Contents

Volume 4 - Analytical Framework & Model Descriptions: Part C

Part C Road Map

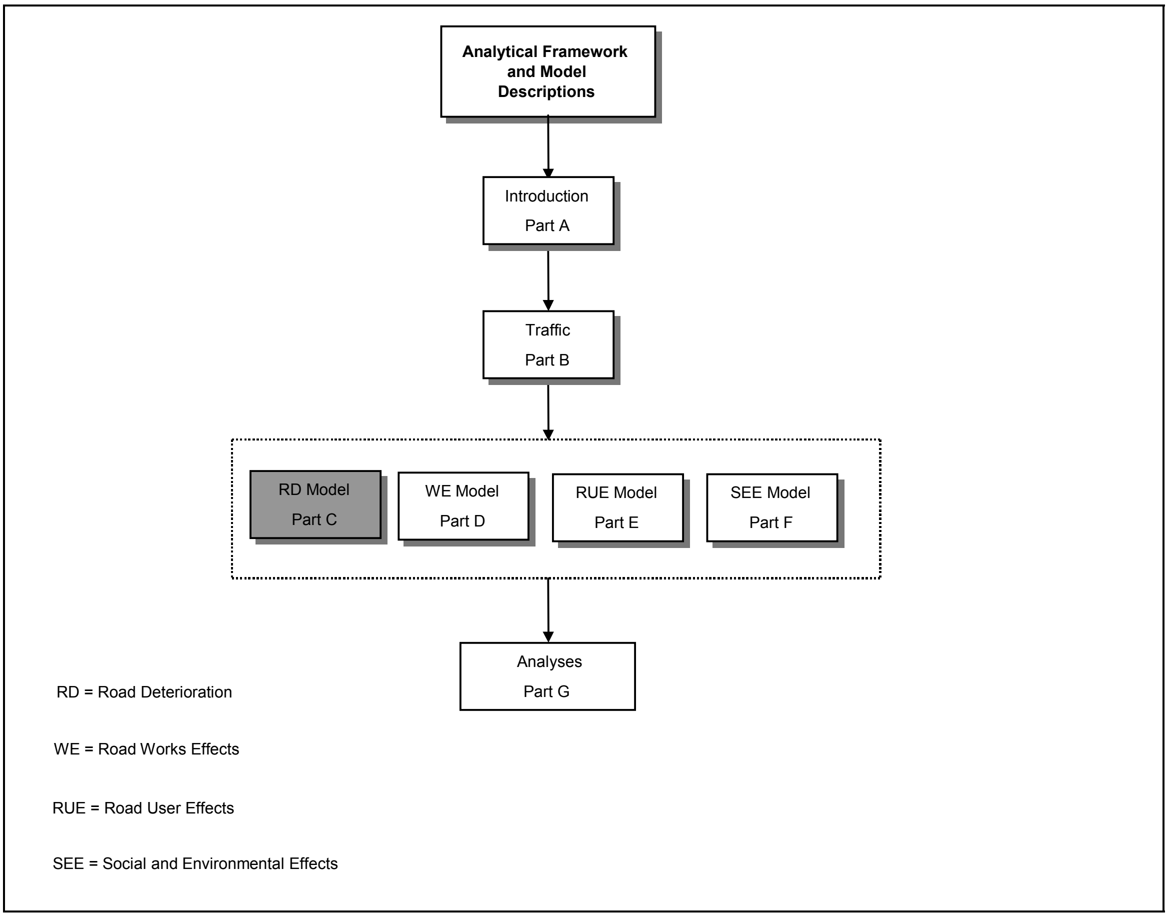

Figure C Analytical Framework and Model Descriptions Road Map

C1 Modelling concepts and approach

1 Introduction

Road deterioration is broadly a function of the original design, material types, construction quality, traffic volume, axle load characteristics, road geometry, environmental conditions, age of pavement, and the maintenance policy pursued.

HDM-4 includes relationships for modelling Road Deterioration (RD) and Road Works Effects (WE). These are used for the purpose of predicting annual road condition and for evaluating road works strategies. The relationships should link standards and costs for road construction and maintenance to road user costs through road user cost models. In HDM-III these relationships were combined into a single module called the Road Deterioration and Maintenance Effects (RDME), as described by *Watanatada et al. (1987)*. In HDM-4, this module has been separated in order to address properly the expanded scope of modelling Road Deterioration and Works Effects. The analysis now includes:

- **Physical environments (climatic zones)**

  Encompassing cold (freeze/thaw) climates, very high temperatures and a very wide range of temperature variations such as desert conditions, and very high moisture regimes and arid conditions.

- **Rigid/concrete and semi-rigid pavements, and a wide range of flexible pavements.**

- **Models for the following distresses:**

  Edge-break, texture depth and skid resistance.

- **Road shoulders and the impact on non-motorised transport, and side-drains and the effects on pavement strength.**

- **Road capacity improvement and a broader range of maintenance techniques for different pavement types.**

This chapter deals with the pavement classification system used, and describes the RD modelling approach for the different road surface classes considered in HDM-4 (see Figure C1.1. The key variables that affect road deterioration (in particular those that are associated with climate and environment) are also discussed. A comprehensive classification of climate in terms of temperature and moisture is also given. The modelling of Road Works Effects is described in Part D.

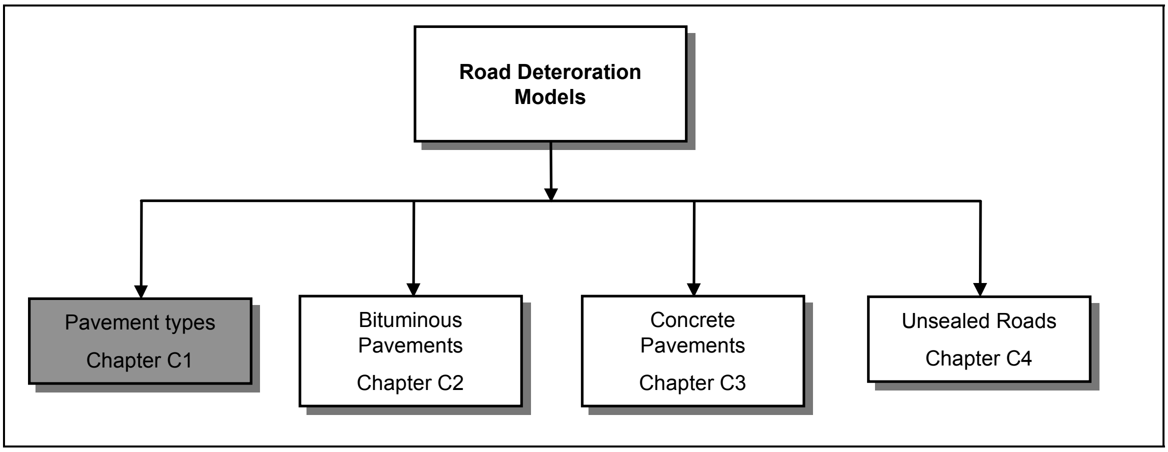

Figure C1.1 Road Deterioration Models

2 Pavement classification

A versatile framework of pavement classification is required to cater for the expanded scope of RD and WE analysis. A system of classifying pavements has therefore been formulated which uses broad definitions of road surfacing and roadbase types as illustrated in Table C1.1 (*NDLI, 1995*).

Table C1.1 Pavement classification system in HDM-4

Surface category	Surface class	Pavement type	Surface type	Surface material	Base type	Base material
Paved	Bituminous	AMGB	AM	AC, HRA,	GB	NG, CRS, WBM, etc.
		AMAB		RAC, PA,	AB	AB, EB, etc.
		AMSB		CM, etc.	SB	CS, LS, etc.
		AMAP			AP	TNA, FDA, etc.
		AMRB			RB	JUC, RBC, CUC, etc.
		STGB	ST	SBSD, PM,	GB	NG, CRS, WBM, etc.
		STAB		DBSD, SL,	AB	AB, EB, etc.
		STSB		CAPE, etc.	SB	CS, LS, etc.
		STAP			AP	TNA, FDA, etc.
		STRB			RB	JUC, RBC, CUC, etc.
	Concrete	JPGB	JP	VC, RC,	GB	NG, CRS, WBM, etc.
		JPAB		FC, PC,	AB	AB, EB, etc.
		JPSB		etc.	SB	CS, LS, etc.
		JPAP			AP	TNA, FDA, etc.
		JPRB			RB	JUC, RBC, CUC, etc.
		JRGB	JR	VC,	GB	NG, CRS, WBM, etc.
		JRAB		FC, etc.	AB	AB, EB, etc.
		JRSB			SB	CS, LS, etc.
		JRAP			AP	TNA, FDA, etc.
		JRRB			RB	JUC, RBC, CUC, etc.
		CRGB	CR	VC,	GB	NG, CRS, WBM, etc.
		CRAB		FC, etc.	AB	AB, EB, etc.
		CRSB			SB	CS, LS, etc.
		CRAP			AP	TNA, FDA, etc.
		CRRB			RB	JUC, RBC, CUC, etc.
	Block	CBSG	CB	CB	SG	SA, NG, etc.
		BRLC	BR	BR	LC	LC
		SSGB	SS	SS	CG	LC, NG, etc.
Unpaved	Unsealed	GRUP	GR	LT, QZ, etc.		
		EAUP	EA	EA	UP	
		SAUP	SA	SA		

Key:

Surface type		Surface material	
AM	Asphaltic Mix	AC	Asphalt Concrete
ST	Surface Treatment	HRA	Hot Rolled Asphalt
JP	Jointed Plain	RAC	Rubberised Asphalt Concrete
JR	Jointed Reinforced	PA	Porous Asphalt
CR	Continuously Reinforced	CM	Cold Mix (Soft Bitumen Mix)
CB*	Concrete Block	SBSD	Single Bituminous Surface Dressing
BR*	Brick	PM	Penetration Macadam
SS*	Set Stone	DBSD	Double Bituminous Surface Dressing
GR	Gravel	SL	Slurry Seal
EA*	Earth	CAPE	Cape Seal
SA*	Sand	VC	Vibrated Concrete
		RC	Rolled Concrete
		FC	Fibre Concrete
		PC	Porous Concrete
		LT	Lateritic Gravel
		QZ	Quartzitic Gravel

Note:

Asterik (*) indicates that different types of material or construction pattern may be defined.

Base type		Base material	
GB	Granular Base	NG	Natural Gravel
AB	Asphalt Base	CRS	Crushed Stone
SB	Stabilised Base	WBM	Water Bound Macadam
AP	Asphalt Pavement	EB	Emulsified Base
RB	Rigid (Concrete) Base	CS	Cement Stabilised
SG	Sand/Gravel	LS	Lime Stabilised
LC	Lean Concrete	TNA	Thin Asphalt Surfacing
CG	Concrete/Gravel	FDA	Full Depth Asphalt
UP	Unpaved – base types not applicable	JUC	Jointed Unbonded Concrete
		RBC	Reinforced Bonded Concrete
		CUC	Continuously Unbonded Concrete

The definitions are as follows:

- **Surface category**

  Divides all pavements into two groups:

  - paved
  - unpaved

  These are mainly used for the reporting of network statistics.

- **Surface class**

  Subdivides the paved category into bituminous, concrete and block surfaces; together with the unsealed class there are thus four classes that are used to define the family of distress models used for performance modelling.

- **Pavement type**

  Integrates surface and roadbase types. Each type is designated by a four-character code, combining the surface and roadbase type codes.

- **Surface type**

  Divides bituminous surfacings into two types:

  - asphaltic mix (AM)
  - surface treatment (ST)

  Divides concrete surfacings into three types:

  - jointed plain (JP)
  - jointed reinforced (JR)
  - continuously reinforced (CR)

  Divides three types of block:

  - concrete (CB)
  - brick (BR)
  - set stone (SS)

  Divides three types of unsealed surfacings:

  - gravel (GR)
  - earth (EA)
  - sand (SA)

  A surface type is designated by a two-character code.

- **Base type**

  There are eight generic types, including those which allow for overlays of asphalt on concrete and vice versa. Each base type is designated by a two-character code.

- **Surface material**

  Defines more specific surface types (for example, different types of asphalt mixes). These are user definable.

- **Base material**

  Allows the user to specify more detailed characteristics of roadbase types.

Note that during an analysis period, the road surface class and pavement type might change depending on the types of works applied to the pavement (see Part D). For example, the initial pavement type for a section may be AMGB (asphaltic mix surface on granular base); if an asphaltic overlay is applied the pavement type will change to AMAP (asphaltic mix surface on asphalt pavement) and different model parameters will apply. If the same initial pavement is given a surface treatment it will change to STAP (surface treatment on asphalt pavement).

3 The modelling approach

3.1 Classes and types of models

The two general classes of models used for Road Deterioration (RD) and Works Effects (WE) analyses are mechanistic and empirical (*NDLI, 1995*). Mechanistic models use sound fundamental theories of pavement behaviour for their development; but they are usually very data intensive and rely on parameters which are difficult to quantify in the field. Empirical models are usually based on statistical analyses of locally observed deterioration trends, and may not be applicable outside the specific conditions upon which they are based.

To minimise these problems *Paterson (1987)* adopted a **structured empirical** approach for developing the HDM-III RDME model. This was based on identifying the functional form and primary variables from external sources and then used various statistical techniques to quantify their impacts. This had the advantage that the resulting models combined both the theoretical and experimental bases of mechanistic models with the behaviour observed in empirical studies. The RD and WE relationships included, in HDM-4, are therefore mainly structured empirical models.

There are two types of models that can be used for predictive purposes:

- **Absolute models**
- **Incremental models**

Absolute models predict the condition (or distress) at a particular point in time as a function of the independent variables. Incremental models give the change in condition from an initial state as a function of the independent variables.

The families of pavement performance models used are based on the road surface classes:

Bituminous	incremental models (described in Chapter C2)
Concrete	absolute models (described in Chapter C3)
Unsealed	incremental models (described in Chapter C4)
Block	incremental models (not included in this software release)

3.2 Pavement distresses

Pavement deterioration manifests itself in various kinds of distresses, each of which should be modelled separately. Table C1.2 gives a summary of the pavement defects that are modelled in HDM-4. As each mode of distress develops and progresses at different rates in different environments, it is important that the RD relationships should be calibrated to local conditions before using them for road investment analyses. To facilitate this, the relationships include a number of user-definable deterioration factors to change the scale of a particular distress. The model coefficients should be used to adjust the rates of deterioration for different types of pavement material.

In order to model road deterioration properly it is required that homogeneous road sections in terms of physical attributes and condition should be identified so that a particular set of RD relationships can be applied. The basic unit of analysis is therefore the **homogeneous** road section, to which several investment options can be assigned for analysis.

Table C1.2 Pavement defects modelled in HDM-4

Bituminous	Concrete	Block*	Unsealed
Cracking	Cracking	Rutting	Gravel loss
Ravelling	Joint spalling	Surface texture	Roughness
Potholing	Faulting	Roughness	
Edge break	Failures		
Rutting	Serviceability loss		
Surface texture	Roughness		
Skid resistance			
Roughness			

* Not implemented in this HDM-4 release

3.3 Side-drains deterioration

The condition of the drains will deteriorate unless they are maintained adequately through, for example, routine maintenance. The deterioration of side-drains has the effect of reducing pavement strength and accelerating its deterioration. Drain life is expressed as a function of terrain, drain type, climate type and the maintenance policy pursued. A number of drain types are considered in RD modelling (see Chapters C2 and C3).

3.4 Shoulders deterioration

The modelling of road shoulder deterioration is required in order to assess the effect on the rate of pavement deterioration; and the impact on non-motorised transport and traffic flow in terms of Road User Costs. It is proposed to include this feature in a future release of HDM-4.

4 Key variables affecting deterioration

The key variables that are used in the various deterioration models are associated with the following:

- **Climate and environment**
- **Traffic**
- **Pavement history**
- **Road geometry**
- **Pavement structural characteristics**
- **Material properties**

4.1 Climate and environment

The climate in which a road is situated has a significant impact on the rate at which the road deteriorates. Important climatic factors are related to temperature, precipitation and winter conditions. This section describes the principal climatic data that is used to model the deterioration of the different categories of roads considered in HDM-4.

4.1.1 Classification

It is necessary for the user to define climatic and environment information as per Table C1.3 and Table C1.4:

Table C1.3 Moisture classification

Moisture classification	Description	Thornthwaite moisture index	Annual precipitation (mm)
Arid	Very low rainfall, high evaporation	-100 to -61	< 300
Semi-arid	Low rainfall	-60 to -21	300 to 800
Sub-humid	Moderate rainfall, or strongly seasonal rainfall	-20 to +19	800 to 1600
Humid	Moderate warm seasonal rainfall	+20 to +100	1500 to 3000
Per-humid	High rainfall, or very many wet-surface days	> 100	> 2400

Table C1.4 Temperature classification

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

4.1.2 Precipitation

The Mean Monthly Precipitation (MMP) is used in the modelling of bituminous pavement deterioration and unsealed road deterioration, and is expressed in mm/month. The annual average precipitation (PRECIP) is used in the modelling of concrete pavement deterioration and is expressed in inches/year.

4.1.3 Thornthwaite moisture index

The **Thornthwaite moisture index (MI)** is defined as follows (*LAST, 1996*):

$$MI = I_h - 0.6 * I_a = \frac{100 * SWAT - 60 * DWAT}{NWAT} \qquad ...(4.1)$$

where:

MI	Thornthwaite moisture index
I_h	humidity index
I_a	aridity index
SWAT	excess of water (mm)
DWAT	water deficiency (mm)
NWAT	necessary water (mm)

It is important to know if a given place is continually wet or dry, or if it is wet in a given season and dry in another. The Moisture Index is capable of indicating how wet or dry is a given climate zone, but it is not capable of distinguishing climates with or without seasonal dampness variations.

Wet climates will have a positive Moisture Index; on the other hand dry climates will have a negative index. The Thornthwaite Moisture Index indicates the free humidity in a particular area.

4.1.4 Freezing index

The **freezing index (FI)** is defined as the difference between the mean ambient temperature and 0ºC (degrees per day). The freezing index is **negative** when the ambient temperature is below 0ºC and **positive** otherwise.

The freezing index is calculated as:

$$FI = \sum_{i=1}^{ndays} ABS[MIN(TEMP,0)] \qquad ...(4.2)$$

where:

FI	freezing index
TEMP	temperature (^{o}C)
Ndays	number of days in one freezing season

Note that FI is only required as input data for the two temperate temperature zones, and are used in modelling the performance of concrete pavements.

4.1.5 Temperature range

Temperature range (TRANGE) is defined as the mean monthly ambient temperature range. Its calculation is based on the temperature ranges for each of the 12 months of the year, hence the difference between the maximum and minimum temperature for each month. The 12 values obtained are then averaged to obtain the variable TRANGE, which is used for modelling concrete pavements.

4.1.6 Days with temperatures greater than 90ºF

The number of days, in a year, in which the ambient temperature exceeds 90ºF (32ºC) is denoted as DAYS90. This variable is required for modelling the performance of concrete pavements.

4.2 Traffic

The primary traffic-related variables that affect road deterioration include the number and types of vehicles using the road, and axle loading characteristics of the different vehicle types. Details of the variables required are described in Part B, and the way in which they are used in the various deterioration relationships is defined in the appropriate Sections of this document.

4.3 Pavement history

The required variables refer to the age of pavement. These variables are related to the previous maintenance, rehabilitation and construction works carried out on the pavement, and have been discussed in the appropriate sections of this document.

4.4 Other road related variables

The other key variables affecting pavement performance are related to road geometry, pavement structural characteristics and material properties. These are also described in the appropriate Sections.

5 References

LAST (1996)

Modelling road design and maintenance effects for pavements in HDM-4
Final Report, FICEM, Latin American Study Team, International Study of Highway Development and Management Tools, Santiago, Chile

NDLI (1995)

Modelling Road Deterioration and Maintenance Effects in HDM-4
Final Project Report, Asian Development Bank, RETA 5549
N. D. Lea International Limited, Vancouver, Canada

Paterson W. D. O., (1987)

Road Deterioration and Maintenance Effects: Models for Planning and Management
The Highway Design and Maintenance Standards Series
World Bank, Johns Hopkins Press, Baltimore, USA

Watanatada T., Harral C.G., Paterson W.D.O., Dhareshwar A.M., Bhandari A., and Tsunokawa K., (1987)

The Highway Design and Maintenance Standards Model - Volume 1 Description
World Bank, Johns Hopkins Press, Baltimore, USA

C2 Bituminous Pavements

1 Introduction

This chapter describes the detailed modelling of **bituminous pavement** deterioration in HDM-4 (see Figure C2.1).

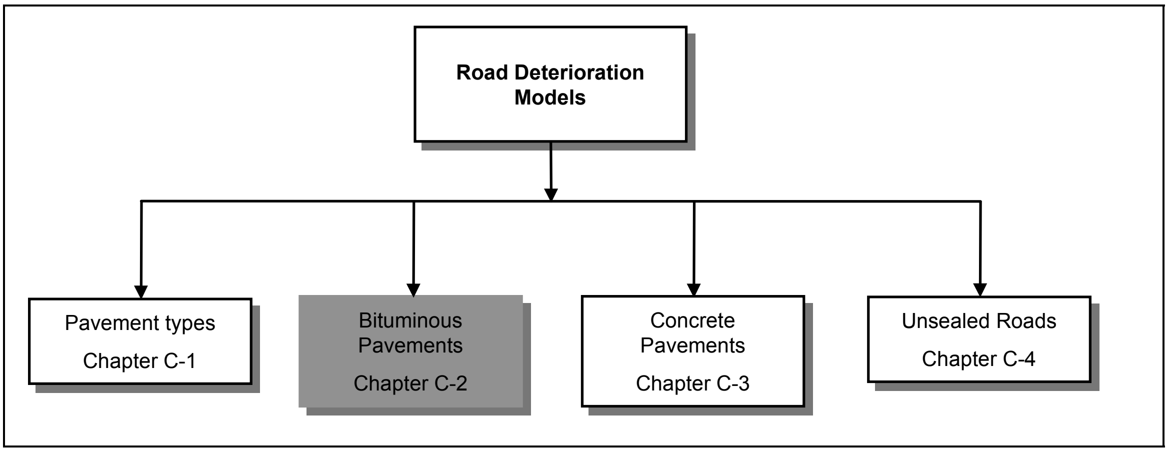

Figure C2.1 Road Deterioration Modules

The development of the road deterioration models was based on three previous documents:

1 *Riley and Bennett (1995 & 1996)* - based on *Paterson (1987)*

2 *Watanatada et al. (1987)*

3 *NDLI (1995)*

A series of formal workshops followed. These were held at the University of Birmingham (UoB), UK, in April 1996, December 1996 and October 1997. Informal meetings were then held in Washington, the University of Birmingham and the Transport Research Laboratory (TRL), UK that supplemented these workshops.

As a result of these discussions, major changes were regularly proposed to the road deterioration models which were presented in various versions of the fourth and fifth draft specifications (*Morosiuk, 1996 & 1998a*). Following on from the beta testing of the HDM-4 software in November 1998, a sixth then a seventh draft of specifications were produced. A further workshop was held at TRL in June 1999 to resolve outstanding issues. The decisions reached at this workshop have been incorporated in the eighth draft version of the specifications. The main contributors to the specifications were *Paterson (IBRD)*, *Morosiuk (TRL)*, *Riley (Riley Partnership)*, *Odoki and Kerali (UoB)*.

For HDM-4 Version 2 a further review of the road deterioration models was performed by Joubert (LCPC), Morosiuk (TRL), Riley (Riley Partnership), and Toole (ARRB). A number of improvements to the models was agreed and are included in this documentation.

Section 2 provides an overview of the Road Deterioration modelling framework. This is followed by the relationships and default coefficient values for each of the distresses to be modelled. The model coefficient values are stored in data files instead of having them hard coded into the model. This facilitates local calibration and adaptation. The HDM-4 model has more calibration factors than the previous HDM-III model.

A list of research documents referenced from this chapter is given in Section 14.

2 Model framework and logic

2.1 Classification and concepts

The road deterioration framework developed for HDM-4 is much more flexible than in HDM-III and is able to handle a wider range of pavement types. This has been accomplished by providing a single set of generic models whose coefficient values are altered based on the surface and base type. The pavement classification system that forms the basis for defining the model framework is shown in Table C2.1.

The formal structure of the framework is comprised of the non-shaded cells in Table C2.1. The pavement type is defined by the combination of the surface type and base type. This is given in the right hand column of Table C2.1.

Within a given pavement type, there are various combinations of surface and base materials. As the performance of the pavement can be anticipated to be a function of these materials, the user is able to associate model coefficients with each combination of surface and base materials. The same basic models for the pavement type is used with the different coefficient values.

Table C2.1 HDM-4 Bituminous pavements classification system

Surface type	Surface material	Base type	Base material	Pavement type
AM	AC	GB	CRS	AMGB
	HRA		GM	
	PMA	AB	AB	AMAB
	RAC	SB	CS	AMSB
	CM		LS	
	PA	AP	TNA	AMAP
	SMA		FDA	
	Xx			
ST	CAPE	GB	CRS	STGB
	DBSD		GM	
	SBSD	AB	AB	STAB
	SL	SB	CS	STSB
	PM		LS	
	Xx	AP	TNA	STAP
			FDA	

Note: The modelling of AM and ST surfacings on concrete pavements, that is, AMRB and STRB, is not included in this software release.

The abbreviations in Table C2.1 are described in Table C2.2.

Table C2.2 Descriptions of surface and base materials

Surface type		Surface materials	
Abbreviation	**Description**	**Abbreviation**	**Description**
AM	Asphalt Mix	AC	Asphaltic Concrete
		CM	Soft Bitumen Mix (Cold Mix)
		HRA	Hot Rolled Asphalt
		PA	Porous Asphalt
		PMA	Polymer Modified Asphalt
		RAC	Rubberised Asphalt Concrete
		SMA	Stone Mastic
ST	Surface Treatment	CAPE	Cape Seal
		DBSD	Double Bituminous Surface Dressing
		PM	Penetration Macadam
		SBSD	Single Bituminous Surface Dressing
		SL	Slurry Seal
Base types		**Base materials**	
Abbreviation	**Description**	**Abbreviation**	**Description**
AB	Asphalt Base	CRS	Crushed Stone
AP	Asphalt Pavement	NG	Natural Gravel
GB	Granular Base	CS	Cement Stabilisation
SB	Stabilised Base	LS	Lime Stabilisation
		TNA	Thin Asphalt Surfacing
		FDA	Full Depth Asphalt

The available models are constructed from different factors. Many are created from surface and base types, whilst others are created from surface materials. Accordingly, the modelling is done in terms of surface material and base type, even though base materials can be specified. HDM-4 contains default coefficient values for the bituminous pavement types given in Table C2.3.

Table C2.3 Generic HDM-4 bituminous pavement types

Pavement type	Surface type	Base type	Description of pavement types
AMGB	AM	GB	Asphalt Mix on Granular Base
AMAB	AM	AB	Asphalt Mix on Asphalt (Dense Bitumen Macadam) Base
AMSB	AM	SB	Asphalt Mix on Stabilised Base
AMAP	AM	AP	Asphalt Mix on Asphalt Pavement
STGB	ST	GB	Surface Treatment on Granular Base
STAB	ST	AB	Surface Treatment on Asphalt (Dense Bitumen Macadam) Base
STSB	ST	SB	Surface Treatment on Stabilised Base
STAP	ST	AP	Surface Treatment on Asphalt Pavement

Currently, there are no coefficient values to differentiate between the performances of different base materials, so all materials of a given base type are assigned the same coefficient values. Each combination of surface and base material results in a set of coefficient values associated with the pavement.

NDLI (1995) gives definitions of the characteristics used to define different types of pavements into the above framework and alternative terminology applied to the same pavement materials.

The resets for pavement type after maintenance works are discussed in detail in the Road Works Effects (see Part D). These resets are summarised in Table C2.4.

Table C2.4 Pavement type resets after maintenance works

Works activity	Existing pavement type							
	AMGB	AMSB	AMAB	AMAP	STGB	STSB	STAB	STAP
Routine works	AMGB	AMSB	AMAB	AMAP	STGB	STSB	STAB	STAP
Preventive treatment	AMGB	AMSB	AMAB	AMAP	STGB	STSB	STAB	STAP
Reseal	STAP	STAP / STSB	STAP	STAP	STGB	STSB	STAB	STAP
Overlay	AMAP	AMAP / AMSB	AMAP	AMAP	AMGB	AMSB	AMAB	AMAP
Inlay	AMGB	AMSB	AMAB	AMAP	STGB	STSB	STAB	STAP
Mill & replace to intermediate surface layer	**AP	**AP	**AP	**AP	N/A	**SB	**AB	**AP
Mill & replace to base	**GB	**SB	**AB	**AP	**GB	**SB	**AB	**AP

Source: *NDLI (1995)*

Notes:

** Indicates that this, two character variable, is dependent on the specific works activity, that is, operation

N/A not applicable

2.2 Computational logic

2.2.1 Pavement distress modes

Road deterioration is predicted through eight separate distress modes, namely:

- **Cracking** (see Section 5)
- **Ravelling** (see Section 6)
- **Potholing** (see Section 7)
- **Edge-break** (see Section 0)
- **Rutting** (see Section 10)
- **Roughness** (see Section 11)
- **Texture depth** (see Section 12.1)
- **Skid resistance** (see Section 12.2)

These are defined in Table C2.5, and can be considered under the following three categories:

- **Surfacing distress**

This category comprises:

- ❑ Cracking
- ❑ Ravelling
- ❑ Potholing
- ❑ Edge-break

The first three distress modes are characterised by two phases referred to as initiation and progression. The initiation phase is the period before surfacing distress of a given mode or severity develops. The progression phase refers to the period during which the area and severity of distress increases. Edge-break is modelled only through its continuous progression.

- **Deformation distress**

This category comprises:

- ❑ Rutting
- ❑ Roughness

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

- **Surface texture**

This category comprises:

- ❑ Texture depth
- ❑ Skid resistance

Surface texture distress modes are continuous, and like deformation distress modes they are modelled only through their progression.

2.2.2 Primary modelling parameters

The primary variables used from one analysis year to the next may be grouped as described below. The road characteristics at the beginning of the analysis year are initialised either from input data if it is the first year of the analysis or the first year after construction, or otherwise from the result of the previous year's maintenance and improvement works.

Pavement structural characteristics

These include measures of pavement strength, layer thickness, material types, construction quality, and subgrade stiffness.

The RD models require as input data the thickness of **new** and **old** bituminous surfacing layers. An original pavement that has not been resurfaced or overlaid since it was constructed/reconstructed has a new surfacing and no old surfacing. For a pavement that has been resurfaced or overlaid, the following relationship applies:

$$HSOLD_2 = HSNEW_1 + HSOLD_1 - MLLD \qquad \text{...(2.1)}$$

$$HSNEW_2 = \text{user specified value} \qquad \text{...(2.2)}$$

where:

$HSOLD_2$	thickness of old surfacing after works (mm)
$HSNEW_1$	thickness of the most recent surfacing (mm)
$HSOLD_1$	total thickness of previous underlying surfacing layers (mm)
MLLD	mill depth (mm)
$HSNEW_2$	thickness of new surfacing after works (mm). This is the user-specified thickness when an intervention is to be made

Table C2.5 Definitions of distress measures

Measure	Definition
Area (of distress)	Sum of rectangular areas circumscribing manifest distress (line cracks are assigned a width of 0.5 m), expressed as a percentage of the carriageway area
All cracking	Narrow and wide structural cracking inclusive
Narrow cracking	Interconnected or line cracks of 1-3 mm crack width (equivalent to AASHTO Class 2)
Wide cracking	Interconnected or line cracks of 3 mm crack width or greater, with spalling (equivalent to AASHTO Class 4)
Indexed cracking	Normalised sum of AASHTO Classes 2 to 4 cracking weighted by class, see Section 5.3
Transverse thermal cracking	Unconnected crack running across the pavement
Ravelling	Loss of material from wearing surface
Pothole	Open cavity in the road surface with at least 150 mm diameter and at least 25 mm depth
Edge-break	Loss of bituminous surface material (and possibly base materials) from the edge of the pavement
Rutting	Permanent or unrecoverable traffic-associated deformation within pavement layers which, if channelled into wheelpaths, accumulates over time and becomes manifested as a rut
Rut depth	Maximum depth under 2 m straightedge placed transversely across a wheelpath
Roughness	Deviations of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads and drainage (ASTM E-867-82A) – typically in the ranges of 0.1 to 100 m wavelength and 1 to 100 mm amplitude
IRI	International Roughness Index, the reference measure expressing roughness as a dimensionless average rectified slope statistic of the longitudinal profile and defined in *Sayers et al. (1986)*
Mean Texture depth	The average depth of the surface of a road surfacing expressed as the quotient of a given volume of standardised material [sand (sand patch test), glass spheres] and the area of that material spread in a circular patch on the surface being tested, (*PIARC, 1997*)
Skid resistance	Resistance to skidding expressed by the sideways force coefficient (SFC) measured using the Sideways Force Coefficient Routine Investigation Machine (SCRIM)

Source: *Watanatada et al. (1987)*

Road condition

Pavement and side-drain condition data at the beginning of the first analysis year or the first year after construction are required. The data items for surfacing and deformation distress modes and surface texture, are as described in Table C2.5.

The pavement conditions at the end of the year (that is, before road works) are predicted as follows:

$$[CONDITION]_b = [CONDITION]_a + \Delta[CONDITION] \quad ...(2.3)$$

$$[CONDITION]_{av} = 0.5 * \{[CONDITION]_a + [CONDITION]_b\} \quad ...(2.4)$$

where:

$[CONDITION]_b$	condition at the end of the year
$[CONDITION]_a$	condition at the start of the year
$\Delta[CONDITION]$	change in condition during the year
$[CONDITION]_{av}$	average condition for the year

Pavement history

The required data items refer to pavement ages, and these are related to the previous maintenance, rehabilitation and construction works carried out on the pavement.

There are four variables defining the age of the pavement used in the models; AGE1, AGE2 AGE3 and AGE4. These variables are defined as follows:

1 **AGE1** is referred to as the preventive treatment age. It is defined as the time, in number of years, since the latest preventive treatment, reseal, overlay (or rehabilitation), pavement reconstruction or new construction activity.

2 **AGE2** is referred to as the surfacing age. It is defined as the time, in number of years, since the latest reseal, overlay, pavement reconstruction or new construction activity.

3 **AGE3** is referred to as the rehabilitation age. It is defined as the time, in number of years, since the latest overlay, pavement reconstruction or new construction activity.

4 **AGE4** is referred to as the base construction age. It is defined as the time, in number of years, since the latest reconstruction that involves the construction of a new base layer or new construction activity.

Road geometry and environment

These include carriageway and shoulder widths, vertical alignment and the mean monthly precipitation.

Traffic

The required traffic data are the flow of all vehicle axles (YAX), the flow of equivalent standard axle loads (YE4), both expressed on an annual basis in millions per lane, the flow of heavy commercial vehicles per lane per day (QCV), and the number of equivalent light vehicles passes per year (NELV). These data are calculated for each analysis year based on the user-specified traffic and vehicle characteristics. The annual average traffic speed and the average speed of heavy vehicles are also required in some RD relationships.

2.2.3 Computational procedure

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

1 Initialise input data and the conditions at the beginning of the year

2 Compute pavement strength parameters

3 Calculate the amount of change in each surfacing distress mode during the analysis year in the following order:

(a) **Cracking**

(b) **Ravelling**

(c) **Potholing**

(d) **Edge-break**

4 Check that the total damaged and undamaged carriageway surface area equals 100% based on the limits defined for each distress mode, and determine the amount of each surfacing distress mode at the end of the year and the average value for the year

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

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

7 Store results for use in subsequent modules (that is, RUE, WE, SEE) and in the following analysis year, and for reporting

3 Pavement strength

3.1 Adjusted structural number

Pavement strength is characterised by the adjusted structural number, SNP, (*Parkman and Rolt, 1997*). This has been derived from the modified structural number, which was adopted as the pavement strength descriptor for HDM-III. The adjusted structural number applies a weighting factor, which reduces with increasing depth, to the sub-base and subgrade contributions so that the pavement strength for deep pavements is not over-predicted (a concern with the use of the modified structural number). It is calculated as:

$$SNP_s = SNBASU_s + SNSUBA_s + SNSUBG_s \qquad ...(3.1)$$

$$SNBASU_s = 0.0394 \sum_{i=1}^{n} a_{is} \, h_i \qquad ...(3.2)$$

$$SNSUBA_s = 0.0394 \sum_{j=1}^{m} a_{js} \left\{ \left(\frac{b_0 \exp(-b_3 z_j)}{-b_3} + \frac{b_1 \exp(-(b_2 + b_3) z_j)}{(b_2 + b_3)} \right) - \left(\frac{b_0 \exp(-b_3 z_{j-1})}{-b_3} + \frac{b_1 \exp(-(b_2 + b_3) z_{j-1})}{(b_2 + b_3)} \right) \right\} \qquad ...(3.3)$$

$$SNSUBG_s = [b_0 - b_1 \exp(-b_2 z_m)]\,[\exp(-b_3 z_m)]\,\left[3.51 \log_{10} CBR_s - 0.85 (\log_{10} CBR_s)^2 - 1.43\right] \qquad ...(3.4)$$

where:

SNP_s	adjusted structural number of the pavement for season *s*
$SNBASU_s$	contribution of surfacing and base layers for season *s*
$SNSUBA_s$	contribution of the sub-base or selected fill layers for season *s*
$SNSUBG_s$	contribution of the subgrade for season *s*
n	number of base and surfacing layers ($i = 1, 2, \ldots, n$)
a_{is}	layer coefficient for base or surfacing layer *i* for season *s*
h_i	thickness of base or surfacing layer *i* (mm)
m	number of sub-base and selected fill layers ($j = 1, 2, \ldots, m$)
z	depth parameter measured from the top of the sub-base (underside of base) (mm)
z_j	depth to the underside of the j^{th} layer ($z_0 = 0$) (mm)
CBR_s	in situ subgrade CBR for season *s*
a_{js}	layer coefficient for sub-base or selected fill layer *j* for season *s*
b_0, b_1, b_2, b_3	model coefficients

The values of the model coefficients b_0 to b_3 are given in Table C2.6 and the values of the layer coefficients a_i and a_j are given in Table C2.7.

Table C2.6 Adjusted structural number model coefficients

Pavement type	b_0	b_1	b_2	b_3
All pavement types	1.6	0.6	0.008	0.00207

Table C2.7 Pavement layer strength coefficients

Layer	Layer type	Condition	Coefficient
Surfacing	ST	Usually 0.2	$a_i = 0.20$ to 0.40
	AM	$h_i <$ 30 mm, low stability and cold mixes	$a_i = 0.20$
		$h_i >$ 30 mm, MR_{30} = 1500 MPa	$a_i = 0.30$
		$h_i >$ 30 mm, MR_{30} = 2500 MPa	$a_i = 0.40$
		$h_i >$ 30 mm, $MR_{30} \geq$ 4000 MPa	$a_i = 0.45$
Base	GB	Default	$a_i = (29.14\ CBR - 0.1977\ CBR^2 + 0.00045\ CBR^3)\ 10^{-4}$
		CBR > 70, cemented sub-base	$a_i = 1.6\ (29.14\ CBR - 0.1977\ CBR^2 + 0.00045\ CBR^3)\ 10^{-4}$
		CBR < 60, max. axle load > 80kN	$a_i = 0$
	AB, AP	Dense graded with high stiffness	$a_i = 0.32$
	SB	Lime or cement	$a_i = 0.075 + 0.039\ UCS - 0.00088(UCS)^2$
Sub-base		Granular	$a_j = -0.075 + 0.184(\log_{10} CBR) - 0.0444(\log_{10} CBR)^2$
		Cemented UCS > 0.7 MPa	$a_j = 0.14$

Source: *Watanatada et al. (1987)*

Notes:

1 The table reproduces information from the source, with the exception of the granular sub-base coefficient

2 If the user quotes a CBR value for a stabilised (lime or cement) layer, the corresponding granular coefficient should be used

3 Unconfined Compressive Strength (UCS) is quoted in MPa at 14 days

4 MR_{30} is the resilient modulus by the indirect tensile test at 30 °C

5 CBR is the California Bearing Ratio

Equation 3.4 above predicts negative values for the subgrade contribution below CBR = 3. This is different to HDM-III where the values were set to 0 and reflects the detrimental impact of weak subgrades on pavement performance.

3.2 Seasonal and drainage effects

It has been well recognised that pavement strength changes during the course of a year due to climatic effects. Hence, both seasonal and drainage effects have been included in the modelling of road deterioration in HDM-4. The average annual strength of the pavement is used in the deterioration models. This is estimated from the strength of the pavement during the dry season and during the wet season, and the duration of each season. The user is required to input the dry season SNP (or the wet season SNP) and the length of the dry season.

The average annual SNP is derived as follows:

$$SNP = f_s \, SNP_d \qquad ...(3.5)$$

where:

$$f_s = \frac{f}{\left[(1-d) + d\left(f^p\right)\right]^{1/p}} \qquad ...(3.6)$$

and:

SNP	Average annual adjusted structural number
SNP_d	dry season SNP
f	SNP_w / SNP_d ratio
d	length of dry season as a fraction of the year
p	Exponent of SNP specific to the appropriate deterioration model (see Table C2.8)

Table C2.8 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*) should be used to calculate the wet/dry season SNP ratio. This relationship will also be used to calculate the wet/dry season SNP ratio for each year of the analysis period, taking into account changes in the drainage factor and the amount of cracking.

$$f = K_f \left\{ 1 - \frac{\left[1 - \exp\left(a_0 MMP\right)\right]}{a_1} \left(1 + a_2 DF_a\right)\left(1 + a_3 ACRA_a + a_4 APOTa\right) \right\} \qquad ...(3.7)$$

where:

f	SNP_w / SNP_d ratio

SNP_w	wet season SNP
SNP_d	dry season SNP
MMP	mean monthly precipitation (mm/month)
DF_a	Drainage factor at start of analysis year
$ACRA_a$	total area of cracking at the start of the analysis year (% of total carriageway area)
K_f	Calibration factor for wet/dry season SNP ratio (range 0.1 to 10)

The default coefficient values a_0 to a_4 are given in Table C2.9.

Table C2.9 Default 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 will be required to input the type of drain (as listed in Table C2.10) and the condition of the drain as **excellent**, **good**, **fair**, **poor** or **very poor**.

Table C2.10 Suggested range of drainage factor values

Drain type	Drain condition	
	Excellent DF_{min}	Very poor DF_{max}
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

The minimum (**excellent**) and maximum (**very poor**) values for DF suggested for various types of drain are given in Table C2.10. The value of DF for drains in a **good**, **fair** or **poor** condition is determined by linearly interpolating between these values.

In some instances there will be an absence of drains. In situations where a drain is required the value of DF ranges between 3 and 5, whereas in situations where a drain is unnecessary a value of 1 for DF is suggested.

The condition of the drains will deteriorate unless they are well maintained, for example, through routine maintenance. The incremental annual change in DF due to deterioration is given in Equation 3.8 below: (the change in DF due to maintenance, ΔDF_w, is detailed in Part D - Road Works Effects).

$$\Delta DF_d = MAX\{0, MIN[K_{ddf}\ ADDF, (DF_{max} - DF_a)]\} \quad ...(3.8)$$

and:

$$ADDF = \frac{(DF_{max} - DF_{min})}{Drain\ Life} \quad ...(3.9)$$

where:

ΔDF_d annual change in DF due to deterioration

K_{ddf} calibration factor for drainage factor

ADDF annual deterioration of DF

Drain Life life of the drain (years) (see Table C2.8)

Drain life has been expressed as a function of the terrain as given below. The proposed default coefficient values a_0 and a_1 are given in Table C2.11 (*Morosiuk, 1998b*) for the climatic categories classified by moisture (see Chapter C1 - Section 4).

$$Drain\ Life = K_{drain}\ a_0\ (1 + a_1 RF) \quad ...(3.10)$$

where:

RF rise and fall (m/km)

K_{drain} calibration factor for drain life

Table C2.11 Default coefficient values for drain life

Drain type	Arid		Semi-arid		Sub-humid		Humid		Per-humid	
	a_0	a_1	a_0	a_1	a_0	a_1	a_0	a_1	a_0	a_1
Fully lined and linked	20	-0.0033	20	-0.0033	13	-0.0031	6	-0.0022	5	-0.0027
Surface lined	20	-0.0033	15	-0.0031	8	-0.0017	5	-0.0027	4	-0.0033
V-shaped - hard	20	-0.0033	15	-0.0031	10	-0.0027	6	-0.0022	4	-0.0033
V-shaped - soft	15	-0.0031	8	-0.0033	6	-0.0022	5	-0.0027	4	-0.0033
Shallow - hard	15	-0.0031	6	-0.0022	5	-0.0027	4	-0.0033	3	-0.0022
Shallow - soft	10	-0.0033	5	-0.0027	4	-0.0033	3	-0.0022	3	-0.0033
No drain - but required	3.5	-0.0029	2.5	-0.0027	2	-0.0033	1.5	-0.0044	1.5	-0.0044
No drain - not required	50	0	50	0	50	0	50	0	50	0

3.3 User input options

Pavement strength may be input in the following forms:

1 SNP

2 Benkelman beam deflections

3 FWD deflections

4 Layer thickness, strength coefficients and subgrade CBR

Option 4 has been described by Equations 3.1 above to 3.4 above. If either of options 2 or 3 are used, the model will convert the input data to SNP as follows:

- **Option 2 - Benkelman beam deflections**

The relationships used to convert Benkelman beam deflections (DEF) to SNP values are based on those in HDM-III (*Paterson, 1987*) and are given below:

- Base is not cemented

$$SNP_s = 3.2\,(DEF_s)^{-0.63} + dSNPK \qquad ...(3.11)$$

- Base is cemented

$$SNP_s = 2.2\,(DEF_s)^{-0.63} + dSNPK \qquad ...(3.12)$$

and:

$$dSNPK = 0.0000758\,\{MIN\,(63, ACX_a)\,HSNEW + MAX\,[MIN\,(ACX_a - PACX, 40), 0]\,HSOLD\} \qquad ...(3.13)$$

where:

DEF_s	benkelman beam rebound deflection under 80 kN axle load, 520 kPa tyre pressure and 30°C average asphalt temperature for season *s* (mm)
dSNPK	reduction in adjusted structural number due to cracking
ACX_a	area of indexed cracking at the start of the analysis year (% of total carriageway area)
HSNEW	thickness of the most recent surfacing (mm)
PACX	area of previous indexed cracking in old surfacing (% of total carriageway area); that is, 0.62 (PCRA) + 0.39 (PCRW)
HSOLD	total thickness of previous underlying surfacing layers (mm)

Some models need Benkelman beam deflection values. Where these are not user input, DEF values are derived from SNP values using relationships originated from those in HDM-III, that is:

- Base is not cemented

$$DEF_s = 6.5\,(SNPK_s)^{-1.6} \qquad ...(3.14)$$

- Base is cemented

$$DEF_s = 3.5\,(SNPK_s)^{-1.6} \qquad \text{...(3.15)}$$

and:

$$SNPK_s = SNP_s - dSNPK \qquad \text{...(3.16)}$$

where:

$SNPK_s$ adjusted structural number due to cracking for season *s*

- **Option 3 - FWD deflections**

The central FWD deflection at 700 kPa is used as the equivalent Benkelman beam deflection. The equations in Option 2 are then used to calculate SNP.

4 Construction quality

Poor construction quality results in greater variability in material properties and performance. HDM-4 does not provide a method of modelling proportions of road that are classified as **good**, **fair** and **poor**, so only an average level of construction defects is usually included. The construction defects indicators (CDS and CDB) used in the deterioration models are described below.

The relative compactions of the base, sub-base and selected subgrade layers (COMP) is important in predicting the initial densification of rut depth. *Paterson (1987)* gives an equation to calculate COMP, but it is proposed that users are also able to estimate it based on the values in Table C2.12.

Table C2.12 Default values for relative compaction

Compliance	Relative Compaction COMP (%)
Full compliance in all layers	100
Full compliance in some layers	95
Reasonable compliance in most layers	90
Poor compliance in most layers	85
HDM-4 default value	97

The initiation (and in some cases progression) of certain distresses is more accurately attributed to problems in material handling, preparation, or construction than to structural weakness in the pavement. In HDM-III, a construction quality code (CQ) was used in the crack initiation and ravelling models. However, in HDM-4 the construction defects are input through two indicators:

- **CDS**

  Construction defects indicator for bituminous surfacings

- **CDB**

  Construction defects indicator for the base

CDS is a factor indicating the general level of binder content and stiffness relative to the optimal material design for the specified bituminous mixture. It is used as an indicator to illustrate whether a bituminous surfacing is prone to cracking and ravelling (low value of CDS), or prone to rutting through plastic deformation (high value of CDS).

CDS is a continuous variable, generally ranging in value between 0.5 and 1.5 as shown in Table C2.13. Intermediate values are chosen by judgement. This may involve back-analysis to verify that the plastic deformation and cracking predictors are valid (see A Guide to Calibration and Adaptation).

Table C2.13 Selection of construction defect indicator for bituminous surfacings

Surfacing condition		CDS
Dry (Brittle)	Nominally about 10% below design optimal binder content	0.5
Normal	Optimal binder content	1.0
Rich (Soft)	Nominally about 10% above design optimal binder content	1.5

For potholing, the base construction defects indicator (CDB) is used. CDB is a continuous variable ranging between 0 (no construction defects) and 1.5 (several defects). The type of defects that should be considered in setting a value of CDB is given in Table C2.14.

Table C2.14 Selection of construction defect indicator for base

Construction defect	CDB
Poor gradation of material	0.5
Poor aggregate shape	0.5
Poor compaction	0.5

5 Crack modelling

Cracking is one of the most important distresses in bituminous pavements. Fatigue and ageing have been identified as the principal factors which contribute to cracking of a bituminous pavement layer. The propagation of cracking is accelerated through the embrittlement resulting from ageing and the ingress of water, which can significantly weaken the underlying pavement layers.

There are two types of cracking considered in HDM-4:

- **Structural cracking**

  This is effectively load and age/environment associated cracking (see Section 5.1).

- **Transverse thermal cracking**

  This is generally caused by large diurnal temperature changes or in freeze/thaw conditions, and therefore usually occurs only in certain climates (see Section 5.2).

For each type of cracking, separate relationships are given for predicting the time to initiation and then the rate of progression. These relationships include the construction defects indicator for bituminous surfacings, CDS, as a variable (see Section 4).

5.1 Structural cracking

Structural cracking is modelled as **All** and **Wide** cracking, based on the relationships derived by *Paterson (1987)*.

5.1.1 Initiation of All structural cracking

Crack initiation is said to occur when 0.5% of the carriageway surface area is cracked. Initiation of All structural cracking depends on the base:

- **Stabilised base**

  if HSOLD = 0 (that is, original surfacings)

$$ICA = K_{cia}\left\{CDS^2\ a_0\ \exp\begin{bmatrix}a_1\ HSE + a_2\ \log_e CMOD \\ + a_3\ \log_e DEF + a_4 (YE4)\ (DEF)\end{bmatrix} + CRT\right\} \quad ...(5.1)$$

  if HSOLD > 0 (that is, overlays or reseals)

$$ICA = K_{cia}\left\{CDS^2\begin{bmatrix}(0.8\,KA + 0.2KW)\ (1 + 0.1HSE) + (1 - KA)\ (1 - KW)a_0 \\ *\ \exp\begin{bmatrix}a_1\ HSE + a_2\ \log_e CMOD \\ + a_3\ \log_e DEF + a_4 (YE_4)\ (DEF)\end{bmatrix}\end{bmatrix} + CRT\right\}$$

...(5.2)

- **Other bases**

  if HSOLD = 0 (that is, original surfacing)

$$ICA = K_{cia}\left\{CDS^2\ a_0\ \exp\left[a_1\ SNP + a_2\ \left(\frac{YE4}{SNP^2}\right)\right] + CRT\right\} \quad ...(5.3)$$

if HSOLD > 0 (that is, overlays or reseals)

- For all surface materials **except** CM, SL and CAPE

$$ICA = K_{cia}\left\{CDS^2\left[MAX\left(\begin{array}{l} a_0 \exp\left[a_1\, SNP + a_2\left(\dfrac{YE4}{SNP^2}\right)\right] \\ * MAX\left(1-\dfrac{PCRW}{a_3},0\right), a_4 HSNEW \end{array}\right)\right]+CRT\right\} \quad ...(5.4)$$

- For surface materials - CM, SL and CAPE

$$ICA = K_{cia}\left\{CDS^2\left[MAX\left(\begin{array}{l} a_0 \exp\left[a_1\, SNP + a_2\left(\dfrac{YE4}{SNP^2}\right)\right] \\ * MAX\left(1-\dfrac{PCRA}{a_3},0\right), a_4 \end{array}\right)\right]+CRT\right\} \quad ...(5.5)$$

5.1.2 Initiation of Wide structural cracking

$$ICW = K_{ciw}\, MAX[(a_0 + a_1\, ICA), a_2\, ICA] \quad ...(5.6)$$

where:

ICA	time to initiation of All structural cracks (years)
ICW	time to initiation of Wide structural cracks (years)
CDS	construction defects indicator for bituminous surfacings
YE4	annual number of equivalent standard axles (millions/lane)
SNP	average annual adjusted structural number of the pavement
DEF	mean Benkelman beam deflection in both wheelpaths (mm)
CMOD	resilient modulus of soil cement (GPa) (in the range between 0 and 30 GPa for most soils)
HSNEW	thickness of the most recent surfacing (mm)
HSOLD	total thickness of previous underlying surfacing layers (mm)
PCRA	area of All cracking before latest reseal or overlay (% of total carriageway area)
PCRW	area of Wide cracking before latest reseal or overlay (% of total carriageway area)
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
K_{ciw}	calibration factor for initiation of Wide structural cracking
CRT	crack retardation time due to maintenance (years) (see Part D)

The proposed default coefficient values a_0 to a_4 for the initiation of All cracking are given in Table C2.15, and those of a_0 to a_2 for the initiation of Wide cracking are given in Table C2.16.

Table C2.15 Default coefficient values for initiation of All structural cracking models

Pavement type	Surface material	HSOLD value	Equ^n	a_0	a_1	a_2	a_3	a_4
AMGB	All	0	5.3	4.21	0.14	-17.1		
	All except CM	> 0	5.4	4.21	0.14	-17.1	30	0.025
	CM	> 0	5.5	13.2	0	-20.7	20	1.4
AMAB	All	0	5.3	4.21	0.14	-17.1		
		> 0	5.4	4.21	0.14	-17.1	30	0.025
AMAP	All	> 0	5.4	4.21	0.14	-17.1	30	0.025
AMSB	All	0	5.1	1.12	0.035	0.371	-0.418	-2.87
		> 0	5.2	1.12	0.035	0.371	-0.418	-2.87
STGB	All	0	5.3	13.2	0	-20.7		
	All except SL, CAPE	> 0	5.4	13.2	0	-20.7	20	0.22
	SL, CAPE	> 0	5.5	13.2	0	-20.7	20	1.4
STAB	All	0	5.3	13.2	0	-20.7		
	All except SL, CAPE	> 0	5.4	4.21	0.14	-17.1	20	0.12
	SL, CAPE	> 0	5.4	4.21	0.14	-17.1	30	0.025
STAP	All	> 0	5.4	4.21	0.14	-17.1	20	0.12
STSB	All	0	5.1	1.12	0.035	0.371	-0.418	-2.87
		> 0	5.2	1.12	0.035	0.371	-0.418	-2.87

Table C2.16 Default coefficient values for initiation of Wide structural cracking models

Pavement type	Surface material	HSOLD value	a_0	a_1	a_2
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

5.1.3 Progression of All structural cracking

The general form of the model for the progression of All structural cracking is given below:

$$dACA = K_{cpa}\left[\frac{CRP}{CDS}\right]Z_A\left[\left(Z_A\ a_0\ a_1\ \delta t_A + SCA^{a1}\right)^{1/a1} - SCA\right] \qquad ...(5.7)$$

Progression of All structural cracking commences when $\delta t_A > 0$ or $ACA_a > 0$

where:

if $ACA_a > 0$ $\delta t_A = 1$ otherwise $\delta t_A = MAX\{0, MIN[(AGE2 - ICA), 1]\}$

if $ACA_a \geq 50$ then: $z_A = -1$ otherwise: $z_A = 1$

$$ACA_a = MAX(ACA_a, 0.5)$$

$$SCA = MIN[ACA_a, (100 - ACA_a)]$$

$$Y = \left[a_0\ a_1\ Z_A\ \delta t_A + SCA^{a1}\right] \qquad ...(5.8)$$

- **if Y < 0**

  then:

$$dACA = K_{cpa}\left[\frac{CRP}{CDS}\right](100 - ACA_a) \qquad ...(5.9)$$

- **if $Y \geq 0$**

  then:

$$dACA = K_{cpa}\left[\frac{CRP}{CDS}\right] Z_A \left(Y^{1/a1} - SCA\right) \qquad ...(5.10)$$

- **if** $ACA_a \leq 50$ **and** $ACA_a + dACA > 50$

  then:

$$dACA = K_{cpa}\left[\frac{CRP}{CDS}\right]\left(100 - c_1^{1/a1} - ACA_a\right) \qquad ...(5.11)$$

where:

$$c_1 = MAX\left\{\left[2\left(50^{a1}\right) - SCA^{a1} - a_0\, a_1\, \delta t_A\right], 0\right\} \qquad ...(5.12)$$

5.1.4 Progression of Wide structural cracking

The general form of the model for the progression of Wide structural cracking is given below:

$$dACW = K_{cpw}\left[\frac{CRP}{CDS}\right] Z_w \left[\left(Z_w\, a_0\, a_1\, \delta t_w + SCW^{a1}\right)^{1/a1} - SCW\right] \qquad ...(5.13)$$

where:

$$dACW = MIN\left[ACA_a + dACA - ACWa,\ dACW\right] \qquad ...(5.14)$$

Progression of Wide structural cracking commences when $\delta t_W > 0$ or $ACW_a > 0$

where:

if $ACW_a > 0 \quad \delta t_W = 1$ otherwise $\delta t_w = MAX\{\,0, MIN\left[(AGE2 - ICW), 1\right]\,\}$

The initiation of Wide structural cracking is constrained so that it does not commence before the area of All structural cracking (ACA_a) exceeds 5% as follows:

$\delta t_W = 0$ if $ACA_a \leq 5$ and $ACW_a \leq 0.5$ and $\delta t_W > 0$

If patching of Wide structural cracking was performed in the previous analysis year, reducing the area of Wide cracking to below 1% but with the area of All structural cracking remaining at over 11% at the start of the current analysis year (that is, $ACW_a \leq 1$ and $ACA_a > 11$), then the rate of progression of Wide structural 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 structural cracking, ACW_{temp} is defined to be 5% less than ACA_a; that is:

$ACW_{temp} = ACA_a - 5$ if $ACW_a \leq 1$ 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:

if $ACW_a \geq 50$ $ACW_a \geq 50$ then: $z_w = -1$

otherwise: $z_w = 1$

$$ACW_a = MAX(ACW_a, 0.5)$$

$$SCW = MIN\left[ACW_a, (100 - ACW_a)\right]$$

$$Y = \left[a_0\ a_1\ Z_w\ \delta t_w + SCW^{a1}\right] \quad ...(5.15)$$

- **if Y < 0**

then:

$$dACW = K_{cpw}\left[\frac{CRP}{CDS}\right] MIN\left[(ACA_a + dACA - ACW_a), (100 - ACW_a)\right] \quad ...(5.16)$$

- **if Y ≥ 0**

then:

$$dACW = K_{cpw}\left[\frac{CRP}{CDS}\right] MIN\left[(ACA_a + dACA - ACW_a), Z_w\left(Y^{1/a1} - SCW\right)\right] \quad ...(5.17)$$

- **if** $ACW_a \leq 50$ **and** $ACW_a + dACW > 50$

then:

$$dACW = K_{cpw}\left[\frac{CRP}{CDS}\right] MIN\left[(ACA_a + dACA - ACW_a), \left(100 - c_1^{1/a1} - ACW_a\right)\right]$$

...(5.18)

where:

$$c_1 = MAX\left\{\left[2\left(50^{a1}\right) - SCW^{a1} - a_0\ a_1\ \delta t_w\right], 0\right\} \quad ...(5.19)$$

and:

dACA	incremental change in area of All structural cracking during the analysis year (% of total carriageway area)
dACW	incremental change in area of Wide structural cracking during the analysis year (% of total carriageway area)
ACA_a	area of All structural cracking at the start of the analysis year
ACW_a	area of Wide structural cracking at the start of the analysis year (% of total carriageway area)
δt_A	fraction of analysis year in which All structural cracking progression applies

δt_W	fraction of analysis year in which Wide structural cracking progression applies
AGE2	pavement surface age since last reseal, overlay, reconstruction or new construction (years)
ICA	time to initiation of All structural cracking (years)
ICW	time to initiation of Wide structural cracking (years)
K_{cpa}	calibration factor for progression of All structural cracking
K_{cpw}	calibration factor for progression of Wide structural cracking
CRP	retardation of cracking progression due to preventative treatment, given by CRP = 1 - 0.12 CRT

The proposed default coefficient values a_0 and a_1 for the progression of All cracking and those for the progression of Wide cracking are given in Table C2.17.

Table C2.17 Default coefficient values for progression of All and Wide structural cracking

Pavement type	Surface material	HSOLD value	All cracking		Wide cracking	
			a_0	a_1	a_0	a_1
AMGB	All	0	1.84	0.45	2.94	0.56
	All except CM	> 0	1.07	0.28	2.58	0.45
	CM	> 0	2.41	0.34	3.40	0.35
AMAB	All	0	1.84	0.45	2.94	0.56
		> 0	1.07	0.28	2.58	0.45
AMAP	All	> 0	1.07	0.28	2.58	0.45
AMSB	All	0	2.13	0.35	3.67	0.38
		> 0	2.13	0.35	3.67	0.38
STGB	All	0	1.76	0.32	2.50	0.25
		> 0	2.41	0.34	3.40	0.35
STAB	All	0	1.76	0.32	2.50	0.25
	All except SL, CAPE	> 0	2.41	0.34	3.40	0.35
	SL, CAPE	> 0	1.07	0.28	2.58	0.45
STAP	All	> 0	2.41	0.34	3.40	0.35
STSB	All	0	2.13	0.35	3.67	0.38
		> 0	2.41	0.34	3.40	0.35

5.2 Transverse thermal cracking

Transverse thermal cracking is modelled as cracking intensity expressed as the number of cracks per kilometre. A coefficient of thermal cracking (CCT) is used as a variable to predict time to initiation of thermal cracks for the various climate zones described in Chapter C1. Suggested values of CCT are given in Table C2.18. Table C2.19 gives the proposed values of the maximum number of thermal cracks (NCT_{eq}) per kilometre of road and the time since initiation to reach this level of cracking (T_{eq}), for the various climate zones.

Table C2.18 Proposed default values of CCT

Model parameter	Tropical	Sub-tropical hot	Sub-tropical cool	Temperate cool	Temperate freeze
Arid	100	5	100	100	2
Semi-arid	100	8	100	100	2
Sub-humid	100	100	100	100	1
Humid	100	100	100	100	1
Per-humid	100	100	100		

Table C2.19 Proposed default values of NCT_{eq} and T_{eq}

Model parameter	Tropical	Sub-tropical hot	Sub-tropical cool	Temperate cool	Temperate freeze
NCTeq	0	100	0	0	20
Teq	50	7	50	50	7

5.2.1 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.

- **if HSOLD = 0** (that is, original surfacings)

$$ICT = K_{cit}\, MAX[a_0, (CDS)(CCT)] \qquad ...(5.20)$$

- **if HSOLD > 0** (that is, overlays or reseals)

$$ICT = K_{cit}\, MAX\,[a_0, CDS\,(CCT + a_1 + a_2\, HSNEW)] \qquad ...(5.21)$$

5.2.2 Progression of transverse thermal cracking

Progression of transverse thermal cracking commences when $\delta t_T > 0$

where:

$$\text{if } ACT_a > 0 \quad \delta t_T = 1 \qquad \text{otherwise } \delta t_T = MAX\{\, 0, MIN[(AGE2 - ICT), 1\,]\,\}$$

- **if HSOLD = 0** (that is, original surfacings)

$$dNCT = K_{cpt}\left[\frac{1}{CDS}\right]MAX\left\{0, MIN\left[(NCT_{eq} - NCT_a), \left(\frac{2\,NCT_{eq}(AGE3 - ICT - 0.5)}{(T_{eq})^2}\right)\right]\right\}\delta t_T \quad ...(5.22)$$

- **if HSOLD > 0** (that is, overlays or reseals)

$$dNCT = K_{cpt}\left[\frac{1}{CDS}\right]MIN\left\{(NCT_{eq} - NCT_a), MAX\left[\begin{array}{l} MIN(a_0\,PNCT, (PNCT - NCT_a)), \\ \left(\frac{2\,NCT_{eq}(AGE3 - ICT - 0.5)}{(T_{eq})^2}\right), 0\end{array}\right]\right\}\delta t_T \quad ...(5.23)$$

A transverse thermal crack is assumed to traverse the full width of the carriageway. Thus the area of transverse thermal cracking is given by:

$$dACT = \frac{dNCT}{20} \quad ...(5.24)$$

where:

ICT	time to initiation of transverse thermal cracks (years)
dNCT	incremental change in number of transverse thermal cracks during the analysis year (n^o/km)
CDS	construction defects indicator for bituminous surfacings
dACT	incremental change in area of transverse thermal cracking during the analysis year (% of total carriageway area)
CCT	coefficient of thermal cracking (see Table C2.18)
PNCT	number of transverse thermal cracks before latest overlay or reseal (n^o/km)
NCT_a	number of (reflected) transverse thermal cracks at the start of the analysis year (n^o/km)
NCT_{eq}	maximum number of thermal cracks (n^o/km) (see Table C2.19)
T_{eq}	time since initiation to reach maximum number of thermal cracks (years) (see Table C2.19)
HSNEW	thickness of the most recent surfacing (mm)
K_{cit}	calibration factor for initiation of transverse thermal cracking
K_{cpt}	calibration factor for progression of transverse thermal cracking

The default coefficient values a_0 to a_2 for the initiation, and those of a_0 for the progression of transverse thermal cracks, are given in Table C2.20.

Table C2.20 Default coefficient values for transverse thermal cracking

Pavement type	Initiation			Progression
	a_0	a_1	a_2	a_0
All pavement types except STGB and STSB	1.0	-1.0	0.02	0.25
STGB and STSB	100	-1.0	0.02	0.25

5.3 Total areas of cracking

The above cracking models predict areas of All and Wide structural cracking (ACA and ACW respectively) and transverse thermal cracking (ACT). In several of the deterioration models, areas of cracking other than ACA, ACW or ACT are required. These are defined in Sections 5.3.1 and 5.3.2.

5.3.1 Area of indexed cracking

The area of indexed cracking is a weighted average of All and Wide structural cracking, defined by *Paterson (1987)* as follows:

$$ACX = 0.62\ ACA + 0.39\ ACW \qquad ...(5.25)$$

where:

ACX area of indexed cracking (% of total carriageway area)

ACA area of All structural cracking (% of total carriageway area)

ACW area of Wide structural cracking (% of total carriageway area)

5.3.2 Total area of cracking

The total area of cracking combines the structural and transverse thermal cracking and is defined as follows:

$$ACRA = ACA + ACT \qquad ...(5.26)$$

where:

ACRA total area of carriageway cracked (% of total carriageway area)

ACA area of All structural cracking (% of total carriageway area)

ACT area of transverse thermal cracking (% of total carriageway area)

6 Ravelling

Ravelling is the progressive loss of surface material through weathering and/or traffic abrasion. The occurrence of ravelling varies considerably among different regions and countries according to construction methods, specifications, available materials, and local practice. Ravelling is a common distress in poorly constructed, thin bituminous layers such as surface treatment, but it is rarely seen in high quality, hot-mix asphalt.

The construction defects indicator for bituminous surfacings, CDS, (see Section 4) is used as a variable in the ravelling models. The initiation model is basically as proposed by *Paterson (1987)*, with CDS replacing the original construction quality variable CQ. The progression model is also based on that proposed by *Paterson (1987)* but with a traffic variable introduced as proposed by *Riley (1999)*.

6.1 Initiation

Ravelling is said to occur on a given road section when 0.5% of the carriageway surface area is classified as ravelled. The initiation is given as:

$$IRV = K_{vi}\ CDS^2\ a_0\ RRF \exp(a_1\ YAX) \qquad ...(6.1)$$

where:

IRV	time to ravelling initiation (years)
CDS	construction defects indicator for bituminous surfacings
YAX	annual number of axles of all motorised vehicle types in the analysis year (millions/lane)
K_{vi}	calibration factor for ravelling initiation
RRF	ravelling retardation factor due to maintenance (see Part D)

The proposed default coefficient values a_0 to a_1 for the ravelling initiation model is given in Table C2.21.

Table C2.21 Default coefficient values for ravelling initiation model

Surface type	Surface material	a_0	a_1
AM	All except CM	10.0	0.0
	CM	8.0	-0.156
ST	All except SL, CAPE	10.0	0.0
	SL, CAPE	12.0	0.0

6.2 Progression

The general form of the model for the progression of ravelling is given below:

$$dARV = \left[\frac{K_{vp}}{RRF}\right]\left[\frac{1}{CDS^2}\right]Z\left[\left(Z(a_0 + a_1 YAX)a_2\ \delta t_v + SRV^{a2}\right)^{1/a2} - SRV\right] \quad ...(6.2)$$

Progression of ravelling commences when $\delta t_v > 0$ or $ARV_a > 0$

where:

if $ARV_a > 0$ $\delta t_v = 1$ otherwise $\delta t_v = MAX\{0, MIN[(AGE2 - IRV), 1]\}$

if $ARV_a \geq 50$ then: $z = -1$

otherwise: $z = 1$

$$ARV_a = MAX(ARV_a, 0.5)$$

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

$$YAX = MAX[MIN(YAX, 1), 0.1]$$

$$Y = \left[(a_0 + a_1 YAX)a_2\ Z\ \delta t_v + SRV^{a2}\right] \quad ...(6.3)$$

- **if Y < 0**

  then:

$$dARV = \left[\frac{K_{vp}}{RRF}\right]\left[\frac{1}{CDS^2}\right](100 - ARV_a) \quad ...(6.4)$$

- **if Y ≥ 0**

  then:

$$dARV = \left[\frac{K_{vp}}{RRF}\right]\left[\frac{1}{CDS^2}\right]Z\left(Y^{1/a2} - SRV\right) \quad ...(6.5)$$

- **if ARVa ≤ 50 and ARVa + dARV > 50**

  then:

$$dARV = \left[\frac{K_{vp}}{RRF}\right]\left[\frac{1}{CDS^2}\right]\left(100 - c_1^{1/a2} - ARV_a\right) \quad ...(6.6)$$

and:

$$c_1 = MAX\left\{\left[2\left(50^{a2}\right) - SRV^{a2} - (a_0 + a_1 YAX)a_2\ \delta t_v\right], 0\right\} \quad ...(6.7)$$

where:

dARV	change in area of ravelling during the analysis year (% of total carriageway area)
ARV_a	area of ravelling at the start of the analysis year (% of total carriageway area)
δt_v	fraction of analysis year in which ravelling progression applies

AGE2 pavement surface age since last reseal, overlay, reconstruction or new construction (years)

Kvp calibration factor for ravelling progression

IRV time to ravelling initiation (years)

Other parameters are as defined for ravelling initiation.

The proposed default coefficient values a_0 and a_2 for the ravelling progression model is given in Table C2.22.

Table C2.22 Default coefficient values for ravelling progression model

Pavement type	a_0	a_1	a_2
All pavement types	0.3	1.5	0.352

7 Potholing

Potholes usually develop in a surface that is either cracked, ravelled, or both. The presence of water accelerates pothole formation both through a general weakening of the pavement structure and lowering the resistance of the surface and base materials to disintegration.

The potholing models use the construction defects indicator for the base, CDB, as a variable (see Section 4). In the models, potholing is expressed in terms of the number of **pothole units** of area 0.1 m^2. The volume of each of these pothole units is assumed to be 10 litres (that is, 100 mm in depth). The relationships for the initiation and progression of potholing have been modified from those given in the *NDLI (1995)* and *Riley (1996b*).

7.1 Initiation

Initiation of potholes due to cracking only arises when the following condition is met:

$$ACWa > ACWpi$$

where:

ACWa Wide structural cracking at the start of the analysis year

ACWpi The user-defined percentage at which wide structural cracking initiated potholes arise (default = 20%)

The time to initiation of potholing due to wide structural cracking is given by the following model:

$$IPT_c = K_{pic} * a_0 \left[\frac{(1 + a_1 HS)}{(1 + a_2 CDB)(1 + a_3 YAX)(1 + a_4 MMP)} \right] \qquad ...(7.1)$$

where:

IPT_c time between the initiation of Wide structural cracking and the initiation of potholes (years)

HS total thickness of bituminous surfacing (mm)

CDB construction defects indicator for the base

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

MMP mean monthly precipitation (mm/month)

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

Initiation of potholes due to ravelling only arises when the following condition is met:

$$ARVa > ARVpi$$

where:

ARVa	Ravelling at the start of the analysis year
ARVpi	The user-defined percentage at which ravelling initiated potholes arise (default = 30%)

The time to initiation of potholing due to wide structural cracking is given by the following model:

$$IPT_r = K_{pir} * a_0 \left[\frac{(1 + a_1 HS)}{(1 + a_2 CDB)(1 + a_3 YAX)(1 + a_4 MMP)} \right] \quad ...(7.2)$$

where:

IPTr	time between the initiation of ravelling and the initiation of potholes (years)
K_{pir}	calibration factor for pothole initiation due to ravelling

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 proposed default coefficient values a_0 to a_4 for the pothole initiation model is given in Table C2.23.

Table C2.23 Default coefficient values for pothole initiation model

Cause of pothole initiation	Pavement type	a_0	a_1	a_2	a_3	a_4
Cracking	AMGB, STGB	2.0	0.05	1.0	0.5	0.01
	All except GB bases	3.0	0.05	1.0	0.5	0.01
Ravelling	AMGB, STGB	2.0	0.05	1.0	0.5	0.01
	All except GB bases	3.0	0.05	1.0	0.5	0.01

7.2 Progression

Pothole progression arises from potholes due to cracking, ravelling and the enlargement of existing potholes. The progression of potholes is affected by the patching policy assigned to the section.

The annual incremental increase in the number of pothole units due to each of these three distresses is calculated as:

$$dNPT_i = K_{pp} a_0 (ADIS_i)(PEFF_i)\left(\frac{ELANES}{2}\right)\left[\frac{(1+a_1 CDB)(1+a_2 YAX)(1+a_3 MMP)}{(1+a_4 HS)}\right] \quad ...(7.3)$$

Pothole progression from wide cracking or ravelling commences as follows:

- If at the start of the first year of the analysis period $ACW_a = 0$, then potholing progression from wide cracking commences when:

  $AGE2 > ICW + IPT$ and $ACW_a > ACW_{pi}$

- If at the start of the first year of the analysis period $ARV_a = 0$, then potholing progression from ravelling commences when:

  $AGE2 > IRV + IPT$ and $ARV_a > ARV_{pi}$

- If at the start of the first year of the analysis period $0 < ACW_a \leq ACW_{pi}$, then potholing progression from wide cracking commences when $ACW_a > ACW_{pi}$
- If at the start of the first year of the analysis period $0 < ARV_a \leq ARV_{pi}$, then potholing progression from ravelling commences when $ARV_a > ARV_{pi}$
- If at the start of the first year of the analysis period $ACW_a > ACW_{pi}$, then potholing progression from wide cracking commences immediately
- If at the start of the first year of the analysis period $ARV_a > ARV_{pi}$, then potholing progression from ravelling commences immediately
- If during the analysis period ARV_a becomes $< ARV_{pi}$, because of ravelling areas reverting to other distressed areas, then potholing still progresses from ravelling
- Potholing progression from enlargement commences if $NPT_a > 0$ at the start of an analysis year

The total annual increase in the number of pothole units per kilometre of road length is given by:

$$dNPT = \sum_{i=1}^{3} dNPT_i \qquad ...(7.4)$$

where:

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

ADIS$_i$ the percentage area of Wide structural cracking at the start of the analysis year, or the percentage area of ravelling at the start of the analysis year, or number of existing potholes per km at the start of the analysis year

PEFF$_i$ patching policy factor for distress type $_i$ (see below)

dNPT total number of additional potholes per km during the analysis year

ELANES effective number of lanes for the road section

K$_{pp}$ calibration factor for pothole progression

Other parameters are as defined previously.

The proposed default coefficient values a_0 to a_4 for the pothole progression model are given in Table C2.24.

Table C2.24 Default coefficient values for pothole 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

7.2.1 Patching Policy Factor

A correction factor (Patching Policy Factor) is introduced in equation 7.3. This modified model recognises that a new pothole has to reach a certain size before it is deemed to need repair, that patching may be performed at regular intervals during the year and that for each patching campaign, partial patching can be carried out. The correction factor is calculated as:

$$PEFF_i = 1 - \frac{Ppt}{100}(1 - TLF_i) \qquad ...(7.4)$$

where

PEFF$_i$ patching policy factor for distress type $_i$ ($0 < PEFF_i \leq 1$)

Ppt percentage of potholes to patch ($0 < Ppt \leq 100$)

TLF_i effects of pothole patching frequency ($0 < TLF_i \leq 1$)

If no patching operation is assigned to the section, the default value for $PEFF_i$ is 1.

TLF_i is calculated as:

$$TLF_i = a_0 + (1 - a_0)\left(\frac{Fpat}{365}\right)^{a1} \qquad ...(7.5)$$

where

TLF_i effects of pothole patching frequency for distress type i ($0 < TLF_i \leq 1$) (see Table C2.26)

Fpat interval between pothole patching campaigns, in days; Fpat is selected by the user in a limited list of values.

Both Ppt and Fpat parameters are user specified in works standards (see part D2).

The coefficient values of a_0 and a_1 are given in Table C2.25.

Table C2.25
Coefficient values for TLF_i relationship

Cause of pothole progression	a_0	a_1
Cracking & Ravelling	0.2	1.5
Enlargement	0	1.5

The default value of 0.2 for a_0 has been set, but is likely to vary from agency to agency. In the case of enlargement of potholes existing at the start of the year, there is no intercept in the function as enlargement will be a continuous process until repairs are executed.

Table C2.26 Tabulated values for TLF_i

		TLF_i	
Number of patching campaigns per year	**Pothole patching interval**	**Cracking & Ravelling**	**Enlargement**
24	2 weeks	0.21	0.01
12	1 month	0.22	0.02
6	2 months	0.25	0.07
4	3 months	0.30	0.12
3	4 months	0.35	0.19
2	6 months	0.48	0.35
1	12 months	1.00	1.00

8 Edge-break

Edge-break can be defined as the loss of surface and base materials at the pavement edge, caused by shear failure and attrition. This commonly arises on narrow roads with unsealed shoulders, where vehicle wheels pass on or close to a pavement edge.

The measure for edge-break that the user provides as input into the model, and the corresponding output data, will be in square metres per km, and not in cubic metres per km. The value in square metres is then multiplied by ESTEP, specified below, (defined as part of a sections calibration attributes) to obtain the volume of edge-break in cubic metres for modelling purposes.

The edge-break model predicts that edge-break occurs on roads with a carriageway width of up to a user defined maximum width of CW_{max}. The default value of CW_{max} is 7.2 metres and an upper limit of CW_{max} has been set to 7.5 metres (that is, no edge-break is predicted for roads with a carriageway width greater than 7.5 metres).

The edge-break model is as follows:

$$dVEB = K_{eb}\ a_0\ PSH\,(AADT)^2 ESTEP(S)^{a1}\left[a_2 + \frac{MMP}{1000}\right]10^{-6} \qquad ...(8.1)$$

and:

$$PSH = MAX\left\{MIN\left[MAX\left(a_3 + a_4\ CW, \frac{CW_{max} - CW}{a_5}\right), 1\right], 0\right\} \qquad ...(8.2)$$

where:

dVEB	annual loss of edge material (m^3/km)
PSH	proportion of time vehicles use the shoulder due to road width
AADT	annual average daily traffic (veh/day)
ESTEP	elevation difference from pavement to shoulder (mm) (default = 10mm)
MMP	mean monthly precipitation (mm/month)
S	average traffic speed (km/h)
CW	carriageway width (metres)
CW_{max}	maximum carriageway width for the occurrence of edge-break (metres) (default = 7.2)
K_{eb}	calibration factor for edge-break progression

The proposed default coefficient values a_0 to a_5 for the edge-break model are given in Table C2.24.

Table C2.24 Default 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

9 Damaged and undamaged surface area

In modelling pavement deterioration, it is important to ensure that the sum of damaged and undamaged surface area must be equal to 100%, in any given analysis year. The total road surface consists of the following:

- **Edge-break**
- **Potholes (including those potholes that were patched during the analysis year, in case of frequent patching)**
- **Cracking**
- **Ravelling**
- **Undamaged**
  This area consists of the original road surface which is still in good condition since the last surfacing and the area which has been patched.

The logic devised for calculating the distress values at the end of an analysis year is described below (*Odoki, 1998*).

9.1 The logic

For modelling purposes, the above types of deterioration need to be converted to the equivalent surface area and these are assumed to be mutually exclusive. Therefore the sum of the surface area with edge-break, potholes, cracking, ravelling and undamaged must equal 100%.

It is acknowledged that an area of road can be both cracked and ravelled. However, the hierarchy employed in HDM-4 classifies cracking above ravelling because cracking is considered to be a more severe distress than ravelling. Once substantial amounts of damaged area are being modelled, the area of ravelling will therefore be re-classified as area of cracking. This will result in the reported area of ravelling decreasing, although this re-classified area could be regarded as both cracked and ravelled.

In devising a logic that satisfies the constraint of 100% total surface area, the following simplifying assumptions are made:

- Cracking develops first from the undamaged area and then, after the latter is exhausted, from the ravelled area if any. Furthermore, an area once cracked can develop potholes but cannot ravel.
- Ravelling can only develop from the undamaged area. After an area is ravelled it can also crack, at which stage it is reclassified from ravelled to cracked. (**Note**: this does not mean that ravelled areas would physically disappear).
- Potholes can only develop from cracked, ravelled and undamaged areas (as reflected in the formulas for computing the change in the number of potholes), and unless it is repaired, an area potholed cannot revert to cracking, ravelling or undamaged.
- An upper limit of 10% is imposed on the potholed area. This is because above this level the pavement surface becomes ill defined and the roughness function becomes invalid.
- Edge-break can only develop from cracked, ravelled and undamaged areas, and unless it is repaired, an area of edge-break cannot revert to potholes, cracking, ravelling or undamaged.

- An upper limit of 18% is imposed on the area of edge-break. The upper value of 18% is based on the assumption that edge-break will not extend beyond 0.5 metres from either edge of a pavement of 5.5 metres width.

9.2 Distress values at the end of the year

The assumptions given in Section 0 lead to Equations 9.1below - 9.20 below (see Sections 9.2.1 to 9.2.4) for computing the damaged areas at the end of each analysis year and before road works.

9.2.1 Edge-break

$$AVEB_b = MIN\,[\,18, (AVEB_a + dAVEB)\,] \qquad ...(9.1)$$

where:

$AVEB_b$	area of edge-break at the end of the analysis year (% of total carriageway area)
$AVEB_a$	area of edge-break at the start of the analysis year (% of total carriageway area)
dAVEB	unadjusted increase in the area of edge-break during the analysis year (% of total carriageway area)

Equation 9.1 above requires that the volume of edge-break, VEB, be converted into an area of edge-break measured as a percentage of total carriageway area. The area of edge-break expressed as a percentage of the total carriageway area, AVEB, is obtained from the following

expression:

$$AVEB = \frac{VEB}{CW} \qquad ...(9.2)$$

where:

AVEB	area of edge-break (% of total carriageway area)
VEB	volume of edge-break per km (m^3/km)
CW	carriageway width (metres)

Thus, by substituting VEB in Equation 9.2 above with VEB_a the value of $AVEB_a$ is obtained, and by substituting VEB with dVEB, the value of dAVEB is obtained;

where:

VEB_a	volume of edge-break per km at the start of the analysis year (m^3/km)
dVEB	unadjusted increase in volume of edge-break per km during the analysis year (m^3/km)

9.2.2 Potholes

$$APOT_b = MIN[10, (APOT_a + dAPOT)] \qquad ...(9.3)$$

where:

$APOT_b$	Total area of potholes at the end of the analysis year (% of total carriageway area), including potholes patched during the analysis year
$APOT_a$	area of potholes at the start of the analysis year (% of total carriageway area)
dAPOT	unadjusted increase in the area of potholes during the analysis year (% of total carriageway area)

Equation 9.3 above requires that the number of potholes per km be converted into area of potholes in per cent of total carriageway area. The area of potholes expressed as a percentage of total carriageway area (APOT) is obtained from the expression:

$$APOT = \frac{(NPT)\,(STDAPOT)}{10(CW)} \qquad ...(9.4)$$

where:

APOT	area of potholes (% of total carriageway area)
NPT	number of potholes per km
STDAPOT	Default area of a standard pothole (m^2) (default = 0.1)

Thus, by substituting NPT in Equation 9.4 above with NPT_a the value of $APOT_a$ is obtained, and by substituting NPT with dNPT the value of dAPOT is obtained;

where:

NPT_a	number of potholes per km at the start of the analysis year (n^o/km)
dNPT	unadjusted increase in the number of potholes per km during the analysis year

9.2.3 Cracking

Total area of cracking

$$ACRA_b = MAX\{0, MIN[(100 - AVPD_b), (ACRA_a + dACRA - dAVEBCR - dAPOTCR)]\} \qquad ...(9.5)$$

where:

$ACRA_b$	total area of cracking at the end of the analysis year (% of total carriageway area); that is, $ACA_b + ACT_b$
$ACRA_a$	total area of cracking at the start of the analysis year (% of total carriageway

	area); that is, $ACA_a + ACT_a$
dACRA	unadjusted increase in total area of cracking during the analysis year (% of total carriageway area); that is, dACA + dACT
$AVPD_b$	$AVEB_b + APOT_b$
dAVEBCR	increase in area of edge-break arising from cracked area during the analysis year (% of total carriageway area)
dAPOTCR	increase in area of potholes arising from cracked area during the analysis year (% of total carriageway area)
ACA_b	area of All structural cracking at the end of the analysis year (% of total carriageway area)
ACA_a	area of All structural cracking at the beginning of the analysis year (% of total carriageway area)
dACA	unadjusted increase in area of All structural cracking during the analysis year (% of total carriageway area)
ACT_b	area of transverse thermal cracking at the end of the analysis year (% of total carriageway area)
ACT_a	area of transverse thermal cracking at the start of the analysis year (% of total carriageway area)
dACT	unadjusted increase in area of transverse thermal cracking during the analysis year (% of total carriageway area)

The value of dAVEBCR is obtained as follows:

- **if ACRAa > 0**

then:

$$dAVEBCR = 0.01(VBCR)(\Delta AVEB) \quad ...(9.6)$$

otherwise:

$$dAVEBCR = 0$$

and:

$$\Delta AVEB = AVEB_b - AVEB_a$$

where:

ΔAVEB	adjusted increase in the area of edge-break during the analysis year (% of total carriageway area)
VBCR	percentage of dAVEB arising from cracked areas (default = 20)

The value of dAPOTCR is obtained as follows:

- **if dNPT > 0**

$$dAPOTCR = \frac{\Delta NPT_c\,(STDAPOT)}{10\,(CW)} \qquad ...(9.7)$$

otherwise:

$$dAPOTCR = 0$$

$$\Delta NPT_C = \left[\frac{dNPT_c}{dNPT}\right]\Delta NPT \qquad ...(9.8)$$

and:

$$\Delta NPT = NPT_b - NPT_a$$

where:

ΔNPT	adjusted total increase in the number of potholes per km during the analysis year
ΔNPT_c	adjusted increase in the number of potholes per km derived from Wide structural cracking during the analysis year
$dNPT_c$	unadjusted increase in the number of potholes per km derived from Wide structural cracking during the analysis year (see Section 7)
NPT_b	total number of potholes per km at the end of the analysis year
NPT_a	total number of potholes per km at the start of the analysis year

Other parameters are as defined previously.

All structural cracking

$$ACA_b = MIN\left[(ACA_a + \Delta ACA), ACRA_b\right] \qquad ...(9.9)$$

- **if ACRAa > 0**

$$\Delta ACA = MAX\left[0, \frac{\left[dACA - q(dAVEBCR) - dAPOTCR\right]}{(dACRA - dAVEBCR - dAPOTCR)}\Delta ACRA\right] \qquad ...(9.10)$$

- **if ACRAa = 0 and ACRAb > 0**

$$\Delta ACA = \frac{(dACA - dAPOTCR)}{(dACRA - dAPOTCR)}\Delta ACRA \qquad ...(9.11)$$

otherwise:

$$\Delta ACA = 0$$

and:

$$q = \left[1 - \frac{ACT_a}{ACRA_a}\right] \qquad ...(9.12)$$

and:

$$\Delta ACRA = ACRA_b - ACRA_a \qquad ...(9.13)$$

where:

$\Delta ACRA$ adjusted increase in total area of cracking during the analysis year (% of total carriageway area)

ΔACA adjusted increase in area of All structural cracking during the analysis year (% of total carriageway area)

Other parameters are as defined previously.

Wide structural cracking

$$ACW_b = MAX\{\,0, MIN\ [ACW_a + dACW - dAPOTCR - q\,(dAVEBCR), ACA_b\,]\,\} \qquad ...(9.14)$$

where:

ACW_b area of Wide structural cracking at the end of the analysis year (% of total carriageway area)

ACW_a area of Wide structural cracking at the start of the analysis year (% of total carriageway area)

dACW unadjusted increase in area of Wide structural cracking during the analysis year (% of total carriageway area)

Other parameters are as defined previously.

Transverse thermal cracking

- **if ACTa + dACT > 0**

then:

$$ACT_b = ACRA_b - ACA_b \qquad ...(9.15)$$

otherwise:

$$ACT_b = 0$$

All the parameters are as defined previously.

9.2.4 Ravelling

$$ARV_b = MAX\left\{0, MIN\begin{bmatrix}(100 - AVPC_b), \\ (ARV_a + dARV - dAVEBRV - dAPOTRV - dACRARV)\end{bmatrix}\right\}$$

...(9.16)

where:

ARV_b	area of ravelling at the end of the analysis year (% of total carriageway area)
ARV_a	area of ravelling at the start of the analysis year (% of total carriageway area)
dARV	unadjusted increase in area of ravelling during the analysis year (% of total carriageway area)
$AVPC_b$	$AVEB_b + APOT_b + ACRA_b$
dAVEBRV	increase in area of edge-break arising from ravelled area during the analysis year (% of total carriageway area)
dAPOTRV	increase in area of potholes arising from ravelled area during the analysis year (% of total carriageway area)
dACRARV	increase in area of cracking arising from ravelled area during the analysis year (% of total carriageway area)

Other parameters are as defined previously.

The value of dAVEBRV is obtained as follows:

- **if ARVa > 0**

  then:

$$dAVEBRV = 0.01\ (VBRV)(\Delta AVEB) \qquad ...(9.17)$$

  otherwise:

$$dAVEBRV = 0$$

  where:

VBRV	percentage of dAVEB arising from ravelled area (default = 20)

The value of dAPOTRV is obtained as follows:

- **if dNPT > 0**

  then:

$$dAPOTRV = \frac{\Delta NPT_r (STDAPOT)}{10(CW)} \qquad ...(9.18)$$

  otherwise:

$$dAPOTRV = 0$$

  and:

$$\Delta NPT_r = \left[\frac{dNPT_r}{dNPT}\right]\Delta NPT \qquad ...(9.19)$$

where:

ΔNPT_r adjusted increase in the number of potholes per km derived from ravelling during the analysis year

$dNPT_r$ unadjusted increase in the number of potholes per km derived from ravelling during the analysis year (see Section 7)

Other parameters are as defined previously.

The value of dACRARV is obtained as follows:

- **if ARVa > 0**

then:

$$dACRARV = 0.01\ (CRV)(\Delta ACRA) \qquad ...(9.20)$$

otherwise:

$$dACRARV = 0$$

where:

CRV percentage of dACRA arising from the ravelled area (default = 10)

9.3 Total damaged surface area

The total non-patched damaged surface area at any time is calculated from the following expression:

$$ADAMR_b = AVEB_b + APOT_b + ACRA_b + ARV_b \qquad ...(9.21)$$

where:

$ADAMR_b$ total non-patched damaged surface area at the end of the analysis year (% of total carriageway area)

Other parameters are as defined previously.

Severely damaged surface area that can be patched is given by the expression:

$$ADAMS_b = APOT_b + ACW_b + ARV_b \qquad ...(9.22)$$

where:

$ADAMS_b$ severely damaged surface area at the end of the analysis year (% of total carriageway area)

Other parameters are as defined previously.

10 Rut depth

Rutting is defined as the permanent or unrecoverable traffic-associated deformation within pavement layers which, if channelised into wheelpaths, accumulates over time and becomes manifested as a rut (*Paterson, 1987*).

Rut depth modelling is performed after the values of all the surface distresses (that is, cracking, ravelling, potholing and edge-break) at the end of the year have been calculated.

The rut depth model is based on four components of rutting:

- **Initial densification** (see Section 10.1)
- **Structural deformation** (see Section 10.2)
- **Plastic deformation** (see Section 10.3)
- **Wear from studded tyres** (see Section 10.4)

The rut depth at any time is the sum of the four components.

For HDM-4 the rut depths have been standardised to a 2.0 m straight-edge. Since HDM-III was based on a 1.2 m straight-edge, the default model coefficients have been changed accordingly.

10.1 Initial densification

The initial densification depends upon the degree of relative compaction of the base, sub-base and selected subgrade layers; that is, COMP. Suggested values of COMP have been given in Section 4.

The initial densification is:

$$RDO = K_{rid}\left[a_0\left(YE4\,10^6\right)^{(a_1 + a_2\,DEF)} SNP^{a_3}\ COMP^{a_4}\right] \qquad ...(10.1)$$

where:

RDO	rutting due to initial densification (mm)
YE4	annual number of equivalent standard axles (millions/lane)
DEF	average annual Benkelman beam deflection (mm)
SNP	average annual adjusted structural number of the pavement
COMP	relative compaction (%) (see Section 4)
K_{rid}	calibration factor for initial densification

The proposed default coefficient values a_0 to a_4 for the initial densification model is given in Table C2.25.

Table C2.25 Default coefficient values for initial densification model

Pavement type	a_0	a_1	a_2	a_3	a_4
AMGB, AMAB, AMSB, STGB, STAB, STSB	51740	0.09	0.0384	-0.502	-2.30
AMAP, STAP	0	0	0	0	0

Initial densification only applies to new construction or reconstruction that involves the construction of a new base layer (that is, from when AGE4 = 0), for a period of time of one year. AGE4 is defined as follows:

AGE4 age since reconstruction (including base) or new construction (years)

10.2 Structural deformation

The structural deformation model used in HDM-III has been simplified into a linear form for inclusion in HDM-4 (*Morosiuk, 1998c*). Separate terms are proposed for structural deformation without cracking and structural deformation after cracking as follows:

- **Structural deformation without cracking**

$$\Delta RDST_{uc} = K_{rst}\left(a_0\ SNP^{a_1}\ YE4^{a_2}\ COMP^{a_3}\right) \qquad ...(10.2)$$

- **Structural deformation after cracking**

$$\Delta RDST_{crk} = K_{rst}\left[a_0\ SNP^{a_1}\ YE4^{a_2}\ MMP^{a_3}\ ACX_a{}^{a4}\right] \qquad ...(10.3)$$

The total annual incremental increase in structural deformation is as follows:

- **if ACRA = 0**

then:

$$\Delta RDST = \Delta RDST_{uc} \qquad ...(10.4)$$

- **if ACRA > 0**

then:

$$\Delta RDST = \Delta RDST_{uc} + \Delta RDST_{crk} \qquad ...(10.5)$$

where:

$\Delta RDST$ total incremental increase in structural deformation in the analysis year (mm)

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

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

MMP mean monthly precipitation (mm/month)

ACX_a	area of indexed cracking at the beginning of the analysis year (% of total carriageway area)
SNP	average annual adjusted structural number of the pavement
YE4	annual number of equivalent standard axles (millions/lane)
K_{rst}	calibration factor for structural deformation

The proposed default coefficient values a_0 to a_4 for the structural deformation models are given in Table C2.26.

Table C2.26 Default 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

10.3 Plastic deformation

The plastic deformation model includes a variable, CDS, which indicates whether the surfacing is prone to plastic deformation.

A more accurate method of determining the plastic deformation of a bituminous surfacing is detailed in A Guide to Calibration and Adaptation. The method includes the use of variables to predict changes in material properties, such as the softening point of the binder and voids in the mix, to model the incremental increase in plastic deformation.

The general plastic deformation model (that is, without material properties) is given by:

$$\Delta RDPD = K_{rpd}\ a_0\ CDS^{a_1}\ \ YE4\ Sh^{a_2}\ [MIN(HS, HSLIM)]^{a_3} \qquad ...(10.6)$$

where:

ΔRDPD	incremental increase in plastic deformation in the analysis year (mm)
CDS	construction defects indicator for bituminous surfacings
YE4	annual number of equivalent standard axles (millions/lane)
Sh	speed of heavy vehicles (km/h)
	If the road section traffic does not include heavy vehicles Sh = 80 km/h will be used.
HS	total thickness of bituminous surfacing (mm)
HSLIM	Maximum thickness of bituminous surfacing in which plastic flow effects develop (mm, default 100)
K_{rpd}	calibration factor for plastic deformation

The proposed default coefficient values for the plastic deformation model are given in **Error! Reference source not found.**

Table C2.29 Default coefficient values for plastic deformation model

Surface type	a_0	a_1	a_2	a_3
AM	0.3	3.27	-0.78	0.71
ST	0.0	3.27	-0.78	0.71

10.4 Surface wear

The surface wear model (*Djarf, 1995*) is applied to environments where vehicles use studded tyres during the freezing period.

$$RDW = K_{rsw}\left[a_0\ PASS^{a_1}\ W^{a_2}\ S^{a_3}\ SALT^{a_4}\right] \qquad ...(10.7)$$

where:

ΔRDW	incremental increase in rut depth due to studded tyres in the analysis year (mm)
PASS	annual number of vehicle passes with studded tyres in one direction (1000s)
S	average traffic speed (km/h)
SALT	variable for salted or unsalted roads (2 = salted; 1 = unsalted)
W	road width (m) (carriageway plus total shoulder width)
K_{rsw}	calibration factor for surface wear

The proposed default coefficient values a_0 to a_4 for the surface wear model is given in Table C2.27.

Table C2.27 Default 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

10.5 Total rut depth

The annual incremental increase in total rut depth, ΔRDM, is derived as follows:

- if $AGE4 \leq 1$

$$\Delta RDM = RDO + \Delta RDPD + \Delta RDW \qquad ...(10.8)$$

otherwise:

$$\Delta RDM = \Delta RDST + \Delta RDPD + \Delta RDW \qquad ...(10.9)$$

where:

ΔRDM	incremental increase in total mean rut depth in both wheelpaths in the analysis year (mm)
RDO	rutting due to initial densification in the analysis year (mm)
$\Delta RDST$	incremental increase in structural deformation in the analysis year (mm)
$\Delta RDPD$	incremental increase in plastic deformation in the analysis year (mm)
ΔRDW	incremental increase in wear by studded tyres in the analysis year (mm)

The total rut depth, RDM_b, at any given time is given as:

$$RDM_b = MIN\left[(RDM_a + \Delta RDM), 100\right] \quad ...(10.10)$$

where:

RDM_b	total mean rut depth in both wheelpaths at the end of the analysis year (mm)
RDM_a	total mean rut depth in both wheelpaths at the start of the analysis year (mm)

10.6 Standard deviation of rut depth

The standard deviation of rut depth is used in the roughness model. It is calculated from the mean total rut depth as:

$$RDS_b = RDS_a + \Delta RDS \quad ...(10.11)$$

where:

RDS_b	rut depth standard deviation at the end of the analysis year (mm)
RDS_a	rut depth standard deviation at the start of the analysis year (mm)
ΔRDS	incremental change in rut depth standard deviation in analysis year (mm)

$$\Delta RDS = K_{rds} \max\left[a_0, a_1 - a_2\left(RDM_b\right)\right]\Delta RDM \quad ...(10.12)$$

where:

RDM_b	mean rut depth at end of analysis year, in mm
ΔRDM	change in mean rut depth during analysis year, in mm
K_{rds}	calibration factor for rut depth standard deviation

The coefficient values of a_0 to a_2 for the rut depth standard deviation model are given in Table C2.28.

Table C2.28: Coefficient values for RDS model

a_0	a_1	a_2
0.2	0.65	0.03

RDS_a is derived from RDS_b at the end of previous year and after works effects, if any, have been applied.

For first year of analysis:

$$RDS_a = RDS_0$$

or, by default

$$RDS_a = 0.35RDM_0 - 0.0015RDM_0^2 \qquad ...(10.13)$$

where

RDS_0 Standard deviation of rut depth at the beginning of first year of analysis, supplied by user,

RDM_0 Mean rut depth at the beginning of first year of analysis, supplied by user.

11 Roughness

The roughness model consists of several components of roughness (cracking, disintegration, deformation and maintenance). The total incremental roughness is the sum of these components. The surface distress values used in predicting roughness are those that have been adjusted so that the total damaged surface area plus the undamaged area equals 100%.

11.1 Structural

The structural component of roughness relates to the deformation in the pavement materials under the shear stresses imposed by traffic loading. It is given by:

$$\Delta RI_s = K_{gs} a_0 \exp\left(m\, K_{gm}\ AGE3\right)\left(1 + SNPK_b\right)^{-5} YE4 \qquad ...(11.1)$$

and:

$$SNPK_b = MAX\left[(SNP_a - dSNPK), 1.5\right] \qquad ...(11.2)$$

and:

$$dSNPK = K_{snpk}\ a_0 \begin{Bmatrix} MIN\,(a_1, ACX_a)\,HSNEW + \\ MAX\left[MIN\,(ACX_a - PACX, a_2), 0\right] HSOLD \end{Bmatrix} \qquad ...(11.3)$$

where:

ΔRI_s	incremental change in roughness due to structural deterioration during the analysis year (IRI m/km)
dSNPK	reduction in adjusted structural number of pavement due to cracking
$SNPK_b$	adjusted structural number of pavement due to cracking at the end of the analysis year
SNP_a	adjusted structural number of pavement at the start of the analysis year
ACX_a	area of indexed cracking at the start of the analysis year (% of total carriageway area)
PACX	area of previous indexed cracking in the old surfacing (% of total carriageway area); that is, 0.62 (PCRA) + 0.39 (PCRW)
HSNEW	thickness of the most recent surfacing (mm)
HSOLD	total thickness of previous underlying surfacing layers (mm)
AGE3	pavement age since last overlay (rehabilitation), reconstruction or new construction (years)
YE4	annual number of equivalent standard axles (millions/lane)
m	environmental coefficient (see Table C2.29)
K_{gs}	calibration factor for structural component of roughness
K_{gm}	calibration factor for environmental coefficient

K_{snpk} calibration factor for SNPK

The default values for the environmental coefficient *m* are given in Table C2.29.

Table C2.29 Roughness environmental coefficient 'm' by climate zones

Moisture classification	Temperature classification				
	Tropical	Sub-tropical hot	Sub-tropical cool	Temperate cool	Temperate freeze
Arid	0.005	0.010	0.015	0.020	0.030
Semi-arid	0.010	0.015	0.020	0.030	0.040
Sub-humid	0.020	0.025	0.030	0.040	0.050
Humid	0.025	0.030	0.040	0.050	0.060
Per-humid	0.030	0.040	0.050		

The environmental coefficient *m* value is assigned to section according to its selected climate zone which associates the moisture and temperature classification.

11.2 Cracking

The incremental change in roughness due to cracking is given by:

$$\Delta RI_c = k_{gc} a0 \ \Delta ACRA \qquad ...(11.4)$$

where:

ΔRI_c incremental change in roughness due to cracking during the analysis year (IRI m/km)

$\Delta ACRA$ incremental change in area of total cracking during the analysis year (% of total carriageway area)

k_{gc} calibration factor for the cracking component of roughness

11.3 Rutting

The incremental change in roughness due to variation of rut depth is given by:

$$\Delta RI_r = k_{gr} a_0 \ \Delta RDS \qquad ...(11.5)$$

where:

ΔRI_r incremental change in roughness due to rutting during the analysis year (m/km IRI)

ΔRDS incremental change in standard deviation of rut depth during the analysis year (mm) (= $RDS_b - RDS_a$)

k_{gr} calibration factor for the rutting component

11.4 Potholing

The potholing effect depends upon the number of vehicles that actually hit the potholes, which in turn depends upon the traffic volume and the freedom to manoeuvre. A freedom to manoeuvre variable (FM), ranging between 0 and 1, is used and is predicted using Equation 11.6 below:

$$FM = (MAX\{\ MIN[0.25\,(CW - 3)\,,1\,]\,,0\}\ )\left\{MAX\left[\left(1 - \frac{AADT}{5000}\right), 0\right]\right\} \qquad ...(11.6)$$

where:

FM freedom to manoeuvre

CW carriageway width (m)

AADT annual average daily traffic (veh/day)

A frequent patching policy has a very important effect on pavement roughness, as average number of potholes perceived by user over the year will be much less. It is assumed that the total number of potholes patched during the year is equally distributed in each of the patching campaigns.

$$PATQ = NPT_b * \frac{Ppt}{100} * \frac{Fpat}{365} \qquad ...(11.7)$$

where

PATQ Patching quantity at each patching campaign

NPT_b number of pothole units per km at end of the analysis year, including potholes patched during the analysis year,

Ppt percentage of potholes to patch over the year

Fpat number of days between two patching campaigns (365/Fpat is an integer)

The user perceived end of year value for potholes is given by:

$$NPT_{bu} = NPT_{ayn+1} + PATQ \qquad ...(11.8)$$

where

NPT_{bu} number of pothole units per km at end of the analysis year, as seen by the road user (total number of unpatched potholes at end of year)

PATQ Patching quantity at each patching campaign

NPT_{ayn+1} number of pothole units per km at start of year following the analysis year

NPT_{ayn+1} is given by:

$$NPT_{ayn+1} = NPT_b * \left(1 - \frac{Ppt}{100}\right) \qquad ...(11.9)$$

And therefore

$$NPT_{bu} = NPT_b * \left[1 - \frac{Ppt}{100}\left(1 - \frac{Fpat}{365}\right)\right] \qquad ...(11.10)$$

The change in roughness is calculated as follows:

$$\Delta RI_p = k_{gp} a_0 \,(a_1 - FM)\left[NPT_{bu}{}^{a2} - NPT_a{}^{a2}\right] \qquad ...(11.11)$$

where:

FM freedom to manoeuvre

ΔRI_p incremental change in roughness due to potholing during the analysis year (IRI m/km)

NPT_a number of potholes per km at the start of the analysis year

NPT_{bu} number of potholes per km at the end of the analysis year, as seen by the road user

k_{gp} calibration factor for potholing component of roughness

11.5 Environment

The environmental component of roughness is due to factors which include temperature and moisture fluctuations, and also foundation movements (for example, subsidence). It is given by:

$$\Delta RI_e = m * K_{gm} \, RI_a \qquad ...(11.12)$$

where:

ΔRI_e incremental change in roughness due to the environment during the analysis year (IRI m/km)

RI_a roughness at the start of the analysis year (IRI m/km)

m environmental coefficient

K_{gm} calibration factor for the environmental component

11.6 Total change in roughness

The total change in the roughness of the pavement is derived as:

$$\Delta RI = [\Delta RI_s + \Delta RI_c + \Delta RI_r + \Delta RI_t] + \Delta RI_e \qquad ...(11.13)$$

where:

ΔRI total incremental change in roughness during the analysis year (IRI m/km)

The default coefficient values for the various roughness components are given in Table C2.30.

The roughness of the pavement at the end of the analysis year is given by:

$$RI_b = MIN[(RI_a + \Delta RI), a_0] \qquad ...(11.14)$$

where:

RI_b roughness of the pavement at the end of the analysis year (IRI m/km)

RI_a roughness of the pavement at the start of the analysis year (IRI m/km)

a_0 upper limit of pavement roughness, specified by the user (default = 16 IRI m/km)

Table C2.30 Default coefficient values for roughness components

Pavement type	Roughness component	Equation	a_0	a_1	a_2
All pavement types	Structural	11.1	134		
	dSNPK	11.3	0.0000758	63.0	40.0
	Cracking	11.4	0.0066		
	Rutting	11.5	0.088		
	Potholing	11.7	0.00019	2.0	1.5

The annual average roughness of the pavement for a given analysis year is calculated as:

$$RI_{av} = 0.5\,(RI_a + RI_b) \qquad ...(11.15)$$

where:

RI_{av} annual average roughness of the pavement for the analysis year (IRI m/km). This is the roughness used in the Road User Effects model.

12 Pavement surface texture

Pavement texture is perhaps the most important variable which determines the magnitude of longitudinal and lateral forces at the tyre-road interface. 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. The changes in macrotexture due to wear and compaction from traffic action have important safety and economic consequences since rolling resistance is a function of texture.

12.1 Texture depth

This refers to the macrotexture of pavement. *Cenek and Griffith-Jones (1997)* proposed an incremental macro-texture model that can be expressed as:

$$\Delta TD = K_{td}\left\{ITD - TD_a - a_0\, ITD \log_{10}\left(10^{\left[(ITD - TD_a)/(a_0\, ITD)\right]} + \Delta NELV\right)\right\} \qquad ...(12.1)$$

where:

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

The proposed default coefficient values for a_0 for the texture depth model are given in Table C2.31. This table also includes values for the initial texture depth (ITD) which is used as default when resetting pavement surface type. These can be replaced by user definable values.

Table C2.31 Default parameter values for texture depth model

Surface type	Surface material	Parameter	
		ITD	a_0
AM	AC	0.7	0.005
	HRA	0.7	0.005
	PMA	0.7	0.005
	RAC	0.7	0.005
	CM	0.7	0.005
	SMA	0.7	0.005
	PA	1.5	0.008
ST	SBSD	2.5	0.120
	DBSD	2.5	0.120
	CAPE	0.7	0.006
	SL	0.7	0.006
	PM	1.5	0.008

The texture depth at the end of the analysis year is given by the following relationship:

$$TD_b = MAX[(TD_a + \Delta TD), 0.1] \qquad ...(12.2)$$

where:

TD_b texture depth at the end of the analysis year (mm)

TD_a texture depth at the start of the analysis year (mm)

ΔTD incremental change in texture depth during the analysis year (mm)

The annual average texture depth for a given analysis year is calculated as follows:

$$TD_{av} = 0.5\,(TD_a + TD_b) \qquad ...(12.3)$$

where:

TD_{av} annual average texture depth for the analysis year (mm). This is the texture depth used in the Road User Effects model.

12.2 Skid resistance

This is strongly influenced by the microtexture, which is a measure of the degree of polishing of a pavement surface or of the aggregate and the surface. The proposed skid resistance model is as follows:

$$\Delta SFC_{50} = K_{sfc}\ a_0\ MAX[0, \Delta QCV] \qquad ...(12.4)$$

where:

ΔSFC_{50} incremental change in sideway force coefficient during the analysis year, measured at 50 km/h

ΔQCV annual incremental increase in the flow of commercial vehicles (veh/lane/day)

K_{sfc} calibration factor for skid resistance

The proposed default coefficient values for a_0 for the skid resistance model are given in Table C2.32.

Table C2.32 Default coefficient values for skid resistance model

Surface type	Surface material	Coefficient a_0
AM	AC	-0.663×10^{-4}
	HRA	-0.663×10^{-4}
	PMA	-0.663×10^{-4}
	RAC	-0.663×10^{-4}
	CM	-0.663×10^{-4}
	SMA	-0.663×10^{-4}
	PA	-0.663×10^{-4}
ST	SBSD	-0.663×10^{-4}
	DBSD	-0.663×10^{-4}
	CAPE	-0.663×10^{-4}
	SL	-0.663×10^{-4}
	PM	-0.663×10^{-4}

The skid resistance measured at 50 km/h at the end of the analysis year is given by the following expression:

$$SFC_{50b} = MAX[(SFC_{50a} + \Delta SFC_{50}), 0.35] \qquad ...(12.5)$$

where:

SFC_{50b} sideway force coefficient, measured at 50 km/h, at the end of the analysis year

SFC_{50a} sideway force coefficient, measured at 50 km/h, at the start of the analysis year

ΔSFC_{50} incremental change in sideway force coefficient, measured at 50 km/h, during the analysis year

The annual skid resistance value for a given analysis year is calculated as follows:

$$SFC_{50av} = 0.5\,(SFC_{50a} + SFC_{50b}) \qquad ...(12.6)$$

where:

SFC_{50av} annual average sideway force coefficient, measured at 50 km/h, for the analysis year

All the other parameters are as defined previously.

The average skid resistance value at a given annual average traffic speed is calculated as follows:

$$SFCs = K_{sfcs}\left[\frac{SFC_{50av}\left\{400 - \left[2 - MIN\left(TD_{av}, 2\right)\right]\left[MAX\left(50, S\right) - 50\right]\right\}}{400}\right] \qquad ...(12.7)$$

where:

SFCs sideway force coefficient at an average traffic speed of S km/h

TD_{av} annual average texture depth for the analysis year (mm)

S average traffic speed (km/h)

K_{sfcs} calibration factor for skid resistance speed effects

The user needs to define a value of SFC_{50} in order for skid resistance modelling to be performed. This will also need to be supplied after maintenance treatments.

13 Calibration factors

The deterioration models contain calibration factors to facilitate local calibration. These factors have default values of 1.0 and are summarised in Table C2.33.

Table C2.33 Calibration factors used in the deterioration models

Deterioration model	Calibration factor
Wet/dry season SNP ratio	K_f
Drainage deterioration factor	K_{ddf}
Drain life factor	K_{drain}
All structural cracking - initiation	K_{cia}
Wide structural cracking - initiation	K_{ciw}
All structural cracking - progression	K_{cpa}
Wide structural cracking - progression	K_{cpw}
Transverse thermal cracking - initiation	K_{cit}
Transverse thermal cracking - progression	K_{cpt}
Rutting - initial densification	K_{rid}
Rutting - structural deterioration	K_{rst}
Rutting - plastic deformation	K_{rpd}
Rutting - surface wear	K_{rsw}
Rutting - calibration factor for rut depth standard deviation	K_{rds}
Ravelling - initiation	K_{vi}
Ravelling - progression	K_{vp}
Pothole – initiation due to cracking	K_{pic}
Pothole – initiation due to ravelling	K_{pir}
Pothole - progression	K_{pp}
Edge-break	K_{eb}
Roughness - environmental coefficient	K_{gm}
Roughness - SNPK	K_{snpk}
Roughness - structural	K_{gs}
Roughness – cracking	K_{gc}
Roughness – rutting	K_{gr}
Roughness – potholing	K_{gp}
Texture depth - progression	K_{td}
Skid resistance	K_{sfc}
Skid resistance - speed effects	K_{sfcs}

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C3 Concrete Pavements

1 Introduction

The prediction models for concrete pavement deterioration included in HDM-4 are based on research carried out by the Latin American Study Team, in Chile (1996). The research involved a comprehensive bibliographical review of the existing concrete pavement deterioration models. The models implemented in HDM-4 are mainly based on previous work carried out by *SHRP (1993)*, *Al-Omari and Darter (1994)*, *Lee and Darter (1994)* and *ERES Consultants (1995)*.

The process of model selection considered the following aspects:

- **Validity range of the model**

  Modelled in terms of types of climate, traffic range, and pavement structure.

- **Statistical parameters**

  Number of observations, correlation coefficients, estimated errors, etc.

- **Sensitivity analysis**
- **Year of model development**

This chapter describes the Road Deterioration (RD) models for concrete pavements included in HDM-4 (see Figure C3.1). It commences with an overview of the modelling framework, followed by a brief analysis of the concrete pavement types and distress modes considered, and finally a complete description of the models.

It should be noted that the RD models for concrete pavements are basically absolute models (as opposed to incremental models used for bituminous pavements). The models have been developed in imperial units. However, for consistency within HDM-4 user-interface, the data required for modelling is initially input in metric. This data is then converted into imperial units for use in the relationships, and then the results of calculations are converted back into metric for reporting purposes.

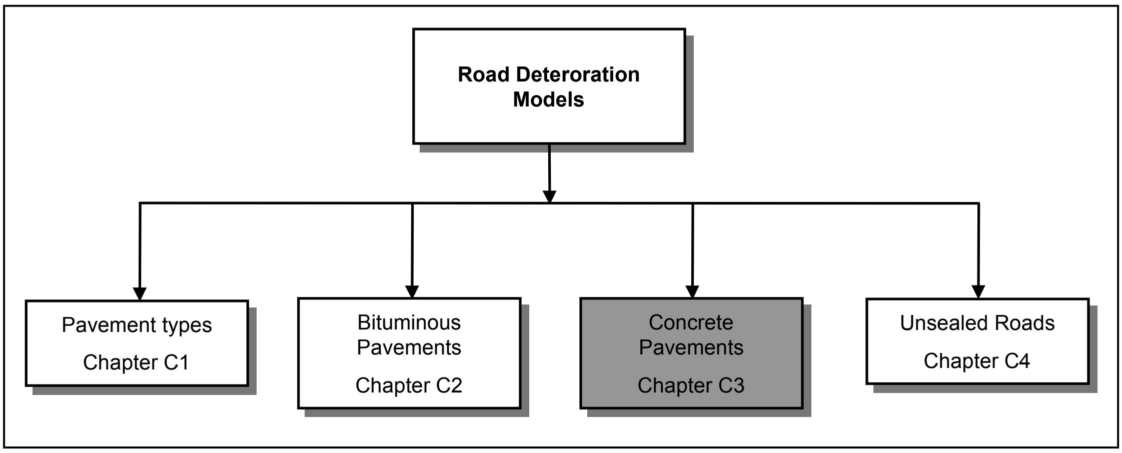

Figure C3.1 Road Deterioration Modules

2 Modelling framework and logic

The framework used for concrete pavement modelling conforms to the general HDM-4 pavement classification system described in Chapter C1. This is a versatile framework that is able to handle a wide range of pavement types. The formal structure of concrete pavement classification is shown in Table C3.1.

Table C3.1 Structures of concrete pavements

Surface type	Base type	Pavement type	Description
JP	GB	JPGB	Jointed Plain Concrete over Granular Base
JP	AB	JPAB	Jointed Plain Concrete over Asphalt Base
JP	AP	JPAP	Jointed Plain Concrete over Asphalt Pavement
JP	SB	JPSB	Jointed Plain Concrete over Stabilised Base
JP	RB	JPRB	Jointed Plain Concrete over Rigid/concrete Base
JR	GB	JRGB	Jointed Reinforced Concrete over Granular Base
JR	AB	JRAB	Jointed Reinforced Concrete over Asphalt Base
JR	AP	JRAP	Jointed Reinforced Concrete over Asphalt Pavement
JR	SB	JRSB	Jointed Reinforced Concrete over Stabilised Base
JR	RB	JRRB	Jointed Reinforced Concrete over Rigid/concrete Base
CR	GB	CRGB	Continuously Reinforced Concrete over Granular Base
CR	AB	CRAB	Continuously Reinforced Concrete over Asphalt Base
CR	AP	CRAP	Continuously Reinforced Concrete over Asphalt Pavement
CR	SB	CRSB	Continuously Reinforced Concrete over Stabilised Base
CR	RB	CRRB	Continuously Reinforced Concrete over Rigid/concrete Base

There are different sets of deterioration models for concrete pavements included in HDM-4 that are based on pavement surface type and construction type (see Table C3.2). Calibration parameters have also been provided to account for variations in surface material and to facilitate local adaptation of the models.

Table C3.2 Surface types upon which the concrete RD models are based

Surface type	Description
JP	Jointed Plain concrete pavement - **without** load transfer dowels
JP	Jointed Plain concrete pavement - **with** load transfer dowels
JR	Jointed Reinforced concrete pavement
CR	Continuously Reinforced concrete pavement

The modelling of concrete pavement performance is considered in two separate phases:

- **Phase 1**

  Refers to the time before any major periodic maintenance or reconstruction.

- **Phase 2**

  Refers to the time after the pavement has received a major maintenance or has been reconstructed.

Phase 1 models are described in this chapter; Phase 2 models are discussed in Chapter D3.

2.1 Concrete pavement structure

In rigid pavement roads, the thickness of concrete slab often ranges from 15 cm for light traffic to 30 cm for heavy traffic. Thicker slabs in excess of 28 cm may be constructed without a base course. A brief description of the concrete pavement structures considered in HDM-4 is given below.

2.1.1 Jointed plain concrete pavements without load transfer dowels

This type of JP concrete pavement (JPCP n/d) is built using short slabs without reinforcement steel (see Figure). Spacing between transverse joints (or slab length) is such that the strains induced by changes in temperature and/or moisture content do not produce intermediate cracking between the joints. The maximum spacing between joints is limited to minimise slab movement and maximise load transfer. Typical values of slab length vary between 3.0 and 6.0 metres for this type of pavement. Transverse load transfer from one slab to the next one is accomplished through aggregate interlock.

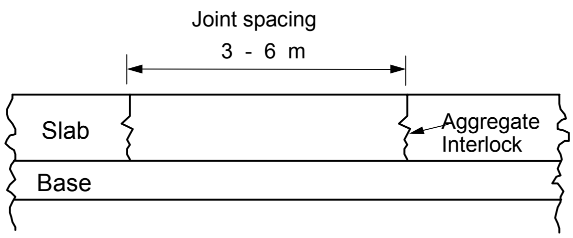

Figure C3.2 Jointed plain concrete pavements without dowels

2.1.2 Jointed plain concrete pavements with load transfer dowels

This type of JP concrete pavement is similar to (JPCPn/d) described above, except that dowel bars are added in the transverse joints to help load transfer (see Figure).

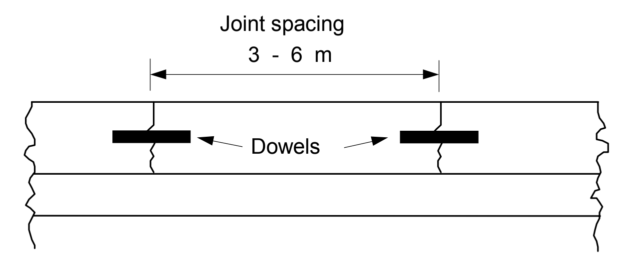

Figure C3.3 Jointed plain concrete pavements with dowels

2.1.3 Jointed reinforced concrete pavements

This type of concrete pavement is designed with a quantity of longitudinal reinforcement steel, which permits longer slab lengths, between 10 and 20 m (see Figure). Reinforcement steel control transverse cracking that could occur due to movements of the foundation subgrade, and/or strains produced by temperature or humidity changes. Load transfer in transverse joints is accomplished through load transfer dowels.

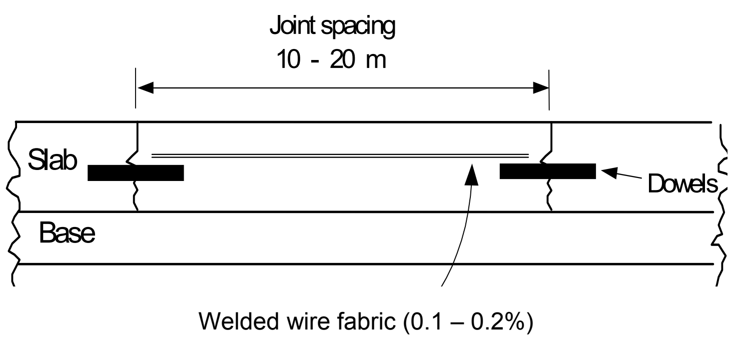

Figure C3.4 Jointed reinforced concrete pavements

2.1.4 Continuously reinforced concrete pavements

This type of concrete pavement has longitudinal reinforcement throughout its length; therefore, it has no transverse joints. The objective of the longitudinal reinforcement steel is to control the cracks that are produced in the pavement due to shrinkage in the concrete (see Figure).

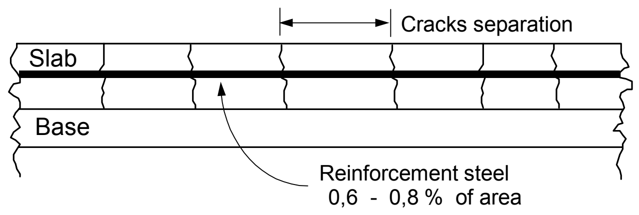

Figure C3.5 Continuously reinforced concrete pavements

2.2 Concrete pavement distress modes

There are six concrete pavement distress modes modelled in HDM-4. These distresses together with the pavement surface type to which they apply are presented in Table C3.3.

Table C3.3 Distress modes modelled in HDM-4

No.	Distress mode	Units of measurement	Pavement surface type
1	Cracking	Percent of slabs cracked	JP
		Number per mile	JR
2	Faulting	inches	JP and JR
3	Spalling	Percent of spalled joints	JP and JR
4	Failures	Number per mile	CR
5	Serviceability loss	Dimensionless	JR and CR
6	Roughness	Inches per mile (or m/km)	JP, JR and CR

Each of these distress modes is described in the following sections (2.2.1 - 2.2.6).

2.2.1 Cracking

There are three types of cracking distress that are commonly identified on concrete pavements:

1 **Transverse cracking**

2 **Longitudinal cracking**

3 **Durability cracks**

Transverse cracks are predominantly perpendicular to the central axis of the road (see Figure). They manifest three severity levels, according to *SHRP (1993)*:

- **Low**

  Cracks with a width of less than 3 mm, without visible spalling or faulting; or well sealed, with a non-determinable width.

- **Medium**

  Cracks with a width between 3 and 6 mm, or with spalling less than 75 mm, or faulting less than 6 mm.

- **High**

  Cracks with a width greater than 6 mm, or spalling greater than 75 mm, or faulting greater than 6 mm.

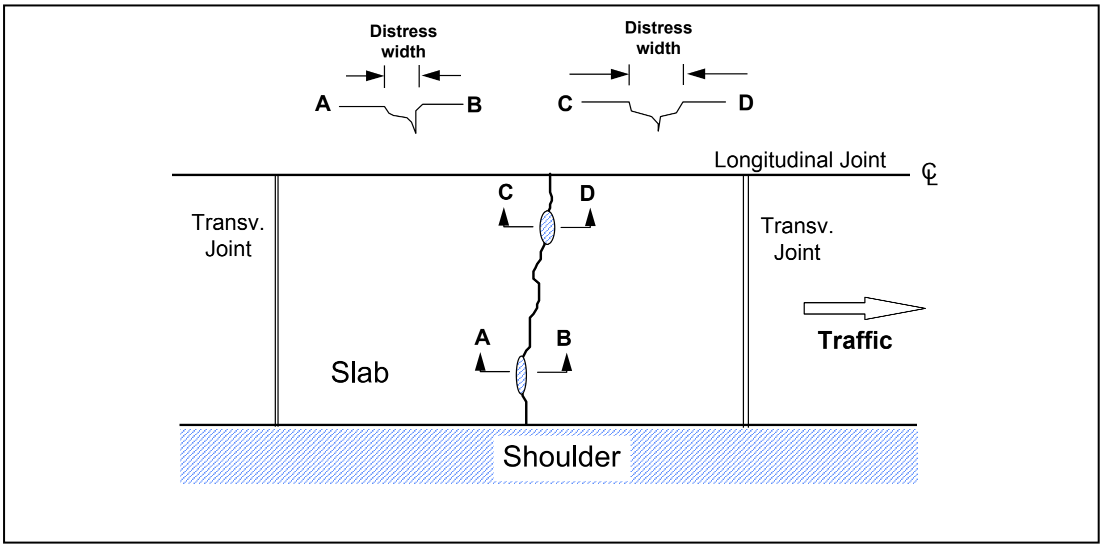

Figure C3.6 Transverse cracking

Transverse cracking may have significant impact on the riding quality and are therefore modelled in HDM-4.

Longitudinal cracks are predominantly parallel to the axis of the road. Durability cracks are fine cracks, slightly spaced, and often occur adjacent to joints, cracks, or free edges. They begin in the slab corner as a group of obscure cracks just as in the surrounding area.

2.2.2 Faulting of transverse joints and cracks

This distress refers to a joint or crack having a difference in elevation between both sides of the joint or crack (see Figure). Faulting causes significant increases in road roughness. Faulting is measured as the average fall of all transverse joints within the pavement section under consideration.

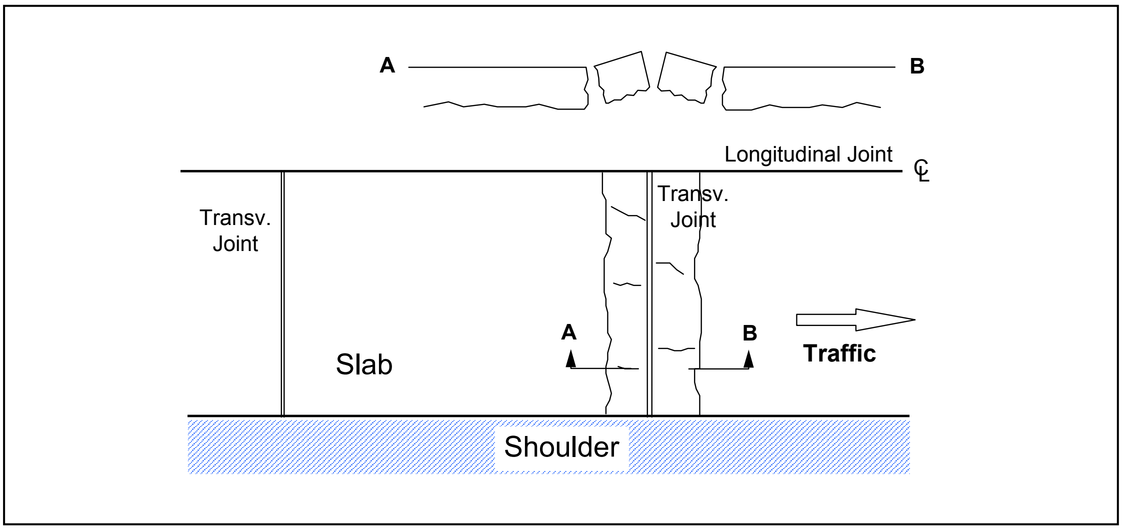

Figure C3.7 Faulting of transverse joints and cracks

2.2.3 Spalling of transverse joints

These are breaks or cracks of the joint edge, occurring within a maximum distance of 0.6 m from the transverse joint (see Figure). Spalling occurs on Jointed Plain and Jointed Reinforced concrete pavements. They manifest three severity levels, according to *SHRP (1993)*:

- **Low**

  Spalling of less than 75 mm of distress width, measured from centre of the joint, with or without loss of material.

- **Medium**

  Spalling of between 75 and 150 mm of distress width, measured from centre of the joint, with loss of material.

- **High**

  Spalling of greater than 150 mm of distress width, measured from the centre of the joint, with loss of material.

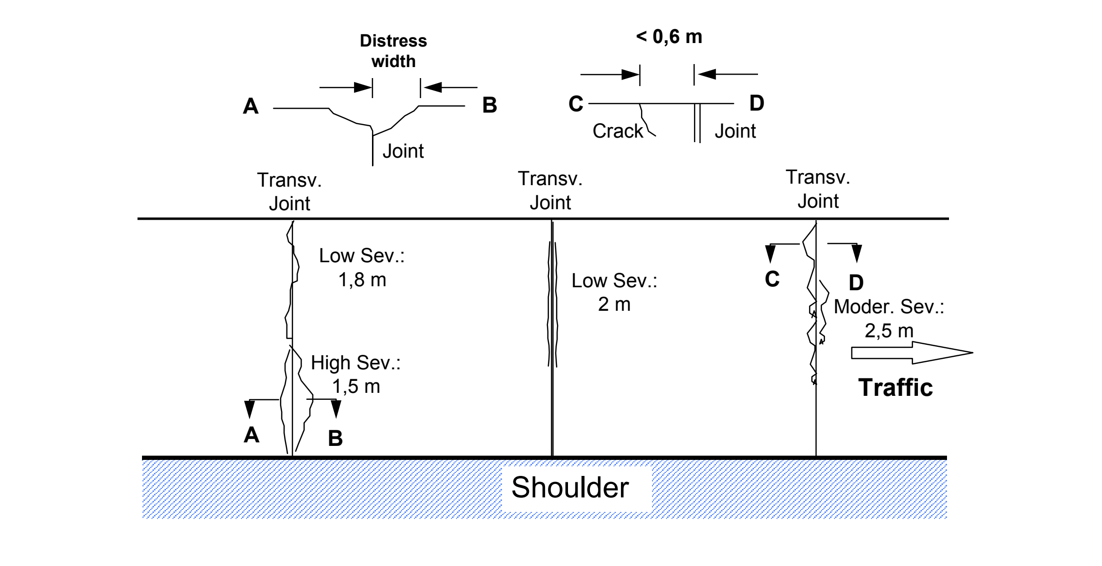

Figure C3.8 Spalling of transverse joints

2.2.4 Failures

This distress is a major defect that occurs in Continuously Reinforced (CR) concrete pavements. Located failures include loosening and breaking of reinforcement steel and transverse crack spalling. Failures are measured in number per mile (or km).

Many of the maintenance activities in CR concrete pavements are directly related to failures. Often, it is necessary to estimate the quantity of these distresses for the purpose of preventive designs and rehabilitation planning.

2.2.5 Serviceability loss

Present Serviceability Rating (PSR) is a subjective user rating of the existing ride quality of pavement condition. The ratings based on key distress types (for example, transverse distortions, cracking, spalling, faulting and surface deterioration) range from 0 for extremely poor condition to 5 for extremely good condition, as shown in Table C3.4. This concept for rating pavement surface condition was developed by engineers at the *AASHO Road Test (1962)*, and it has since been correlated with various roughness indicators, such as slope variance and International Roughness Index (IRI).

Table C3.4 Scale of PSR values used

PSR	Condition
0 - 1	Very poor
1 - 2	Poor
2 - 3	Fair
3 - 4	Good
4 - 5	Very good

2.2.6 Roughness

This is a measure of the deviations of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads and drainage (*ASTM E-867-82A*) - typically in the ranges of 0.1 to 100 m wavelength and 1 to 100 mm amplitude.

International Roughness Index (IRI), is the reference measure expressing roughness as a dimensionless average rectified slope statistic of the longitudinal profile and defined in *Sayers et al. (1986)*.

2.2.7 Other defects

There are several other defects on concrete pavements that include the following:

- **Scaling**
- **Polishing of aggregate**
- **Popouts**
- **Blow-up**
- **Punchouts**
- **Water bleeding and pumping**
- **Patch-deterioration**
- **Deterioration of transverse joints**

2.3 Primary modelling parameters

The primary variables used for modelling the performance of concrete pavements may be considered under pavement structural characteristics, condition, history, traffic, road geometry and the environment. The road characteristics at the beginning of the analysis year is initialised either from input data if it is the first year of the analysis or the first year after construction, or otherwise from the result of the previous year's maintenance and improvement works.

Pavement structural characteristics

These include measures of pavement strength, slab thickness, material types and properties, amount of reinforcement steel, the presence of tied concrete shoulders and widened outside lanes, and subgrade stiffness. These parameters are described in detail in Section 3.

Road condition

The pavement and side-drain condition data at the beginning of the first analysis year or the first year after construction are required as inputs. The data for pavement condition is as described above in Section 2.2.

The average pavement condition indicators in a given analysis year (that is, before road works) are predicted using absolute models. Absolute models predict condition (or distress) at a particular point in time as a function of the independent variables, and can be represented as follows:

$$(\mathrm{CONDITION})_t = f[\,(\mathrm{TIME}),(\mathrm{TRAFF}),(\mathrm{STRENGTH}),(\mathrm{ENVIRON}),\mathrm{ETC.}] \qquad ...(2.1)$$

where:

$(CONDITION)_t$	condition at time *t*
(TIME)	time since the construction year of pavement
(TRAFF)	cumulative traffic loading since the construction year of pavement
(STRENGTH)	pavement strength parameters
(ENVIRON)	Environment/climate related parameters

Pavement history

The required data items refer to pavement age and the year of previous major maintenance and construction works carried out on the pavement.

Road geometry and environment

The key road geometry data required are carriageway and shoulder widths. Several environment-related parameters are used for concrete pavement deterioration modelling. These include the mean annual precipitation, freezing index, Thornthwaite moisture index, temperature range, and number of days with temperature greater than 32°C (90°F). These parameters are described in detail in Chapter C1.

Traffic

The required traffic data are the annual flow of equivalent standard axle loads (ESAL) and the cumulative equivalent standard axle loads (NE4), both expressed in millions per lane. These data are calculated for each analysis year based on the user-specified traffic and vehicle characteristics.

2.4 Computational procedure

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

1. Initialise input data and the conditions at the beginning of the year
2. Convert input data from metric to imperial units
3. Compute pavement strength parameters
4. Calculate the amount of each distress mode in the analysis year, in the following order depending on the pavement surface type:
   (a) Cracking
   (b) Faulting
   (c) Spalling
   (d) Failures
5. Calculate present serviceability rating (PSR) if pavement type is JR or CR
6. Calculate average roughness value for the analysis year
7. Store results in imperial units for use in Works Effects (WE) module and in the following analysis year

8 Convert the required outputs into metric for use in the RUE and SEE modules and for reporting

3 Structural characteristics

3.1 Introduction

This section describes the principal pavement structural data that are necessary to predict the deterioration of concrete pavements. These include the following:

- **Properties of the materials** (see Section 3.2)
- **Drainage conditions** (see Section 3.3)
- **Percentage of reinforcement steel** (see Section 3.4)
- **Load transfer efficiency** (see Section 3.5)
- **Widened outside lanes** (see Section 3.6)

3.2 Properties of the materials

- **Modulus of elasticity of concrete** (E_c)

  The Modulus of elasticity of concrete denoted by E_c can be obtained from an analysis of deflection measures or from a laboratory testing (for example, according to the procedure described in *ASTM C469*). It can also be estimated from a correlation with the compressive strength of concrete expressed by Equation 3.1 below *(Pauw, 1960)*:

  $$E_c = 57000(f'_c)^{0.5} \qquad \text{...(3.1)}$$

  where:

E_c	elasticity Modulus of concrete (psi)
f'_c	compressive strength of concrete, in psi, as determined using procedures *AASHTO (T22-92)*, *AASHTO (T140-92)* or *ASTM C39*

  The value of the Modulus of elasticity of concrete used in the pavement deterioration models is 5,000,000 psi

- **Modulus of Rupture of concrete** (MR28)

  Stresses in concrete pavements are mainly caused by the effects of traffic and environmental action. The Modulus of Rupture is a measure of the concrete flexural strength in providing a sustained resistance to the stresses. During the useful life of the pavement the stress levels may exceed the Modulus of Rupture at certain points, causing fatigue damage and cracking in the slabs.

  The Modulus of Rupture measured after 28 days and denoted by MR28 can be determined using *AASHTO T97* and *ASTM C78* procedures, or estimated from the compressive strength of concrete, as follows:

  $$MR28 = RUP * \left(f'_c\right)^{0.5} \qquad \text{...(3.2)}$$

  where:

MR28	modulus of Rupture of concrete after 28 days (psi)
f'_c	compressive strength of concrete, in psi, as determined using procedures *AASHTO (T22-92)*, *AASHTO (T140-92)* or *ASTM C39*
RUP	model parameter (varies between 8 and 10, default = 9)

The Modulus of Rupture can also be estimated using the modulus of elasticity of concrete, which can be obtained from Falling Weight Deflectometer (FWD) test results or from laboratory testing. The empirical equation (*Foxworthy, 1985*) for estimating the Modulus of Rupture is as follows:

$$MR = 43.5 * \left(\frac{E_c}{10^6} \right) + 488.5 \qquad \text{...(3.3)}$$

where:

MR	modulus of Rupture of concrete (psi)
E_c	modulus of Elasticity of concrete (psi)

The deterioration models for concrete pavements consider the modulus of rupture (MR) in the long term. The long-term value is estimated by increasing the modulus of rupture at 28 days (MR28) by 11%.

- **Thermal coefficient of concrete** (α)

The thermal coefficient of expansion is used to determine the warping (or curling) stresses produced in a concrete pavement when it is subjected to a temperature difference between the top and the bottom of the slab. The stresses are greatest at the edges of the slab, and may result in slab cracking usually near its mid point.

The thermal coefficient of expansion varies with such factors as water to cement ratio, concrete age, richness of the mix, relative humidity, and the type of aggregate in the mix. Table C3.5 shows typical values of the **Thermal coefficient of concrete** according to the type of aggregate. A value of 5.5 x 10^{-6} per ºF is commonly used in concrete pavement analysis.

Table C3.5 Typical values of the thermal coefficient of concrete

Type of aggregate	Thermal coefficient of concrete (α) (10^{-6} per ºF)
Quartz	6.6
Sandstone	6.5
Gravel	6.0
Granite	5.3
Basalt	4.8
Limestone	3.8

Source: *AASHTO (1993)*

- **Drying shrinkage coefficient of concrete** (γ)

Concrete pavement slabs are subjected to daily variations in temperature and humidity with associated expansion or contraction effects. A slab with unrestricted movement in the horizontal direction would not develop stresses under the effects of expansion and contraction. However, in real site conditions there exists some resistance between the slab and the underlying base.

Shrinkage in concrete pavements is caused by the loss of water in the drying process. The drying shrinkage coefficient (γ) is used in the evaluation of the opening and closing of the joints caused by variations in the mean temperature to which the slabs are subjected.

The shrinkage and strength of concrete are strongly dependent upon the water to cement ratio. High values of water to cement ratio will reduce the strength of the concrete and increase the drying shrinkage potential. Shrinkage can therefore be considered to be inversely proportional to the indirect tensile strength of the concrete. Table C3.6 can be used to estimate the drying shrinkage coefficient of concrete.

Table C3.6 Approximate relationship between shrinkage coefficient and indirect tensile strength of Portland cement concrete

Indirect tensile strength (psi)	Shrinkage coefficient (in/in)
300 (or less)	0.0008
400	0.0006
500	0.00045
600	0.0003
700 (or greater)	0.0002

Source: *AASHTO (1993)*

- **Poisson's ratio for concrete** (μ)

For most cement treated materials, the value of μ normally varies between 0.10 and 0.25, with 0.15 generally accepted as a representative value.

- **Modulus of elasticity of dowel bars** (E_s)

Dowel bars may be used to transfer (or distribute) load across discontinuities such as transverse joints. The value of the Modulus of elasticity of load transfer dowels assigned within the deterioration model is $2.9*10^7$ psi (or $2.0*10^5$ MPa).

- **Modulus of elasticity of bases** (E_{base})

The stiffness of base influences the overall behaviour of concrete pavements, mainly as a result of the support provided to the slabs. Drainage effects also have a significant influence on the behaviour of the base, as discussed below. A more rigid base will generally provide better support to the slab, and this should reduce the occurrence of faulting at transverse joints. However, a very rigid base may increase the warping effect induced by changes in temperature or humidity, and transverse cracking will increase. Table C3.7 gives typical values of Modulus of elasticity for different base types.

Table C3.7 Elasticity Modulus by base type

Base type	Elasticity Modulus, Ebase (in psi)
Granular (GB)	25,000
Asphalt treated (AB)	600,000
Cement treated (SB)	400,000
Lean concrete (RB)	1,000,000

Source: *AASHTO (1993)*

The effects of a stabilised base is considered in the cracking model, see Section 4.

- **Modulus of subgrade reaction (KSTAT)**

The modulus of reaction of a material is an elastic constant that defines the stiffness of the material or resistance to deformation, under certain loading conditions. The Modulus of subgrade reaction (KSTAT) is defined by Equation 3.4 below:

$$KSTAT = \frac{RPRESS}{DEF} \qquad ...(3.4)$$

where:

KSTAT	modulus of subgrade reaction (pci)
RPRESS	reactive pressure (psi)
DEF	deflection of the plate (inches)

The value of KSTAT can be determined through the plate load test, where the deflection is the displacement of a circular plate of 30 inches diameter subjected to a static pressure. KSTAT is expressed in pounds per cubic inch (pci). The assumption associated with the determination of the value of KSTAT is that the plate is in complete contact with the subgrade soil and that the soil is elastic.

3.3 Drainage conditions

It is widely recognised that drainage is a major factor that influences the performance of many concrete pavements. Water which infiltrates through cracks and joints in a concrete slab often results in the loss of uniform subgrade support and in pavement faulting due to redistribution of base/sub-base material.

The effect of drainage on concrete pavement performance is incorporated in the HDM-4 deterioration models through the use of a drainage coefficient (Cd). The drainage coefficient is defined by the quality of drainage and the percentage of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation (*AASHTO, 1986*). The quality of drainage is based on the speed at which water is removed from the pavement structure, and is determined by:

The time that a base needs for draining 50% of the free saturation water.

This is equivalent to the saturation time (T_{50}) given in Table C3.8, and the associated values for the drainage coefficient Cd are given in Table C3.9 (*AASHTO, 1986*).

Table C3.8 Relationship between drainage time and the quality of drainage

Drainage quality	Free water removed within, (T_{50})
Excellent	2 hours
Good	1 day
Fair	7 week
Bad	1 month
Very bad	(water will not drain)

Source: *AASHTO (1993)*

Table C3.9 Recommended values of the drainage coefficient (Cd) for concrete pavements

Drainage quality	Percentage of time that the pavement structure is exposed to humidity levels near to saturation			
	Less than 1%	1 - 5%	5 - 25%	Greater than 25%
Excellent	1.25 - 1.20	1.20 - 1.15	1.15 - 1.10	1.10
Good	1.20 - 1.15	1.15 - 1.10	1.10 - 1.00	1.00
Regular	1.15 - 1.10	1.10 - 1.00	1.00 - 0.90	0.90
Bad	1.10 - 1.00	1.00 - 0.90	0.90 - 0.80	0.80
Very Bad	1.00 - 0.90	0.90 - 0.80	0.80 - 0.70	0.70

Source: *AASHTO (1993)*

Table C3.10 shows a simplified matrix that can be used to estimate the drainage coefficient (*FWHA, 1995*).

Table C3.10 Modified AASHTO simplified matrix for drainage coefficient (Cd)

Lateral drains	Precipitation levels	Fine subgrade		Coarse subgrade	
		Impermeable base	Permeable base	Impermeable base	Permeable base
No	Wet (humid)	0.85 - 0.95	0.70 - 0.90	0.75 - 0.95	0.90 - 1.00
	Dry (arid)	0.95 - 1.05	0.90 - 1.10	0.90 - 1.15	1.00 - 1.10
Yes	Wet (humid)	1.00 - 1.10	0.75 - 0.95	0.90 - 1.10	1.05 - 1.15
	Dry (arid)	1.10 - 1.20	0.95 - 1.15	1.10 - 1.20	1.15 - 1.20

Source: *FHWA (1995)*

Notes:

1	(a)	Coarse Subgrade	Groups A-1 to A-3
	(b)	Fine Subgrade	Groups A-4 to A-7 , according to AASHTO Soil Classification System
2		Permeable Base	k = 1000 ft/day (305 m/day) or Cu $\leq$ 6
3	(a)	Wet (humid) Climate	Precipitation >25 in/year (635 mm/year)
	(b)	Dry (arid) Climate	Precipitation $\leq$ 25 in/year (635 mm/year)

3.4 Percentage of reinforcement steel

The purpose of distributed steel reinforcement in reinforced concrete pavement is to resist cracking due to induced tensile stresses and to reduce the opening of any cracks that may form, thus maintaining the pavement as an integral unit. The amount of reinforcement required is expressed as a percentage of concrete cross-sectional area, denoted as PSTEEL.

The requirement of steel reinforcement in concrete pavement varies between JR and CR type construction.

3.5 Load transfer efficiency

3.5.1 Efficiency of load transfer in the transverse joints

The effective transfer of traffic loads from one slab to another reduces tensile stress levels in the slabs and the associated deformations of the slabs at the joints. This situation helps to decrease deterioration by reducing pumping, loss of support and breaking of slab edges. Load transfer through transverse joints can be effected through dowel bars, aggregate interlock or a combination of both mechanisms.

Load transfer in the joints can be evaluated with equipment such as the FWD, by registering the deformations from both the loaded and unloaded sides of the joint. The percentage of load transferred across a joint, denoted by LT, is expressed as follows:

$$LT = \left(\frac{DEF_{unld}}{DEF_{load}}\right) * 100 \qquad ...(3.5)$$

where:

LT	percentage of load transferred across a joint
DEF_{unld}	deflection in the unloaded side of the joint (inches)
DEF_{load}	deflection in the loaded side of the joint (inches)

The efficiency of load transfer is used in the calculation of the maximum bearing stress of the dowel-concrete system. Theoretically, if a dowel is 100% efficient it is capable of assigning half of the applied load to each adjacent slab. However, a reduction in load transfer efficiency would occur over the pavement life, either due to the loss of bond in the zone where the load transfer device is imbedded in the concrete slab or due to the deterioration of the aggregate interlock mechanism. Generally, the reduction in load transfer efficiency increases as traffic loads increase since aggregate load transfer decreases with load repetitions. The reduction in the load transfer efficiency can be assumed to be around 5% to 10%, therefore the value of LT used in the deterioration model is 45%.

3.5.2 Efficiency of load transfer between slab and shoulder

Tied concrete shoulders contribute substantially to improve the overall performance of the pavement, by providing a reduction in slab stress and an increase in the service life. These effects are considered in the cracking model through the efficiency of load transfer (LTE_{sh}) between the slab and shoulder defined in terms of stress. The variable LTE_{sh} is given by the expression:

$$LTE_{sh} = \left(\frac{STRESS_{unld}}{STRESS_{load}}\right) * 100 \qquad \text{...(3.6)}$$

where:

LTE_{sh}	efficiency of load transfer between slab and shoulder (%)
$STRESS_{unld}$	stress in the unloaded side of the joint (psi)
$STRESS_{load}$	stress in the loaded side of the joint (psi)

If tied concrete shoulders are provided in the original pavement construction, a value for LTE_{sh} = 20% should be used. If the shoulders are provided on an existing pavement the value of LTE_{sh} should be taken as 10%.

3.6 Widened outside lanes

This refers to an original construction that incorporates a wider lane (or standard lane with hard strip) adjacent to the shoulders. The main benefit associated with the provision of a wider outer lane is stress reduction at the outer edge of the slab since wheel loads are kept at a distance from the pavement edge.

The effects of widened outside lanes on concrete pavement performance are considered in both the cracking and faulting models.

4 Cracking

The HDM-4 cracking model considers transverse cracking in concrete pavements due to high stress levels in the slabs or defects originating from material fatigue. The stresses are caused generally by the combined effect of thermal curling, moisture-induced curling and traffic loading.

Separate relationships are given for predicting the amounts of transverse cracking over the life cycle of Jointed Plain concrete pavements and Jointed Reinforced concrete pavements. The models are deterministic and predict the expected average deterioration based on the input variables.

4.1 Jointed plain concrete pavements

Transverse cracking is modelled as a function of cumulative fatigue damage in the slabs (*ERES Consultants, 1995*).

The percentage of slabs cracked is given by:

$$PCRACK = Kjp_c * \frac{100}{1 + 1.41 * FD^{-1.66}} \qquad ...(4.1)$$

where:

PCRACK	percent of slabs cracked
FD	cumulative fatigue damage, dimensionless
Kjp_c	calibration factor (default = 1.0)

4.1.1 Fatigue damage determination

The cumulative fatigue damage is calculated in terms of Miner's damage analysis, by summing the damage index over each slab thermal condition or temperature gradient and axle load distribution as follows:

$$FD = \sum_{tg=1}^{G} \frac{n_{tg}}{N_{tg}} \qquad ...(4.2)$$

where:

FD	cumulative fatigue damage
tg	temperature gradient (*tg* = 1, …., G)
n_{tg}	number of 18 kip equivalent single axle load passes during temperature gradient *tg* (ESALs per lane)
N_{tg}	maximum number of 18 kip equivalent standard axle load repetitions during temperature gradient *tg* before flexural failure occurs (ESALs per lane)

According to Miner's theory, transverse cracking is expected to occur when the cumulative fatigue damage (FD) approaches 1.0.

- **Temperature gradients**

  The variations in concrete pavement temperature over the year can be represented by a distribution of temperature gradients. The average temperature gradient is defined as the difference between the temperature at the top and at the bottom of the slab divided by the slab thickness. A positive gradient indicates the top of the slab is warmer than the bottom, which normally occurs during the daytime. Negative gradient condition typically occurs during the cooler hours of the evening. In all types of climate the positive temperature gradients occur with greater frequency than the negative temperature gradients.

  Owing to the difficulty that may be experienced in obtaining field data on temperature gradient distribution, a default data set based on climate zones is provided in HDM-4 as illustrated in Table C3.11.

Table C3.11 Temperature gradient distribution

Temperature difference	Frequency (FREQ)			
(ΔT) in °F	Dry with freezing	Dry without freezing	Wet with freezing	Wet without freezing
-8	0.086660	0.073237	0.090494	0.086209
-6	0.092003	0.067994	0.094611	0.072691
-4	0.076447	0.057834	0.081522	0.052129
-2	0.058163	0.039585	0.067007	0.039496
0	0.057014	0.031803	0.052426	0.033466
2	0.034749	0.029573	0.036817	0.030790
4	0.036162	0.024472	0.039393	0.031347
6	0.037122	0.019472	0.033196	0.021113
8	0.031273	0.021223	0.033254	0.024858
10	0.036200	0.028565	0.032462	0.032160
12	0.021978	0.027069	0.026291	0.025427
14	0.037272	0.029359	0.034706	0.038571
16	0.026134	0.036464	0.029423	0.037274
18	0.032394	0.030194	0.034758	0.038976
20	0.033724	0.037439	0.032034	0.038803
22	0.023131	0.032684	0.017874	0.037385
24	0.009683	0.036172	0.006422	0.027180
26	0.000047	0.024021	0.000078	0.011631
28	0.000000	0.013717	0.000000	0.001188

Note: The frequencies do not add up to 1.0 because the data relative to temperature differences of negative 8 ºF do not provide significant (meaningful) information to the concrete models.

There are factors other than temperature that cause curling (which may be concave) in slabs. A correction to the difference in temperature measured in the slab is applied according to climate zones as follows, *Eisenmann and Leykauf (1990)*:

$$\Delta T_s = \Delta T - \left[a0 + \frac{a1 * (SLABTHK - 2)}{SLABTHK^3} \right] \qquad ...(4.3)$$

where:

ΔT_s adjusted difference in temperature at the top and bottom of the slab (ºF)

ΔT difference between the temperature measured at the top and bottom of the slab (ºF)

(= T_{top} - T_{bottom})

SLABTHK slab thickness (inches)

a0 and a1 model coefficients based on climate zones

The model coefficient values are given in Table C3.12.

Table C3.12 Model coefficient for temperature correction

Climate type	a0	a1
Dry with freezing	6.29	436.36
Dry without freezing	7.68	436.36
Wet with freezing	5.03	327.27
Wet without freezing	6.66	218.18

- **Distribution of total traffic loading according to temperature gradients**

The total traffic loading since the construction of pavement is distributed over the temperature gradients as follows:

$$n_{tg} = \frac{NE4}{LCR_{tg}} * FREQ_{tg} \qquad ...(4.4)$$

where:

n_{tg} number of 18 kip equivalent standard axle load passes during temperature gradient *tg* (ESALs per lane)

NE4 cumulative number of ESALs since construction of pavement, in millions 18-kip axles per lane

$FREQ_{tg}$ frequency of each temperature gradient *tg*

LCR_{tg} lateral coverage ratio of traffic, for temperature gradient *tg*

- **Determination of the lateral coverage ratio of traffic**

The lateral coverage ratio of traffic (LCR) is simply a measure of the likelihood of the wheel loading passing through the critical edge location. The edge loading location is considered critical for jointed plain concrete pavements as this is the location of the maximum stress and will be the point of crack initiation.

Assuming that the average location of vehicle wheels is 22 inches from the edge of the slab, with a standard deviation of 8.4 inches, the following regression equation can be used to calculate LCR:

$$LCR_{tg} = 418.9 - 1148.6 * SR_{tg} + 1259.9 * SR_{tg}^{2} - 491.55 * SR_{tg}^{3} \qquad ...(4.5)$$

$$SR_{tg} = \frac{SIGMA_{tg}}{MR} \qquad ...(4.6)$$

where:

LCR_{tg}	lateral coverage ratio of traffic, for temperature gradient *tg*
SR_{tg}	ratio between combined stress in slab and the Modulus of Rupture of concrete, for temperature gradient *tg*
$SIGMA_{tg}$	combined stress in the slab edge due to loading and curling for temperature gradient *tg* (psi)
MR	modulus of Rupture of concrete (psi)

- **Maximum number of N_{tg}**

The maximum number of load repetitions to the failure of concrete slab (N) applied during temperature gradient *tg* depends on the induced stress level, and is calculated through the law of fatigue as follows:

$$Log_{10}(N_{tg}) = 2.13 * SR_{tg}^{-1.2} \qquad ...(4.7)$$

where:

N_{tg}	maximum number of 18 kip equivalent standard axle load repetitions during temperature gradient *tg* before flexural failure occurs (ESALs per lane)
SR_{tg}	ratio between combined stress in slab and the Modulus of Rupture of concrete, for temperature gradient *tg*

4.1.2 Calculation of stresses

The combined stress due to curling and loads, for each temperature gradient *tg*, is obtained from Equation 4.8 below:

$$SIGMA_{tg} = f_{SB} * \left(\sigma_{load(tg)} + R_{tg} * \sigma_{curl(tg)} \right) \qquad ...(4.8)$$

where:

$SIGMA_{tg}$	combined stress in slab edge for temperature gradient *tg* (psi)
f_{SB}	adjustment factor for stabilised bases
$\sigma_{load(tg)}$	stress in slab edge due to traffic loading (psi)
R_{tg}	regression coefficient
$\sigma_{curl(tg)}$	stress in slab edge due to curling (psi)

- **Calculation of load induced stress**

The stress at the slab edge produced by traffic loads, for each temperature gradient, is expressed as follows:

$$\sigma_{load} = f_{ES} * f_{WL} * \sigma_e \qquad ...(4.9)$$

where:

σ_{load}	stress in slab edge due to traffic loading (psi)
f_{ES}	adjustment factor for edge support (for example, shoulder)
F_{WL}	adjustment factor for widened outside lanes
σ_e	edge stress obtained from Westergaard's equations (psi)

- **Calculation of edge stress** (σ_e)

Edge stress in the slab is calculated using Westergaard's equation (*Westergaard, 1948*) for a circular load, in which the load application radius for a simple axle is replaced by the equivalent radius for a single axle dual wheel, as follows:

$$\sigma_e = \frac{3 * (1+\mu) * P}{\pi(3+\mu) * SLABTHK^2}\left[\ln\left(\frac{E_c * SLABTHK^3}{100 * KSTAT * a_{eq}^4}\right) + 1.84 - \frac{4\mu}{3} + \frac{1-\mu}{2} + 1.18 * (1+2\mu)\frac{a_{eq}}{\ell}\right] \qquad ...(4.10)$$

where:

σ_e	edge stress obtained from Westergaard's equation (psi)
μ	Poisson's ratio
P	total load applied by each wheel of a single-axle dual wheel (lb), default = 9000
SLABTHK	slab thickness (inches)
E_c	modulus of elasticity of concrete (psi)
KSTAT	modulus of subgrade reaction (pci)
a_{eq}	equivalent load application radius for a dual-wheel single axle (inches)
ℓ	radius of relative stiffness of the slab-foundation system (inches)

The equivalent load application radius (a_{eq}) is calculated from the following expression:

$$\frac{a_{eq}}{a} = \begin{bmatrix} 0.909 + 0.339485 * \left(\frac{SP}{a}\right) + 0.103946 * \left(\frac{a}{\ell}\right) - 0.017881 * \left(\frac{SP}{a}\right)^2 - 0.045229 * \left(\frac{SP}{a}\right)^2 * \frac{a}{\ell} \\ + 0.000436 * \left(\frac{SP}{a}\right)^3 - 0.301805 * \frac{SP}{a} * \left(\frac{a}{\ell}\right)^3 + 0.034664 * \left(\frac{SP}{\ell}\right)^2 + 0.001 * \left(\frac{SP}{a}\right)^3 * \frac{a}{\ell} \end{bmatrix}$$

...(4.11)

Limits: $0 \leq SP/a \leq 20$

$0 \leq a/\ell \leq 0.5$

where:

a_{eq}	equivalent load application radius for a dual-wheel single axle (inches)
a	load application radius for a single-wheel single axle, in inches. This is given by the square root of ($P/\pi*p$)
p	tyre pressure (psi)
SP	spacing between central wheels of dual wheel single axle (inches)
ℓ	radius of relative stiffness of the slab-foundation system (inches)

The radius of relative stiffness of the slab-foundation system is calculated from the following expression:

$$\ell = \left[\frac{E_c * SLABTHK^3}{12 * (1 - \mu^2) * KSTAT}\right]^{0.25}$$ ...(4.12)

where:

ℓ	radius of relative stiffness of the slab-foundation system (inches)
E_c	modulus of elasticity of concrete (psi)
SLABTHK	slab thickness (inches)
μ	Poisson's ratio
KSTAT	modulus of subgrade reaction (pci)

- **Calculation of the adjustment factor for shoulders** (f_{ES})

  In pavement sections with concrete shoulders or other forms of edge support (such as kerb), load stress should be multiplied by the edge support adjustment factor, calculated as follows:

$$f_{ES} = \frac{100}{100 + LTE_{sh}} \qquad \text{...(4.13)}$$

where:

f_{ES}	adjustment factor for the edge support
LTE_{sh}	efficiency of load transfer between slab and edge support (for example, shoulder), (%) Default: = 20 if concrete shoulders are placed during initial construction = 10 if concrete shoulders are placed after initial construction

- **Calculation of the adjustment factor for widened outside lanes** (f_{WL})

  In sections with widened lanes, the load stress should be multiplied by an adjustment factor, calculated as follows (*Benekohal et al., 1990*):

$$f_{WL} = 0.454147 + \frac{0.013211 * \ell}{DW} + 0.386201 * \left(\frac{a}{DW}\right) - 0.24565 * \left(\frac{a}{DW}\right)^2 + 0.053891 * \left(\frac{a}{DW}\right)^3 \qquad \text{...(4.14)}$$

where:

f_{WL}	adjustment factor for widened outside lanes
ℓ	radius of relative stiffness of the slab-foundation system (inches)
DW	average wheels location, given by the average distance of the exterior wheel to slab edge (inches)
a	load application radius for single-wheel single axle (inches)

- **Calculation of stresses produced by curling**

  Curling stress is given by Equation 4.15 below:

$$\sigma_{curl} = \frac{COEF * E_c * \alpha * \Delta T_s}{2} \qquad \text{...(4.15)}$$

where:

σ_{curl}	stress in slab edge due to curling (psi)
COEF	curling stress coefficient
E_c	modulus of elasticity of concrete (psi)
α	thermal coefficient of concrete (default = $5.5*10^{-6}$), (per ºF)
ΔT_s	adjusted difference in temperature at the top and bottom of the slab (ºF)

The curling stress coefficient (COEF) in Equation 4.15 above is obtained from Equation 4.16 below developed by *Westergaard (1926)* and *Bradbury (1938)*:

$$\mathrm{COEF} = 1 - \left[\frac{2 * \cos(\lambda) * \cosh(\lambda)}{(\sin(2\lambda) + 2 * \sinh(\lambda) * \cosh(\lambda))}\right] * \left[\tan(\lambda) + \left(\frac{\sinh(\lambda)}{\cosh(\lambda)}\right)\right] \qquad ...(4.16)$$

and:

$$\lambda = \frac{12 * \mathrm{JTSPACE}}{\ell * \sqrt{8}} \qquad ...(4.17)$$

where:

λ	intermediate parameter expressed in sexagesimal degrees
JTSPACE	average transverse joint spacing (ft)
ℓ	radius of relative stiffness of the slab-foundation system (inches)

- **Calculation of the regression coefficient** (R)

The load induced stresses and curling stresses cannot be added directly since curling produces a debonding effect between the slab and the base. This effect is taken into account by the regression coefficient (R_{tg}) in Equation 4.8 above.

The regression coefficient is calculated for each temperature gradient using the following equation:

$$R = \begin{bmatrix} 86.97 * Y^3 - \left(1.051 * 10^{-9} * E_c * dT * KSTAT + 1.7487 * dT\right) * Y^2 \\ - \left(1.068 - 0.387317 * dT - 1.84 * 10^{-11} * E_c * dT^2 * KSTAT + 8.16396 * dT^2\right) * Y \\ + \left(1.062 - 1.5757 * 10^{-2} * dT - 8.76 * 10^{-5} * KSTAT + (1.17 - 0.181 * dT) * 10^{-11} * E_c * dT * KSTAT\right) \end{bmatrix} \qquad ...(4.18)$$

The intermediate parameters Y and dT in Equation 4.18 above are expressed as follows:

$$Y = \frac{12 * \mathrm{JTSPACE}}{100 * \ell} \qquad ...(4.19)$$

$$dT = \alpha * \Delta T_s * 10^5 \qquad ...(4.20)$$

where:

KSTAT	modulus of subgrade reaction (pci)
E_c	modulus of elasticity of concrete (psi)
JTSPACE	average transverse joint spacing (ft)
ℓ	radius of relative stiffness of the slab-foundation system (inches)

α thermal coefficient of concrete (per ºF)

ΔT_s adjusted difference in temperature at the top and bottom of the slab (ºF)

- **Calculation of the adjustment factor for stabilised bases** (f_{SB})

The effect of stabilised bases on the performance of concrete pavement structure is considered in Equation 4.8 above through the adjustment factor f_{SB}. The adjustment factor is based on the effective slab thickness, which represents the equivalent thickness of a plain concrete slab that would give the same structural response of the current pavement (that is, the slab and the base).

The adjustment factor is calculated as follows:

$$f_{SB} = \frac{2 * (SLABTHK - NAXIS)}{EFFETHK} \quad \text{...(4.21)}$$

where:

f_{SB} adjustment factor for stabilised bases

= 1.0 if EFFETHK is equal to SLABTHK

SLABTHK slab thickness (inches)

NAXIS location of the neutral axis

EFFETHK effective slab thickness (inches)

The location of the neutral axis (NAXIS) and the effective slab thickness (EFFETHK) are calculated from Equations 4.22 below and 4.23 below, respectively:

$$NAXIS = \left[\frac{0.5 * SLABTHK^2 + \frac{E_{base}}{E_c} * BASETHK * (SLABTHK + 0.5 * BASETHK)}{SLABTHK + \frac{E_{base}}{E_c} * BASETHK} \right]$$

...(4.22)

$$EFFETHK = \left[SLABTHK^2 + BASETHK^2 * \frac{E_{base} * BASETHK}{E_c * SLABTHK} \right]^{0.5}$$

...(4.23)

where:

SLABTHK slab thickness (inches)

E_{base} modulus of elasticity of stabilised base (psi)

E_c modulus of elasticity of concrete (psi)

BASETHK thickness of the stabilised base (inches)

4.1.3 Key factors

The following factors have significant effects on the propagation of transverse cracking:

- Slab thickness
- Joint spacing
- Concrete flexural strength
- Climate/environment

4.2 Jointed reinforced concrete pavements

Low severity transverse cracks usually occur in JR concrete pavements due to shrinkage, curling and contractions of concrete caused by variations in the mean temperature. The reinforcement steel in a JR concrete pavement holds the cracks tightly closed and ensures load transfer by aggregate interlock, thus reducing distress progression. However, crack propagation may increase due to repetitive traffic loading and environmental effects (leading to corrosion of reinforcement). Only medium and high severity transverse cracks in JR concrete pavements are modelled in HDM-4, since these types may increase road roughness significantly.

The number of deteriorated transverse cracks per mile is given by the following relationship (*ERES Consultants, 1995*):

$$DCRACK = Kjr_c * AGE^{2.5} * \begin{bmatrix} 6.88 * 10^{-5} * FI/SLABTHK + NE4 * (0.116 - 0.073 * BASE) \\ * (1 - \exp(-0.032 * MI) \\ * \exp[7.5518 - 66.5 * PSTEEL - (1 - 5 * PSTEEL) * E_c * 10^{-6}] \end{bmatrix}$$

...(4.24)

where:

DCRACK	number of deteriorated transverse cracks per mile
AGE	number of years since pavement construction
FI	freezing Index (ºF-days)
SLABTHK	slab thickness (inches)
NE4	cumulative ESALs since pavement construction (millions 18-kip axles per lane)
BASE	base type: 0 if non stabilised 1 if stabilised
MI	Thornthwaite moisture index
PSTEEL	percentage of longitudinal steel reinforcement
E_c	modulus of elasticity of concrete (psi)
Kjr_c	calibration factor (default = 1.0)

Note that this model does not use the spacing between joints to predict crack deterioration. The model highlights the following:

- Crack deterioration increases with pavement age and traffic.
- A significant increase in the quantity of longitudinal steel reinforcement (greater than 0.15%) reduces the number of deteriorated cracks.
- Crack deterioration occurs at a higher rate in cold and wet climates (higher values of MI).
- Stabilised bases gives less cracking than non-stabilised bases.
- Crack deterioration can be reduced by using higher strength concrete or by increasing slab thickness.

5 Faulting

Faulting is caused by the loss of fine material under a slab and the increase in fine material under nearby slabs. This flow of fine material is called **pumping**, and is caused by the presence of high levels of free moisture under a slab carrying heavy traffic loading. The effects of thermal and moisture-induced curling and lack of load transfer between slabs increase pumping.

The HDM-4 pavement deterioration model considers faulting in jointed plain concrete pavements (with and without load transfer dowels) and jointed reinforced concrete pavements as described in Sections 5.1, 5.2 and 5.3.

5.1 JP concrete pavements without load transfer dowels

The relationship for modelling transverse joint faulting in jointed plain concrete pavements without load transfer dowels is as follows (*ERES Consultants, 1995*):

$$FAULT = Kjpn_f * NE4^{0.25} * \begin{bmatrix} 0.2347 - 0.1516 * Cd - 0.00025 * \left(SLABTHK^2 / JTSPACE^{0.25}\right) \\ -\left(0.0115 * BASE + 7.78 * 10^{-8} * FI^{1.5} * PRECIP^{0.25}\right) \\ -\left(0.002478 * DAYS90^{0.5} - 0.0415 * WIDENED\right) \end{bmatrix} \quad ...(5.1)$$

where:

FAULT	average transverse joint faulting (inches)
NE4	cumulative ESALs since pavement construction (millions 18-kip axles per lane)
Cd	drainage coefficient, modified AASHTO
SLABTHK	slab thickness (inches)
JTSPACE	average transverse joint spacing (ft)
BASE	base type:
	0 if not stabilised
	1 if stabilised
FI	freezing Index (ºF-days)
PRECIP	annual average precipitation (inches)
DAYS90	number of days with mean temperature greater than 90ºF
WIDENED	widened lane:
	0 if not widened
	1 if widened
$Kjpn_f$	calibration factor (default = 1.0)

The following design characteristics can be used to reduce faulting in jointed plain concrete pavements without load transfer dowels:

- **Provision of better drainage conditions**

  For example, use of longitudinal drains, and more permeable bases.

- **Use of stabilised bases (or lean concrete bases)**
- **Use of widened lanes or concrete shoulders**
- **Provide shorter joint spacing or use thicker slabs**

Since this model predicts the average faulting, it is recommended that the critical level of intervention should be set fairly low (at around 0.07 in) to provide some safety factor. In situations where faulting in JP concrete pavements without load transfer dowels is predicted to be excessive, the use of load transfer dowels has to be considered.

5.2 JP concrete pavements with load transfer dowels

The use of dowels as a load transfer mechanism reduces transverse joint faulting in concrete pavements. Transverse joint faulting in JP concrete pavements with load transfer dowels is predicted as described below, (*ERES Consultants, 1995*):

$$FAULT = Kjpd_f * NE4^{0.25} * \begin{bmatrix} 0.0628 * (1 - Cd) + 3.673 * 10^{-9} * BSTRESS^2 \\ + (4.116 * 10^{-6} * JTSPACE^2 + 7.466 * 10^{-10} * FI^2 * PRECIP^{0.5}) \\ - (0.009503 * BASE - 0.01917 * WIDENED + 0.0009217 * AGE) \end{bmatrix} \quad ...(5.2)$$

where:

FAULT	average transverse joint faulting (inches)
NE4	cumulative ESALS since pavement construction (millions 18-kip axles per lane)
Cd	drainage coefficient, modified AASHTO
BSTRESS	maximum concrete bearing stress, in the dowel-concrete system (psi)
JTSPACE	average transverse joint spacing (ft)
FI	freezing Index (ºF-days)
PRECIP	annual average precipitation (inches)
BASE	base type:
	0 if not stabilised
	1 if stabilised
WIDENED	widened lane:
	0 if not widened
	1 if widened or shoulders provided during initial construction
	0.5 if concrete shoulders are placed after initial construction

AGE number of years since pavement construction

$Kjpd_f$ calibration factor (default = 1.0)

The value of the maximum concrete bearing stress (BSTRESS) has a significant impact on the predictions of the faulting model, and is calculated as follows (*Heinrichs et al., 1989*):

$$BSTRESS = \frac{DFAC * P * LT * Kd * (2 + BETA * OPENING)}{4 * E_s * INERT * BETA^3} \quad ...(5.3)$$

where:

BSTRESS maximum concrete bearing stress, in the dowel-concrete system (psi)

DFAC distribution factor, given by $24/(\ell + 12)$

ℓ radius of relative stiffness of the slab-foundation system (inches). This is calculated using Equation 4.12 above

P total load applied by each wheel of a single-axle dual wheel (lb) (default = 9,000)

LT percentage of load transfer between joints (default = 45)

Kd modulus of dowel support, in pci (default = $1.5*10^6$ psi/in)

BETA relative stiffness of the dowel-concrete system

OPENING average transverse joint opening (inches)

E_s modulus of elasticity of dowel bar (psi)

INERT moment of inertia of the transverse section of the dowel bar (in^4)

The relative stiffness of the dowel-concrete system (BETA) is obtained from Equation 5.4 below:

$$BETA = \left[\frac{Kd * DOWEL}{4 * E_s * INERT}\right]^{0.25} \quad ...(5.4)$$

where:

BETA relative stiffness of the dowel-concrete system

Kd modulus of dowel support (pci) (default = $1.5*10^6$ psi/in)

DOWEL dowel diameter (inches)

E_s modulus of elasticity of dowel bar (psi)

INERT moment of Inertia of the transverse section of the dowel bar (in^4)

The average transverse joint opening (OPENING) is given by:

$$OPENING = 12 * CON * JTSPACE * \left[\left(\frac{\alpha * TRANGE}{2} \right) + \gamma \right] \quad ...(5.5)$$

where:

BETA relative stiffness of the dowel-concrete system

CON adjustment factor due to base/slab frictional restraint:

0.80 if non stabilised base

0.65 if stabilised base

JTSPACE average transverse joint spacing (ft)

α thermal coefficient of concrete (per °F)

TRANGE temperature range (the mean monthly temperature range obtained from data on the difference between the maximum and the minimum temperature for each month) (°F)

γ drying shrinkage coefficient of concrete

The moment of inertia of the dowel bar (INERT) is given by:

$$INERT = 0.25 * \pi * \left(\frac{DOWEL}{2} \right)^4 \quad ...(5.6)$$

where:

INERT moment of Inertia of the transverse section of the dowel bar (in^4)

DOWEL dowel diameter (inches)

Based on model results, the following design characteristics can reduce faulting:

- **Use of load transfer dowels of greater diameter to reduce the stress levels at the dowel-concrete support system**
- **Provision of better drainage conditions**

  For example, use of longitudinal drains or permeable bases.
- **Use of widened outside lanes**
- **Use of concrete shoulders**
- **Shorter spacing between transverse joints**

Faulting in JP concrete pavements is more likely to occur in cold and wet climates than in warm climates.

5.3 Jointed reinforced concrete pavements

The relationships for modelling faulting in jointed reinforced concrete pavements are the same as those used for jointed plain concrete pavements with load transfer dowels with the exception of the calibration factor Kjr_f.

$$FAULT = Kjr_f * NE4^{0.25} * \begin{bmatrix} 0.0628 * (1 - Cd) + 3.673 * 10^{-9} * BSTRESS^2 \\ + (4.116 * 10^{-6} * JTSPACE^2 + 7.466 * 10^{-10} * FI^2 * PRECIP^{0.5}) \\ - (0.009503 * BASE - 0.01917 * WIDENED + 0.0009217 * AGE) \end{bmatrix}$$

...(5.7)

where:

FAULT	average transverse joint faulting (inches)
NE4	cumulative ESALS since pavement construction (millions 18-kip axles per lane)
Cd	drainage coefficient, modified AASHTO
BSTRESS	maximum concrete bearing stress, in the dowel-concrete system (psi)
JTSPACE	average transverse joint spacing (ft)
FI	freezing Index (ºF-days)
PRECIP	annual average precipitation (inches)
BASE	base type:
	0 if not stabilised
	1 if stabilised
WIDENED	widened lane:
	0 if not widened
	1 if widened or shoulders provided during initial construction
	0.5 if concrete shoulders are placed after initial construction
AGE	number of years since pavement construction
Kjr_f	calibration factor (default = 1.0)

The following design characteristics can reduce faulting in JR concrete pavements and JP concrete pavements with dowels:

- **Use of load transfer dowels of greater diameter to reduce the stress levels at the dowel-concrete support system**
- **Provision of better drainage conditions**

  For example, use of longitudinal drains or permeable bases.
- **Use of widened outside lanes**
- **Shorter spacing between transverse joints**
- **Use of stabilised bases**

Note that the model does not include the effects of shoulder types.

6 Spalling

Transverse joint spalling is the cracking or breaking of the edge of the slab up to a maximum of 0.6 metres from the joint. Spalling generally does not extend through the whole thickness of the slab, but intercepts the joint at an angle. Transverse joint spalling can be caused by a variety of factors including:

- **Presence of incompressible materials**

  The presence of incompressible materials in the joint which produces excessive stress in the joint. This produces a fracture or detachment of the joint edges when the slab expands in warm conditions.

- **Disintegration of concrete under high traffic loads**
- **Improper consolidation of the concrete in the joint**
- **Wrongly designed or built load transfer system**

The HDM-4 model considers medium and high severity spalled transverse joints.

6.1 Jointed plain concrete pavements

Transverse joint spalling in jointed plain concrete pavements is predicted using Equation 6.1 below (*ERES Consultants, 1995*):

$$SPALL = Kjp_s * AGE^2 * JTSPACE * 10^{-6} * \begin{bmatrix} 549.9 - 895.7 * (LIQSEAL + PREFSEAL) \\ + 1.11 * DAYS90^3 * 10^{-3} + 375 * DWLCOR \\ + (29.01 - 27.6 * LIQSEAL) * FI \\ - (28.59 * PREFSEAL + 27.09 * SILSEAL) * FI \end{bmatrix} \quad ...(6.1)$$

where:

SPALL	percentage of spalled transverse joints
AGE	age since pavement construction (years)
JTSPACE	average transverse joint spacing (ft)
LIQSEAL	presence of liquid sealant in joint:
	0 if not present
	1 if present
PREFSEAL	presence of pre-formed sealant in joint:
	0 if not present
	1 if present
DAYS90	number of days with temperature greater than 90ºF

DWLCOR dowel corrosion protection:

0 if no dowels exist, or are protected from corrosion

1 if dowels are not protected from corrosion

FI freezing Index (ºF-days)

SILSEAL presence of silicone sealant in joint:

0 if not present

1 if present

Kjp_s calibration factor (default = 1.0)

The following observations can be made regarding the behaviour of the JP concrete pavement model:

- **The transverse joint spalling increases with the pavement age to the second power**
- **Pre-formed seals are more effective in reducing joint spalling than other types of seals considered**
- **Transverse joints without seals, show a great quantity of spalling**
- **Liquid sealant gives a better performance than the silicon seals**
- **An increase in joint spacing increases the percentage of spalled joints**
- **An appropriate dowel protection against corrosion reduces joint spalling**

The model also shows that the ageing of JP concrete pavements has a significant effect on joint spalling. Effective maintenance strategies that include joint cleaning and resealing at regular intervals can reduce the ageing effect, and therefore, reduce joint spalling significantly.

6.2 Jointed reinforced concrete pavements

Transverse joint spalling in jointed reinforced concrete pavements is predicted using Equation 6.2 below (*ERES Consultants, 1995*):

$$SPALL = Kjr_s * AGE^3 * JTSPACE * 10^{-5} * \begin{bmatrix} 1.94 * DWLCOR + 8.819 * BASE * (1 - PREFSEAL) \\ + 7.01 * FI * 10^{-3} \end{bmatrix} \quad ...(6.2)$$

where:

SPALL percentage of spalled transverse joints

AGE age since pavement construction (years)

JTSPACE average transverse Joint Spacing (ft)

DWLCOR dowel corrosion protection:

0 if no dowels exist, or are protected from corrosion

1 if dowels are not protected from corrosion

BASE	base type:
	0 if not stabilised
	1 if stabilised
PREFSEAL	presence of pre-formed sealant in joint:
	0 if not present
	1 if present
FI	freezing Index (ºF-days)
Kjr_s	calibration factor (default =1.0)

The following observations can be made regarding the behaviour of the JR concrete pavement model:

- **The percentage of spalled joints increases rapidly with the age of the pavement**
- **Transverse joint spalling is more likely to occur in cold climates than in warm climates**
- **An increase in joint spacing results in an increase of the percentage of spalled joints**
- **Protecting dowels against corrosion will reduce spalling**
- **The use of a stabilised dense base increases joint spalling**
- **Pre-formed seals reduce spalling in pavements with stabilised bases**

7 Failures

This is the main deterioration mode that occurs in Continuously Reinforced concrete pavements. Located failures include loosening and breaking of reinforcement steel and transverse crack spalling. These are caused by high tensile stresses induced in the concrete and reinforcement steel by traffic loading and changes in environmental factors. Most maintenance activities carried out on CR concrete pavements are directly related to failures.

Failures in continuously reinforced concrete pavements are predicted using Equation 7.1 below (*Lee et al., 1991*):

$$\mathrm{Log_e}(\mathrm{FAIL}) = \mathrm{Kcr_f} * \begin{bmatrix} 6.8004 - 0.0334 * \mathrm{SLABTHK}^2 - 6.5858 * \mathrm{PSTEEL} \\ + 1.2875 * \log_e(\mathrm{NE4}) - 1.1408 * \mathrm{AB} - 0.9367 * \mathrm{SB} \\ - 0.8908 * \mathrm{GB} - 0.1258 * \mathrm{CHAIRS} \end{bmatrix} \quad ...(7.1)$$

where:

FAIL	number of failures per mile in the more trafficked lane (number/mile)	
SLABTHK	slab thickness (in)	
PSTEEL	percentage of longitudinal reinforcement steel (%)	
NE4	cumulative equivalent standard axle load (ESALs) since pavement construction (millions per lane)	
AB	1	if base type is asphaltic
	0	in other cases
SB	1	if base type is cement stabilised
	0	in other cases
GB	1	if base type is granular
	0	in other cases
CHAIRS	1	if chairs are used for installation of the reinforcement
	0	if tubes are used
$\mathrm{Kcr_f}$	calibration factor (default = 1.0)	

Slab thickness and percentage of reinforcement steel have a significant effect on the number of failures in CR concrete pavements. Installation of reinforcement steel with **chairs** produces fewer failures than installation using **tubes**. The use of stabilised or asphaltic bases further reduces the risk of failures.

8 Serviceability loss

Present Serviceability Rating (PSR) is a subjective user rating of the existing ride quality of pavement condition. PSR has been correlated with various roughness indicators, such as slope variance and IRI. It is a reflection of the user response to pavement condition.

8.1 Jointed reinforced concrete pavements

PSR values for JR concrete pavements are predicted in HDM-4 using the following relationship (*ERES Consultants, 1995*):

$$PSR = 4.165 - 0.06694 * TFAULT^{0.5} - 0.00003228 * DCRACK^{2} - 0.1447 * SPALL^{0.25} \quad ...(8.1)$$

where:

PSR	present serviceability rating
TFAULT	total transverse joint faulting per mile (in/mile)
DCRACK	number of deteriorated transverse cracks per mile
SPALL	percentage of spalled joints

The total joint faulting per mile (TFAULT) is calculated as:

$$TFAULT = \frac{FAULT * 5280}{JTSPACE} \quad ...(8.2)$$

where:

TFAULT	total transverse joint faulting per mile (in/mile)
FAULT	average transverse joint faulting (in)
JTSPACE	average transverse joint spacing (ft)

It should be noted that cracking has a very significant influence on the predictions of the PSR model.

8.2 Continuously reinforced concrete pavements

The model of serviceability loss for continuously reinforced concrete pavements predicts loss of serviceability in the traditional scale of 0 to 5, based on pavement age since construction, cumulative equivalent standard axle loads and slab thickness.

The model is as follows (*Lee et al., 1991*):

$$\mathrm{Log}_{10}(PSR_0 - PSR_t) = \begin{bmatrix} 0.79 - 1.3121 * \log_{10}(SLABTHK) \\ + 0.1849 * \log_{10}(AGE) + 0.2634 * \log_{10}(NE4) \end{bmatrix} \quad ...(8.3)$$

where:

PSR_0	initial PSR at the time of pavement construction (default = 4.5)
PSR_t	predicted PSR value at time *t*
SLABTHK	slab thickness (in)
AGE	age since pavement construction (years)
NE4	cumulative equivalent standard axle load (millions 18-kip axles per lane)

9 Roughness

9.1 Jointed plain concrete pavements

Roughness on JP concrete pavements is calculated as a function of faulting, spalling and transverse cracking (*ERES Consultants, 1995*):

$$RI_t = Kjp_r * (RI_0 + 2.6098 * TFAULT + 1.8407 * SPALL + 2.2802 * 10^{-6} * TCRACKS^3) \quad ...(9.1)$$

where:

RI_t	roughness at time *t* (in/mile)
RI_0	initial roughness at the time of pavement construction (in/mile), (default = 98.9)
TFAULT	total transverse joint faulting per mile (in/mile) (calculated from Equation 8.2 above)
SPALL	percentage of spalled joints
TCRACKS	total number of cracked slabs per mile
Kjp_r	calibration factor (default = 1.0)

The total number of cracked slabs per mile (TCRACKS) is calculated as:

$$TCRACKS = \frac{PCRACK * 5280}{JTSPACE * 100} \quad ...(9.2)$$

where:

TCRACKS	total number of transverse cracks per mile
PCRACK	percent of slabs cracked
JTSPACE	average transverse joint spacing (ft)

9.2 Jointed reinforced concrete pavements

Roughness on JR concrete pavements is calculated as a function of PSR *(Al-Omari and Darter, 1994*):

$$RI_t = Kjr_r * \left[-\log_e \left(\frac{0.2 * PSR_t}{0.0043} \right) \right] \quad ...(9.3)$$

where:

RI_t	roughness at time *t* (in/mile)

PSR_t serviceability rating at time *t*

Kjr_r calibration factor (default = 1.0)

9.3 Continuously reinforced concrete pavements

Roughness on CR concrete pavements is calculated as a function of *PSR (Al-Omari and Darter, 1994*):

$$RI_t = Kcr_r * \left[-\log_e \left(\frac{0.2 * PSR_t}{0.0043} \right) \right] \quad \text{...(9.4)}$$

where:

RI_t roughness at time *t* (in/mile)

PSR_t serviceability rating at time *t*

Kcr_r calibration factor (default = 1.0)

10 Calibration factors

The deterioration models contain calibration factors to facilitate local calibration. The calibration factors have default values of 1.0 and are summarised in Table C3.13.

Table C3.13 Calibration factors used in the deterioration models of concrete pavements

Pavement surface type	Calibration factor	Deterioration model
JP	Kjp_c	Transverse cracking calibration factor
	$Kjpn_f$	Faulting calibration factor in JP concrete pavements without dowels
	$Kjpd_f$	Faulting calibration factor in JP concrete pavements with dowels
	Kjp_s	Joint spalling calibration factor
	Kjp_r	Roughness (IRI) progression calibration factor
JR	Kjr_c	Cracking deterioration calibration factor
	Kjr_f	Faulting calibration factor
	Kjr_s	Joint spalling calibration factor
	Kjr_r	Roughness progression calibration factor
CR	Kcr_f	Failures calibration factor
	Kcr_r	Roughness progression calibration factor

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C4 Unsealed Roads

1 Introduction

This chapter describes the detailed modelling of unsealed road deterioration (see Figure C4.1).

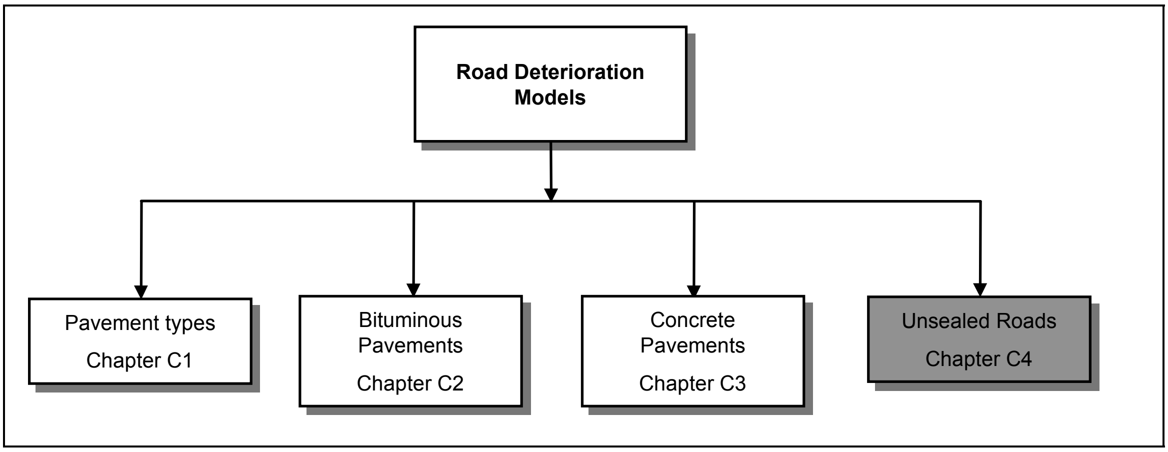

Figure C4.1 Road Deterioration Modules

The HDM-4 Road Deterioration model for unsealed roads is based on the specifications given in the HDM-III documentation by *Watanatada et al. (1987)* reproduced with the approval of the World Bank. Minor modifications have been made in the text and to the models by incorporating calibration factors to facilitate local calibration and adaptation. The background of the model is given in *Paterson (1987)*.

For HDM-4 Version 2 a further review of the road deterioration models was performed by Joubert (LCPC), Morosiuk (TRL), and Toole (ARRB). A number of improvements to the models was agreed and are included in this documentation.

A list of research documents referenced from this chapter is given in Section 7.

2 Modelling logic

2.1 Classification, concepts, and logic

Unpaved roads comprise the lower classes of the road network hierarchy, and generally carry low volumes of traffic ranging from a few vehicles up to several hundred vehicles per day. The geometric standards vary considerably and it is necessary to make a primary classification of unpaved roads into engineered roads, which have controlled alignment, formation width, cross-section profile and drainage; and tracks, which are essentially ways formed by trafficking along natural contours with or without the removal of topsoil. Unpaved roads classified in a country's network are usually engineered or partly engineered, and tracks are usually not classified.

The analysis of unsealed road deterioration is designed primarily for engineered unpaved roads, of either gravel or earth surfacing, because the empirical models are based on a variety of such roads. When necessary it is possible to use the relationship also for tracks as a first estimate, but the user needs to be aware that the environmental effects of drainage and rainfall may be poorly represented.

The deterioration of unpaved roads is characterised primarily by roughness and by material loss from the surfacing. The prediction relationships for these are based on analyses of the Brazil-UNDP study (*Visser, 1981;* and *Paterson, 1987*). Wheelpath ruts also develop under traffic but the ruts are usually not straight forward, often being mixed with water-induced surface erosion. Thus the concept of rut depth was not used in HDM-III and is subsumed in the property of roughness; prediction relationships may be found in *Visser (1981)*. The looseness of surfacing material, which was analysed in the Kenya study (*Hodges et al., 1975*), was also observed in the Brazil-UNDP study (*GEIPOT, 1982*). But as it was found to have no substantial effect on vehicle speed, no prediction relationships were incorporated to HDM-III. Finally, road passability is an important criterion for upgrading tracks or earth roads to gravel roads. HDM-4 allows for an increase in vehicle operating costs by a factor specified by the user. This is to reflect the economic effects of reduced passability when the gravel thickness drops below a minimum level (see Part E).

The periodic grading of unpaved roads is usually undertaken on a more-or-less regular basis for management purposes, either seasonally or frequently enough to keep the roughness within tolerable limits.

These repeated cycles of roughness deterioration and grading maintenance are treated as continual by the model. The average roughness during each analysis year is computed as a function of the roughness at the beginning of the year, of material, traffic, geometry and rainfall parameters and the specified grading frequency. Over a period of time depending on the traffic volume and frequency of grading, the annual average roughness tends towards a long-term average roughness that is also computed.

Maintenance of the gravel surfacing is accounted each analysis year through the surfacing thickness and the net change from material loss, **spot regravelling** and **gravel resurfacing** maintenance. The material loss from earth roads, although computed, is accounted only for the purpose of predicting spot regravelling quantities and is otherwise ignored.

The computational logic described above is simplified by considering that an unpaved road comprises two layers, a gravel surfacing and a subgrade. A gravel road has both layers, but an earth road has a zero thickness of gravel surfacing and its surface characteristics are those of the subgrade. When a gravel road loses all of its gravel surfacing, then its classification reverts to that of an earth road. Upon gravel resurfacing, all unpaved roads become gravel roads by definition of the new surfacing layer.

Deterioration is predicted using the properties of the surfacing layer, whether that is **gravel** or **subgrade**, as it is defined for the analysis year. Thus the user must specify the physical properties of both gravel surfacing and subgrade for unpaved roads.

The pavement classification framework for defining the modelling of unsealed road deterioration and works effects is shown in Table C4.1.

Table C4.1 Generic HDM-4 unsealed pavement types

Pavement type	Surface type	Base type	Description
GRUP	GR	n/a	Granular Unsealed Pavement (for example, gravel road)
EAUP	EA	n/a	Earth Unsealed Pavement

n/a not applicable for unsealed pavements

NDLI (1995) give definitions of the characteristics used to define different types of pavements into the above framework and alternative terminology applied to the same pavement materials (see also Chapter C1).

2.2 Primary model parameters

2.2.1 Material properties

Previously, deterioration relationships have been categorised by material type (lateritic, quartzitic, coral, volcanic, etc.), but from the Brazil-UNDP study it has been possible to replace these by material properties that should improve the transferability of the relationships.

The material properties, which were found to affect the rate of deterioration in Brazil, include the maximum particle size, the particle size distribution and the soil plasticity (*Paterson, 1987*). The specific soil properties are used subsequently to define various summary metrics of the particle size distribution, which are parameters in the deterioration prediction equations. The minimum and maximum levels of roughness (RIMIN and RIMAX) are predicted endogenously from the soil properties but the user may override those by specifying input values. The soil properties are defined for both the **gravel** and **subgrade** layers, and this is denoted by the subscript j**,** where j = g for **gravel surfacing** layer, and j = s for **subgrade** (or earth road surfacing) layer, in Table C4.2.

Table C4.2 Definition of primary variables for unpaved roads

Variable	Definition
ADH	the average daily heavy vehicle traffic (GVW ≥ 3,500 kg) in both directions (veh/day)
ADL	the average daily light vehicle traffic (GVW < 3,500 kg) in both directions (veh/day)
AADT	the annual average daily traffic in both directions (veh/day)
C	the average horizontal curvature of the road (deg/km)
$D95_j$	the maximum particle size of the material, defined as the equivalent sieve opening through which 95% of the material passes (mm)
MGD_j	dust ratio of material gradation, see Section 3.2
MG_j	slope of mean material gradation, see Section 3.3
PI_j	the plasticity index of the material (%)
$P075_j$	the amount of material passing the 0.075 mm sieve (or ASTM No. 200 sieve) (% by mass)
$P425_j$	the amount of material passing the 0.425 mm sieve (or ASTM No. 40 sieve) (% by mass)
$P02_j$	the amount of material passing the 2.0 mm sieve (or ASTM No. 10) (% by mass)
RI_{avg}	average roughness during analysis year (m/km)
$RI_{(ag)}$	roughness after grading (m/km)
$RI_{(bg)}$	roughness before grading (m/km)
$RIMIN_j$	the minimum roughness of the material (either estimated in Section 3.3 or specified) (m/km)
$RIMAX_j$	the maximum roughness of the material (either estimated in Section 3.2 or specified) (m/km)
RF	the average absolute rise plus fall of the road (m/km) Note: RF = 10 times average absolute gradient (%)
SW	average width of shoulder (m)

2.2.2 Traffic loading measures

The three traffic-loading variables used in predicting unpaved road deterioration are simply those of two-way motorised traffic MT counts:

1 **All vehicles** (AADT)

Used in the prediction of material loss. It equals ADL plus ADH,

2 **Light vehicles** (ADL)

Used in the prediction of roughness,

3 **Heavy vehicles** (ADH)

Used in the prediction of roughness,

as defined in Table C4.2 and also in Part B of this Manual.

2.2.3 Road geometry measures

The geometric characteristics found to influence the deterioration of unpaved roads in the Brazil-UNDP study were horizontal curvature (C) and longitudinal gradient (here represented by the rise plus fall variable, RF). Roughness progression, and in particular the maximum roughness, is influenced by both characteristics. In material loss prediction the horizontal curvature affects the rate of traffic-induced material whip-off and the gradient interacts with rainfall in causing erosion. Cross-sectional geometry, including crown, camber and superelevation, were not measured in the study and are discussed in the following section. The average shoulder width (SW) is used to compute the amount of gravel used in **spot regravelling** and **gravel resurfacing**. The variables RF, C and SW are defined in Table C4.2.

2.2.4 Environment: Climate and Drainage

While the climate of the Brazil-UNDP study area is classed as humid to warm- or wet-humid, the rainfall pattern was seasonal, ranging from:

- Precipitation of less than 20 mm per month and air humidity less than 40% during a continuous six to eight months of a year,

to

- Precipitation's of 200 to 600 mm per month and air humidity in excess of 60% over four months of a year.

The effects of the full range of highly seasonal rainfall were analysed in the study, and are represented by the average monthly rainfall in the deterioration prediction relationship. The predictions of annual average roughness and material loss transforms this to an annual average rainfall and thus makes no specific distinction between uniform and seasonal-rainfall climates.

Geometric cross-sectional characteristics, particularly crown, camber, table side-drains and run-off points, have pronounced effects on drainage and road deterioration during high rainfall. In the study area, roughness levels on level, tangent sections that were poorly drained were very high during wet periods due largely to the rapid development of potholes. On vertical slopes, roughness levels were frequently low despite extensive erosion by surface water run-off because the longitudinal profile was affected less than the transverse profile. The study sections generally had moderate drainage facilities and maintenance, and positive crowns. The prediction relations therefore apply to unpaved roads with moderate to good cross-sectional geometry and for dry to wet conditions but may not apply to **bathtub** type roads with negative crown or lack of surface drainage in high rainfall conditions.

2.3 Basic computational procedure

The model assumes that the grading operations and **spot regravelling** specified for each year, both for gravel and earth roads, are distributed uniformly throughout the year. However, the **gravel resurfacing** operation, when it occurs, is performed at the end of the year. Like the periodic paved road maintenance operations, gravel resurfacing is not permitted in an effective construction completion year. The computational procedure for road deterioration of the unsealed roads for each analysis year comprises the following steps:

1. Initialise road characteristics and traffic loading variables at the beginning of the analysis year.

2. If **earth road** skip to step 3. Otherwise, check whether the gravel thickness is zero (that is, no gravel remaining) at the beginning of the analysis year. If the thickness is zero, reset the **road type** to **earth**.

3. If **grading** is specified compute the annual average road roughness as a function of the grading frequency, traffic volume, environmental conditions, and attributes of the gravel (if gravel road) or the **subgrade** (if earth road). Otherwise, if no grading is specified, set the average roughness equal to the predicted maximum roughness ($RIMAX_j$) (see Section 3.2).

4 Compute the depth of material loss during the analysis year as a function of the traffic volume, monthly rainfall, and road geometry and the attributes of the **gravel** (if gravel road) or the **subgrade** (if earth road) (see Section 4).

5 Store the results for later use in the RUE (see Part E) and WE (see Part D) models and in the evaluation and reporting phase.

2.4 Initialisation of variables

At the beginning of the analysis year the traffic variables are computed based on the user-specified traffic data. The values of the environment, road geometry, and material property variables are provided in one of the following three ways:

1 **From the preceding analysis year**

If the analysis year is neither the first year of the analysis period nor an improvement/construction opening year;

2 **From the existing section characteristics data**

If the analysis year is the first year of the analysis period;

3 **From the improvement/construction option data**

If the analysis year is a construction opening year.

The only history variable for unpaved roads is the **gravel age**, denoted by GAGE, which is relevant only for gravel roads. It is defined as the number of years elapsed since the last gravel surfacing or resurfacing. It is initialised as follows:

- When the analysis year is not a construction opening year, the value of GAGE is provided either from:
  - the preceding year - if the analysis year is the second or a subsequent year of the analysis period, or
  - the existing section characteristics data - if the analysis is the first year of the analysis period and is increased by one year; and
- When the analysis year is a construction-opening year (of a gravel road project), the value of GAGE is set to **1** and the unpaved road surface type to **gravel**, irrespective of the previous surface type.

3 Road roughness

3.1 General

The roughness of unpaved roads increases through the shear, mechanical disintegration, and erosion of the surfacing material caused by traffic and surface water runoff. Roughness levels are usually 4 to 15 m/km IRI (50 to 200 QI) although lower levels sometimes occur with fine materials. Roughness in excess of 13 m/km IRI (180 QI) is usually related to depressions, potholes or transverse erosion gullies, and levels above 22 m/km IRI (300 QI), which correspond to numerous wheel-sized potholes, are very rare and usually apply only on short sections or unclassified tracks. The roughness modelled for economic analysis is the profile in the wheel-paths of the traffic, since this generates the vehicle operating costs. The location of the wheel-paths tends to vary when roughness reaches high levels as vehicles seek to minimise the dynamic impact. Hence the prediction of roughness progression must take this self-regulating tendency into account. On account of the high variability of:

- **Material properties,**
- **Drainage,**
- **Surface erosion, and**
- **High roughness levels of unpaved roads,**

prediction errors tend to be large, in the order of 1.5 to 2.5 m/km IRI (20 to 32 QI) standard error, or equivalent to 95 percentile confidence intervals of 20% to 40%.

A number of different model forms have been applied to roughness progression and to the effects of maintenance grading (*Hodges et al., 1975*; *Visser, 1981; Paterson, 1987*). As the objective of policy analyses can be satisfied by computation of the average roughness resulting from a specified policy, the model selected for predicting roughness was one which both represented the progression and grading phases of the roughness cycle realistically and also permitted a closed-form solution. The model form and its derivation are described in detail elsewhere (*Paterson, 1987*). The primary principles and parameter estimates are described in Sections 3.2 - 3.5.

3.2 Roughness progression

In previous models, progression followed either cubic (*Hodges et al., 1975*; *Visser, 1981*; *Paterson, 1987*) concave curves which, unless restrained, led to unrealistically high predictions of roughness for policies of infrequent grading. The model form adopted here constrains the roughness to a high upper limit, or maximum roughness ($RIMAX_j$), by a convex function in which the rate of progression decreases linearly with roughness to zero at RIMAX conforms well with practical observations. The predictions of both forms differ significantly only at high levels of roughness; at low levels of roughness the concave curve is often more realistic in shape, but quantitatively there is little difference between the two. From the Brazil-UNDP study (*GEIPOT, 1982*):

- **Maximum roughness** was found to be a function of material properties and road geometry.

- **The HDM-4 roughness progression** relationship is given by:

$$RI_{TG2} = RI_{max} - b * [RI_{max} - RI_{TG1}] \quad \text{...(3.1)}$$

where

$$RI_{max} = \max\left\{\left[21.5 - 32.4 * (0.5 - MGD_j)^2 + 0.017 * HC - \frac{0.764 * RF * MMP}{1000}\right], 11.5\right\} \quad \text{...(3.2)}$$

$$b = \exp[c * (TG_2 - TG_1)] \quad \text{...(3.3)}$$

where 0 < b < 1

$$c = K_c * \min[1, COMPGR * t * \max(1, n^{0.33})] * \left[-0.001 * \left(0.461 + 0.0174 * ADL + 0.0114 * ADH - \frac{0.0287 * ADT * MMP}{1000}\right)\right] \quad \text{...(3.4)}$$

where

RI_{TG1}	roughness at time TG_1, in m/km IRI
RI_{TG2}	roughness at time TG_2, in m/km IRI
RI_{max}	maximum allowable roughness for specified material, in m/km IRI
TG_1, TG_2	time elapsed since latest grading, in days
ADL	average daily light traffic (GVW < 3500kg) in both directions, in veh/day
ADH	average daily heavy traffic (GVW ≥ 3500kg) in both directions, in veh/day
ADT	average daily vehicular traffic in both directions, in veh/day
MMP	mean monthly precipitation, in mm/month
HC	average horizontal curvature of the road, in deg/km
RF	average rise plus fall of the road, in m/km
t	time since regravelling or construction, in years (integer, with a minimum value of 1)
DG	Number of days between grading
n	frequency of grading, in cycles/year, n = 365/DG
COMPGR	type of compaction used during construction or regravelling: No mechanical compaction during construction or regravelling: COMPGR = 1.0; Mechanical compaction during construction or regravelling: 0.1 <= COMPGR <= 0.5 (default = 0.25)
K_c	section calibration factor for roughness progression
MGD_j	material *j* gradation dust ratio

The material gradation dust ratio is defined as follows:

if $P425_j = 0$, then:

$$MGD_j = 1 \quad \text{…(3.5)}$$

if $P425_j > 0$, then:

$$MGD_j = \frac{P075_j}{P425_j} \quad \text{…(3.6)}$$

where:

$P425_j$	the amount of material *j* passing the 0.425 mm sieve (% by mass)
$P075_j$	the amount of material *j* passing the 0.075 mm sieve (% by mass)

Note: The standard error of this prediction on the original database was 1.5 m/km IRI (19.8 QI).

3.3 Effect of grading

The effect of grading maintenance on roughness was found to depend on the roughness before grading, the material properties and the minimum roughness ($RIMIN_j$) (*Paterson, 1987*). The effect of grading is also dependent on grading type. The minimum roughness, below which grading cannot reduce roughness, increases as the maximum particle size increases and the gradation of the surfacing material worsens. The prediction of roughness after grading is expressed a linear function of the roughness before grading, dust ratio and the minimum roughness, as given by:

$$RI_{ag} = \min\left[RIMIN_j + a * \left(RI_{bg} - RIMIN_j\right), RI_{bg}\right] \quad \text{…(3.7)}$$

where

$$a = K_a * \max\left\{0.5, \min\left[GRAD * \left(0.553 + 0.23 * MGD_j\right), 1\right]\right\} \quad \text{…(3.8)}$$

$$RIMIN_j = \max\left\{0.8, \min\left[7.7, 0.36 * D95_j * \left(1 - 2.78 * MG_j\right)\right]\right\} \quad \text{...(3.9)}$$

and

RI_{ag}	roughness after grading, in m/km IRI
RI_{bg}	roughness before grading, in m/km IRI
$RIMIN_j$	minimum allowable roughness after grading for material j, in m/km IRI
$D95_j$	maximum particle size of the material j, defined as the equivalent sieve size through which 95 per cent of the material passes, in mm
MG_j	slope of mean material gradation for material j
MGD_j	material gradation dust ratio for material j
GRAD	dependent on type of grading (GRAD values are given in **Error! Reference source not found.**C4.3)

K_a section calibration factor for roughness effect of grading

Descriptions of the types of grading (GRAD) are given in Table C4.3.

Table C4.3 Default GRAD values for various types of grading

Type of Grading	GRAD
Non-motorised grading, bush or tyre dragging	1.4
Light motorised grading, little or no water, no roller compaction	1.0
Heavy motorised grading with water and light roller compaction	0.75

NB Maintenance of gravel roads such as full re-processing of the wearing course with water and heavy roller compaction have been observed to produce GRAD values of 0.2. However, as this type of grading is unusual, it has not been included in the default options. Users can obtain lower values of 'a' than the minimum value of 0.5 through the calibration factor K_a

The slope mean material gradation is calculated as follows:

$$MG_j = MIN\left(MGM_j, 1 - MGM_j, 0.36\right) \quad \ldots(3.10)$$

$$MGM_j = \frac{\left(MG075_j + MG425_j + MG02_j\right)}{3} \quad \ldots(3.11)$$

The value of parameter $MG075_j$ is obtained as follows:

if $D95_j > 0.4$

then:

$$MG075_j = \frac{\log_e\left(P075_j / 95\right)}{\log_e\left(0.075 / D95_j\right)} \quad \ldots(3.12)$$

otherwise:

$$MG075_j = 0.3$$

The value of parameter $MG425_j$ is obtained as follows:

if $D95_j > 1.0$

then:

$$MG425_j = \frac{\log_e\left(P425_j / 95\right)}{\log_e\left(0.425 / D95_j\right)} \quad \ldots(3.13)$$

otherwise:

$$MG425_j = 0.3$$

The value of parameter $MG02_j$ is obtained as follows:

if $D95_j > 4.0$

then:

$$MG02_j = \frac{\log_e\left(P02_j / 95\right)}{\log_e\left(2.0 / D95_j\right)} \quad \ldots(3.14)$$

otherwise:

$$MG02_j = MG425_j$$

Note: The standard error of this prediction on the original database was 2.4 m/km IRI (31.6 QI).

3.4 Average roughness during the analysis year

The average roughness during the analysis year is computed by combining the progression and grading-effect relationships and integrating (see *Paterson, 1987*). The year's average is expressed in terms of the roughness at the beginning of the year and the parameters in the previous expressions (see Sections 3.2 - 3.3) as follows:

Case 1: if (t*n) ≥ 1

The average roughness during year *t* (RI_{avg}) is computed as follows:

$$RI_{avg} = RIMAX_j * (1 - y) + \left[\frac{y * NS}{n}\right] \quad \ldots(3.14)$$

$$y = \frac{(b - 1) * n}{365 * c} \quad \ldots(3.15)$$

$$NS = \frac{\left\{n * k + \left[1 - (a * b)^n\right] * QI_a - \left[k\left(1 - (a * b)^n\right) / (1 - a * b)\right]\right\}}{(1 - a * b)} \quad \ldots(3.16)$$

$$k = (1 - a) * RIMIN_j + a * (1 - b) * RIMAX_j \quad \ldots(3.17)$$

where:

RI_{avg}	average roughness during year *t* (m/km)
$RIMAX_j$	maximum roughness for material *j* (m/km)
$RIMIN_j$	minimum roughness for material *j* (m/km)
n	frequency of grading (cycles/year)
RI_a	roughness at beginning of year *t* (m/km)
a	as defined above (see Equation 3.8 **Error! Reference source not found.**)
b	as defined above (see Equation 3.3 **Error! Reference source not found.**)
c	as defined above (see Equation 3.4 **Error! Reference source not found.**)

The roughness at the beginning of the year is obtained as follows:

- **First year of analysis period**

  For the first year of analysis period when *t* = 1, $RI_a = RI_o$ (the value specified by the user).

- **Subsequent analysis year**

  For any subsequent analysis year *t*, $RI_a = RI_b$ (= roughness at end of the previous year *t*-1, as given below in m/km).

In any given analysis year *t*, the roughness at the end of the year (RI_b) is given by:

$$RI_b = (a * b)^n * RI_a + \frac{k * \left[1 - (a * b)^n\right]}{(1 - a * b)} \qquad \text{...(3.18)}$$

where:

All the parameters are as defined previously.

Case 2: if (t*n) < 1

The average roughness during year *t* (RI_{avg}) is given by:

$$RI_{avg} = RIMAX_j - (RIMAX_j - RI_a) * \frac{[\exp(365 * c) - 1]}{365 * c} \qquad \text{...(3.19)}$$

The roughness at the end of the year (RI_b) is given by:

$$RI_b = RIMAX_j - (RIMAX_j - RI_a) * \exp(365 * c) \qquad \text{...(3.20)}$$

where:

All the parameters are as defined previously.

3.5 Roughness cycle 'steady state'

When grading is performed regularly at constant time intervals, or a fixed roughness level, or fixed traffic intervals, the process of roughness change described by these relationships without restriction eventually leads to a steady state, as shown in *Paterson (1987)*. This steady state is characterised by a saw-toothed pattern of roughness-time profile, in which the highs and lows represent the roughness immediately before and after grading, respectively. These highs and lows, denoted by RIH and RIL, are given by:

$$RIH = \frac{\left[RIMAX_j * (1 - b) + RIMIN_j * (1 - a) * b\right]}{(1 - a * b)} \qquad \text{...(3.21)}$$

$$RIL = \frac{\left[RIMIN_j * (1 - a) + RIMAX_j * a * (1 - b)\right]}{(1 - a * b)} \qquad \text{...(3.22)}$$

where:

RIH	roughness immediately before grading (m/km)
RIL	roughness immediately after grading (m/km)

All the other parameters are as defined previously.

The long-term average roughness, denoted by RI_{lta}, at this steady state under a maintenance policy is dependent on the grading frequency (embodied in the variable *b* above) and is obtained by integration over the roughness-time profile, so the annual average roughness tends to the following:

$$RI_{avg} \rightarrow RI_{lta} \qquad \ldots(3.23)$$

and:

$$RI_{lta} = RIMAX_j + (1 - a) * (1 - b) * \frac{\lfloor RIMAX_j - RIMIN_j \rfloor}{[(1 - a * b) * \log_e b]} \qquad \ldots(3.24)$$

where:

All the parameters are as defined previously.

4 Material loss

From the Brazil-UNDP study the following relationship for predicting the annual quantity of material loss as a function of monthly rainfall, traffic volume, road geometry and characteristics of the **gravel** (if a gravel road) and the **subgrade** (if an earth road) was obtained (*Paterson, 1985*):

$$MLA = K_{gl} * 3.65 \left(3.46 + 2.46 * MMP * RF * 10^{-4} + KT * AADT\right) \qquad \ldots(4.1)$$

where:

MLA	the predicted annual material loss (mm/year)
RF	average rise plus fall of the road (m/km)
MMP	mean monthly precipitation (mm/month)
AADT	annual average daily traffic (veh/day)
KT	the traffic-induced material whip-off coefficient
K_{gl}	gravel material loss calibration factor

The traffic-induced material whip-off coefficient is expressed as a function of rainfall, road geometry and material characteristics, as follows:

$$KT = K_{kt} * MAX\left\{0, \left[\begin{array}{l} 0.022 + \left(\dfrac{0.969 * C}{57300}\right) + 3.42 * MMP * P075_j * 10^{-6} \\ - 9.2 * MMP * PI_j 10^{-6} - 1.01 * MMP * 10^{-4} \end{array}\right]\right\} \qquad \ldots(4.2)$$

where:

C	average horizontal curvature of the road (deg/km)
PI_j	the plasticity index of material *j* where:
	$j = g$ if a gravel road
	$j = s$ if an earth road
K_{kt}	traffic-induced material loss calibration factor

These predictions are illustrated in *Watanatada et al. (1987)*.

5 Passability

Passability is the quality of the road surface that ensures the safe passage of vehicles. In the vehicle operating cost model, provision has been made to determine the economic impact of a partial reduction in passability through factors augmenting the operating costs of the various vehicle types (see Part E). This augmentation comes into effect when the gravel surfacing thickness drops below a minimum, and relates to the risk of the **subgrade** material being impassable.

The user however must determine endogenously whether passability will be a problem in the subgrade material, because no physical estimation of it is made within the model. The following criteria from *Visser (1981)* are adequate for ensuring passability and surface stability:

1 **Passability**

is a function of the shear strength of the saturated material, and is satisfactory when:

$$SFCBR \geq 8.25 + 3.75 * \log_{10} AADT \qquad \text{…(5.1)}$$

2 **Surfacing stability**

relates to **ravelling** and looseness, is satisfactory when:

$$P075 \geq 14 \qquad \text{…(5.2)}$$

where:

SFCBR the (minimum) soaked California Bearing Ratio at standard Proctor laboratory compaction for ensuring passability

All other parameters are as defined previously.

6 Calibration factors

The deterioration models contain calibration factors to facilitate local calibration. These factors have default values of 1.0 and are summarised in Table C4.4.

Table C4.4 Calibration Factors used in the Deterioration Models

Calibration factor	Deterioration model
K_{gl}	Gravel loss factor
K_{kt}	Traffic-induced material loss factor
K_a	section calibration factor for roughness effect of grading
K_c	Section calibration factor for roughness progression

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