpa). Considering the few sections that showed the trend in cracking, the small amounts of area of cracking that could not be used for calibration purposes, and the reasonableness of the predictions with the default value, the crack progression factor can be kept at '1' at this stage.

### 10.3.5 Ravelling Calibration Coefficients

Ravelling calibration coefficients showed relatively very little sensitivity to all the parameters except ravelling itself. Besides, the ravelling progression factor is taken as '0' in the NZ dTIMS Setup because HDM-III ravelling models were found to be not suitable for New Zealand conditions as explained earlier (Section 9.6). Hence, the calibration study on ravelling was carried out to find if the HDM-4 ravelling models better reflect the New Zealand condition.

The percent of ravelled sections were plotted against surfacing age. No trend was found in either snapshot and time-series methods. Figure 10.7 presents the analysis for the time-series method. Analysis showed that 30 – 50% of the road sections were ravelled irrespective of the age (in years).

Figure 10.7 Observed and predicted ravelling using the time-series method over 25 years.



The predicted ravelling initiation period may vary a good deal, if it is based on the value of the construction defect indicator (CDS) and calibration coefficients in which CDS is dominating (Figure 10.7).

The field visit of the sites had shown that most of the ravelling (scabbing) area recorded in RAMM was found outside the wheelpath area. This could have occurred because of improper compaction of the surface layers. After application of the reseal, the surface layer is not fully compacted, and is opened for traffic to do the last compaction. Hence, compaction was found to have taken place only in the wheelpath

area (and in some of the places flushing was observed). Other portions of the surface layer are left without the required compaction and hence, as a result, contribute to ravelling (locally known as 'scabbing') within 1 or 2 years. Discussion with local maintenance practitioners indicated that ravelling in general does not generally grow to form potholes and has very little impact on pavement performance.

Time-series method was considered for calibrating the ravelling progression. The area of ravelling appeared to be very scattered over the surfacing age and did not yield any meaningful trend that could be used for calibration purposes. This could be attributed to local repairs that generally are not reported in the RAMM database, and the inconsistency in the recording of scabbing by different enumerator field staff while carrying out the rating survey.

The progression of ravelling was also very much dependent on construction quality, and to calibrate for ravelling progression factor the construction quality indicator is considered necessary. Similarly, rating the ravelling located only in the wheelpath area may give some reasonableness to the model used.

Considering all these issues, calibration for ravelling coefficients was not done in detail at this stage of the project, and the default values used in the NZ dTIMS Setup were adopted instead.

### **10.3.6 Pothole Calibration Coefficients**

Calibration for the pothole coefficients for use in New Zealand, were not considered essential as previous studies have shown that the potholed/patched areas were very small in number and the potholing did not coincide with cracking or ravelling in many instances (Riley 1998). Hallett & Tapper (2000) also did not consider the calibration coefficients of potholes.

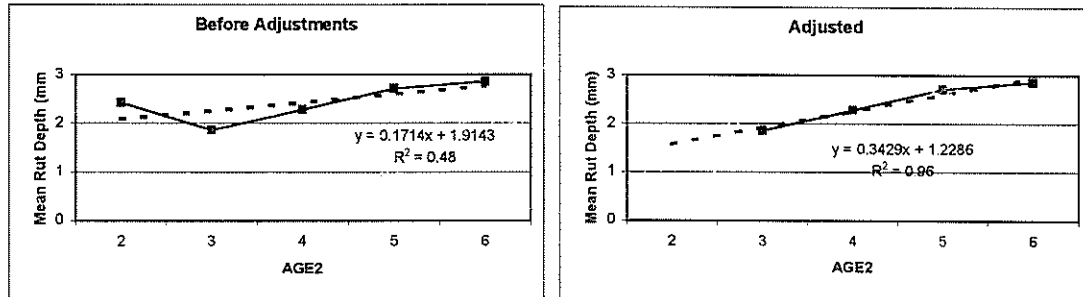
The dataset prepared for the calibration had very few sections with potholes. The maintenance regime in New Zealand warrants that the road should be resealed soon after structural cracks start to show on the surface and, hence, are not allowed to develop into a pothole. Pothole calibration coefficients were found to have very low sensitivity to roughness progression, and therefore the default value of '1' was used for the analysis.

### **10.3.7 Rutting Calibration Coefficients**

The sensitivity analysis carried out in the early stages of this study showed that the coefficients are quite sensitive to rutting progression, but showed very little sensitivity to the overall roughness prediction.

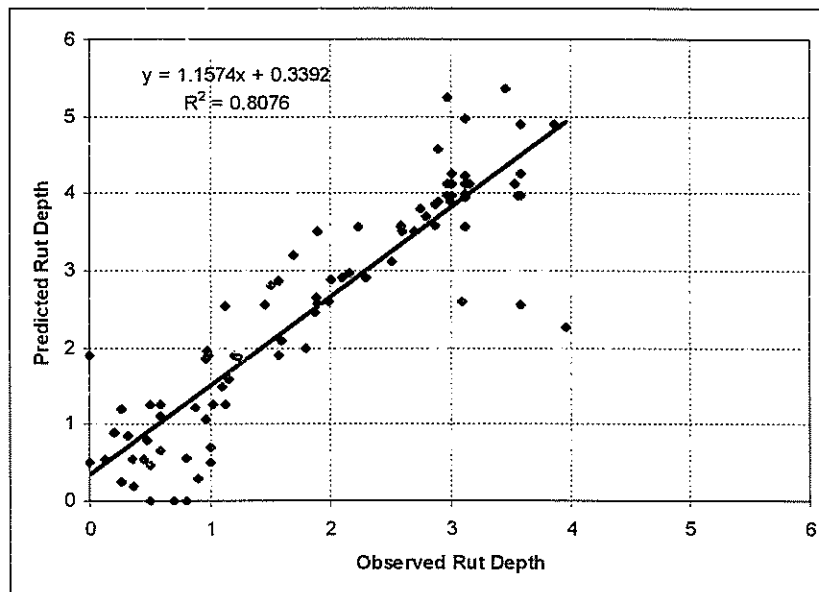
The snap-shot method used for rutting calibration did not yield satisfactory results. Time-series HSD rutting data for 5 years showed some kind of trend in the rutting progression and good correlation between the surface age and distress was achieved by eliminating the outliers. Figure 10.8 shows the trend for HSD rutting before and after adjustments using the time-series method.

**Figure 10.8** Rutting based on time-series method, showing before and after adjustments over 6 years of pavement life.



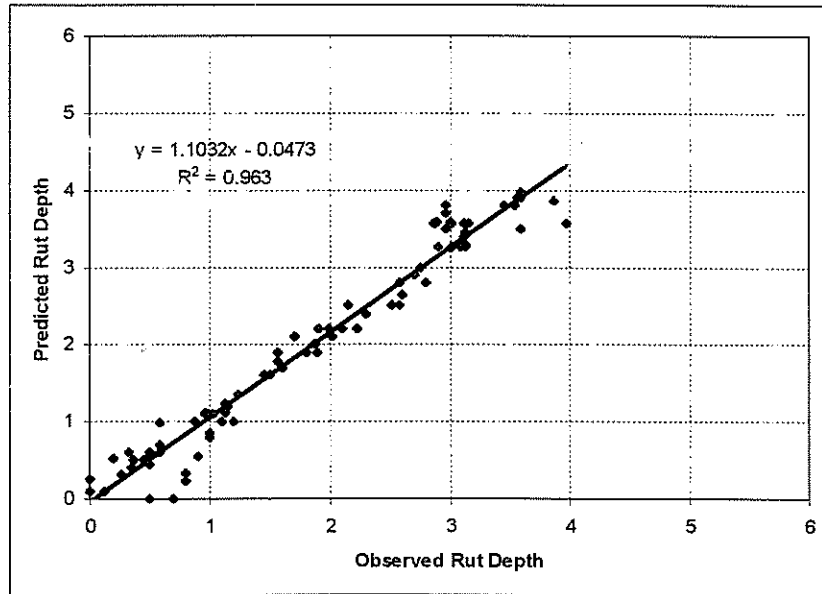
In the HDM-4 model, the rut depth progression is based on the plastic deformation (for AC pavement only) and structural deformation (excluding surface wear which is not relevant to New Zealand conditions). However, RAMM database includes only rut depth data. It is not possible to separate the rutting due to plastic deformation and that due to structural deformation. Hence, the same value was used for both coefficients. The time-series analysis of the rutting had shown that predicted rutting was greater than that observed, and the best fit was found when the calibration coefficient was taken as 0.7. Figures 10.9 and 10.10 show the observed versus predicted rut depths for both default and calibrated coefficients respectively.

**Figure 10.9** Observed v predicted rut depth (mm) with default rutting coefficient.



As Figure 10.10 shows less scatter, the rutting calibration coefficient chosen was 0.7 for this study.

Figure 10.10 Observed v predicted rut depth (mm) with calibrated rutting coefficient.



### 10.3.8 Roughness Calibration Coefficients

#### 10.3.8.1 Snap-shot Method for Roughness Calibration

In the snap-shot method, data has been segregated based on construction age (AGE3). For example, the sections having AGE3 of '1' were grouped in one category, and so on. After segregating the sections in different age groups these sections were amalgamated as one representative section for a particular traffic range (500 to 2000 veh/day), and pavement strength and pavement age (AGE3). Figure 10.11 presents a typical case for Transit New Zealand data plotted for right lane roughness using AGE3.

Figure 10.11 Roughness progression based on AGE3, in right lane of sample road sections (from Transit New Zealand database).

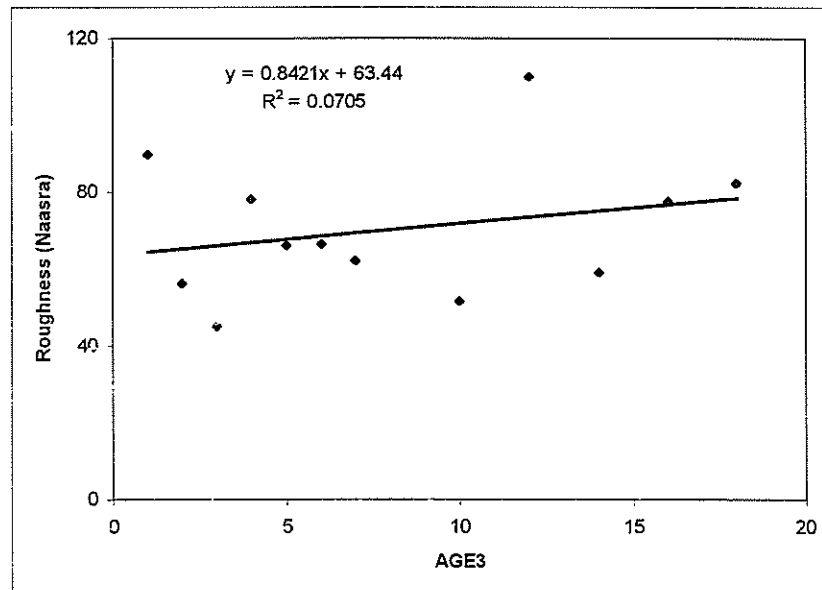
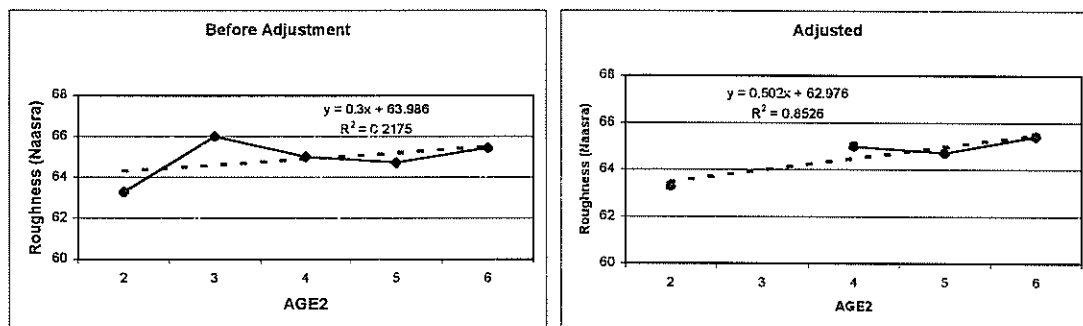


Figure 10.11 shows that no specific trend was observed in the roughness progression, which could be because different values of initial roughness were observed after rehabilitation work. The calibration of the works effects (WE) model had shown that the initial roughness after the reconstruction works can vary significantly (Section 10.4.3.3), and that could be the reason why a definite trend was not observed. Hence, it was decided not to consider the snap-shot method for the roughness model calibration.

### 10.3.8.2 Time-Series Method for Roughness Calibration

Better correlation was found using time-series analysis. Figure 10.12 shows an example of the observed trend for HSD roughness for RAMM traffic category '3'. The roughness trend after the adjustments (eliminating outliers) is also shown in Figure 10.12.

**Figure 10.12** Roughness trend analysis before and after adjustment, over 6 years of pavement life.



Keeping in view the pavement deterioration nature (S-curve), it was felt best not to extrapolate through a linear or polynomial equation. Hence, effectively 5 years data (HSD available from 1995 onwards) were available for calibration purpose.

Two calibration coefficients are available for calibrating the roughness progression model in HDM-4 version 1.1, as follows:

- Environmental coefficient (Kgm); and
- General roughness coefficient (Kgp).

Generally the environmental coefficient (Kgm) is calibrated first, and rarely is the general roughness progression factor (Kgp) attempted for calibration. Preliminary analysis of the data showed that the general roughness factor (Kgp) had more influence on roughness progression than Kgm in some cases, particularly for high volume traffic roads. Hence, calibration was carried out for both general (Kgp) and environmental (Kgm) coefficients in this study.

In the context of New Zealand, cracking and potholing can usually be ignored, as they are insignificant in the component for roughness incremental model (HTC 2000). Time-series roughness and standard deviation of rut depth data, and average values of structural number, axle loading and construction age were compiled for Transit New Zealand network from the Transit database.

The following steps explain the detailed methodology adopted for calibration:

**Step 1:** Obtain the best estimate of the environmental variable (m) for New Zealand conditions from the HDM-4 tables given in HDM documentation series.

**Step 2:** Calculate the mean incremental values of IRI and RDS. SLOPE () function was used for this purpose.

**Step 3:** Calculate the mean absolute values of IRI

**Step 4:** Calculate the predicted values of the structural term using the structural number, axle loading, construction age and the initial estimate of 'm' from Step 1:

$$\frac{\exp(m \text{ AGE } 3) \text{ YE } 4}{(1 + \text{SNC})^5}$$

**Step 5:** Fit a multiple linear regression between observed mean incremental IRI against the following terms:

- predicted structural component of roughness increment from Step 4;
- observed mean increment in RDS from Step 2; and
- observed mean absolute value of IRI from Step 3.

The intercept was set to zero when carrying out the regression.

The regression coefficients gave a modified version of the component incremental model as follows:

$$\text{IRI} = a1 \frac{\exp(m \text{ AGE } 3) \text{ YE } 4}{(1 + \text{SNC})^5} + a2 \text{ RDS} + a3 \text{ IRI}_a$$

**Step 6:** Calculate Kgm from 'm' value (obtained from Step 5) as follows:

$$\text{Kgm} = m / 0.023$$

Whenever the derived value of m (regression coefficient a3) differed significantly from the value considered in step 1, analysis was done again using the value a3 in step 1.

During the analysis the coefficients a1 and a2 were observed to differ significantly from their default values (a1 = 134 and a2 = 0.088) used for roughness prediction model. However, the m (a3) value given by the above equation is dependent on a1, a2 and a4. Hence, the above equation is slightly modified by incorporating Kgp (Kgp = 1 is taken in the above equation) as follows:

$$\text{IRI} = \text{Kgp} \{134 \exp(m \text{ AGE } 3) \text{ YE } 4 / (1 + \text{SNC})^5 + 0.088 \text{ dRDS}\} + a3 \text{ IRI}_a + a4 \text{ Pat}$$

'Kgp' was unable to adjust for structural and rutting components simultaneously, and the calibration results were improved by using different coefficients for different components (structural and rutting). Morosuik & Riley (2001) proposed to include different calibration coefficients for different roughness components in the HDM-4 roughness model.

The following roughness model incorporating separate calibration coefficients for structural, rutting, environmental and patching components was used in our present study.

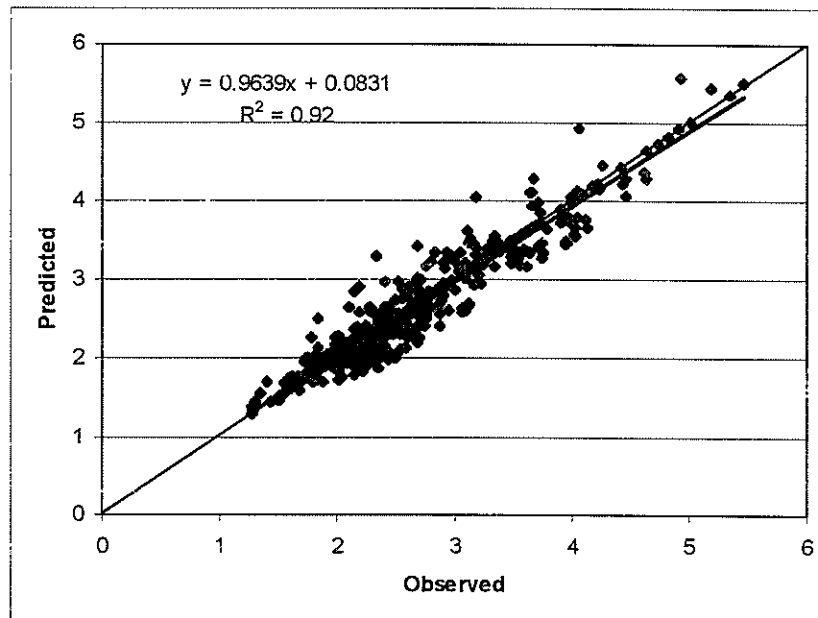
$$IRI = \{Kgs\ 134\ \exp(mAGE3)\ YE4 / (1+SNC)^5\} + \{Kgr\ 0.088\ dRDS\} + \{Kgm\ 0.023\ IRI_{aj}\} + \{Kgp\ Pat\}$$

The following calibration factors were derived for the analysis.

Coefficient	Code	Value
Structural component	Kgs	1.7
Rutting component	Kgr	2.4
Environmental component	Kgm	0.56
Patching component	Kgp	0.03

Using these coefficients, roughness values were predicted and compared against the observed values, and the comparison of these values excluding the outliers is shown in Figure 10.13.

Figure 10.13 Observed v predicted roughness (calibrated factors using coefficients Kgs 1.7, Kgr 2.4, Kgm 0.56, Kgp 0.03).



## 10.4 Customisation of HDM-4 Parameters to New Zealand Conditions

In HDM-4, some of the parameters were modified or added and some expressions were replaced by simple relationships. These parameters or relationships were verified and customised for New Zealand conditions in this study.

### 10.4.1 Drainage Factor

One of the most distinctive enhancements of the HDM-4 model is the incorporation of the drainage factor. Morosiuk et al. (2000) recommended drainage factors for different types of drains and their condition (Appendix D gives applicable values for



drainage factor). Different types of drain were generally found within an analysis section considered for modelling. Hence, for the network level analysis, only 'drainage inadequacy' was considered for defining the drainage deficiency factor.

The drainage deficiency (DD) was calculated from the drainage condition data available in RAMM database using the following expression:

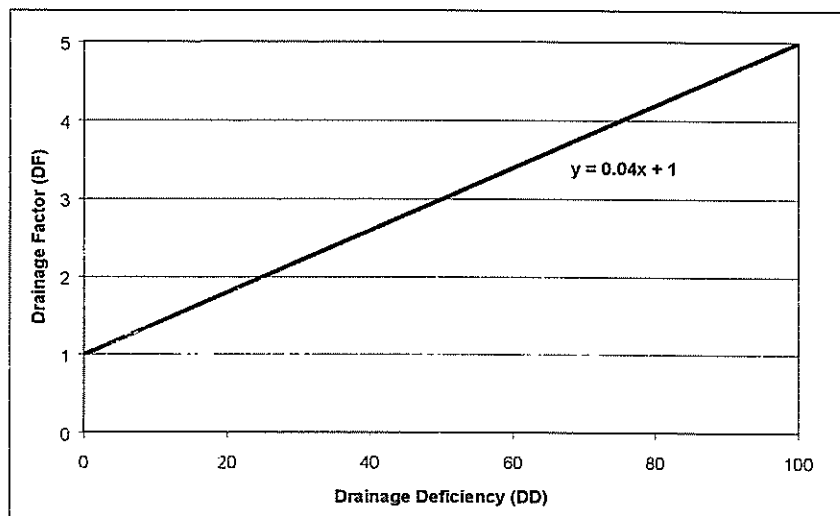
$$DD = \text{Max} (100, (\text{surf\_broke\_lhs} + \text{surf\_highlip\_lhs} + \text{surf\_brokesurf\_lhs} + \text{surf\_blocked\_lhs} + \text{surf\_uphill\_lhs} + \text{earth\_block\_lhs} + \text{earth\_inadeq\_lhs} + \text{ineffsh\_lhs} + \text{surf\_broke\_rhs} + \text{surf\_highlip\_rhs} + \text{surf\_brokesurf\_rhs} + \text{surf\_blocked\_rhs} + \text{surf\_uphill\_rhs} + \text{earth\_block\_rhs} + \text{earth\_inadeq\_rhs} + \text{ineffsh\_rhs}) / (2 * \text{Insp\_length}) * 100)$$

where:

insp_length	is the inspection length in the rating section.
surf_broke_lhs	is the ineffective drainage length due to broken channel on lhs (left hand side) or rhs (right hand side).
surf_highlip_lhs	is the ineffective drainage length due to high channel lip (lhs, rhs)
surf_brokesurf_lhs	is the length of drainage ineffective due to broken c/w surface channel (lhs, rhs)
surf_blocked_lhs	is the length of drainage ineffective due to blocked channel (lhs, rhs)
surf_uphill_lhs	is the length of drainage ineffective due to uphill grade (lhs, rhs)
earth_block_lhs	is the length of the blocked earth drainage (lhs, rhs)
earth_inadeq_lhs	is the length of inadequate earth channel (lhs, rhs)
ineffsh_lhs	is the ineffective shoulder (lhs, rhs)

A drainage factor was assigned depending on the drainage deficiency (DD) estimated as above. A regression analysis was done between the DD and DF as shown in Figure 10.14.

**Figure 10.14** Drainage factor v drainage inadequacy for the analysed road sections.



The DF was calculated using the following linear relationship:

$$DF = 1 + 0.04 * DD$$

However, more detailed study on the influence of the drainage factor and drainage deficiency on the road condition is recommended.

#### 10.4.2 Standard Deviation of Rutting

In NZ dTIMS and HDM-III, standard deviation of rutting is calculated through an empirical relationship involving traffic factor, mean rut depth and pavement strength. In HDM-4, a simplified expression is introduced for the calculation of standard deviation of rutting based on mean rut depth value as proposed by NDLI (1995). The model relationship is given below:

$$RDS = a_0 RDM$$

where:

- RDS standard deviation of rut depth
- RDM mean rut depth (mm)
- $a_0$  model coefficient

Different values for model coefficient ( $a_0$ ) were suggested for different ranges of mean rut depth in the HDM-4 system. To verify these model coefficients requires a large amount data, and the criteria to select the dataset were as follows:

- Data with good precision; and
- Data covering a range of values applicable to New Zealand conditions.

After the preliminary examination of the different datasets, HSD data for the Transit network were considered for the analysis. More than 51,000 records were used to determine the model coefficient to New Zealand conditions. Mean rut depth ranged from 0mm to as high as 36mm for this dataset. Data were segregated in different ranges of rut depth, and data were analysed to evaluate the model coefficients for different rut depth ranges. The calculated model coefficients are given in Table 10.1.

**Table 10.1 RDS model coefficient value calculated for New Zealand conditions.**

Mean Rut Depth (mm)	A <sub>0</sub>		
	HDM-4	This study	Adjusted for NZ
0 – 5	0.8	0.45	0.5
5 – 15	0.5	0.34	0.4
>15	0.3	0.27	0.3

The model coefficient values that were evaluated with the data broadly followed the HDM-4 trend. After examination of the results, these coefficients were adjusted to be applicable for New Zealand conditions. However, as these values were derived from the data considered for the study, they can be treated as preliminary in nature. Further investigations are recommended to evaluate the applicability of these suggested values to New Zealand conditions.

### 10.4.3 Roughness Reset Value

Roughness reset equations were analysed for different treatment types like surface treatment, overlay and reconstruction, etc.

#### 10.4.3.1 Surface Treatment

Surface treatments generally would not reduce the roughness much after treatment except at very high 'before roughness' values. HDM-III, NZ dTIMS and HDM-4 systems incorporated the similar reset equations for roughness after surface treatments. Analysis was then carried out to determine the applicability of this equation to New Zealand conditions. The model expression available in HDM-III, NZ dTIMS and HDM-4 is given below:

$$RI_a = RI_b + \min\{0, \max[0.3 (5.4 - RI_b), -0.5]\} - 0.0066(ACX)$$

where:

$RI_a$  = roughness in IRI after maintenance treatment

$RI_b$  = roughness in IRI before maintenance treatment

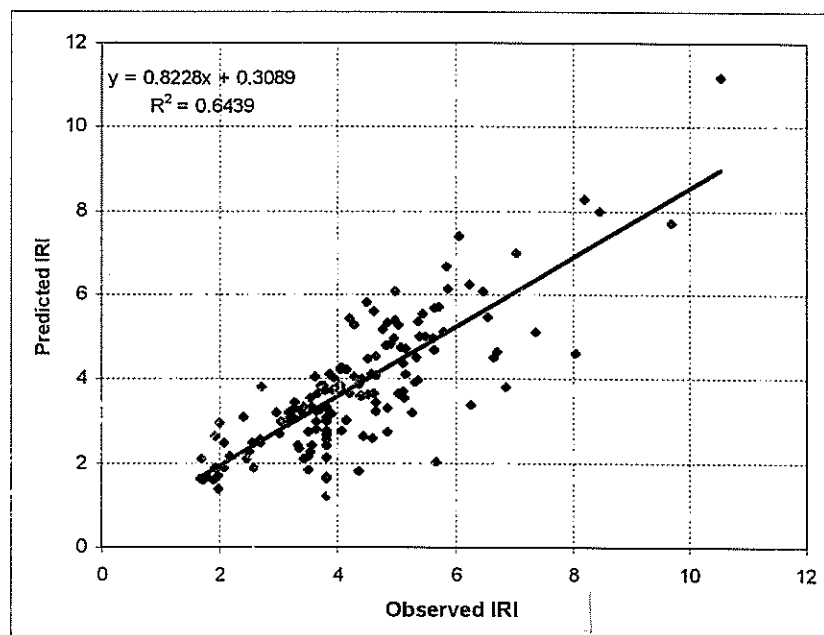
ACX = area of index cracking

Data compiled for Hamilton City Council (HCC) to study the works effects on roughness (Clark 2001) was used for the analysis. Chipsealed (with small chips) sections were taken for the analysis.

The cracking parameter (ACX) was excluded from the above equation to predict the after treatment roughness because the contribution of this parameter would be negligible for New Zealand conditions.

Before and after roughness was compared for about 250 road sections. Figure 10.15 presents the results of the analysis.

**Figure 10.15** Roughness after surface treatment for 250 road sections, comparing predicted and observed IRI.



The analysis showed that observed values are slightly higher than predicted by the reset expression. This suggests that some modifications will need to be applied to the coefficients of the expression for particular networks. This is true as maintenance practice in New Zealand varies considerably in different regions, although for the national setup, the existing expression can be used.

#### 10.4.3.2 Overlay

As discussed in Section 5.4.2, roughness reset expression for overlay in HDM-4 system is simple in comparison to that used in HDM-III and NZ dTIMS Setup. The HDM-III model was considered to be unnecessarily complicated and that a simpler (linear) model form would be easy to calibrate to local conditions (Morosiuk et.al. 2000).

The following equation was provided in the HDM-4 system to reset the roughness after applying an overlay:

$$RI_a = a_0 + a_1 \max(RI_b - a_0, 0) \max(a_2 - H, 0)$$

where:

- $RI_a$  = Roughness after overlay (IRI)
- $RI_b$  = Roughness before overlay (IRI)
- $H$  = Thickness of the overlay (mm)

The default values for model coefficients ( $a_0$ ,  $a_1$  and  $a_2$ ) considered in HDM-4 are given in Table 10.2.

**Table 10.2 Default model coefficients for roughness reset after overlay for use in HDM-4.**

Coefficient	Value
a0	2.0
a1	0.01
a2	80

Generally a thin AC overlay of 25mm is applied as a resurfacing treatment for high traffic roads in New Zealand. Hence, calibrating the HDM-4 overlay roughness reset equation was considered for an overlay of 25mm thick AC. Thus, 'H' in the above equation was taken as 25. In HDM-4,  $a_0$  is taken as 2.0 and that appeared to be applicable for New Zealand conditions.

Considering these values, the above reset equation was modified to the following form for further analysis.

$$RI_a = a_0 + a_1 \max(RI_b - a_0, 0)$$

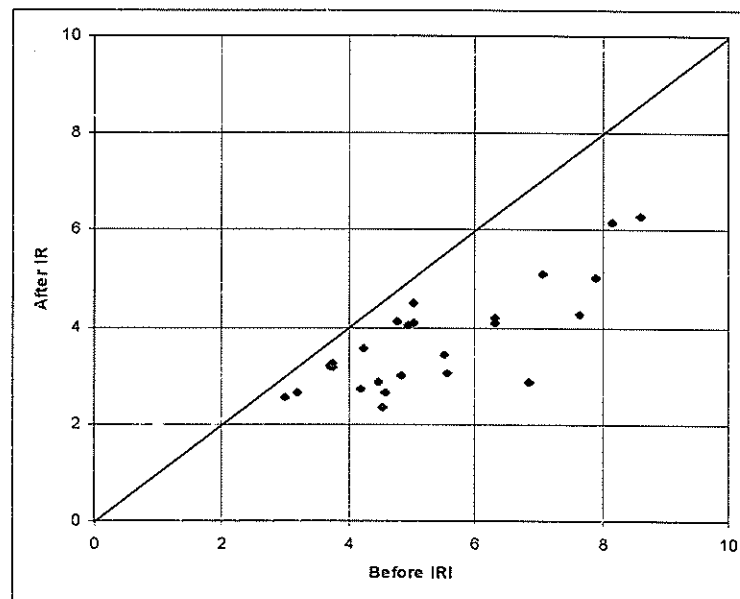
The analysis was carried out to determine the value of ' $a_1$ ' applicable to New Zealand conditions.

Datasets from Auckland City Council (ACC) and HCC (Clark 2001) were considered for the analysis. The before and after overlay roughness values were filtered out, and Figure 10.16 shows the observed before and after overlay roughness for ACC data.

Linear regression analysis was carried out to determine the preliminarily applicable value for 'a<sub>1</sub>'.

In the next step, to capture the effect of 'before roughness' on reset value, analysis was done for different roughness (before overlay) ranges. The results are shown in Table 10.3.

**Figure 10.16 Roughness (IRI) after overlay, observed for Auckland City Council data.**



**Table 10.3 Roughness reset model coefficient (a<sub>1</sub>) for New Zealand conditions, using Auckland (ACC) and Hamilton (HCC) data.**

Range of roughness	HDM-4 <sup>1</sup>	ACC	HCC
2 – 4	0.55	0.67	0.86
4 – 6		0.48	0.59
> 6		0.52	0.51
All		0.51	0.55

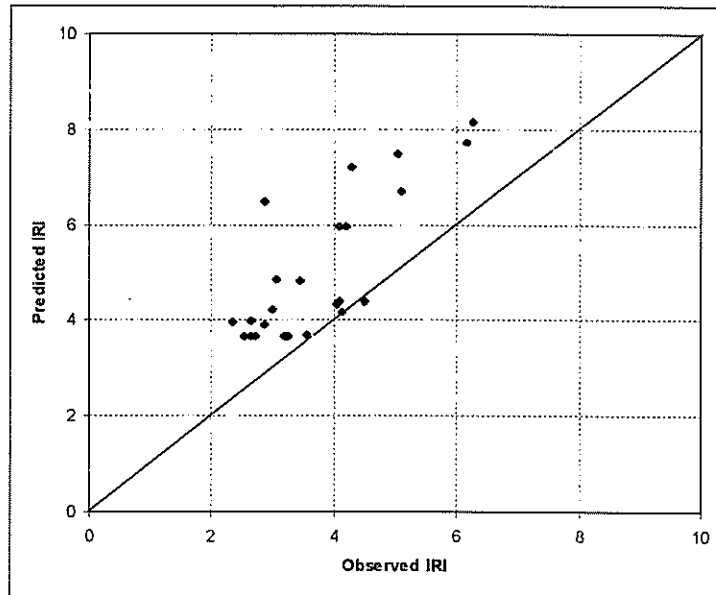
Note: 1 model coefficient (a<sub>1</sub>) calculated for a thickness of 25mm.

The predicted and observed roughness (IRI) values were compared using ACC data in Figure 10.17.

The roughness reset model coefficient for overlay customised for New Zealand appeared to be predicting reasonably well (Figure 10.17).

As the value for model coefficient varies between different networks, then calibration by region of the reset parameters are generally recommended (Table 10.3).

**Figure 10.17** Comparison of observed and predicted roughness (IRI) after overlay, using Auckland data.



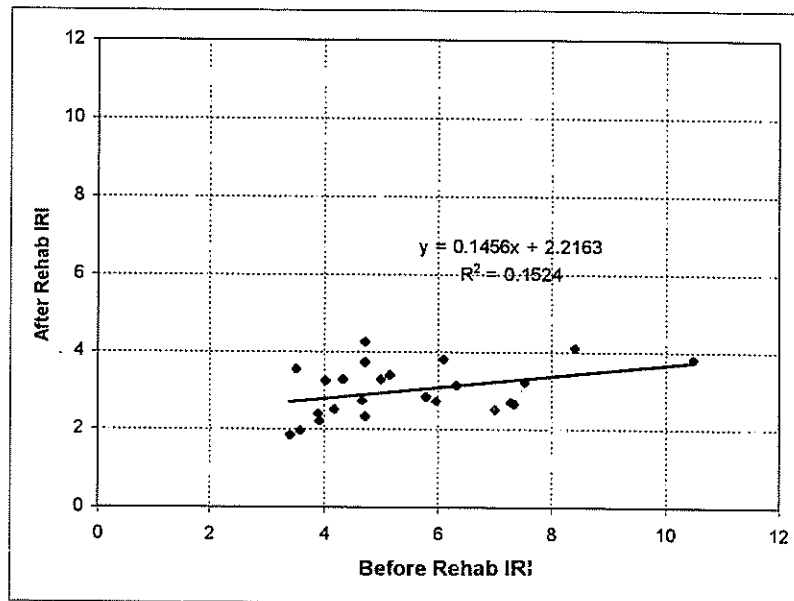
### 10.4.3.3 Reconstruction

A before and after study was carried out for a number of sections in the Auckland City, Northshore City and Franklin District Council networks for the reset value of rip and remake treatment. Rip and remake treatment was considered for the study as it is the most popular treatment used in New Zealand for shape correction. The original pavement is generally ripped off and some new base material is added, and basecourse is stabilised, then covered with chipseal.

Roughness was measured before and after rehabilitation treatment using a ROMDAS bump integrator. Two sets of roughness surveys were conducted, one before and one after the application of the treatment. Each set of surveys consisted of 3 to 4 runs in each direction of the section. About 50m-long untreated control sections were used at the two ends of the treated sections for comparison of roughness, which is shown in Figure 10.18.

Roughness is usually reset at a new value after the reconstruction/rehabilitation treatment, and the value of 2.5 IRI for ST pavements is taken as in the NZ dTIMS Setup. However, the analysis showed that the reset value ranged from 2 to 4 IRI for different sections, and is generally attributed to the construction quality that was maintained, etc. As the roughness reset value could have considerable effect on model prediction after treatment, the reset values after rehabilitation needs to be customised, based on construction quality of works for different regions of New Zealand.

Figure 10.18 Roughness (IRI value) after rehabilitation by rip and remake.



## 10.5 Conclusions

- Sensitivity analysis showed that not all the calibration coefficients are equally sensitive to the pavement deterioration. The calibration coefficients for wet/dry season SNP ratio (Kf), crack initiation (Kcia), environmental (Kgm) and general (Kgp) roughness factors were found to be very sensitive to the roughness progression.
- Using the RAMM historical data, and followed by a field verification and additional data collection, level 2 calibration was carried out for HDM-4 models for the comparison purpose.
- The calibration coefficient for the wet/dry SNP ratio could not be calibrated because specific data required for the exercise were lacking. Considering the high sensitivity of this coefficient a further study is recommended.
- The drainage factor can be estimated for New Zealand conditions from the following equation using the available RAMM data:

$$DF = 1 + 0.04 * DD$$

However, this equation is very preliminary in nature and a detailed study is recommended.

- The following calibration coefficients (Table 10.4) were used for the comparison of model outputs with real network data in this project.

**Table 10.4 HDM-4 model calibration coefficients used for comparison.**

Calibration Factor	RD Model	Values
Kf	Wet/dry season SNP ratio	1
Kddf	Drainage factor	1
Kcia	All structural cracking initiation	1
Kcpa	All structural cracking progression	1
Kciw	Wide structural cracking initiation	1
Kcpw	Wide structural cracking progression	1
Kcit	Transverse thermal cracking initiation	0
Kcpt	Transverse thermal cracking progression	0
Kvi	Ravelling initiation	1
Kvp	Ravelling progression	0
Kpi	Pothole initiation	1
Kpp	Pothole progression	1
Keb	Edge break	1
Krid	Initial densification of rutting	0.5
Krst	Structural deformation of rutting	0.7
Krpd	Plastic deformation of rutting	0.7
Krsw	Surface wear of rutting	0
Kgm	Environmental coefficient of roughness	0.56
Ksnpk	Adjusted structural number for roughness	1
Kgs	Structural component of roughness	1.7
Kgr	Rutting component of roughness	2.4
Kgp	Patching component of roughness	0.03
Ktd	Texture depth progression	1
Ksfc	Skid resistance	1
Ksfcs	Speed effects of skid resistance	1

- The simpler form of standard deviation of rutting model included in the HDM-4 model was found applicable to New Zealand conditions and was customised, based on the available HSD rutting data. Table 10.5 presents the adjusted value of standard deviation of rutting factor found in this study.

**Table 10.5 Rutting standard deviation model coefficient used in the HDM-4 model.**

Mean Rut Depth (mm)	ao	
	HDM-4	Adjusted for NZ
0 – 5	0.8	0.5
5 – 15	0.5	0.4
>15	0.3	0.3



- The simplified version of roughness reset model for overlay included in the HDM-4 model could easily be customised to New Zealand conditions with confidence. The roughness reset coefficients for a 25mm-thick AC overlay determined from the dataset used in this study is given in Table 10.6.

**Table 10.6 Model coefficient for roughness reset after overlay for HDM-4 based on Auckland (ACC) and Hamilton (HCC) data.**

Range of roughness	HDM-4	ACC	HCC
2 – 4	0.01	0.012	0.016
4 – 6		0.009	0.011
> 6		0.009	0.009
All		0.009	0.010

- Analysis indicated that the variation in values of calibration factors, model coefficients and reset values for different treatments within a region also depend on the type of construction and construction quality, etc. Hence, calibration coefficients derived from the study should be treated as preliminary, and more relevant only for the networks (i.e. ACC, HCC) considered for the study.
- A recently initiated Transit New Zealand calibration project is aimed at calibrating the HDM models to New Zealand conditions. This project should consider the additional calibration factors included in the HDM-4 model that need to be calibrated to New Zealand conditions.

## **11. Comparison of Models using Real Network Data**

### **11.1 Introduction**

NZ dTIMS and HDM-4 RDWE models have been compared with synthetic data in Chapter 9. A preliminary conclusion is that both systems (NZ dTIMS and HDM-4) are giving reasonable results with the given input. To increase user confidence, the outputs generated by the NZ dTIMS Setup and HDM-4 system using real network dataset are compared in this chapter.

### **11.2 Preparation for Analysis**

A synthetic database was prepared for the analysis (Chapter 8). The models were calibrated against the historical data of the network (Chapter 10). These preliminary calibrated coefficients were used for comparison of the model outputs, and the analysis was carried out for 'do-nothing' and 'routine maintenance' scenarios.

The following routine maintenance activities were included in the NZ dTIMS Setup and HDM-4 system for comparison purposes for a 'routine' maintenance scenario:

- Crack sealing of all structural cracking (50%);
- Crack sealing of wide structural cracking (80%); and
- Pothole patching (90%).

Cracking is one of the most significant parameters driving the resurfacing action in New Zealand. On the other hand, roughness is the last in the series of the HDM model analysis and hence represents the cumulative effect of the model's prediction. Hence the decision was to use predicted cracking for year 5 and roughness for year 10 as the output parameter for the comparison.

### **11.3 Comparison of Models for Do-Nothing Case**

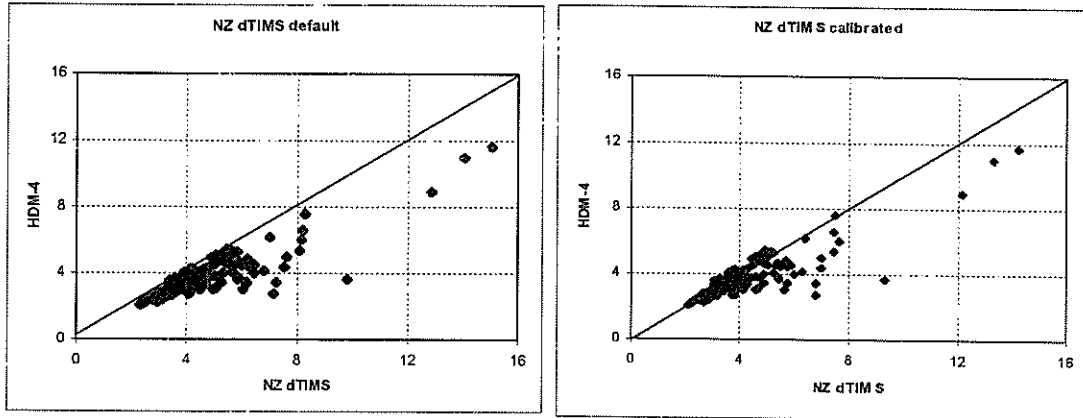
Comparison was carried out between the roughness predicted by the NZ dTIMS Setup and HDM-4 system with no maintenance works (do-nothing case). The analysis showed a good correlation between the roughness predicted by NZ dTIMS and HDM-4. However, NZ dTIMS predicted slightly higher roughness than HDM-4. The comparison was made for two different scenarios as follows:

- Default NZ dTIMS v calibrated HDM-4; and
- Calibrated NZ dTIMS v calibrated HDM-4 (calibration values determined in this study were used in both systems).

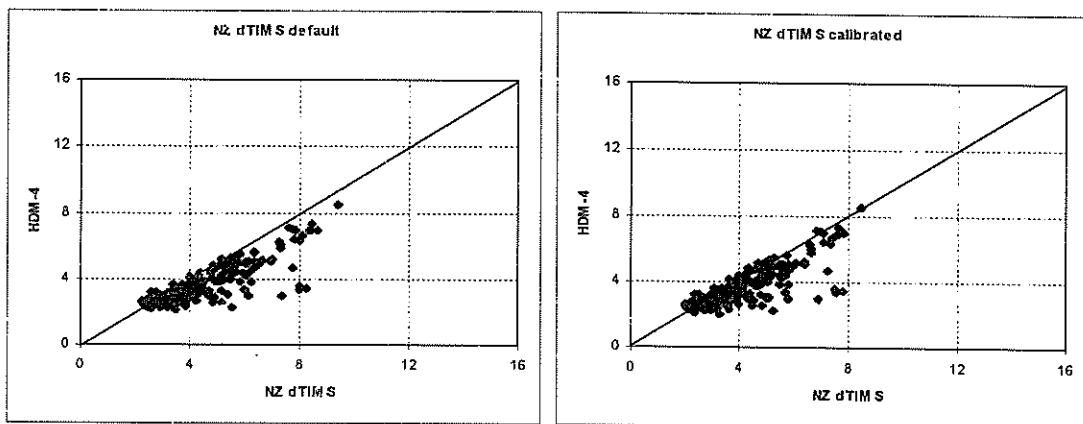
The following Figure 11.1a-c, presents the comparison of the models for the do-nothing scenario applied to roughness for the Transit New Zealand, District Council and City Council network databases respectively.

Figure 11.1 Comparison of HDM-4 and NZ dTIMS models for 'do-nothing' scenario.

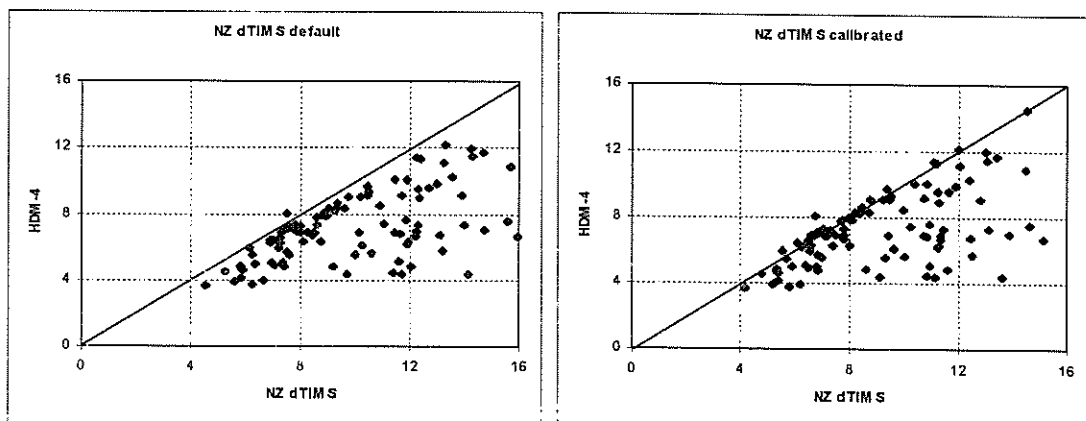
a. Transit New Zealand Network



b. District Council Network



c. City Council Network



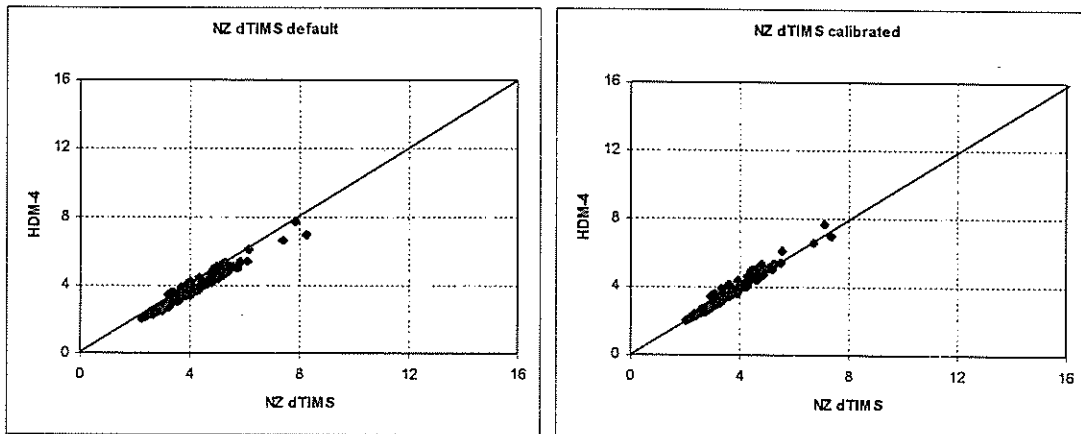
11.4 Comparison of Models for Routine Maintenance

A similar comparison was carried out between the roughness predicted by NZ dTIMS and HDM-4 with routine maintenance treatment. A higher correlation

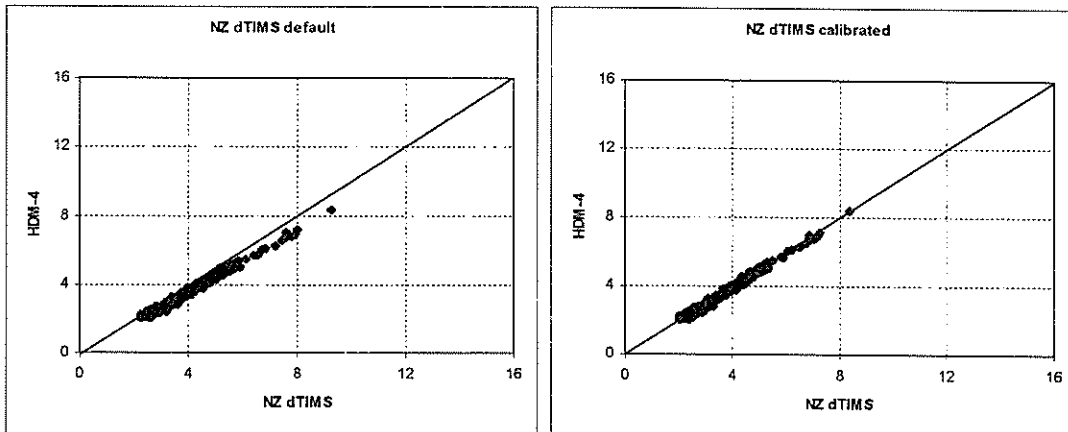
was observed in this case than for the do-nothing case. Figure 11.2a-c presents the analysis results for the three network datasets.

Figure 11.2 Comparison of HDM-4 and NZ dTIMS models for routine maintenance scenario.

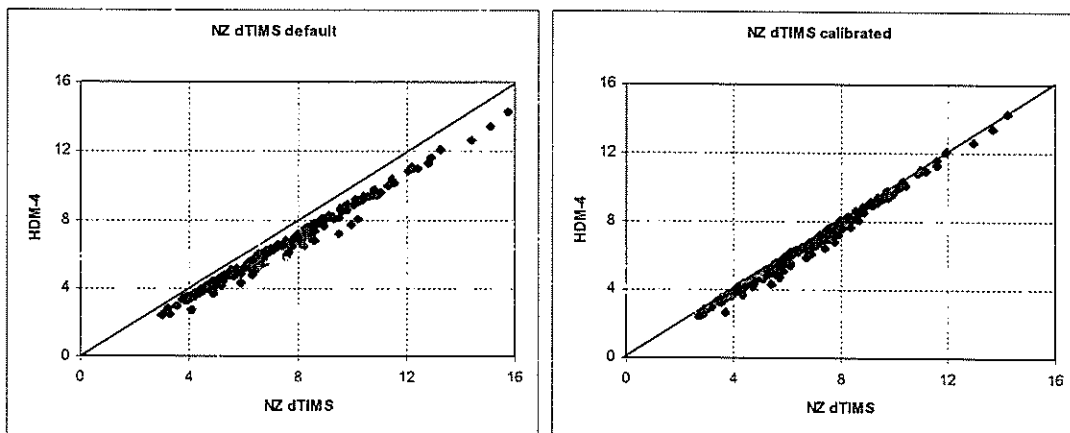
a. Transit New Zealand Network



b. District Council Network

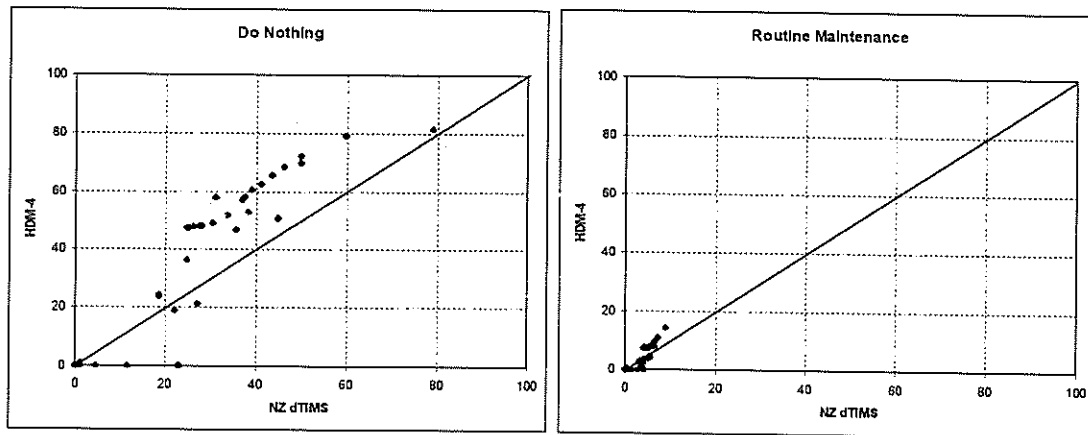


c. City Council Network



The analysis for area of all cracking was also carried out in a similar manner as for roughness. Although the cracking progression was found to be slightly lower in NZ dTIMS, good correlation was found with routine maintenance. However, the comparison between the area of cracking is shown in Figure 11.3 for do-nothing and routine maintenance scenarios with calibrated models for NZ dTIMS and HDM-4.

**Figure 11.3 Comparison of models for area of cracking for do-nothing and routine maintenance scenarios.**



Both models (NZ dTIMS and HDM-4) gave reasonably close predictions for routine maintenance with the real network data because routine maintenance is applied every year over almost the entire New Zealand network. Hence, no major impact in roughness prediction will be observed if the HDM-III model in NZ dTIMS Setup are replaced with HDM-4 model.

## 11.5 Conclusions

- HDM-4 models programmed in HDM-4 interface are giving reasonable results with the real network dataset.
- Similar predictions were observed between the calibrated HDM-4 and NZ dTIMS models for the routine maintenance scenario.
- HDM-4 models can be transformed to NZ dTIMS if required without any major changes to the current setup in the HDM-4 interface.

## 12. Conclusions

- HDM-4 system is more suitable than HDM-III for detailed network analysis as well as for project level analysis, with the enhancements that have been made to the RD and WE models, provided that data of required quality is available.
- Extending the RD and WE models over concrete pavements has increased the scope of the HDM-4 models. Bituminous pavements, block pavements and unsealed roads generally do exist on the New Zealand network. Although modelling for block pavements has not yet been implemented in HDM-4, its inclusion is proposed for the next revisions of HDM-4.
- A comprehensive pavement classification philosophy has been introduced in HDM-4 models which enables the appropriate pavement type to be chosen for analysis. To implement this, some changes in the NZ dTIMS Setup will be required if the HDM-4 pavement classification approach is adopted.
- Pavement strength (SNP) is estimated by considering the effects of the seasonal changes and drainage condition in HDM-4. SNP is adjusted for cracking, potholing and drainage deterioration for predicting the distresses in HDM-4. Presently NZ dTIMS adjusts the SNC for cracking and potholing for roughness modelling only.
- The effects of the drainage condition are quantified in HDM-4 by including the drainage factor. The reduction of the pavement strength was found to be as high as 20% with the deterioration in drainage condition.
- The drainage factor can be estimated for New Zealand conditions from the following equation using the available RAMM data:

$$DF = 1 + 0.04 * DD$$

However, the above equation is very preliminary in nature and a detailed study is recommended.

- Continuous variables (CDS and CDB) are introduced in HDM-4 models to express construction defects, in place of a flag value (CQ) presently used in the NZ dTIMS Setup. These variables were very sensitive to the surface distress prediction and can be used to reflect different qualities of construction.
- In HDM-4, cracking models considered the effects of drainage condition and adjusted structural number. The new cracking type, transverse thermal cracking, introduced in HDM-4 is considered to be not applicable for New Zealand conditions. The reflection cracking model is proposed for inclusion in HDM-4, though a study is required to verify the suitability of this model for New Zealand conditions.

- In HDM-4, the traffic parameter (YAX) was introduced in the ravelling progression model to consider the effect of traffic volume on ravelling progression. Flag value for construction quality indicator used in the NZ dTIMS Setup has been replaced by a continuous variable in HDM-4 ravelling models. With these modifications, HDM-4 ravelling models have become more flexible and applicable to New Zealand conditions.
- Rutting related to both plastic deformation and surface wear has been included in the HDM-4 system. Plastic deformation rutting model appeared to be useful for New Zealand conditions while surface wear component is considered not relevant. Separate calibration factors for each component makes the HDM-4 rutting model flexible for customisation of the different components to the local conditions.
- Pothole component of roughness has been extensively modified in HDM-4 by introducing time lapse in patching of potholes and freedom to manoeuvre factors. However, the effect of this change appeared to be minimal for New Zealand conditions considering the current routine maintenance practice of 100% patching of potholes as soon as they appear.
- Edge break is observed on some narrow roads, particularly in rural areas of New Zealand. Thus the edge break model could be considered for inclusion in the NZ dTIMS Setup.
- HDM-4 models provide greater flexibility to enable customisation to the local conditions, and they include 23 calibration factors for bituminous paved roads, of which 15 were found to be sensitive.
- The calibration coefficients listed in Table 12.1 were used in this study for the comparison of NZ dTIMS and HDM-4 predictions using real network data. These calibration factors were based on the dataset considered for the present study and can generally be used elsewhere in New Zealand. However, regional calibration is recommended for best results.
- Comparison of the model predictions with the real network dataset has shown good correlation between the HDM-4 system and the NZ dTIMS Setup.
- Routine maintenance activities have extensively been modified in the HDM-4 system. The effects of drainage maintenance, patching of wide structural cracking, and time lapse in pothole patching appeared to be significant for the New Zealand condition.
- The reset models for pavement strength and roughness have been modified in HDM-4 for resurfacing and overlay. The customisation carried out in this study revealed that these new models could be calibrated for the New Zealand condition with the data that is presently available. The default values provided in the HDM-4 for these models can be considered at the national level and can easily be customised to local conditions at regional level.



Table 12.1 Initial calibration coefficients recommended for New Zealand conditions.

Calibration Factor	RD Model	Values
Kf	Wet/dry season SNP ratio	1
Kddf	Drainage factor	1
Kcia	All structural cracking initiation	1
Kcpa	All structural cracking progression	1
Kciw	Wide structural cracking initiation	1
Kcpw	Wide structural cracking progression	1
Kcit	Transverse thermal cracking initiation	0
Kcpt	Transverse thermal cracking progression	0
Kvi	Ravelling initiation	1
Kvp	Ravelling progression	0
Kpi	Pothole initiation	1
Kpp	Pothole progression	1
Keb	Edge break	1
Krid	Initial densification of rutting	0.5
Krst	Structural deformation of rutting	0.7
Krpd	Plastic deformation of rutting	0.7
Krsw	Surface wear of rutting	0
Kgm	Environmental coefficient of roughness	0.56
Ksnpk	Adjusted structural number for roughness	1
Kgs	Structural component of roughness	1.7
Kgr	Rutting component of roughness	2.4
Kgp	Patching component of roughness	0.03
Ktd	Texture depth progression	1
Ksfc	Skid resistance	1
Ksfcs	Speed effects of skid resistance	1

## **13. Recommendations**

### **13.1 Introduction**

Recommendations drawn from the present study are divided into two categories as follows:

- Potential enhancements to the NZ dTIMS Setup; and
- Future research requirements.

Potential enhancements to the current NZ dTIMS Setup were limited to the bituminous paved roads. Some of these enhancements can be carried out directly to the current NZ dTIMS Setup while some of them require further research, and these are listed in this chapter.

### **13.2 Potential Enhancements to NZ dTIMS Setup**

The comparison study shows that the outputs of the HDM-4 system and NZ dTIMS Setup are similar while using default parameters. The preliminary calibrated HDM-4 model appeared to be generating reasonable predictions for New Zealand conditions. The predictive capability of the NZ dTIMS Setup can be enhanced by including the HDM-4 model in the current the dTIMS Setup.

Two options for the enhancement of the NZ dTIMS Setup are possible:

- Full implementation of HDM-4 RDWE models for bituminous pavements; and
- Partial implementation of HDM-4 RDWE models.

#### **13.2.1 Full Implementation**

The modelling approach in HDM-4 has been changed significantly. Hence, for full implementation of HDM-4 the basic setup of the current NZ dTIMS needs to be changed. Such modifications are desirable while switching the existing software platform of dTIMS 6.1 to dTIMS CT (Concurrent Transformation version).

#### **13.2.2 Partial Implementation**

Minor modifications to the NZ dTIMS Setup can be done by incorporating the additional models and/or by replacing some of the existing parameters. Although these additional models and parameters will require detailed study for their proper customisation, using engineering judgement would facilitate the initial customisation and future research.

Models that could be considered for implementation are:

- The concept of adjusted structural number in HDM-4 can be introduced to the NZ dTIMS Setup, because SNP considers the effects of seasonal changes and drainage condition on pavement performance.



- Drainage is considered the ‘enemy’ of a pavement. Pavement deteriorates much faster with poor drainage than it would with good drainage maintenance. The drainage factor can be included in the NZ dTIMS Setup to quantify the annual drainage condition. This factor would allow the various RCAs to recognise annual drainage maintenance requirements.
- CQ used in the NZ dTIMS Setup can be replaced by the continuous variables CDS and CDB that were introduced in the HDM-4 system. This would require incorporation of some of the HDM-4 models into the NZ dTIMS Setup.
- Modified reset models for pavement strength, rutting and roughness for periodic maintenance treatments can be included in the works effects modelling of the NZ dTIMS Setup.

### **13.3 Future Research Requirements**

The further research works required for the optimum use of the HDM-4 models have been identified, together with their importance for priority as follows:

- Research is needed to establish the effect of drainage on the pavement strength. It should also include the effect of seasonal variation on pavement strength (high priority).
- Research to define the procedure for defining the values of continuous construction quality indicators (CDS and CDB) for different surface materials and construction quality (high priority).
- Study of the validity and calibration of reflection cracking model to New Zealand conditions (medium priority).
- Study of the rutting related to the plastic deformation and structural deformation in New Zealand pavements (medium priority).
- Study of the HDM-4 ravelling initiation and progression models, and their applicability to New Zealand conditions (low priority).
- Study of the applicability of the edge break model to New Zealand conditions by developing a shoulder deterioration model (high priority).



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## Appendix A: NZ dTIMS Pavement Deterioration Models

### Cracking

#### Crack Initiation Expressions

##### Granular and bituminous bases

$$TYCRA = Kci \left\{ \begin{array}{l} Fc \max \left[ a1 \exp(a2 SNC + a3 \frac{YE4}{SNC^2} (1 + CQ)) \max \left( 1 - \frac{PCRW}{a4}, 0 \right), \right. \\ \left. a5 + a6 HSNEW \right] + CRT \end{array} \right\}$$

##### Cemented bases

$$TYCRA = Kci \left\{ \begin{array}{l} Fc \left[ (a1 KA + a2 KW) (1 + a3 HSE) + (1 - KA) (1 - KW) \right. \\ \left. (a4 \exp(a5 HSE + a6 \ln(CMOD) - a7 \ln(DEF) - a8 YE4 DEF) + CRT) \right] \end{array} \right\}$$

$$TYCRW = Kci \max(a1 + a2 TYCRA, a3 TYCRA)$$

where TYCRA	is the time in years to crack initiation
TYCRW	is the time in years to initiation of wide cracking
Kci	is a user-specified deterioration for crack initiation
Fc	is the occurrence distribution factor for the sub-section
SNC	is the modified structural number
YE4	is the annual axle loading in millions per lane
CQ	is the construction fault indicator for surface treatments 1 if faults exist, 0 otherwise
PCRW	is the area of wide cracks before resurfacing
PCRA	is the area of all cracks before resurfacing
HSNEW	is the thickness of the new surfacing in mm
CRT	is the cracking retardation time if a preventive treatment has been applied
HSE	is the effective thickness of surfacing layers defined as: $\min[100, HSNEW + (1 - KW) HSOLD]$
KW	is a variable for indicating the presence of wide cracking in the old surfacing layers, defined as: $\min[0.05 \max(PCRW - 10, 0), 1]$
KA	is a variable for indicating the presence of all cracking in the old surfacing layers, defined as: $\min[0.05 \max(PCRA - 10, 0), 1]$
CMOD	is the resilient modulus of the cemented base in GPa
DEF	is the deflection under a 40 kN wheel load

### Crack Progression Expressions

The crack progression model is incremental in form and uses three expressions, which give a symmetrical curve. The Brazilian research, described in Paterson (1987) gave models based on time and traffic. The time-based models were selected for use in HDM-III and have the form:

For  $ACR < 50\%$  and  $ACR + \Delta ACR < 50\%$   

$$\Delta ACR = K_{cp} CRP (a b \Delta T + ACR^b)^{1/b} - ACR$$

For  $ACR > 50\%$   

$$\Delta ACR = K_{cp} CRP [100 - ACR - \max(-a b \Delta T + (100 - ACR)^b, 0)^{1/b}]$$

For  $ACR < 50\%$  and  $ACR + \Delta ACR > 50\%$   

$$\Delta ACR = K_{cp} CRP [100 - (2 \cdot 50^b - ACR^b - a b \Delta T)^{1/b} - ACR]$$

where  $ACR$  is the cracked area at the start of the year  
 $\Delta ACR$  is the increase in cracking during time  $\Delta T$   
 $K_{cp}$  is a user-defined coefficient  
 $CRP$  is the retardation due to preventive treatment, given by:  

$$CRP = 1 - 0.12 CRT$$

The parameters used in the above expressions together with the corresponding dTIMS fields are given below.

### Potholes

The initiation period for potholing is a function of traffic and thickness of bituminous layers:

$$TMIN = \max(a1 + a2 HS + a3 YAX, a4)$$

where  $TMIN$  is the time in years from initiation of the triggering distress and the initiation of potholing  
 $HS$  is the total thickness of asphaltic layers, including the base if bituminous

Pothole initiation is further constrained by setting a minimum area of primary distress - 20% for wide cracking and 30% for ravelling.

Annual increase in potholed area is the summation of the amounts derived from wide cracking, ravelling and enlargement of existing potholes:

$$\Delta APOT = \min(\Delta APOTCR + \Delta APOTRV + \Delta APOTP, 10)$$

where  $\Delta POT$  is the total annual increase in percent area.

The increases derived from wide cracking, ravelling and enlargement are given by:

$$\Delta APOTCR = K_{pp} \min \left( \frac{1.6ACRW * YAX * W (1 + CQ)}{SNC * HS * ELANES}, 6 \right)$$

$$\Delta APOTRV = K_{pp} \min \left( \frac{0.32ARAV * YAX * W (1 + CQ)}{SNC * HS * ELANES}, 6 \right)$$

$$\Delta APOTP = \min \{ APOT * YAX (MMP + 0.1) \max(a1 + a2 * HS, a3), 10 \}$$

where W is the pavement width in m  
 ELANES is the number of lanes  
 MMP is the rainfall in m/month

The coefficients in the enlargement component are dependent on the type of base.

Additional parameters required for the modelling are given in the table below.

Parameters	Relevant dTIMS Fields	Description	Remarks
TMIN			
HS	HBASE	Thickness of base (mm)	
YAX	YAX	Annual no. of axles per lane	
W	PAV_WID	Pavement width (m)	
ELANES			Effective width 3 m taken, ELANES not considered
MMP	MMP	Rainfall (m/month)	

Note: 1/ No data input required for composite index as it is calculated based on other parameters

## Rutting

The mean rut depth in the first year after construction of a pavement is given by:

$$RDM = K_{rp} \frac{39800(YE410^6)^{ERM}}{SNC^{0.502} COMP^{2.30}}$$

The increment in mean rut depth in the second and subsequent years is given by:

$$\Delta RDM = K_{rp} * RDM \left( \frac{0.166 + ERM}{AGE3} + 0.0219MMP * \Delta ACRX * \ln(\max(1, AGE3 * YE4)) \right)$$

$$ERM = 0.09 - 0.0009 RH + 0.0384 DEF + 0.00158 MMP * CRX$$

where RDM is the mean rut depth  
 ΔRDM is the annual increment in mean rut depth  
 AGE3 is the time since construction in years

CRX	is the area of indexed cracking in percent given by: ACRA + 0.39 ACRW
$\Delta$ CRX	is the annual increment in indexed cracking
RH	is a rehabilitation indicator (1 if overlay, 0 otherwise)

The standard deviation of rut depth after pavement construction is given by:

$$RDS = Krp \frac{4390 \Delta RDM^{0.532} (YE4 * 10^6)^{ERS}}{SNC^{0.422} COMP^{1.66}}$$

The increment in STD of rut depth in subsequent years is given by:

$$\Delta RDS = Krp RDS \left( \frac{0.532 \Delta RDM}{RDM} + \frac{ERS}{AGE3} + 0.0519 MMP * \Delta CRX \ln(MAX(1, AGE3 * YE4)) \right)$$

$$ERS = -0.0086 RH + 0.00115 MMP * CRX$$

### Roughness

HDM-III predicts the annual increment of roughness progression as several components (hence described as the component incremental model). These components are structural deformation, surface condition and environment. The expression is:

$$\begin{aligned} \Delta IRI = & 134 \exp(m t) YE4 (1 + SNK)^{-5} \quad (\text{structural deformation}) \\ & + 0.114 \Delta RDS + 0.0066 \Delta \text{ACRX} + 0.42 \Delta \text{APOT} (\text{surface condition}) \\ & + m IRI \quad (\text{environment}) \end{aligned}$$

All the parameters used in the roughness increment models are described earlier.

## Appendix B: HDM-4 Pavement Deterioration Models

### Structural Number

$$SNP = f_s = f_s SNP_d$$

where

$$\frac{f}{[(1-d) + d(f^p)]^{1/p}}$$

and

SNP	=	average annual adjusted structural number
SNP <sub>d</sub>	=	dry season SNP
f	=	SNP <sub>w</sub> / SNP <sub>d</sub> ratio
d	=	length of dry season as a fraction of the year
p	=	exponent of SNP specific to the appropriate deterioration model

### Values of Exponent 'p' for Calculating SNP.

Distress	Model	p
Cracking	Initiation of Structural Cracking	2.0
Rut Depth	Initial Densification	0.5
	Structural Deformation	1.0
Roughness	Structural Component	5.0

If only one season's SNP value is available then the following relationship (Riley 1996a, 1999a) should be used to calculate the wet/dry season SNP ratio. This relationship is also used to calculate the wet/dry season SNP ratio for each year of the analysis period, taking into account changes in the drainage and amount of potholing and cracking.

$$F = K_f \left[ 1 - \frac{(1 - \exp(a_0 \text{MMP}))}{a_1} (1 + a_2 \text{DF}_a)(1 + a_3 \text{ACRA}_a + a_4 \text{APOT}_a) \right]$$

where

f	=	SNP <sub>w</sub> / SNP <sub>d</sub> ratio
SNP <sub>w</sub>	=	wet season SNP
SNP <sub>d</sub>	=	dry season SNP
MMP	=	mean monthly precipitation, in mm/month
DF <sub>a</sub>	=	drainage factor at start of analysis year
ACRA <sub>a</sub>	=	total area of cracking at start of analysis year, in %
APOT <sub>a</sub>	=	area of potholing at the start of the analysis year, in %
K <sub>f</sub>	=	calibration factor for wet/dry season SNP ratio

The HDM-4 coefficient values a<sub>0</sub> to a<sub>4</sub> are given in the following table:

**Coefficient Values for the Seasonal SNP Ratio**

Coefficient	$a_0$	$a_1$	$a_2$	$a_3$	$a_4$
Default value	-0.01	10	0.25	0.02	0.05

The drainage factor, DF, is a continuous variable whose value can range between 1 (excellent) and 5 (very poor), depending on the type of drain (Paterson 1998). The user needs to input the type of drain and the condition of the drain as excellent, good, fair, poor or very poor.

### Cracking Model

#### Cracking Initiation

The HDM-4 relationships for predicting the time to initiation of all structural cracking are as follows:

##### *Stabilised Base*

if **HSOLD = 0** (i.e. Original Surfacing)

$$ICA = K_{cia} \{ CDS^2 a_0 \exp[a_1 HSE + a_2 \log_e(CMOD) + a_3 \log_e(DEF) + a_4 (YE4)(DEF)] + CRT \}$$

if **HSOLD > 0** (i.e. Overlays or Reseals)

$$ICA = K_{cia} \{ CDS^2 [(0.8 KA + 0.2 KW)(1 + 0.1 HSE) + (1 - KA)(1 - KW) a_0 \exp(a_1 HSE + a_2 \log_e(CMOD) + a_3 \log_e(DEF) + a_4 (YE4)(DEF))] + CRT \}$$

##### *All Other Bases*

if **HSOLD = 0** (i.e. Original Surfacing)

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

if **HSOLD > 0** (i.e. Overlays or Reseals)

(i) All surface materials except CM, SL and CAPE

$$ICA = K_{cia} \{ CDS^2 [\max(a_0 \exp[a_1 SNP + a_2 (YE4/SNP^2)] \max(1 - PCRW/a_3, 0), a_4 HSNEW)] + CRT \}$$

(ii) Surface materials - CM, SL and CAPE

$$ICA = K_{cia} \{ CDS^2 [\max(a_0 \exp[a_1 SNP + a_2(YE4/SNP^2)]) \max(1 - PCRA/a_3, 0), a_4] + CRT \}$$

where

- ICA = time to initiation of all structural cracks, in years
- CDS = construction defects indicator for bituminous surfacings
- YE4 = annual number of equivalent standard axles, in millions/lane
- SNP = average annual adjusted structural number of the pavement
- DEF = mean Benkelman beam deflection in both wheelpaths, in mm
- CMOD = resilient modulus of soil cement, in GPa (range between 0 and 30 GPa for most soils)
- HSNEW = thickness of the most recent surfacing, in mm
- HSOLD = total thickness of previous underlying surfacing layers, in mm
- PCRA = area of all cracking before latest reseal or overlay, in per cent
- PCRW = area of wide cracking before latest reseal or overlay, in %
- KW =  $\min [0.05 \max (PCRW - 10, 0), 1]$
- KA =  $\min [0.05 \max (PCRA - 10, 0), 1]$
- HSE =  $\min [100, HSNEW + (1 - KW) HSOLD]$
- $K_{cia}$  = calibration factor for initiation of all structural cracking
- CRT = crack retardation time due to maintenance, in years

The HDM-4 coefficient values  $a_0$  to  $a_4$  for the initiation of All cracking are given in the following table:

**Coefficient Values for the Initiation of All Structural Cracking Models**

Pavement Type	Surface Material	HSOLD Value	$a_0$	$a_1$	$a_2$	$a_3$	$a_4$
AMGB	All	0	4.21	0.14	-17.1		
	All except CM	> 0	4.21	0.14	-17.1	30	0.025
	CM	> 0	13.2	0	-20.7	20	1.4
AMAB	All	0	4.21	0.14	-17.1		
		> 0	4.21	0.14	-17.1	30	0.025
AMAP	All	> 0	4.21	0.14	-17.1	30	0.025
AMSB	All	0	1.12	0.035	0.371	-0.418	-2.87
		> 0	1.12	0.035	0.371	-0.418	-2.87
STGB	All	0	13.2	0	-20.7		
	All except SL, CAPE	> 0	13.2	0	-20.7	20	0.22
	SL, CAPE	> 0	13.2	0	-20.7	20	1.4
STAB	All	0	13.2	0	-20.7		
	All except SL, CAPE	> 0	4.21	0.14	-17.1	20	0.12
	SL, CAPE	> 0	4.21	0.14	-17.1	30	0.025
STAP	All	> 0	4.21	0.14	-17.1	20	0.12
STSB	All	0	1.12	0.035	0.371	-0.418	-2.87
		> 0	1.12	0.035	0.371	-0.418	-2.87

**Initiation of Wide Structural Cracking**

$$ICW = K_{ciw} \max [(a_5 + a_6 ICA), a_7 ICA]$$



where

ICW = time to initiation of wide structural cracks, in years  
 $K_{ciw}$  = calibration factor for initiation of wide structural cracking

The HDM-4 coefficient values  $a_5$  to  $a_7$  for the initiation of Wide cracking are given in the following table:

**Coefficient Values for the Initiation of Wide Structural Cracking Models**

Pavement Type	Surface Material	HSOLD value	$a_5$	$a_6$	$a_7$
AMGB	All	0	2.46	0.93	0
	All except CM	> 0	2.04	0.98	0
	CM	> 0	0.70	1.65	0
AMAB	All	0	2.46	0.93	0
		> 0	2.04	0.98	0
AMAP	All	> 0	2.04	0.98	0
AMSB	All	0	1.46	0.98	0
		> 0	0	1.78	0
STGB	All	0	2.66	0.88	1.16
	All except SL, CAPE	> 0	1.85	1.00	0
	SL, CAPE	> 0	0.70	1.65	0
STAB	All	0	2.66	0.88	1.16
	All except SL, CAPE	> 0	1.85	1.00	0
	SL, CAPE	> 0	2.04	0.98	0
STAP	All	> 0	1.85	1.00	0
STSB	All	0	1.46	0.98	0
		> 0	0	1.78	0

**Progression of All Structural Cracking**

Progression of All cracking commences when  $\delta t_A > 0$  or  $ACA_a > 0$

where

$$\delta t_A = 1 \text{ if } ACA_a > 0, \text{ otherwise } \delta t_A = \max \{0, \min [(AGE2 - ICA), 1]\}$$

$$\text{if } ACA_a \geq 50 \text{ then } z_A = -1, \text{ otherwise } z_A = 1$$

$$ACA_a = \max (ACA_a, 0.5)$$

$$SCA = \min [ACA_a, (100 - ACA_a)]$$

$$Y = [a_0 a_1 z_A \delta t_A + SCA^{a_1}]$$

(i) if  $Y < 0$  then

$$dACA = K_{cpa} \left( \frac{CRP}{CDS} \right) (100 - ACA_a)$$

(ii) if  $Y \geq 0$  then

$$dACA = K_{cpa} \left( \frac{CRP}{CDS} \right) z_A (Y^{1/a_1} - SCA)$$

(iii) if  $ACA_a \leq 50$  and  $ACA_a + dACA > 50$  then

$$dACA = K_{cpa} \left( \frac{CRP}{CDS} \right) (100 - c_1^{1/a_1} - ACA_a)$$

where

$$c_1 = \max \left\{ \left[ 2 (50^{a_1}) - SCA^{a_1} - a_0 a_1 \delta t_A \right], 0 \right\}$$

and

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

ACA<sub>a</sub> = area of all cracking at the start of the analysis year, in %

δt<sub>A</sub> = fraction of analysis year in which all cracking progression applies

AGE2 = pavement surface age, in years

K<sub>cpa</sub> = calibration factor for progression of all structural cracking

CRP = retardation of cracking progression due to preventive treatment, given by CRP = 1 - 0.12 CRT

The HDM-4 coefficient values a<sub>0</sub> and a<sub>1</sub> for the progression of All structural cracking are given in the following table.

**Coefficient Values for Progression of All Structural Cracking.**

Pavement Type	Surface Material	HSOLD value	All cracking	
			a <sub>0</sub>	a <sub>1</sub>
AMGB	All	0	1.84	0.45
	All except CM	> 0	1.07	0.28
	CM	> 0	2.41	0.34
AMAB	All	0	1.84	0.45
		> 0	1.07	0.28
AMAP	All	> 0	1.07	0.28
AMSB	All	0	2.13	0.35
		> 0	2.13	0.35
STGB	All	0	1.76	0.32
		> 0	2.41	0.34
STAB	All	0	1.76	0.32
	All except SL, CAPE	> 0	2.41	0.34
	SL, CAPE	> 0	1.07	0.28
STAP	All	> 0	2.41	0.34
STSB	All	0	2.13	0.35
		> 0	2.41	0.34

### Progression of Wide Structural Cracking

Progression of Wide cracking commences when δt<sub>w</sub> > 0 or ACW<sub>a</sub> > 0

where

$$\delta t_w = 1 \text{ if } ACW_a > 0, \text{ otherwise } \delta t_w = \max \{0, \min [(AGE2 - ICW), 1]\}$$

The initiation of Wide cracking is constrained so that it does not commence before the area of All cracking (ACA<sub>a</sub>) exceeds 5% as follows:

$$\delta t_w = 0 \text{ if } ACA_a \leq 5 \text{ and } ACW_a \leq 0.5 \text{ and } \delta t_w > 0$$

If patching of Wide cracking was performed in the previous analysis year, reducing the area of Wide cracking to below 1% but with the area of All cracking remaining at over 11% at the start of the current analysis year (i.e.  $ACW_a \leq 1$  and  $ACA_a > 11$ ), then the rate of progression of Wide cracking is assumed to begin not at the low initial rate, but at a higher rate similar to the rate before patching.

For this situation a temporary value of Wide cracking,  $ACW_{temp}$  is defined to be 5% less than  $ACA_a$ ; i.e.

$$ACW_{temp} = ACA_a - 5 \quad \text{if } ACW_a \leq 1 \text{ and } ACA_a > 11$$

This value is then used as the temporary value of  $ACW_a$  for the computation of  $dACW$  in that analysis year.

$dACW$  is computed each analysis year as follows:

$$\text{if } ACW_a \geq 50 \text{ then } z_w = -1, \text{ otherwise } z_w = 1$$

$$ACW_a = \max(ACW_a, 0.5)$$

$$SCW = \min[ACW_a, (100 - ACW_a)]$$

$$Y = [a_2 a_3 z_w \delta t_w + SCW^{a_3}]$$

(i) if  $Y < 0$  then

$$dACW = K_{cpw} \left( \frac{CRP}{CDS} \right) \min [(ACA_a + dACA - ACW_a), (100 - ACW_a)]$$

(ii) if  $Y \geq 0$  then

$$dACW = K_{cpw} \left( \frac{CRP}{CDS} \right) \min [(ACA_a + dACA - ACW_a), z_w (Y^{1/a_3} - SCW)]$$

(iii) if  $ACW_a \leq 50$  and  $ACW_a + dACW > 50$  then

$$dACW = K_{cpw} \left( \frac{CRP}{CDS} \right) \min [(ACA_a + dACA - ACW_a), (100 - c_1^{1/a_3} - ACW_a)]$$

where

$$c_1 = \max \{ [2 (50^{a_3}) - SCW^{a_3} - a_2 a_3 \delta t_w], 0 \}$$

and

- dACW= incremental change in area of wide structural cracking during analysis year, in per cent of total carriageway area
- ACW<sub>a</sub> = area of wide cracking at the start of the analysis year, in per cent
- δt<sub>w</sub> = fraction of analysis year in which wide cracking progression applies
- K<sub>cpw</sub> = calibration factor for progression of wide structural cracking

The coefficient values a<sub>2</sub> and a<sub>3</sub> for the progression of Wide Structural cracking are given in the following table.

**Coefficient Values for Progression of Wide Structural Cracking.**

Pavement Type	Surface Material	HSOLD Value	Wide cracking	
			a <sub>2</sub>	a <sub>3</sub>
AMGB	All	0	2.94	0.56
	All except CM	> 0	2.58	0.45
	CM	> 0	3.40	0.35
AMAB	All	0	2.94	0.56
		> 0	2.58	0.45
AMAP	All	> 0	2.58	0.45
AMSB	All	0	3.67	0.38
		> 0	3.67	0.38
STGB	All	0	2.50	0.25
		> 0	3.40	0.35
STAB	All	0	2.50	0.25
	All except SL, CAPE	> 0	3.40	0.35
	SL, CAPE	> 0	2.58	0.45
STAP	All	> 0	3.40	0.35
STSB	All	0	3.67	0.38
		> 0	3.40	0.35

**Initiation of Transverse Thermal Cracking**

A distinction is made between the time to initiation of transverse thermal cracking in original surfacings and in overlays or reseals. The relationships for predicting the time to initiation, ICT, in years are as follows:

(a) if **HSOLD = 0** (i.e. Original Surfacing)

$$ICT = K_{cit} \max [1, (CDS)(CCT)]$$

(b) if **HSOLD > 0** (i.e. Overlays or Reseals)

$$ICT = K_{cit} CDS (CCT + a_0 + a_1 HSNEW)$$

- where ICT = time to initiation of transverse thermal cracks, in years
- CCT = coefficient of thermal cracking
- HSNEW = thickness of the most recent surfacing, in mm
- CDS = construction defects indicator for bituminous surfacings
- K<sub>cit</sub> = calibration factor for initiation of transverse thermal cracking

The HDM-4 coefficient values a<sub>0</sub> to a<sub>1</sub> for the initiation of transverse thermal cracks is given in the following table.

**Coefficient Values for the Initiation of Transverse Thermal Cracking**

Pavement Type	HSOLD value	Coefficients	
		$a_0$	$a_1$
All pavement types	> 0	-1.0	0.02

**Progression of Transverse Thermal Cracking**

Progression of Transverse Thermal cracking commences when  $\delta t_T > 0$

where  $\delta t_T = 1$  if  $ACT_a > 0$ ,

otherwise  $\delta t_T = \max \{0, \min [(AGE2 - ICT), 1]\}$

**(i) if HSOLD = 0** (i.e. Original Surfacing)

$$dNCT = K_{cpt} \left( \frac{1}{CDS} \right) \max \left\{ 0, \min \left[ (NCT_{eq} - NCT_a), \left( \frac{2 NCT_{eq} (AGE3 - ICT - 0.5)}{T_{eq}^2} \right) \right] \right\} \delta t_T$$

**(ii) if HSOLD > 0** (i.e. Overlays or Reseals)

$$dNCT = K_{cpt} \left( \frac{1}{CDS} \right) \min \left\{ (NCT_{eq} - NCT_a), \max \left[ \min (a_3 PNCT, (PNCT - NCT_a)), \left( \frac{2 NCT_{eq} (AGE3 - ICT - 0.5)}{T_{eq}^2} \right), 0 \right] \right\} \delta t_T$$

where

- dNCT = incremental change in number of transverse thermal cracks during analysis year, in no/km
- PNCT = number of transverse thermal cracks before latest overlay or reseal, in no/km
- $NCT_a$  = number of (reflected) transverse thermal cracks at the start of the analysis year, in no/km
- $NCT_{eq}$  = maximum number of thermal cracks, in no/km
- $T_{eq}$  = time since crack initiation to reach maximum number of thermal cracks, in years
- AGE3 = age since last overlay or reconstruction, in years
- $K_{cpt}$  = calibration factor for progression of transverse thermal cracking and the other variables are as described in transverse thermal cracking initiation

The HDM-4 coefficient value for the progression of Transverse Thermal cracks is given in the following table.

**Coefficient Value for the Progression of Transverse Thermal Cracking**

Pavement Type	HSOLD value	a <sub>2</sub>
All pavement types	> 0	0.25

**Ravelling Model**

**Initiation of Ravelling**

The HDM-4 ravelling initiation model is given below.

$$IRV = K_{vi} (CDS)^2 a_0 (RRF) \exp[a_1(YAX)]$$

where

- IRV = time to ravelling initiation, in years
- CDS = construction defects indicator for bituminous surfacings
- YAX = annual number of axles of all vehicle classes in the analysis year, in millions/lane
- K<sub>vi</sub> = calibration factor for ravelling initiation
- RRF = ravelling retardation factor due to maintenance

The HDM-4 coefficient values a<sub>0</sub> and a<sub>1</sub> for the ravelling initiation model are given in the following table.

**Coefficient Values for Ravelling Initiation Model.**

Surface Type	Surface Material	a <sub>0</sub>	a <sub>1</sub>
AM	All except CM	100	-0.156
	CM	8.0	-0.156
ST	All except SL, CAPE	10.5	-0.156
	SL, CAPE	14.1	-0.156

**Ravelling Progression**

Progression of ravelling commences when δt<sub>v</sub> > 0 or ARV<sub>a</sub> > 0

where δt<sub>v</sub> = 1 if ARV<sub>a</sub> > 0

otherwise δt<sub>v</sub> = max {0, min [(AGE2 - IRV), 1]}

if ARV<sub>a</sub> ≥ 50 then z = -1, otherwise z = 1

$$ARV_a = \max (ARV_a, 0.5)$$

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

$$YAX = \max [\min (YAX, 1), 0.1]$$

$$Y = [(a_0 + a_1 YAX) a_2 z \delta t_v + SRV^{a_2}]$$

(i) if  $Y < 0$  then

$$dARV = \left( \frac{K_{vp}}{RRF} \right) \left( \frac{1}{CDS^2} \right) (100 - ARV_a)$$

(ii) if  $Y \geq 0$  then

$$dARV = \left( \frac{K_{vp}}{RRF} \right) \left( \frac{1}{CDS^2} \right) z (Y^{1/a_2} - SRV)$$

(iii) if  $ARV_a \leq 50$  and  $ARV_a + dARV > 50$  then

$$dARV = \left( \frac{K_{vp}}{RRF} \right) \left( \frac{1}{CDS^2} \right) (100 - c_1^{1/a_2} - ARV_a)$$

where

$$c_1 = \max \{ [2 (50^{a_2}) - SRV^{a_2} - (a_0 + a_1 YAX) a_2 \delta t_v], 0 \}$$

and

$dARV$  = change in area of ravelling during analysis year, in % of total carriageway area

$ARV_a$  = area of ravelling at the start of the analysis year, in %

$\delta t_v$  = fraction of analysis year in which ravelling progression applies

$AGE_2$  = pavement surface age, in years

$K_{vp}$  = calibration factor for ravelling progression and the other variables are as defined for ravelling initiation

The HDM-4 coefficient values  $a_0$  to  $a_2$  for the ravelling progression model is given in the following table.

**Coefficient Values for Ravelling Progression Model**

Pavement Type	$a_0$	$a_1$	$a_2$
All pavement types	0.6	3.0	0.352

### Pothole Model

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

where

$IPT$  = time between the initiation of wide cracking or ravelling and the initiation of potholes, in years

$HIS$  = total thickness of bituminous surfacing, in mm

$CDB$  = construction defects indicator for the base

$YAX$  = annual number of axles of all vehicle classes in the analysis year, in millions/lane



MMP = mean monthly precipitation, in mm/month  
 K<sub>pi</sub> = calibration factor for pothole initiation

The values for IPT are calculated separately for potholing due to cracking and due to ravelling. The separation between these two mechanisms of potholing is maintained throughout the analysis with the progression being modelled differently for potholes due to cracking, due to ravelling and due to the enlargement of existing potholes.

The HDM-4 coefficient values a<sub>0</sub> to a<sub>4</sub> for the potholing initiation model is given in the following table.

**Coefficient Values for Potholing Initiation Model**

Cause of Pothole Initiation	Pavement Type	a <sub>0</sub>	a <sub>1</sub>	a <sub>2</sub>	a <sub>3</sub>	a <sub>4</sub>
Cracking	AMGB, STGB	2.0	0.05	1	0.5	0.01
	All except GB bases	3.0	0.05	1	0.5	0.01
Ravelling	AMGB, STGB	2.0	0.05	1	0.5	0.01
	All except GB bases	3.0	0.05	1	0.5	0.01

**Pothole Progression**

$$TLF = 1.541 \left[ \exp\left(\frac{T_{pat}}{365}\right) - 1 \right]$$

where

TLF = time lapse factor (0 < TLF ≤ 1)  
 T<sub>pat</sub> = average time between the occurrence and patching of potholes, in days

The annual incremental increase in the number of pothole units is calculated as:

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

$$dNPT = \sum_{i=1}^3 dNPT_i$$

where

dNPT = total number of additional pothole units per km during analysis year  
 dNPT<sub>i</sub> = additional number of pothole units per km derived from distress type i (wide cracking, ravelling, enlargement) during analysis year  
 ADIS<sub>i</sub> = % area of wide cracking at start of the analysis year, or % area of ravelling at start of the analysis year, or number of existing pothole units per km at start of the analysis year  
 TLF = time lapse factor

$K_{pp}$  = calibration factor for pothole progression and the other variables are as defined for potholing initiation

The HDM-4 coefficient values  $a_0$  to  $a_4$  for the potholing progression model is given in the following table.

**Coefficient Values for Potholing Progression Model**

Cause of Pothole Progression	Pavement Type	$a_0$	$a_1$	$a_2$	$a_3$	$a_4$
Cracking	AMGB, STGB	1.0	1.0	10	0.005	0.08
	All except GB bases	0.5	1.0	10	0.005	0.08
Ravelling	AMGB, STGB	0.2	1.0	10	0.005	0.08
	All except GB bases	0.1	1.0	10	0.005	0.08
Enlargement	AMGB, STGB	0.07	1.0	10	0.005	0.08
	All except GB bases	0.035	1.0	10	0.005	0.08

### Edge Break Model

$$dVEB = K_{eb} a_0 PSH (AADT)^2 ESTEP (S)^{a_1} (a_2 + MMP/1000) 10^{-6}$$

where

$$PSH = \max \left\{ \min \left[ \max \left( a_3 + a_4 CW, \frac{CW_{max} - CW}{a_5} \right), 1 \right], 0 \right\}$$

and

- dVEB = annual loss of edge material, in  $m^3/km$
- PSH = proportion of time using shoulder
- AADT = annual average daily traffic
- ESTEP = elevation difference from pavement to shoulder, in mm
- MMP = mean monthly precipitation, in mm/month
- S = average traffic speed, in km/h
- CW = carriageway width, in metres
- $CW_{max}$  = user definable maximum carriageway width for the occurrence of edge break, in metres (default = 7.2, maximum = 7.5)
- $K_{eb}$  = calibration factor for edge break progression

The HDM-4 coefficient values  $a_0$  to  $a_5$  for the edge break model are given in the following table.

**Coefficient Values for Edge Break Model**

Pavement Type	$a_0$	$a_1$	$a_2$	$a_3$	$a_4$	$a_5$
AMGB	50	-1	0.2	2.65	-0.425	10
AMAB, AMSB, AMAP	25	-1	0.2	2.65	-0.425	10
STGB	75	-1	0.2	2.65	-0.425	10
STAB, STSB, STAP	50	-1	0.2	2.65	-0.425	10

## Rutting Model

### Initial Densification

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

where

RDO	=	rutting due to initial densification, in mm
YE4	=	annual number of equivalent standard axles, in millions/lane
DEF	=	average annual Benkelman beam deflection, in mm
SNP	=	average annual adjusted structural number of the pavement
COMP	=	relative compaction, in %
$K_{rid}$	=	calibration factor for initial densification

The HDM-4 coefficient values  $a_0$  to  $a_4$  for the initial densification model is given in the following table.

**Coefficient Values for Initial Densification Model**

Pavement Type	$a_0$	$a_1$	$a_2$	$a_3$	$a_4$
AMGB, AMAB, AMSB, STGB, STAB, STSE	51740	0.09	0.0384	-0.502	-2.30
AMAP, STAP	0	0	0	0	0

### Structural Deformation

Structural deformation without cracking

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

Structural deformation after cracking

$$\Delta RDST_{crk} = K_{rst} [a_0 SNP^{a_1} YE4^{a_2} MMP^{a_3} ACX_a^{a_4}]$$

The total annual incremental increase in structural deformation is as follows:

(i) if  $ACRA = 0$

$$\Delta RDST = \Delta RDST_{uc}$$

(ii) if  $ACRA > 0$

$$\Delta RDST = \Delta RDST_{uc} + \Delta RDST_{crk}$$

where

$\Delta RDST$	=	total incremental increase in structural deformation in analysis year, in mm
$\Delta RDST_{uc}$	=	incremental rutting due to structural deformation without cracking in analysis year, in mm

- $\Delta RDST_{crk}$  = incremental rutting due to structural deformation after cracking in analysis year, in mm  
MMP = mean monthly precipitation, in mm/month  
 $ACX_a$  = area of indexed cracking at the beginning of analysis year, %  
 $K_{rst}$  = calibration factor for structural deformation and the other variables are as defined for initial densification

The HDM-4 coefficient values  $a_0$  to  $a_4$  for the structural deformation models are given in the following table.

**Coefficient Values for Structural Deformation Model**

	Pavement Type	$a_0$	$a_1$	$a_2$	$a_3$	$a_4$
Without Cracking	All pavement types	44950	-1.14	0.11	-2.3	
After Cracking	All pavement types	0.0000248	-0.84	0.14	1.07	1.11

### Plastic Deformation

$$\Delta RDPD = K_{rpd} CDS^3 a_0 YE4 Sh^{a1} HS^{a2}$$

where

- $\Delta RDPD$  = incremental increase in plastic deformation in analysis year (mm)  
CDS = construction defects indicator for bituminous surfacings  
YE4 = annual number of equivalent standard axles, in millions/lane  
Sh = speed of heavy vehicles, in km/h  
HS = total thickness of bituminous surfacing, in mm  
 $K_{rpd}$  = calibration factor for plastic deformation

The HDM-4 coefficient values for the plastic deformation model are given in the following table.

**Coefficient Values for Plastic Deformation Model**

Surface Type	$a_0$	$a_1$	$a_2$
AM	2.46	-0.78	0.71
ST	0	-0.78	0.71

### Surface Wear

$$\Delta RDW = K_{rsw} [ a_0 PASS^{a1} W^{a2} S^{a3} SALT^{a4} ]$$

where

- $\Delta RDW$  = incremental increase in rut depth due to studded tyres in analysis year, in mm  
PASS = annual number of vehicle passes with studded tyres in one direction, in thousands  
S = average traffic speed, in km/h

- SALT = variable for salted or unsalted roads (2 = salted; 1 = unsalted)  
 W = road width, in m (carriageway plus total shoulder width)  
 $K_{rsw}$  = calibration factor for surface wear

The HDM-4 coefficient values  $a_0$  to  $a_4$  for the surface wear model is given in the following table.

**Coefficient Values for Surface Wear Model**

Pavement Type	$a_0$	$a_1$	$a_2$	$a_3$	$a_4$
All pavement types	0.0000248	1.0	-0.46	1.22	0.32

### Total Rut Depth

if  $AGE4 \leq 1$

$$\Delta RDM = RDO + \Delta RDPD + \Delta RDW$$

otherwise

$$\Delta RDM = \Delta RDST + \Delta RDPD + \Delta RDW$$

where

$\Delta RDM$  = incremental increase in total mean rut depth in both wheelpaths in analysis year, in mm

RDO = initial densification, in mm

$\Delta RDST$  = incremental increase in structural deformation in analysis year, in mm

$\Delta RDPD$  = incremental increase in plastic deformation in analysis year, in mm

$\Delta RDW$  = incremental increase in wear by studded tyres in analysis year, in mm

The maximum mean rut depth at the end of the year has been increased to 100 mm in HDM-4 from the 50 mm limit in HDM-III.

The total rut depth,  $RDM_b$ , at any given time is given as:

$$RDM_b = \min [(RDM_a + \Delta RDM), 100]$$

where

$RDM_b$  = total mean rut depth in both wheelpaths at the end of the analysis year, in mm

$RDM_a$  = total mean rut depth in both wheelpaths at the start of the analysis year, in mm

### Standard Deviation of Rut Depth

$$RDS = \max [0.3, (0.9 - 0.04 RDM)]$$

## Roughness Model

### Structural Component

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

where

$$SNPK_b = \max [(SNP_a - dSNPK), 1.5]$$

$$dSNPK = K_{snpk} a_0 [\min(a_1, ACX_a) HSNEW + \max(\min(ACX_a - PACX, a_2), 0) HSOLD]$$

and

- $\Delta RI_s$  = incremental change in roughness due to structural deterioration during analysis year, in m/km IRI
- $dSNPK$  = reduction in adjusted structural number due to cracking
- $SNPK_b$  = adjusted structural number due to cracking at end of analysis year
- $SNP_a$  = adjusted structural number at start of analysis year
- $ACX_a$  = area of indexed cracking at start of analysis year, in %
- $PACX$  = area of previous indexed cracking in old surfacing, in %  
i.e. 0.62 (PCRA) + 0.39 (PCRW)
- $HSNEW$  = thickness of the most recent surfacing, in mm
- $HSOLD$  = total thickness of previous underlying surfacing layers, in mm
- $AGE3$  = age since last overlay or reconstruction, in years
- $YE4$  = annual number of equivalent standard axles, in millions/lane
- $m$  = environmental coefficient
- $K_{gm}$  = calibration factor for environmental coefficient
- $K_{snpk}$  = calibration factor for SNPK

### Cracking Component

$$\Delta RI_c = a_0 \Delta ACRA$$

where

- $\Delta RI_c$  = incremental change in roughness due to cracking during analysis year, in m/km IRI
- $\Delta ACRA$  = incremental change in area of total cracking during analysis year, in %

### Rutting Component

$$\Delta RI_r = a_0 (RDS_b - RDS_a)$$

where

$\Delta RI_r$  = incremental change in roughness due to rutting during analysis year, in m/km IRI

$RDS_b$  = standard deviation of rut depth at end of analysis year, in mm

$RDS_a$  = standard deviation of rut depth at start of analysis year, in mm

### Potholing Component

$$\Delta RI_t = a_0(a_1 - FM) \left\{ [(NPT_a)(TLF) + (\Delta NPT)(TLF/2)]^{a_2} - (NPT_a)^{a_2} \right\}$$

where

$FM = (\max \{ \min [0.25 (CW - 3), 1], 0 \}) (\max [(1 - AADT/5000), 0])$

$\Delta RI_t$  = incremental change in roughness due to potholing during analysis year, in m/km IRI

$\Delta NPT$  = incremental change in pothole units during analysis year, in no/km

$NPT_a$  = number of pothole units per km at start of the analysis year

$TLF$  = time lapse factor

### Environmental Component

$$\Delta RI_e = m K_{gm} RI_a$$

where

$\Delta RI_e$  = incremental change in roughness due to environment during analysis year, in m/km IRI

$RI_a$  = roughness at the start of the analysis year, in m/km IRI

$m$  = environmental coefficient

$K_{gm}$  = calibration factor for the environmental component

### Total Change in Roughness

The total incremental change in roughness in HDM-4 is given by:

$$\Delta RI = K_{gp} [\Delta RI_s + \Delta RI_c + \Delta RI_r + \Delta RI_t] + \Delta RI_e$$

where

$\Delta RI$  = total incremental change in roughness during analysis year, in m/km IRI

$K_{gp}$  = calibration factor for roughness progression

The HDM-4 coefficient values for the various roughness components are given in the following table.



**Coefficient Values for Roughness Components**

Pavement Type	Roughness Component	Equation	a <sub>0</sub>	a <sub>1</sub>	a <sub>2</sub>
All pavement types	Structural	B4.7	134		
	dSNPK	B4.9	0.0000758	63	40
	Cracking	B4.10	0.0066		
	Rutting	B4.11	0.088		
	Potholing	B4.15	0.00019	2	1.5

**Texture Pavement Texture Model**

$$\Delta TD = K_{td} \{ ITD - TD_a - a_0 ITD \log_{10} (10^{[(ITD - TD_a) / (a_0 ITD)]} + \Delta NELV) \}$$

where

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

The HDM-4 coefficient values for a<sub>0</sub> for the texture depth model are given in the following table. This table also includes values for the initial texture depth (ITD) which are used as defaults when resetting pavement surface type. These can be replaced by user definable values.

**Coefficient Values for Texture Depth Model**

Surface Type	Surface Material	Texture Depth	
		ITD	a <sub>0</sub>
AM	AC	0.7	0.005
AM	HRA	0.7	0.005
AM	PMA	0.7	0.005
AM	RAC	0.7	0.005
AM	CM	0.7	0.005
AM	SMA	0.7	0.005
AM	PA	1.5	0.008
ST	SBSD	2.5	0.120
ST	DBSD	2.5	0.120
ST	CAPE	0.7	0.006
ST	SI	0.7	0.006
ST	PM	1.5	0.008

### Skid Resistance Model

$$\Delta SFC_{50} = K_{sfc} \max(0, \Delta QCV) (-0.663 \times 10^{-4})$$

where

$\Delta SFC_{50}$  = incremental change in sideways force coefficient during analysis year, measured at 50 km/h

$\Delta QCV$  = annual incremental increase in the flow of commercial vehicles, in veh/lane/day

$K_{sfc}$  = calibration factor for skid resistance

## Appendix C: HDM-4 Interface

A sample of HDM-4 Setup is given below but the document presented here is not complete. It is aimed basically to provide a brief idea on the sequence of various parameters programmed in HIMS.

### Input Fields

Field	Type	Size	Field Description	Table Reference	Field Reference	Composite Index
PCRW	Number		Area of Wide Cracking before latest reseal or overlay, in percent	Network		No
PCRA	Number		Area of All Cracking before latest reseal or overlay, in percent	Network		No
HSNEW	Number		Thickness of the most recent surfacing, in mm	Network		
HSOLD	Number		Total thickness of previous underlying surfacing layers, in mm	Network		
KW						Y
KA						Y
HSE						Y
CRT			Cracking retardation time due to maintenance, in years	Network		
CMOD	Number		Resilient modulus of soil cement, in GPa	Network		
CDS	Number		Construction defects indicator for bituminous surfacing	Network		
Kcia	Number		Calibration factor for initiation of all structural cracking	Network		
SNP			Annual average adjusted structural of the pavement			Y
DEF	Number		Mean Benkelman beam deflection in both wheelpaths, in mm	Network		
YE4			Annual number of equivalent standard axles, in millions/lane			Y
ADTT			AADT – total	Traffic		
ADT1			Percent of AADT – car	Traffic		
ADT2			Percent of AADT – LCV	Traffic		
ADT3			Percent of AADT – MCV	Traffic		
ADT4			Percent of AADT – HCV1	Traffic		
ADT5			Percent of AADT – HVC2	Traffic		
ADT6			Percent of AADT – bus	Traffic		

Appendix C: HDM-4 Interface

GR1			Growth Rate – car	Traffic		
GR2			Growth Rate –LCV	Traffic		
GR3			Growth Rate –MCV	Traffic		
GR4			Growth Rate –HCV1	Traffic		
GR5			Growth Rate –HCV2	Traffic		
GR6			Growth Rate –bus	Traffic		
ESALF2			Equivalent standard axle load factor – LCV	Traffic		
ESALF3			Equivalent standard axle load factor – MCV	Traffic		

**Constants**

**Initiation of All Structural Cracking**

Code	Value	Filter
CP_S_a0	-0.01	M_ALL
CP_S_a1	10	M_ALL
CP_S_a2	0.25	M_ALL
CP_S_a3	0.02	M_ALL
CP_S_a4	0.05	M_ALL

**Progression of All Structural Cracking**

Code	Value	Filter
CP_dtA2	1	FP_dtA2
CP_zA1	-1	FP_zA1
CP_zA2	1	FP_zA2

**Lookup Expressions**

**Initiation of All Structural Cracking**

Code	Expression
KP_IAS_a0	[LP_Ini_ASC] , [HDM_INTF].[PAVE_TYPE] , [EXP].[EP_CSURF] , [HDM_INTF].[HSOLD] , a0
KP_IAS_a1	[LP_Ini_ASC] , [HDM_INTF].[PAVE_TYPE] , [EXP].[EP_CSURF] , [HDM_INTF].[HSOLD] , a1
KP_IAS_a2	[LP_Ini_ASC] , [HDM_INTF].[PAVE_TYPE] , [EXP].[EP_CSURF] , [HDM_INTF].[HSOLD] , a2
KP_IAS_a3	[LP_Ini_ASC] , [HDM_INTF].[PAVE_TYPE] , [EXP].[EP_CSURF] , [HDM_INTF].[HSOLD] , a3
KP_IAS_a4	[LP_Ini_ASC] , [HDM_INTF].[PAVE_TYPE] , [EXP].[EP_CSURF] , [HDM_INTF].[HSOLD] , a4

**Initiation of Wide Structural Cracking**

Code	Expression
KP_IWS_a5	[LP_Ini_WSC] , [HDM_INTF].[PAVE_TYPE] , [EXP].[EP_CSURF] , [HDM_INTF].[HSOLD] , a5
KP_IWS_a6	[LP_Ini_WSC] , [HDM_INTF].[PAVE_TYPE] , [EXP].[EP_CSURF] , [HDM_INTF].[HSOLD] , a6
KP_IWS_a7	[LP_Ini_WSC] , [HDM_INTF].[PAVE_TYPE] , [EXP].[EP_CSURF] , [HDM_INTF].[HSOLD] , a7

**Formulae**

**Initiation of All Structural Cracking**

Code	Expression	Filter Code
EP_CSURF	$\text{IF}([\text{HDM\_INTF}].[\text{PAVE\_TYPE}] = 0 \text{ And } [\text{HDM\_INTF}].[\text{SURF\_MATRL}] = 4, [\text{HDM\_INTF}].[\text{SURF\_MATRL}],$ $\text{IF}([\text{HDM\_INTF}].[\text{PAVE\_TYPE}] = 4 \text{ or } [\text{HDM\_INTF}].[\text{PAVE\_TYPE}] = 5) \text{ And } ([\text{HDM\_INTF}].[\text{SURF\_MATRL}] = 10 \text{ or } [\text{HDM\_INTF}].[\text{SURF\_MATRL}] = 9), [\text{HDM\_INTF}].[\text{SURF\_MATRL}], 99))$	M_ALL
EP_ICA1	$[\text{HDM\_INTF}].[\text{Kcia}] * ([\text{HDM\_INTF}].[\text{CDS}] ^ 2 * [\text{EXP}].[\text{KP\_IAS\_a0}] * \exp([\text{EXP}].[\text{KP\_IAS\_a1}] * [\text{EXP}].[\text{EP\_HSE}] + [\text{CON}].[\text{CP\_S\_a2}] * \log([\text{HDM\_INTF}].[\text{CMOD}]) + [\text{CON}].[\text{CP\_S\_a3}] * \log([\text{HDM\_INTF}].[\text{DEF}]) + [\text{CON}].[\text{CP\_S\_a4}] * [\text{VAR}].[\text{YE4}] * [\text{HDM\_INTF}].[\text{DEF}]) + [\text{HDM\_INTF}].[\text{CRT}])$	FP_ICA1
EP_ICA2	$([\text{HDM\_INTF}].[\text{Kcia}] * ([\text{HDM\_INTF}].[\text{CDS}] ^ 2 * ((0.8 * [\text{EXP}].[\text{EP\_KA}] + 0.2 * [\text{EXP}].[\text{EP\_KW}]) * (1 + 0.1 * [\text{EXP}].[\text{EP\_HSE}]) + (1 - [\text{EXP}].[\text{EP\_KA}]) * (1 - [\text{EXP}].[\text{EP\_KW}]) * [\text{CON}].[\text{CP\_S\_a0}] * \exp([\text{CON}].[\text{CP\_S\_a1}] * [\text{EXP}].[\text{EP\_HSE}] + [\text{CON}].[\text{CP\_S\_a2}] * \log([\text{HDM\_INTF}].[\text{CMOD}]) + [\text{CON}].[\text{CP\_S\_a3}] * \log([\text{HDM\_INTF}].[\text{DEF}]) + [\text{CON}].[\text{CP\_S\_a4}] * [\text{VAR}].[\text{YE4}] * [\text{HDM\_INTF}].[\text{DEF}]) + [\text{HDM\_INTF}].[\text{CRT}])$	FP_ICA2
EP_ICA3	$[\text{HDM\_INTF}].[\text{Kcia}] * ([\text{HDM\_INTF}].[\text{CDS}] ^ 2 * [\text{EXP}].[\text{KP\_IAS\_a0}] * \exp([\text{EXP}].[\text{KP\_IAS\_a1}] * [\text{VAR}].[\text{SNPcrk}] + [\text{EXP}].[\text{KP\_IAS\_a2}] * ([\text{VAR}].[\text{YE4}] / [\text{VAR}].[\text{SNPcrk}] ^ 2)) + [\text{HDM\_INTF}].[\text{CRT}])$	FP_ICA3
EP_ICA4	$[\text{HDM\_INTF}].[\text{Kcia}] * ([\text{HDM\_INTF}].[\text{CDS}] ^ 2 * (\max([\text{EXP}].[\text{KP\_IAS\_a0}] * \exp([\text{EXP}].[\text{KP\_IAS\_a1}] * [\text{VAR}].[\text{SNPcrk}] + [\text{EXP}].[\text{KP\_IAS\_a2}] * ([\text{VAR}].[\text{YE4}] / [\text{VAR}].[\text{SNPcrk}] ^ 2)), \max(1 - [\text{HDM\_INTF}].[\text{PCRW}] / [\text{EXP}].[\text{KP\_IAS\_a3}], 0), [\text{EXP}].[\text{KP\_IAS\_a4}] * [\text{HDM\_INTF}].[\text{HSNEW}])) + [\text{HDM\_INTF}].[\text{CRT}])$	FP_ICA4
EP_ICA5	$[\text{HDM\_INTF}].[\text{Kcia}] * ([\text{HDM\_INTF}].[\text{CDS}] ^ 2 * (\max([\text{EXP}].[\text{KP\_IAS\_a0}] * \exp([\text{EXP}].[\text{KP\_IAS\_a1}] * [\text{VAR}].[\text{SNPcrk}] + [\text{EXP}].[\text{KP\_IAS\_a2}] * ([\text{VAR}].[\text{YE4}] / [\text{VAR}].[\text{SNPcrk}] ^ 2)), \max(1 - [\text{HDM\_INTF}].[\text{PCRA}] / [\text{EXP}].[\text{KP\_IAS\_a3}], 0), [\text{EXP}].[\text{KP\_IAS\_a4}])) + [\text{HDM\_INTF}].[\text{CRT}])$	FP_ICA5

**Initiation of Wide Structural Cracking**

Code	Expression	Filter Code
EP_ICW	$[\text{HDM\_INTF}].[\text{Kciw}] * \max([\text{EXP}].[\text{KP\_IWS\_a5}] + [\text{EXP}].[\text{KP\_IWS\_a6}] * [\text{VAR}].[\text{ICA}], [\text{EXP}].[\text{KP\_IWS\_a7}] * [\text{VAR}].[\text{ICA}])$	FP_ICW

**Filters**

**Initiation of All Structural Cracking**

Code	Expression	Formula
FP_ICA1	[HDM_INTF].[BASE_TYPE] > 0 And [HDM_INTF].[HSOLD] = 0	EP_ICA1
FP_ICA2	[HDM_INTF].[BASE_TYPE] > 0 And [HDM_INTF].[HSOLD] > 0	EP_ICA2
FP_ICA3	[HDM_INTF].[BASE_TYPE] = 0 And [HDM_INTF].[HSOLD] = 0	EP_ICA3
FP_ICA4	([HDM_INTF].[BASE_TYPE] = 0 And [HDM_INTF].[HSOLD] > 0) And ([HDM_INTF].[SURF_MATRL] <> 4 Or [HDM_INTF].[SURF_MATRL] <> 10 Or [HDM_INTF].[SURF_MATRL] <> 9)	EP_ICA4
FP_ICA5	([HDM_INTF].[BASE_TYPE] = 0 And [HDM_INTF].[HSOLD] > 0) And ([HDM_INTF].[SURF_MATRL] = 4 Or [HDM_INTF].[SURF_MATRL] = 10 Or [HDM_INTF].[SURF_MATRL] = 9)	EP_ICA5

**Initiation of Wide Structural Cracking**

Code	Expression	Formula
FP_IWC	1=1	EP_ICW

**Variables**

**Initiation of All Structural Cracking**

Field	Type	Source	Initiation	Calculation	Differentiation	Filter
ICA		Formula		EP_ICA1		FP_ICA1
		Formula		EP_ICA2		FP_ICA2
		Formula		EP_ICA3		FP_ICA3
		Formula		EP_ICA4		FP_ICA4
		Formula		EP_ICA5		FP_ICA5

**Initiation of Wide Structural Cracking**

Field	Type	Source	Initiation	Calculation	Differentiation	Filter
ICW		Formula		EP_ICW		FP_ICW

## Appendix D: Drainage Factor (DF) Values

Range of drainage factor values

Drain Type	Drain Condition	
	Excellent DF <sub>min</sub>	Very poor DF <sub>max</sub>
Fully lined and linked	1	3
Surface lined	1	3
V-shaped – hard	1	4
V-shaped – soft	1.5	5
Shallow – hard	2	5
Shallow – soft	2	5
No drain - but required	3	5
No drain - not required	1	1





# **Comparison of Predictive Pavement Management Models (HDM-III, HDM-4, NZ dTIMS) for New Zealand Conditions**

**Transfund New Zealand Research Report No. 227**

## 7. HDM-4 Model Calibration Issues

### 7.1 Introduction

To apply pavement deterioration models properly, they need to be customised to local conditions. Although the HDM models were developed from a good dataset using sound principles, as evidenced by their successful application in over 100 countries, model calibration was found to be essential to ensure that the results are reasonable for local and regional conditions.

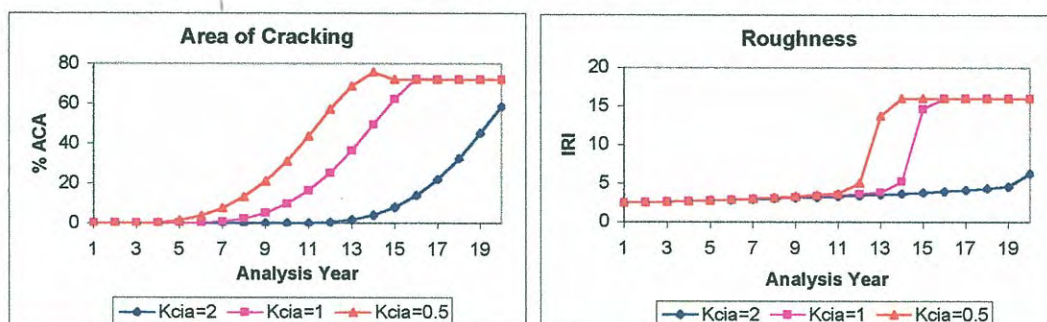
This chapter starts with a brief background on HDM model calibration. Thereafter, a detailed discussion is presented on HDM model calibration factors, additional calibration factors incorporated in HDM-4, and the data required for their estimation. Also included is a note on the past experience on calibration input data and available calibration methods.

### 7.2 An Overview of HDM Model Calibration

#### 7.2.1 Need for HDM Model Calibration

Calibration is required for HDM-4 models to incorporate the local construction techniques and environmental conditions. For this, certain coefficients are purposely embedded in the HDM models that are known as calibration factors (deterioration factors). Figure 7.1 illustrates the change in area of all cracking and roughness with change in crack initiation ( $K_{cia}$ ) and crack progression ( $K_{cpa}$ ) coefficients. The values of these factors are changed for different environmental and physical characteristics.

**Figure 7.1 Impact of values of crack initiation calibration factor ( $K_{cia}$ ) in pavement deterioration, for area of cracking (ACA) and roughness progression (IRI).**



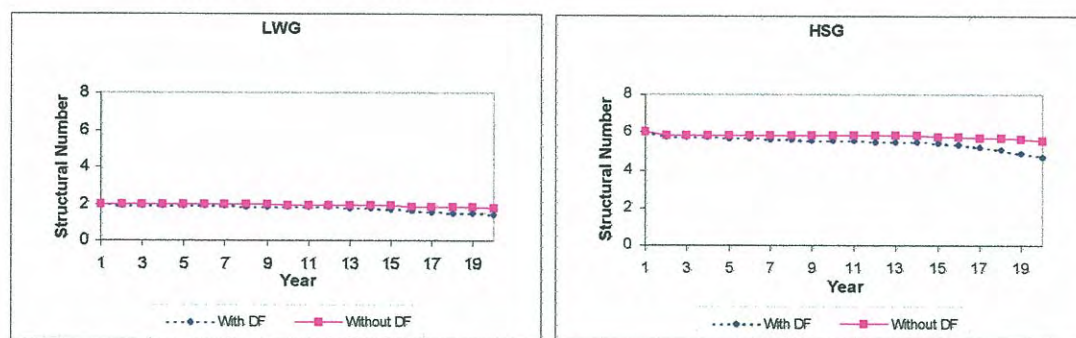
#### 7.2.2 Calibration Levels

Bennett & Paterson (2000) indicated that the HDM model calibration can be divided into the following three levels based on level of effort and time required:



Figure 9.1 shows the variation in pavement strength (structural number) with and without drainage factor scenarios.

**Figure 9.1 Influence of drainage factor (DF) on pavement strength (Structural Number) over years 1 to 19.**



The pavement strength was reducing with the deterioration in drainage condition, and the reduction was as high as 20% for high traffic on the strong pavement (HSG) scenario. Pavement strength reduced even without any changes in drainage factor due to the cracking and potholing developed during the course of the time. The effect of drainage factor is cumulative, in that cracking and potholing causes reduction of SNP if drainage condition is not maintained. The effect of drainage factor on pavement strength increased with the increase in traffic volume.

Drainage factor had very little effect on cracking initiation period (through pavement strength). However, area of cracking was not influenced by drainage factor, because a time-based model for crack progression is used when no change in the year of crack initiation was observed.

There was some impact on rut depth and roughness values due to the drainage factor, because HDM-4 models use an adjusted structural number to predict mean rut depth and roughness.

The effect of the drainage factor on different modelling parameters for two common (LWG and HSG) scenarios in terms of percent variation is given in Table 9.1.

**Table 9.1 Effect of drainage factor (DF, % variation).**

Distress parameter	LWG	HSG
Initiation of all structural cracking	5	5
Mean rut depth	10	4
Roughness	0.5	3

#### 9.4 Structural Number

The desk study showed that adjusted structural number (SNP) used in HDM-4 takes into account the drainage condition and surface distress. SNP is included in the modelling of different distress parameters, i.e. cracking, rutting and roughness, etc. (Table 9.2). On the other hand the SNC that is used in the NZ dTIMS Setup does not



account for drainage condition. Besides, in HDM-III and in the NZ dTIMS Setup, SNC is adjusted based on surface condition only for the roughness modelling.

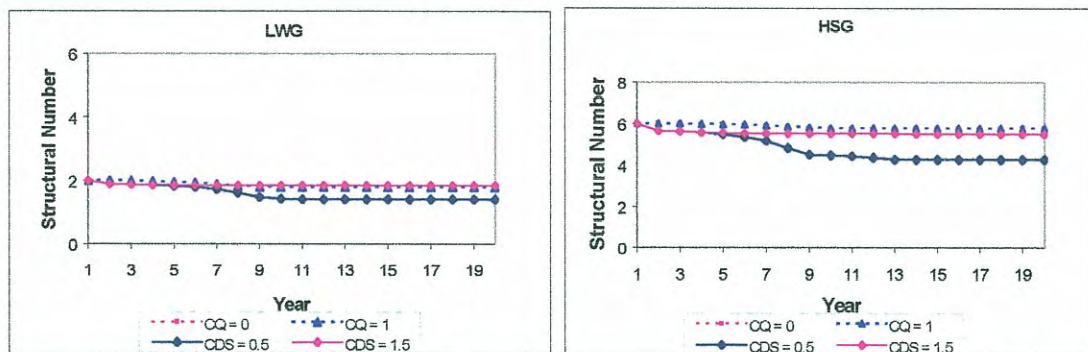
**Table 9.2 Use of annual adjusted structural number in the 3 models.**

Model	HDM-III *	NZ dTIMS *	HDM-4
Cracking			•
Ravelling			
Potholing			
Edgebreak			•
Rutting			•
Roughness	•	•	•

\* Modified structural number adjusted for potholing and cracking is used.

CQ (construction quality) was found to have no impact on pavement strength in the NZ dTIMS Setup, while CDS (construction defects of surfacing) had significant influence on pavement strength in HDM-4. Figure 9.2 shows the influence of construction quality on pavement strength (structural number) in the HDM-4 and NZ dTIMS models. Lower CDS values resulted in a decrease in SNP value, but this could be related to the rapid cracking progression and pothole progression.

**Figure 9.2 Impact of construction quality indicator on structural number for 2 types of pavement (LWG - low traffic, weak, good; HSG - high traffic, strong, good).**



## 9.5 Cracking

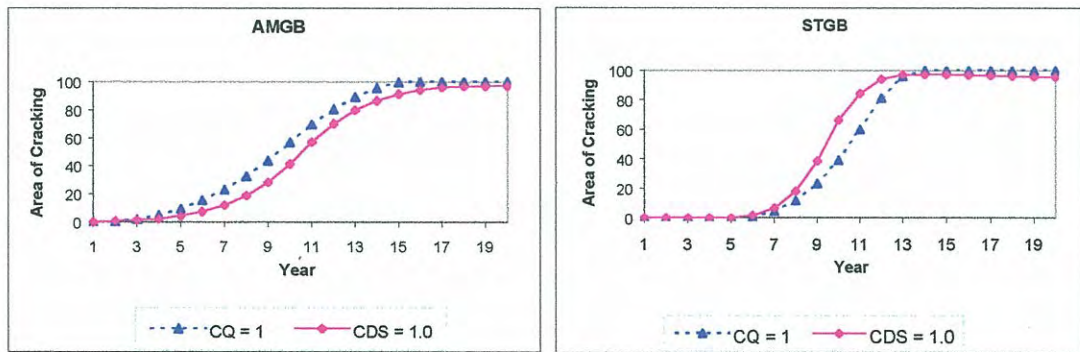
### 9.5.1 Cracking Initiation

Comparison was made between the outputs of crack initiation models used in the NZ dTIMS Setup and HDM-4 system. As stated earlier, the main difference between the NZ dTIMS and HDM-4 crack initiation models is the use of a continuous construction quality indicator for surfacing (CDS) in HDM-4 instead of a flag construction quality indicator (CQ).

The models predicted similar cracking initiation period when default values ('0' for CQ in the NZ dTIMS and '1' for CDS in HDM-4) were used. Changing the flag value (CQ) from '0' to '1' in the NZ dTIMS Setup had little impact on cracking

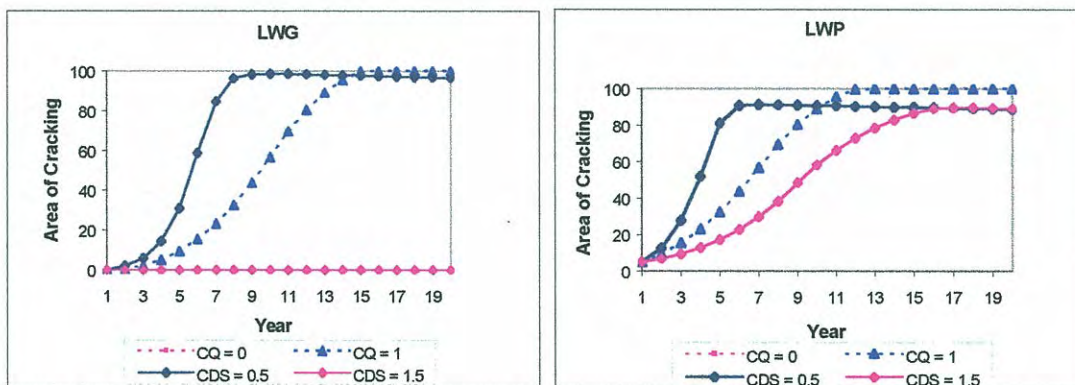


**Figure 9.3** Variation in prediction of area of cracking (%) in 2 types of low volume, weak pavements (AMGB, STGB).



CQ used in the NZ dTIMS Setup did not influence cracking progression. In HDM-4, the rate of progression of cracking decreased as the value of CDS increased from 0.5 to 1.5. As a result, lower values for CDS predicted higher areas of cracking and vice-versa. The variation in the area of cracking was as high as 100% for different values of CDS. Figure 9.4 shows predicted area of cracking when CQ and CDS are different.

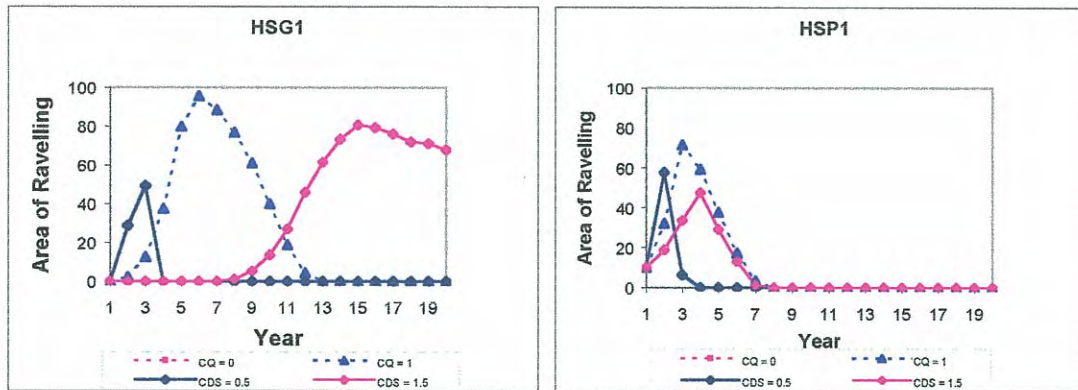
**Figure 9.4** Area of cracking when CQ and CDS is changed for 2 pavement types (LWG, LWP).



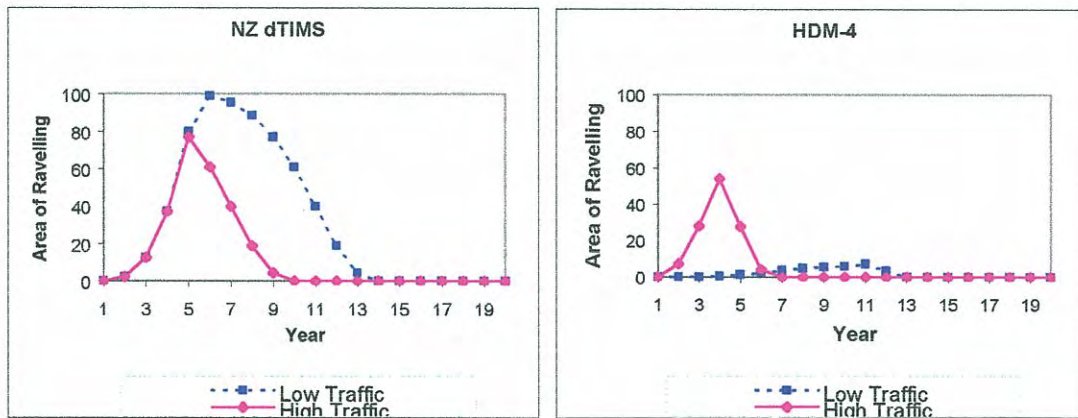
In the case of LWP, the maximum area of cracking (ACA) is always less than 100% in HDM-4, because the limiting condition applied in HDM-4 models of the total damaged area is 100%. Calculated ACA is adjusted based on other surface distresses predicted (potholes and edge break).

Figure 9.4 shows that, with the introduction of CDS, the rate of cracking progression differs a lot. Hence, a detail research study is needed to calibrate the CDS values for different construction quality levels in New Zealand conditions.

**Figure 9.5 Influence of CQ and CDS on ravelling progression for 2 surface treatment pavements (HSG1 good, HSP1 poor).**



**Figure 9.6 Influence of traffic on ravelling progression for NZ dTIMS and HDM-4 models.**



In Figure 9.6 the NZ dTIMS Setup curve also showed some variation in ravelling progression beyond analysis-year 5. Detailed investigation into the output showed that the difference in areas of ravelling for low traffic and high traffic volume scenarios is related to the limiting condition for total area of surface distresses, which is 100%. For example, cracking was found to be progressing faster in the section with high traffic. Otherwise CQ and traffic value do not have any impact on the ravelling progression in the NZ dTIMS Setup.

CDS in HDM-4 had greater influence on the rate of progression of ravelling. The prediction of ravelling progression is enhanced by using the traffic parameter. For example, very little progression in ravelling is observed for the low traffic road. Hence, with these enhancements, the HDM-4 ravelling model can be considered to be applicable for New Zealand conditions, but detailed research work is recommended before implementing the model.



## 9.7 Potholes

### 9.7.1 Pothole Initiation

In the NZ dTIMS Setup, pothole initiation model is a function of traffic flow and thickness of the asphaltic layers. In the case of HDM-4, the CDB indicator for basecourse and rainfall are included in the expression.

The pothole initiation period predicted by HDM-4 model is higher than that predicted by the NZ dTIMS Setup. The variation increased with the increase in the traffic volume. CQ did not have any influence on the initiation of potholes in the NZ dTIMS Setup as expected. On the other hand, significant variation was observed in the initiation of potholing in the case of HDM-4 for different values of CDB (Table 9.5). No impact of the pavement strength on the pothole initiation was observed.

**Table 9.5 Variation in pothole initiation in HDM-4 model for 4 pavement types (LWG, HWG, LSG, HSG).**

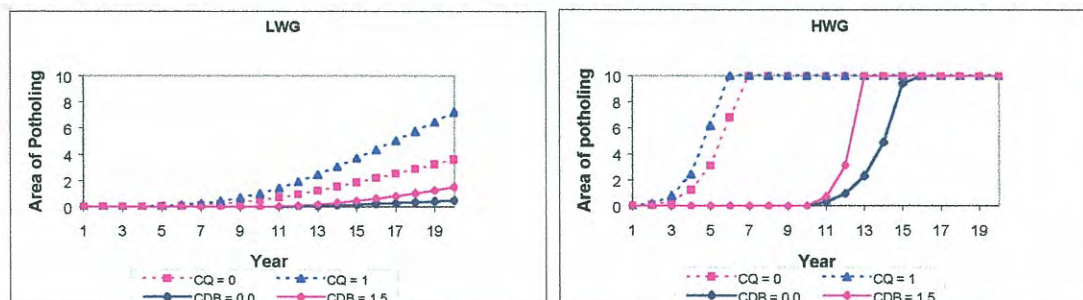
Scenario	Pothole initiation period in years for CDB		% variation in pothole initiation
	0	1.5	
LWG	5.5	2.2	60.0
HWG	2.4	0.9	62.5
LSG	5.5	2.2	60.0
HSG	2.4	0.9	62.5

### 9.7.2 Pothole Progression

The experience gained from using HDM-III models under different physical and environmental conditions showed that the pothole progression modelled in HDM-III over-estimated the area of potholing in some cases. This is particularly true where potholes are patched as soon as they appear, or when signs of potholes appear on the surface, as occurs in New Zealand. A time lapse factor (Odoki 1997, Riley 1997) was introduced in the HDM-4 pothole progression model to account for the response time to patching the potholes.

The rate of pothole progression was higher in the NZ dTIMS Setup than in HDM-4 model for all scenarios. Figure 9.7 shows the variation in areas of potholes as modelled by the NZ dTIMS Setup and HDM-4 system.

**Figure 9.7 Variation in area of potholes for 2 pavement types (LWG, HWG).**

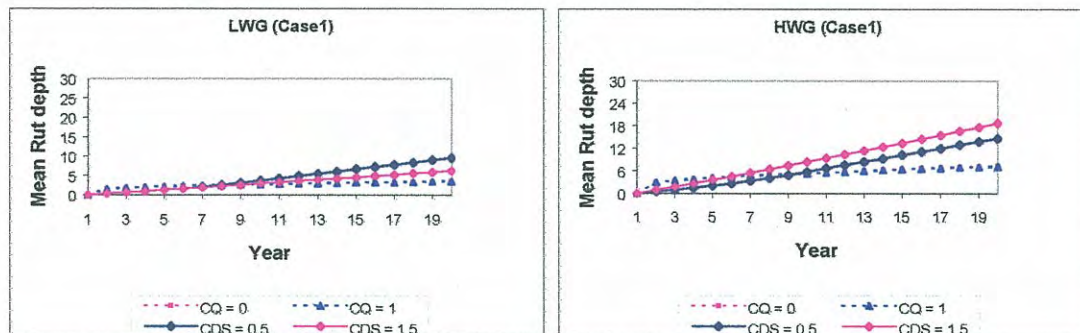




not applicable to New Zealand conditions as studded tyres are generally not used. In the New Zealand case, structural deformation and plastic deformation contributed to the further progression of rutting (from the second year onwards).

CQ had almost no effect on rutting progression except for high traffic volumes in the case of NZ dTIMS. In HDM-4, the CDS has considerable effect on the rutting progression. Figure 9.8 presents the variation in rutting for different CDS values for NZ dTIMS and HDM-4 models.

**Figure 9.8 Variation in rutting in NZ dTIMS and HDM-4 Models for 2 pavement types (LWG, HWG).**



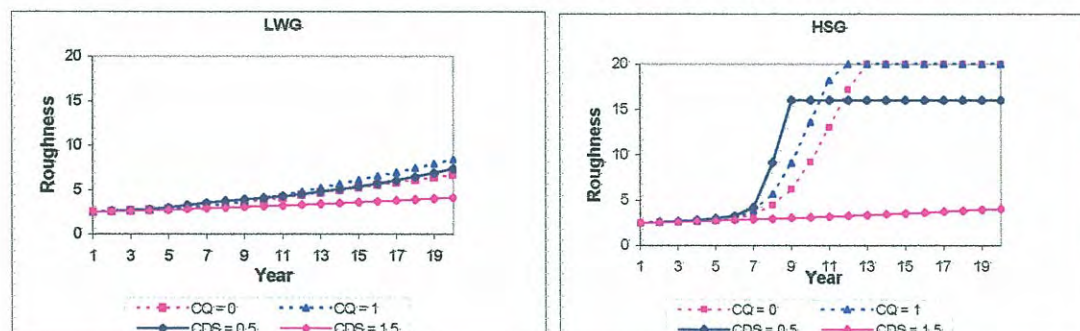
### 9.9 Roughness

The roughness progression model in HDM-4 had the same five components as the HDM-III and NZ dTIMS roughness models, as explained in Chapter 4. The potholing component of roughness in HDM-4 was modified extensively taking into account the effects of time to patch the potholes and freedom to manoeuvre to avoid potholes.

The NZ dTIMS Setup generally predicted higher roughness values than the HDM-4 model. The lower values of roughness in the HDM-4 model could be attributed to the modifications made to the potholing component of roughness, and to other models in the HDM-4 system.

The limiting value for roughness is kept at ‘20’ in the NZ dTIMS Setup and ‘16’ in HDM-4 system. Figure 9.9 shows the influence of construction quality indicator on roughness progression.

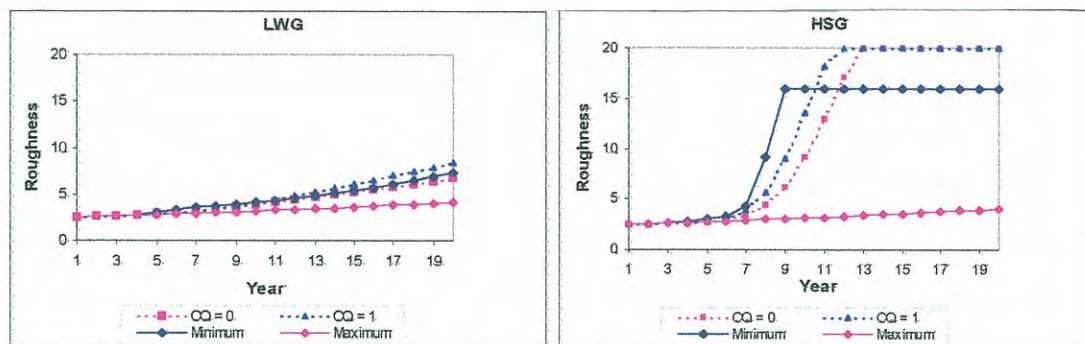
**Figure 9.9 Variation in roughness for 2 pavement types influenced by CQ and CDS.**





Different rates of roughness progression were achieved in HDM-4 by changing the values of CDS and CDB at the same time. The limiting values for CDS and CDB in the HDM-4 system generated different roughness progressions. Figure 9.10 shows the impact of CDS, CDB and drainage factor on roughness progression.

**Figure 9.10 Impact of CDS, CDB and DF on roughness for 2 pavement types (LWG, HSG).**



CDS = 0.5 and CDB = 0 for minimum case. Effect of drainage is considered in this case.  
 CDS = 1.5 and CDB = 1.5 for maximum case. Drainage factor was kept constant at 2.5 in this case.

From Figure 9.10, the CQ indicator clearly influenced roughness progression in both NZ dTIMS and HDM-4 models. However, the variation is small in the NZ dTIMS Setup, while HDM-4 can predict a range of roughness progression applicable to different conditions. With the introduction of CDS, CDB, DF and other enhancements, HDM-4 roughness model has become more flexible and can be customised to a variety of conditions.

## 9.10 Comparison of WE Models

### 9.10.1 Introduction

Maintenance works are applied to road sections to keep the standard of the network at the desired level.

Modifications and enhancements carried out to the HDM-4 WE models for resetting values are explained earlier in Chapter 5 of this report. In the life cycle analysis, various parameters are reset after the treatment, if that considerably affects the further performance of the pavement. Hence, a parametric study of the WE models was carried out.

### 9.10.2 Treatments Selected for Comparison

Although several new models have been included in HDM-4, only maintenance types relevant to New Zealand conditions were selected for comparison at this stage. Table 9.8 explains the types of treatments selected for comparison and the effects of these maintenance operations.



Similarly 80% area of existing wide cracking is repaired in both systems. Drainage condition and edge break reset can only be done with HDM-4 models and they are taken as 75% maintenance for drainage maintenance, and 90% maintenance for edge break.

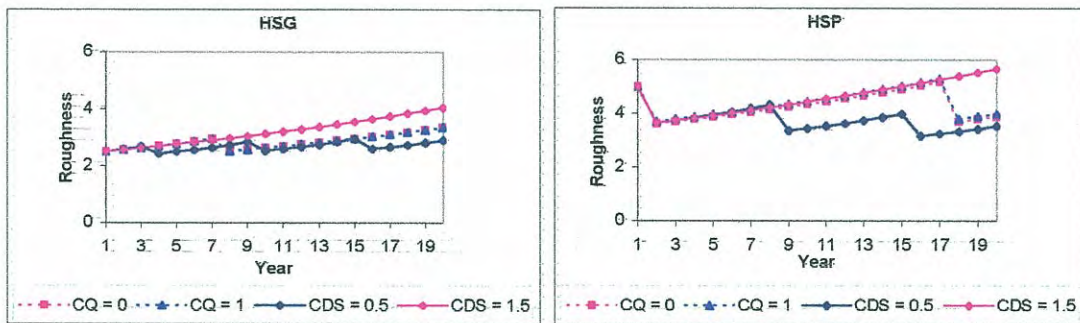
A detailed comparison was made for roughness progression using the synthetic dataset for AC and ST pavements. The HDM-4 roughness progression was slightly lower than that predicted by the NZ dTIMS Setup, and this could be attributed to the introduction of a time-lapse factor and freedom to manoeuvre, and other modifications made to HDM-4 models.

### 9.10.4 Resurfacing and Overlay

Empirical relationships were provided for roughness reset after maintenance for ST and AC pavements in both the NZ dTIMS Setup and HDM-4 system. HDM-4 roughness progression was lower than the NZ dTIMS Setup using its default construction quality indicator. Different roughness progression curves were predicted by the HDM-4 system for different values of CDS using the same trigger limit and reset equations. This is attributed to the value of CDS that takes into effect of the construction defects, if any.

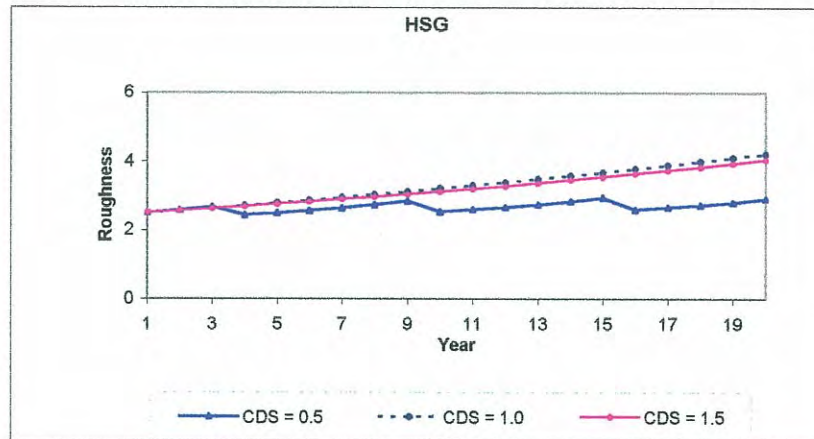
Figure 9.11 presents a typical case which illustrates the different roughness curves predicted by the HDM-4 system for different CDS values. The NZ dTIMS Setup predictions are also included in the figure, in order to show the difference in prediction by the two modelling systems (i.e. NZ dTIMS Setup and HDM-4 system).

**Figure 9.11** Variation in roughness with resurfacing treatment, predicted by the 2 models for 2 pavement types (HSG, HSP).



The number of applications of overlays (resurfacings) also varied with CDS value. A lower number of overlays means less agency cost to be incurred for the same trigger limits. Hence, CDS values have to be chosen carefully so that they represent the field conditions, otherwise the modelling of pavement deterioration and strategies (treatment at a specified time) predicted might not represent real conditions. Then the output produced may over-estimate or under-estimate, not only pavement performance, but the funding requirements as well. Figure 9.12 shows a typical example illustrating different strategies for different CDS values.



**Figure 9.12** Different strategies for applying CDS values to an HSG pavement.

The variation in construction quality value in the NZ dTIMS Setup did not have any impact on strategies predicted. However in HDM-4, three overlays were required to maintain the road at similar standards for a value of '0.5' for CDS, while no overlays were required when CDS equalled to '1' and '1.5'. The test road section synthetic data was maintained within the allowable maintenance standards in all three scenarios.

### 9.10.5 Reconstruction

Two different reconstruction treatments were selected as follows for comparison purposes:

- 150mm granular material and 8mm surface treatment for ST pavements.
- 150mm granular material and 50mm asphalt concrete for AC pavements.

The trigger limit was kept the same for both the NZ dTIMS Setup and HDM-4 system. Roughness (IRI) value after rehabilitation was taken as 2m/km for AC and 2.5m/km for ST pavements. These were the default values in the NZ dTIMS Setup and, hence, were adopted for HDM-4.

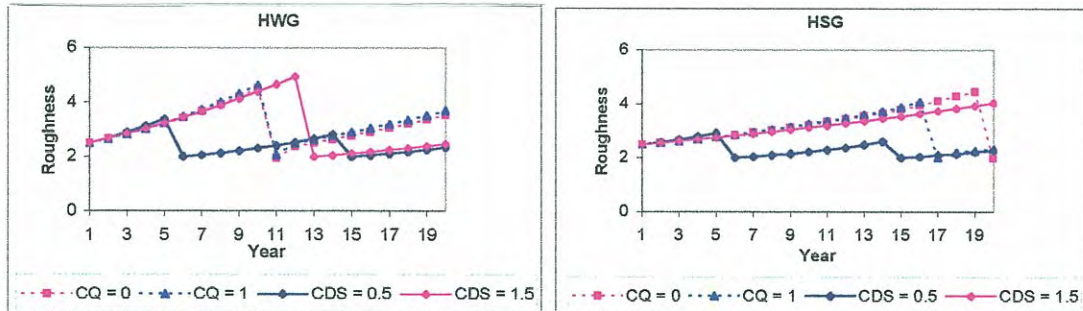
Rut depth and roughness were selected for comparison. The observations made for rehabilitation treatment are as follows:

- Rut depth progression was higher in the case of the HDM-4 system due to the inclusion of plastic deformation of rutting component.
- Roughness progression was lower in the HDM-4 system compared to the NZ dTIMS Setup.
- Different values for CDS produced different strategies for the same set of triggers and reset equations.

Figure 9.13 shows the roughness progression for HDM-4 and the NZ dTIMS Setup for different construction quality values.



**Figure 9.13** Variation of roughness with rehabilitation treatment predicted by the 2 models for 2 pavement types (HWG, HSG).

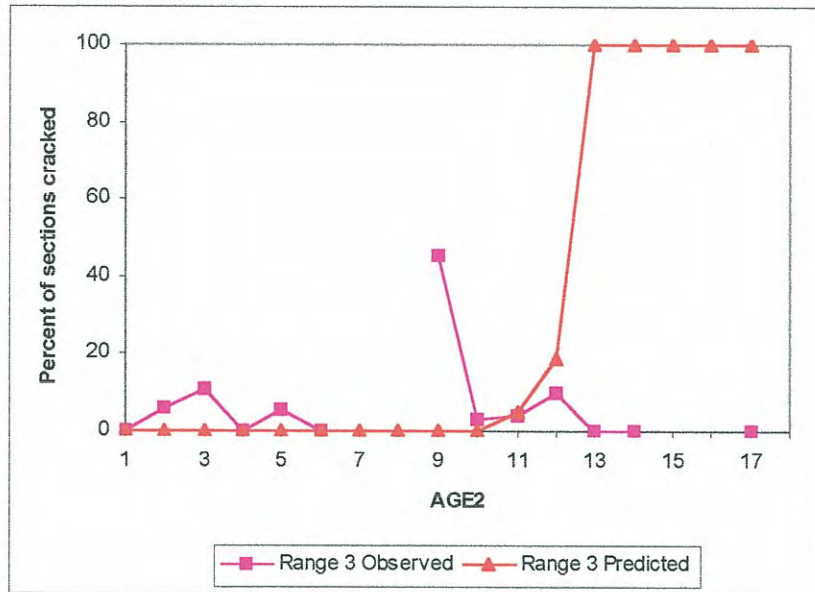


### 9.11 Conclusions

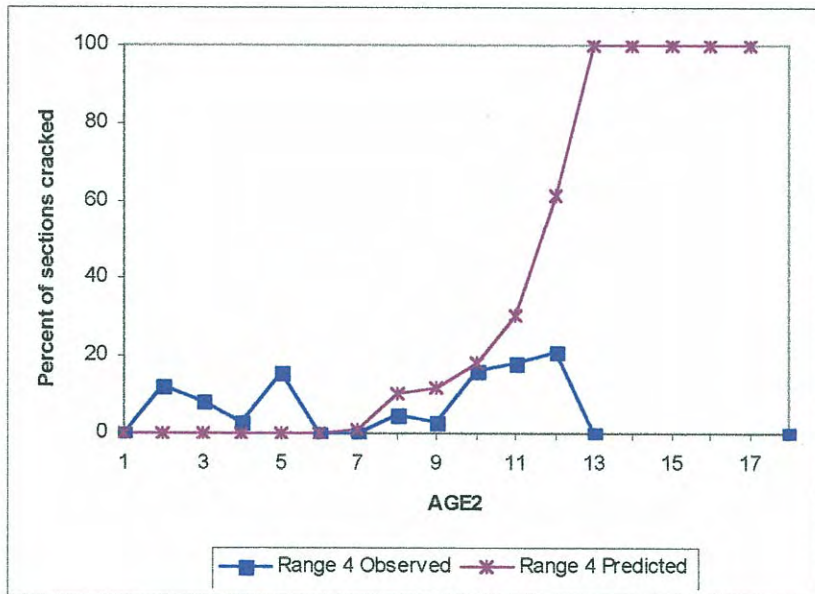
- The drainage factor was introduced in the HDM-4 system to consider the effects of drainage condition. The pavement strength reduced with the deterioration in drainage condition. The reduction was found as high as 20% for a given scenario. Also the effect of drainage factor is cumulative on reduction of SNP, if drainage condition is not maintained.
- CQ did not have any impact on pavement strength in the NZ dTIMS Setup, but CDS influenced pavement strength significantly in HDM-4.
- The new continuous construction quality indicator (introduced in the HDM-4 system) significantly influenced the initiation and progression of distresses, and also in the strategies, while the effect of CQ in the NZ dTIMS Setup was negligible.
- The progression of the distresses in the HDM-4 system was lower compared to that predicted in the NZ dTIMS Setup with its particular default values for construction quality indicator.
- HDM-4 ravelling model has become more flexible and applicable with the introduction of a traffic factor and construction quality. Further study is required to determine the suitability of this model to New Zealand conditions.
- With the introduction of the base construction quality indicator (CDB) in the pothole initiation model, HDM-4 pothole initiation model is flexible to be customised to different conditions. The pothole progression was lower in the HDM-4 system.
- The rutting progression was higher in the HDM-4 system due to inclusion of additional rutting components (plastic deformation and surface wear) in the HDM-4 rutting model.
- Roughness progression was lower in the HDM-4 system. Pothole component of roughness in HDM-4 has been extensively modified by introducing the time lapse factor and freedom to manoeuvre parameter.
- Maintenance works have been modified in HDM-4. Some new user-specified routine maintenance treatments were included in the HDM-4 system, and provide a platform for comprehensive modelling of routine maintenance treatments.



**Figure 10.3 % cracked sections, observed and predicted, versus AGE2 for low traffic range of 500 – 2000 veh/day.**



**Figure 10.4 % cracked sections, observed and predicted, versus AGE2 for high traffic range of 2000 – 4000 veh/day.**



The above figures suggested that the crack initiation period decreased as the traffic volume increased, in both the predicted as well as observed scenarios. The predicted values with Kcia of ‘1’ coincided reasonably with the observed values between pavements of ages 5 and 13 for the two traffic ranges. The cracked sections at early ages were further investigated, and found that the area of cracking that occurs then was negligible and therefore was not considered.

Figure 10.5 shows the comparison between the predicted and observed crack initiation periods for the considered dataset over 18 years.

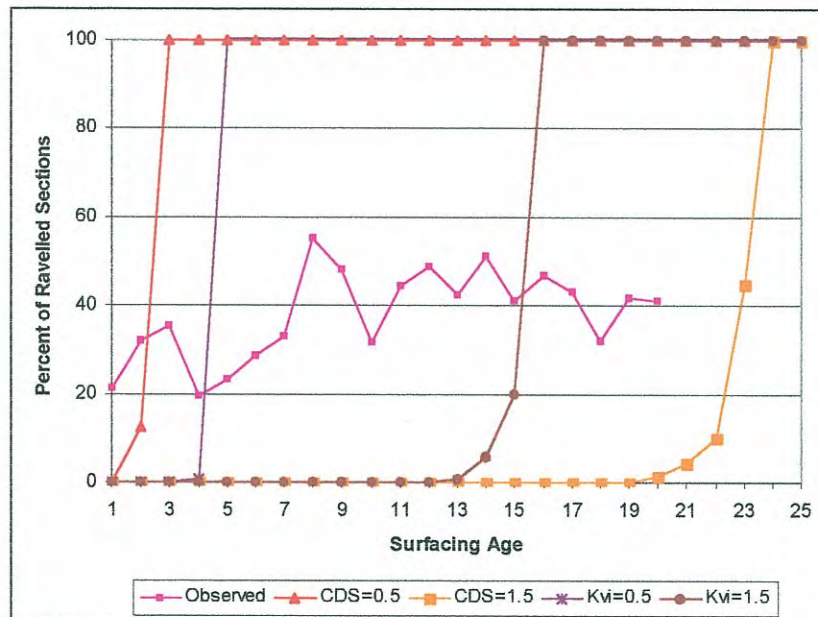
The predictions were found to be reasonable using the default value of '1' for crack progression factor ( $K_{cpa}$ ). Considering the few sections that showed the trend in cracking, the small amounts of area of cracking that could not be used for calibration purposes, and the reasonableness of the predictions with the default value, the crack progression factor can be kept at '1' at this stage.

### 10.3.5 Ravelling Calibration Coefficients

Ravelling calibration coefficients showed relatively very little sensitivity to all the parameters except ravelling itself. Besides, the ravelling progression factor is taken as '0' in the NZ dTIMS Setup because HDM-III ravelling models were found to be not suitable for New Zealand conditions as explained earlier (Section 9.6). Hence, the calibration study on ravelling was carried out to find if the HDM-4 ravelling models better reflect the New Zealand condition.

The percent of ravelled sections were plotted against surfacing age. No trend was found in either snapshot and time-series methods. Figure 10.7 presents the analysis for the time-series method. Analysis showed that 30 – 50% of the road sections were ravelled irrespective of the age (in years).

**Figure 10.7** Observed and predicted ravelling using the time-series method over 25 years.



The predicted ravelling initiation period may vary a good deal, if it is based on the value of the construction defect indicator (CDS) and calibration coefficients in which CDS is dominating (Figure 10.7).

The field visit of the sites had shown that most of the ravelling (scabbing) area recorded in RAMM was found outside the wheelpath area. This could have occurred because of improper compaction of the surface layers. After application of the reseal, the surface layer is not fully compacted, and is opened for traffic to do the last compaction. Hence, compaction was found to have taken place only in the wheelpath