HDM-4

HIGHWAY DEVELOPMENT & MANAGEMENT

volume five

A Guide to Calibration and Adaptation

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AIPCR PIARC

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THE HIGHWAY DEVELOPMENT AND MANAGEMENT SERIES

About This Manual

This Version1.0 edition of A Guide to Calibration and Adaptation provides details on calibrating the HDM model. It is one of five manuals comprising the suite of HDM-4 documentation (see Figure 1). It is intended to be used by specialists interested in technical issues or responsible for setting up the HDM model.

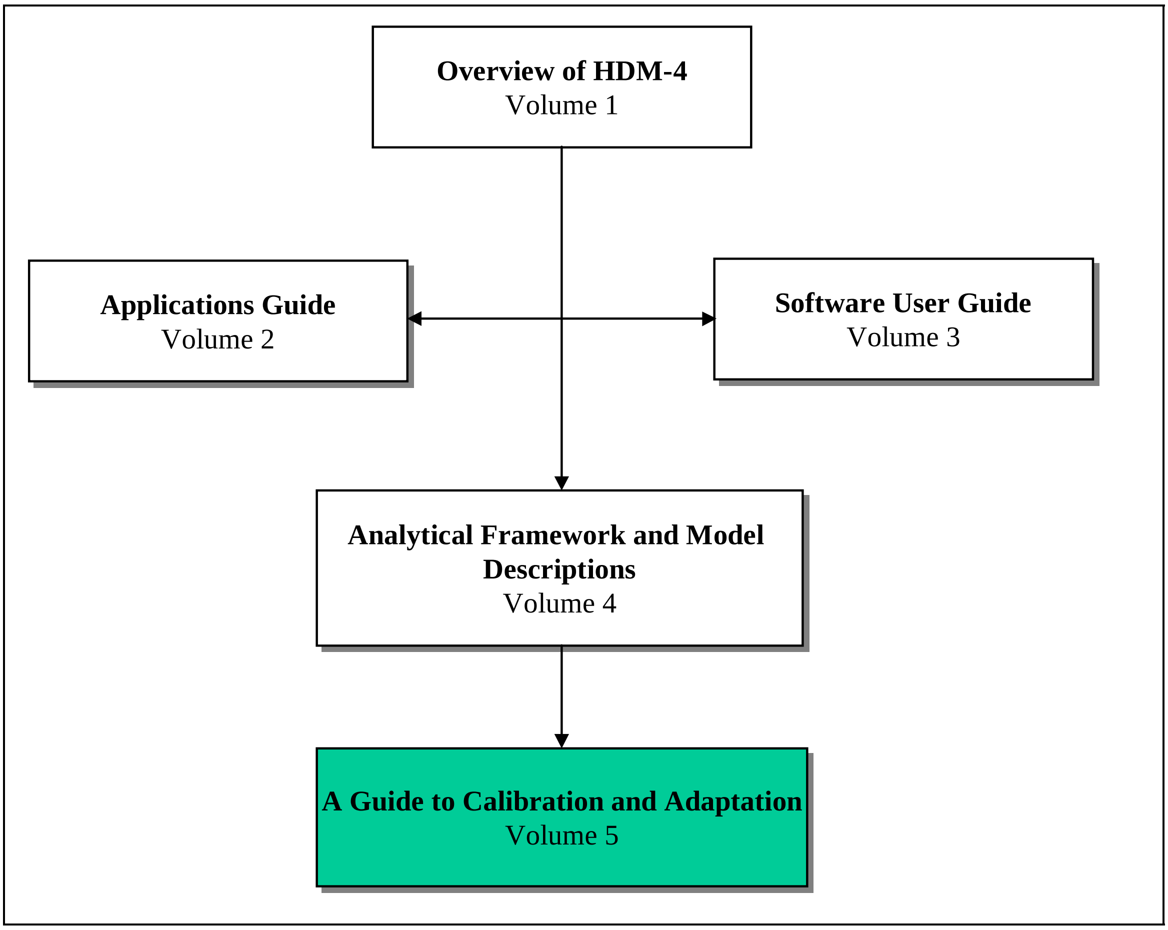

Figure 1 HDM-4 documentation suite

The suite of documents comprise:

- **Overview (Volume 1)**

  A short executive summary describing the HDM-4 system. It is intended to be used by all readers new to HDM-4, particularly high level management within a road organisation.

- **Applications Guide (Volume 2)**

  A task oriented guide describing typical examples of different types of analyses. It is to be used by the frequent user who wishes to know how to perform a task or create a study.

- **Software User Guide (Volume 3)**

  Describes the HDM-4 software. It is a general-purpose document that provides an understanding of the software user interface.

- **Analytical Framework and Model Descriptions (Volume 4)**

  Describes the analytical framework and the technical relationships of objects within the HDM-4 model. It contains very comprehensive reference material describing, in detail, the characteristics of the modelling and strategy incorporated in HDM-4. It is to be used by specialists or experts whose task is to carry out a detailed study for a road management organisation.

- **A Guide to Calibration and Adaptation (Volume 5)**

  Suggests methods for calibrating and adapting HDM models (as used in HDM-III and HDM-4), to allow for local conditions existing in different countries. It discusses how to calibrate HDM-4 through its various calibration factors. It is intended to be used by experienced practitioners who wish to understand the detailed framework and models built into the HDM-4 system.

Notes:

1 Volumes 1, 2 and 3 are designed for the general user.

2 Volumes 4 and 5 will be of greatest relevance to experts who wish to obtain low level technical detail. However, Volume 5, in particular, presents very important concepts, which will be of interest to all users.

Structure of 'A Guide to Calibration and Adaptation'

The information in this document is structured as follows:

- **Chapter 1 - Introduction**

  Provides a general description of HDM-4 and its scope.

- **Chapter 2 – Calibration issues**

  Describes the need for model calibration, different levels of calibration based on available time and resources.

- **Chapter 3 - Reliability concepts**

  Addresses bias and precision of model predictions, input data accuracy, and ways of assessing the reliability of HDM predictions.

- **Chapter 4 – Sensitivity of HDM**

  Presents the results of analyses with HDM to identify the sensitivity of HDM's output to changing the input data or model parameters.

- **Chapter 5 – Adapting data to the model**

  A discussion of how one adapts local data to the HDM model. Appendices A to H address the specifics of how one collects data for use in HDM.

- **Chapter 6 – RUE model calibration**

  Presents details on the calibration of the HDM road user effects model.

- **Chapter 7 – RDWE calibration**

  Presents details on the calibration of the HDM road deterioration and maintenance effects model.

- **Chapter 8 – Summary and conclusions**

  Summarises the main conclusion of the report.

- **Chapter 9 – References**

  Lists the reports referenced in the document.

- **Appendix A - HDM data items and calibration factors**

  Lists HDM data, model parameters and calibration levels.

- **Appendix B - Parameter values used in HDM studies**

  Lists parameter values used in different HDM-III studies.

- **Appendix C - Road user data**

  References road user data

- **Appendix D - Road and pavement data**

  Provides descriptions and discussions of general road data, pavement characteristics, pavement condition, and the maintenance strategies.

- **Appendix E - Traffic data**

  Describes traffic volumes, growth rates and other associated issues.

- **Appendix F - Unit cost data**

  Defines unit cost data for RUE and works effects (WE).

- **Appendix G - Economic data**

  To be included in a subsequent edition of this document.

- **Appendix H - Determining sample sizes**

  Describes the two approaches (the *t* Distribution and the normal distribution) that can be used. The approach is dependent upon the number of samples.

- **Appendix I - Survival curve analysis**

  Describes the calculation of the survival curve.

- **Appendix J - Orthogonal regression**

  Describes orthogonal regression data.

- **Appendix K - HDM Tools user guide**

  Describes HDM Tools, a set of software applications, which is designed to assist in the calibration of HDM-4.

Figure 2 shows the process readers should follow in using this manual.

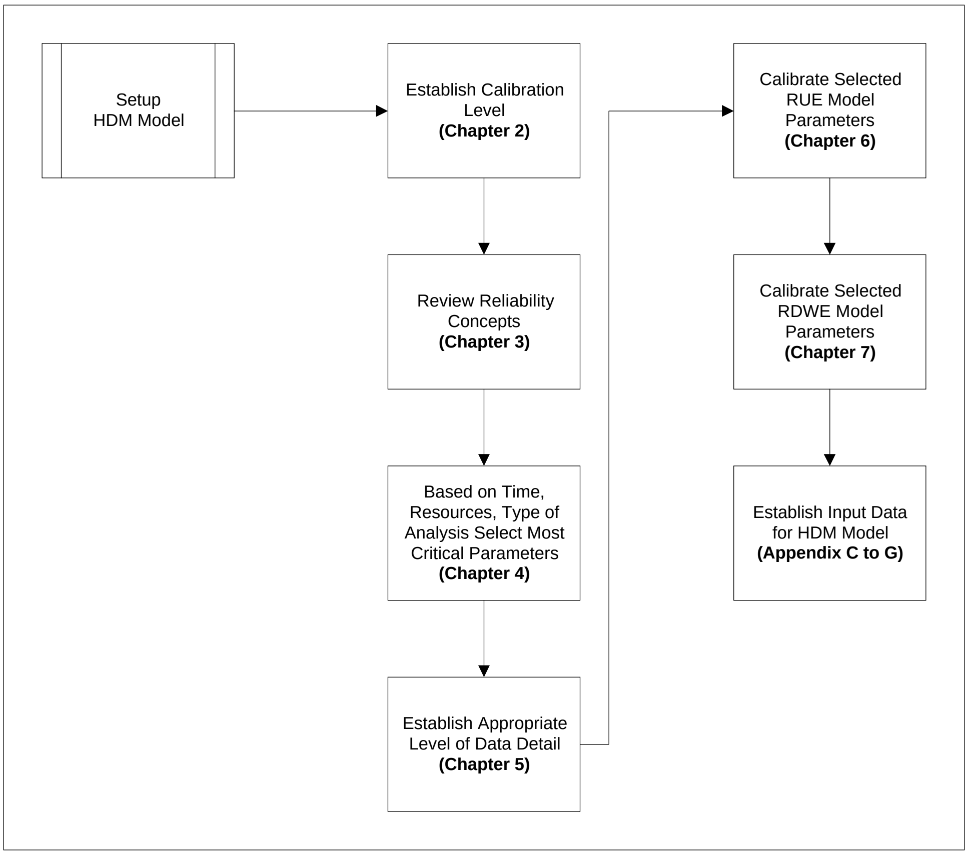

Figure 2 HDM calibration process

ISOHDM products

The products of the International Study of Highway Development and Management Tools (ISOHDM) consist of the HDM-4 suite of software, associated example case study databases, and the Highway Development and Management Series collection of guides and reference manuals. This Volume is a member of that document collection.

Customer contact

Should you have any difficulties with the information provided in this suite of documentation please do not hesitate to report details of the problem you are experiencing. You may send an E-mail or an annotated copy of the manual page by fax to the number provided below.

The ISOHDM Technical Secretariat welcomes any comments or suggestions from users of HDM-4. Comments on A Guide to Calibration and Adaptation should be sent to the following address:

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Change details

This is the first formal edition (Version 1.0) of the HDM-4 documentation.

Related documentation

HDM-4 documents

The Highway Development and Management Series Collection is ISBN: 2-84060-058-7, and comprises:

Volume 1 - Overview of HDM-4, ISBN: 2-84060-059-5

Volume 2 - Applications Guide, ISBN: 2-84060-060-9

Volume 3 - Software User Guide, ISBN: 2-84060-061-7

Volume 4 - Analytical Framework and Model Descriptions, ISBN: 2-84060-062-5

Volume 5 - A Guide to Calibration and Adaptation, ISBN: 2-84060-063-3

Future documentation

The following documents will be issued at a later release:

Volume 6 - Modelling Road Deterioration and Works Effects, ISBN: 2-84060-102-8

Volume 7 - Modelling Road User and Environmental Effects, ISBN: 2-84060-103-6

Terminology handbooks

PIARC Lexicon of Road and Traffic Engineering - First edition. Permanent International Association of Road Congresses (PIARC), Paris 1991. ISBN: 2-84060-000-5

Technical Dictionary of Road Terms - Seventh edition, English - French. PIARC Commission on Terminology, Paris 1997. ISBN: 2-84060-053-6

General reference information

Further details on HDM-4 may be obtained from the following:

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Many of the tools, background reports and additional material for calibration is available from the HDM links available at the following web site:

- HTC Infrastructure Management Ltd.

  Web: http://www.htc.co.nz

Acknowledgements

The work on this Guide to Calibration and Adaptation was initially sponsored by N.D. Lea International Ltd. of Canada. Additional work was sponsored by the Asian Development Bank, HTC Infrastructure Management Ltd., and finally the ISOHDM Study through PIARC.

The authors would like to express their appreciation to all those who assisted in developing this manual:

- Ian Greenwood of Opus International Consultants Ltd. (N.Z.)

  Wrote the HDM Tools calibration applications.

- Rodrigo Archondo-Callao of the World Bank

  Assisted in generating elasticities for the HDM-III model.

- Technical comments were received from a number of reviewers including, but not limited to:
  - Rodrigo Archondo-Callao

- Clive Daniels
- Chris Hoban
- Michael J. Riley
- Robert Thurlow

The development of HDM-4 has been sponsored by several agencies, primarily:

- **Asian Development Bank (ADB)**
- **Department for International Development (DFID) in the United Kingdom**
- **Swedish National Road Administration (SNRA)**
- **The World Bank**

In addition to these, significant contributions were made by:

- **Finnish National Road Administration (Finnra)**
- **Inter-American Federation of Cement Producers (FICEM)**

Many other organisations and individuals in a number of countries have also contributed in terms of providing information, or undertaking technical review of products being produced.

The study has been co-ordinated by the ISOHDM Technical Secretariat at the University of Birmingham in the United Kingdom. A number of organisations participated in the research including:

- **Finnra**

  Specification of the strategic and programme analysis applications.

- **FICEM**

  Development of deterioration and maintenance relationships for Portland cement concrete roads.

- **The Highway Research Group**, School of Civil Engineering, The University of Birmingham

  Responsible for system design and software development.

- **Road Research Institute (IKRAM)** in Malaysia supported by N.D. Lea International (NDLI)

  Responsible for providing updated relationships for road deterioration and road user costs.

- **Transport Research Laboratory (TRL)** in the United Kingdom

  Responsible for review and update of flexible pavement deterioration relationships.

- **SNRA**

  Responsible for developing deterioration relationships for cold climates, road safety, environmental effects, and supporting HRG with system design.

All research organisations received support from local and regional staff, visiting experts and external advisers, to ensure that a high standard of quality and international consensus was achieved. A number of other countries and individuals have supported this work through supplying expert advice and reviewing the products.

Copyright statement

Contents

Volume 5 - A Guide to Calibration and Adaptation

A Guide to Calibration and Adaptation

1 Introduction

The Highway Development and Management Model (HDM), originally developed by the World Bank, has become widely used as a planning and programming tool for highway expenditures and maintenance standards. HDM is a computer model that simulates physical and economic conditions over the period of analysis, usually a life cycle, for a series of alternative alternatives and scenarios specified by the user.

HDM is designed to make comparative cost estimates and economic evaluations of different construction and maintenance options, including different time-staging alternatives, either for a given road project on a specific alignment or for groups of links on an entire network. It estimates the total costs for a large number of alternative project designs and maintenance alternatives year by year, discounting the future costs if desired at different postulated discount rates so that the user can search for the alternative with the lowest discounted total cost.

Three interacting sets of costs (related to construction, maintenance and road use) are added together over time in discounted present values, where the costs are determined by first predicting physical quantities of resource consumption and then multiplying these by unit costs or prices.

The broad concept of HDM is illustrated in Figure 1.1. The user defines a series of alternatives that describe different investment and preservation options for the road. The investments influence the condition of pavement over time and road maintenance costs. The pavement and traffic conditions have an influence on Road User Effects (RUE). The model predicts traffic speeds and the consumption of the RUE components, such as fuel, tyres etc. Multiplying these by the unit costs of the individual components gives the RUE over time. Comparing the cost outputs from various investment alternatives allows assessment of the relative merits, cost savings and benefits of the different alternatives using economic principles.

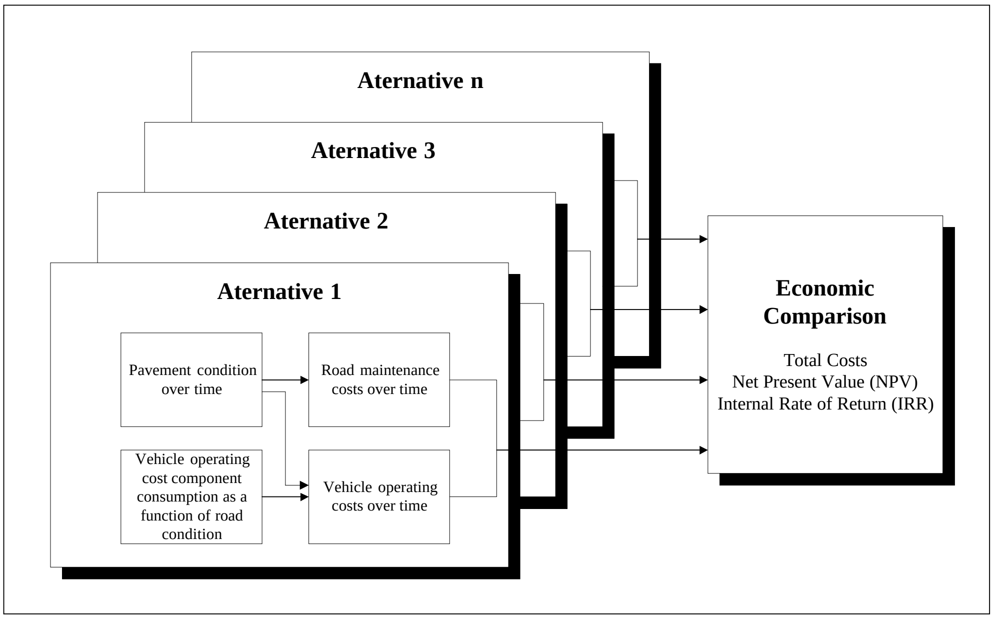


Figure 1.1 Overview of the HDM approach

As illustrated in Figure 1.2, HDM consists of a series of sub-models that address different aspects of the analysis. Each of these sub-models requires certain input data and each produces its own output. In order to apply the model correctly, one needs to ensure that HDM is given the appropriate input data and has been suitably calibrated.

This report presents guidelines for the quality assurance of HDM applications through control of data quality and calibration of the HDM model. It describes the range of options for data collection and input to HDM, as well as accuracy considerations. The sensitivity of HDM to the input data is used to establish those data items that are most critical in the analysis. Similarly, the report discusses how one calibrates HDM through its various calibration factors and the sensitivity of the model to these factors. It presents three levels of effort to achieve calibration based on the available time and resources.

The focus of the report is on HDM-4, although many of the broad principles are applicable to the earlier releases HDM-III and HDM-95.

Box 1.1 Some definitions

- Vehicle Operating Costs (VOC) are the costs of vehicle operation. They are comprised of fuel, tyres, maintenance, labour, depreciation, interest, lubricants, crew, cargo and travel time.
- Road User Effects (RUE) are the VOC along with safety, vehicle emissions, noise and energy balance.
- Road Deterioration and Works Effects (RDWE) encompass pavement deterioration and the effects of works improvements. The latter include maintenance as well as improvements such as widening.

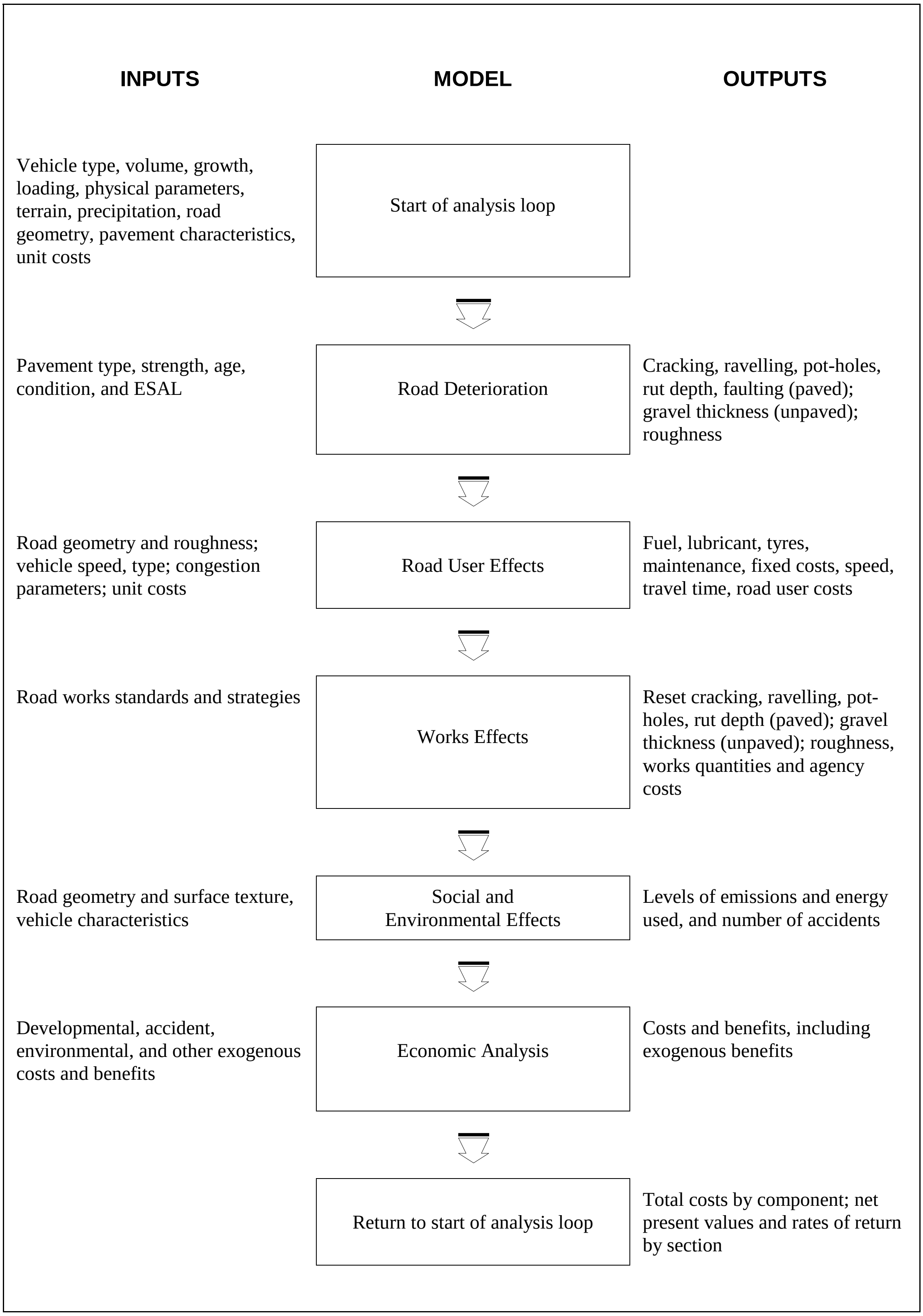


Figure 1.2 Structure of the HDM-4 model

2 Calibration issues

2.1 The need for calibration

As part of the International Study of Highway Development and Management Tools (ISOHDM), a compendium was compiled of the countries where HDM had been applied. HDM or its relationships has been applied in over 100 developed and developing countries having markedly different technological, climatic and economic environments.

Since the model simulates future changes to the road system from current conditions, the reliability of the results is dependent upon two primary considerations:

- How well the data provided to the model represent the reality of current conditions and influencing factors, in the terms understood by the model; and,
- How well the predictions of the model fit the real behaviour and the interactions between various factors for the variety of conditions to which it is applied.

Application of the model thus involves two important steps:

- **Data input -** A correct interpretation of the data input requirements, and achieving a quality of input data that is appropriate to the desired reliability of the results.
- **Calibration of outputs -** Adjusting the model parameters to enhance how well the forecast and outputs represent the changes and influences over time and under various interventions.

The steps in modelling, from data to predictions to the model results are illustrated in Figure 2.1. In the context of this report, the input data and the calibration requirements of the predictive models are dealt with.

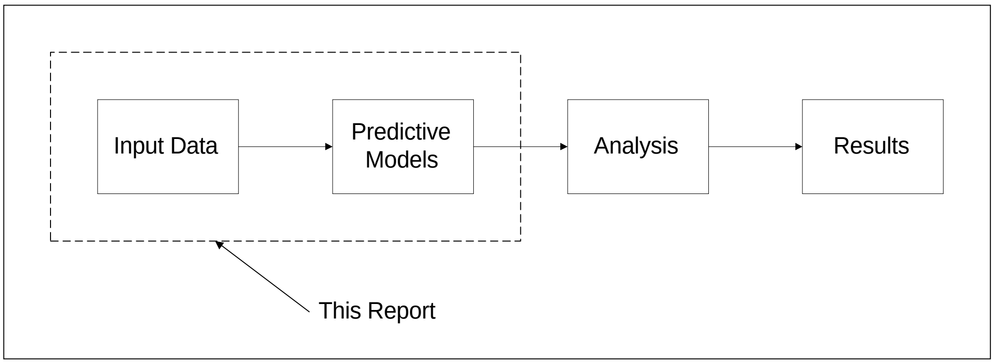


Figure 2.1 HDM modelling process

Calibration of the HDM model focuses on the two primary components that determine the physical quantities, costs and benefits predicted for the analysis, namely:

- **Road User Effects (RUE) -** comprised of vehicle operating costs (VOC), travel time, safety and emissions, and
- **Road deterioration and works effects (RDWE) -** comprised of the deterioration of the pavement and the impact of maintenance activities on pavement condition and the future rate of pavement deterioration.

2.1.1 Model development considerations

Early versions of HDM (HCM and HDM-II) relied on simple empirical regression models that were built on field data collected at specific study sites. However, these simple models lacked transferability because they could not show how the results would change when there was a change in the assumed conditions. In the development of HDM-III and HDM-4, a high degree of transferability across different technological and climatic conditions was built into the model. This was achieved through the use of a structured mechanistic-empirical approach in deriving the underlying predictive relationships. This powerful approach combined various insights provided by the theories of motion and vehicle technology, and of pavement material and structural behaviour under traffic loading, with a rigorous and advanced statistical analysis of real data gathered from a wide range of vehicle types and road conditions. The validity of the vehicle speed and operating costs models was built up from the four major field studies conducted in Kenya, the Caribbean, Brazil and India (*Chesher and Harrison, 1987;* and *Watanatada et al., 1987a*). The validity of the various road deterioration models was demonstrated on independent field data from up to eight different countries and states in climates ranging from hot arid to cold wet (*Paterson, 1987*).

This approach to model building has two principal implications for the user of HDM:

- **Size**

  The number and variety of data items required to feed structured models are potentially large and complex. The HDM modelling chose a middle path, avoiding parameters that require research-level tests and, for road user effects, setting some parameters to have default values that the user could change when analytical resources permitted.

- **Non-modelled effects**

  Even though the field experiments covered wide ranges of conditions, and even though state-of-the-art theory was applied in the model's development, there remain some factors that could not be introduced, because:

  - they were not measured, or
  - would have made the model's input too complex, or
  - their effects could not be determined within the ranges observed

Also, there have been advances in vehicle technology in the period since the studies were done which have lowered consumption and operating costs. For these reasons, some calibration of the HDM model to local conditions is both sensible and desirable.

2.1.2 Case examples

The needs for careful application and some local calibration are discussed in *Curtayne et al. (1987)*. As an example of the needs, Figure 2.2 illustrates the differences arising between costs observed for trucks in Canada and those predicted by the HDM-III model with unadjusted truck default values (*Lea, 1988*). Not only were the total VOC predicted by HDM-III significantly different from those observed, but the relative contributions of the various components to the total operating costs were also very different; which would also arise with HDM-4 were it not properly calibrated. Thus, inadequate local application or calibration can distort both the costs and their allocation that ultimately could distort the indicated allocation of highway expenditures.

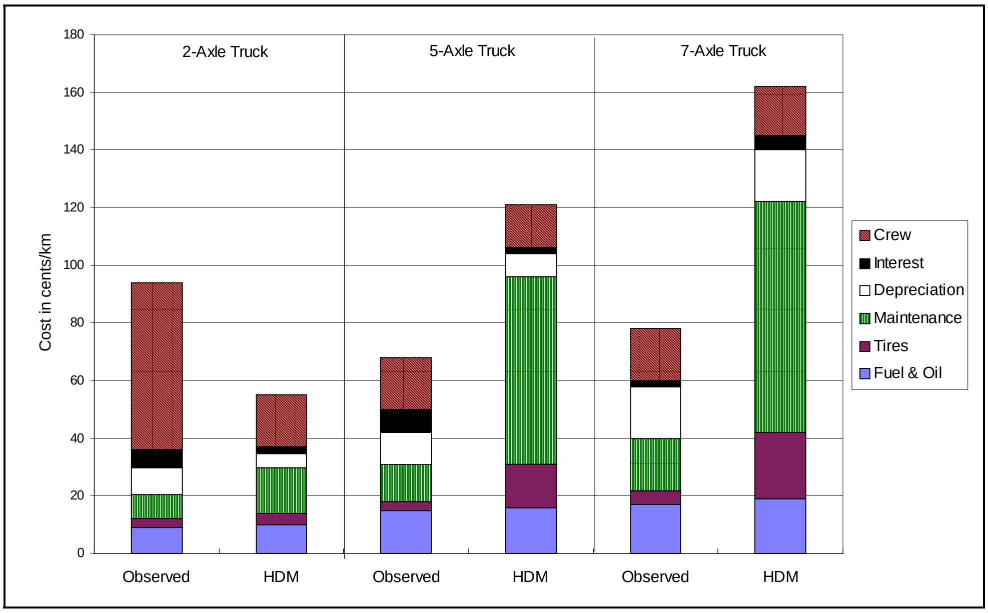


Figure 2.2 Comparison between observed and non-calibrated HDM-III predictions

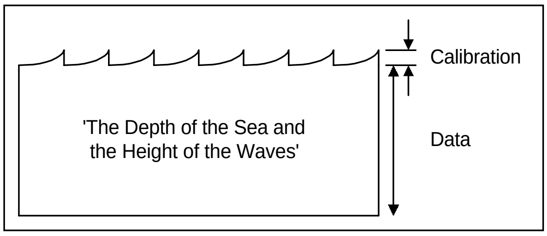
Chesher and Harrison (1987) also point to the need for local calibration through their comparisons of the predictions of the models developed in the various road users cost studies. The relationships not only show significant variations in the magnitudes of the various costs, but also in their sensitivity to pavement and operating conditions. *Finlayson and du Plessis (1991)*, in a comprehensive report containing a number of different papers on VOC, illustrate the importance of calibrating VOC equations to local conditions and discuss how it was done for Southern Africa.

For road deterioration, environmental conditions have a strong influence, affecting deterioration rates by factors of up to 2 or 3 between the extremes of hot arid and cold wet climates. Local construction materials, practices and quality also affect deterioration rates and the effectiveness of maintenance. Both influences can be controlled through calibration factors in the inputs to HDM, in addition to the main traffic and pavement input parameters.

This report presents the ways in which the level of confidence of the HDM predictions can be raised through differing levels of effort that can be matched to the level of application. The advantages of a universal model such as HDM are that it can be used quickly with relatively little or no investment in extensive empirical and statistical research, and that it represents a comprehensive techno-economic framework. Enhancing the reliability thus involves first ensuring the validity of input data and primary parameter values, and second, refining but not reconstructing the predictive relationships to conform to local data. A third stage may be undertaken which sees relationships replaced by new ones developed from local research.

Box 2.1 **Data versus calibration**

An analogy of the sea is useful to illustrate the roles of data and calibration. Data determine the order and magnitude of costs and effects, so these must be of the same order as baseline information, much like the depth of the sea and other attributes like density. Calibration ensures that the height of the waves will be correct under the influence of winds, currents and depths of water.



This report deals with both issues. In the context of HDM-4 data are those items which, when you install the model, require input data. This includes unit costs, road attributes, maintenance alternatives, etc. Where default values are supplied, these are model parameters or coefficients that can be calibrated.

The main text of the report deals with calibration of model parameters and coefficients; Appendix C to Appendix G deals with how one establishes the input data for HDM-4.

2.2 Input data

Input data are the basic data items required to run HDM. These consist of parameters that describe the physical characteristics of the pavements and the network, road user data, traffic data, unit costs and economic data. Appendix A summarises the input data items that are required for HDM applications. Appendix B gives some examples of parameter values adopted in different HDM studies. Appendix C to Appendix G describes how the data are quantified.

In establishing input data, the accuracy required is dictated by the objectives of the analysis. If one is doing a very approximate analysis there is no need to quantify the input data to a very high degree of accuracy. Conversely, if one is doing a detailed analysis it is important to quantify the data as accurately as is practical given the available resources. This is discussed in Chapter 5.

2.3 Calibration

2.3.1 Introduction

Calibration differs from input data since calibration is aimed at adjusting the model predictions. As shown in Appendix A, the HDM RUE and RDWE sub-models contain a large number of parameters that can be adjusted. It will be noted from Appendix A that HDM-4 has many more calibration factors than HDM-III. This is because many values that were **hard coded** in the HDM-III source code can now be altered in HDM-4.

The degree of local calibration appropriate for HDM is a choice that depends very much on the type of application and on the resources available to the user. For example, in planning applications the absolute magnitude of the RUE and road construction costs need to match local costs closely because alternative capital projects with different traffic capacities or route lengths are evaluated on the comparison of the total road transport costs. In road maintenance programming, on the other hand, the sensitivity of RUE to road conditions, particularly roughness, and all the road deterioration and maintenance predictions are the most important aspects.

There are three levels of calibration for HDM, which involve **low**, **moderate** and **major** levels of effort and resources, as follows:

- **Level 1 - Basic Application** (see Section 2.3.2)

  Determines the values of required basic input parameters, adopts many default values, and calibrates the most sensitive parameters with best estimates, desk studies or minimal field surveys.

- **Level 2 - Calibration** (see Section 2.3.3)

  Requires measurement of additional input parameters and moderate field surveys to calibrate key predictive relationships to local conditions. This level may entail slight modification of the model source code.

- **Level 3 - Adaptation** (see Section 2.3.4)

  Undertakes major field surveys and controlled experiments to enhance the existing predictive relationships or to develop new and locally specific relationships for substitution in the source code of the model.

In terms of effort, these three levels can be viewed as **weeks**, **months** and **years**. An analyst should be able to undertake a Level 1 calibration in about one-week. For a Level 2 calibration there is an increase in the amount of effort required so it will take at least a month. Level 3 calibrations require a long-term commitment to basic data collection so their extent spans for a year or more.

Every HDM analysis requires at least a Level 1 calibration.

Figure 2.3 illustrates this concept of increasing effort and increasing resources. It must be appreciated that there is a direct relationship between the time and effort expended in setting up HDM and the reliability and accuracy of its output.

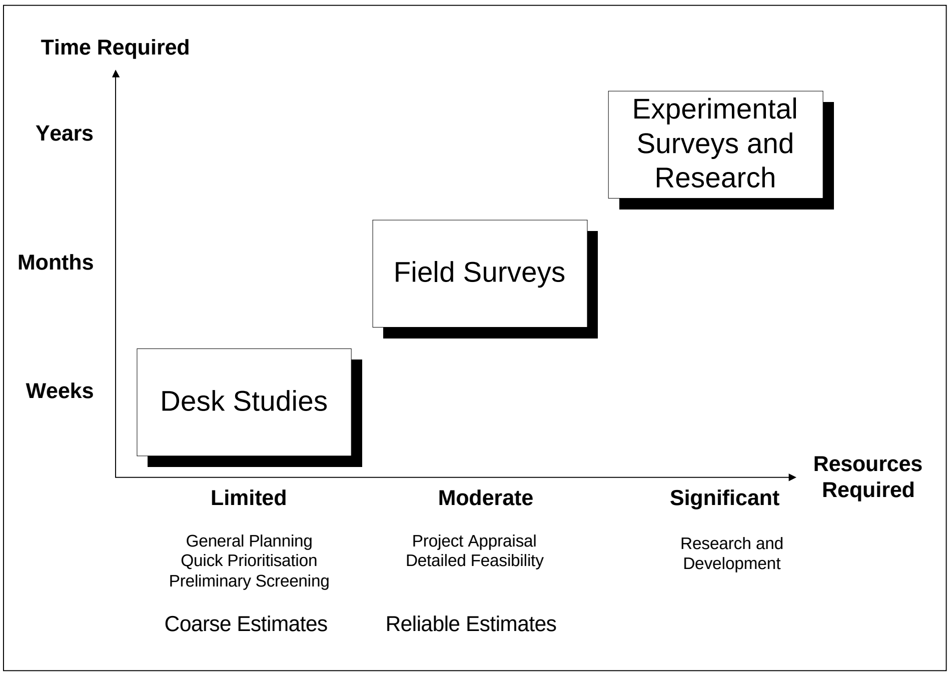


Figure 2.3 Resources and time required for calibrations

As shown in Table 2.1, a related concept of a hierarchy of calibration with increasing levels of activity was suggested by *Curtayne et al. (1987)* in regard to VOC. They noted that calibration activities could be based on both primary and secondary data sources. Primary sources are based on direct comparisons between costs and highway characteristics whereas secondary data provide information about the economic or operating conditions of a region. The calibration of VOC results varies from the selection of available relationships on the basis of few data to the estimation of local relationships using especially collected data.

The following sections outline the components of Level 1 to Level 3 calibrations.

Appendix A gives the data items and model parameters that should be investigated in calibration. It should be noted that not all the items need to be considered, Chapter 4 presents the results of a sensitivity analysis with HDM which shows which are the most important data and parameters in terms of influencing the model predictions. These are where the emphasis should be focused with less sensitive items receiving attention only if time and resources allowing.

2.3.2 Level 1 - Application

In order to run HDM it is always necessary to undertake at least a Level 1 calibration; this can be viewed as a set-up investment for the model. Once this has been done, it generally does not need to be repeated for most of the input data files during future applications in the same country since many data items and most model parameters are relatively stable over time.

A Level 1 calibration is largely based on secondary sources; that is, it is a desk study. For example, the RUE parameters can be estimated using data from sources such as government and industry publications, operator organisations or various RUE reports from previous studies. For road deterioration, the sources would include climate statistics, road traffic and condition statistics, geometric standards, maintenance programmes and budgets.

It can be assumed that the bulk of the default HDM model parameters are appropriate for local conditions so only the most critical ones need to be addressed.

Table 2.1
RUE calibration data sources and hierarchy of resources

Type of data	Sources or needs	Increasing resources required		
Secondary available	Government publications Industry technical reports RUE research reports Manufacturer's literature Consultant's reports Operator organisations Road condition inventories	Choose primary study results which correspond to local conditions	Choose data sets and vehicle types Calibrate mechanistic models for technology changes	Examine intercept and slope values for vehicle types with available cost and road condition data
Survey calibration	Small-scale research collecting cost data from companies operating over a good range of highway characteristics Personnel with industry knowledge Rates survey Route classification	Confirm intercept values and assess slope magnitudes Determine vehicle utilisation by age and road condition	Estimation of tyre and depreciation costs Confirm slope values for total costs	Estimate new local relations, especially for maintenance costs Compare predicted RUE with rates
Experimental calibrations	Small-scale research of vehicle performance and roadway characteristics Trained personnel Analytical capabilities	Speed calibration	Fuel consumption calibration	Estimate new relations, for example for rolling resistance and road roughness

Source: *Curtayne et al. (1987)*

As shown in Appendix A, the following input data should be obtained for a Level 1 calibration:

- **Unit costs** (RUE and RDWE)
- **Certain characteristics of representative vehicles**
- **Economic analysis data** (discount rates and analysis period)
- **Pavement characteristics** (RDWE studies)
- **Traffic composition and growth rates**
- **Regional climatic type**

While HDM often calls for a wide range of input data and calibration parameters, but only the most important need to be established for use with a Level 1 calibration, so the HDM default values should be used almost exclusively.

2.3.3 Level 2 - Calibration

A Level 2 calibration uses direct measurements of local conditions to verify and adjust the predictive capability of the model. It requires a higher degree of data collection and precision than in a Level 1 calibration, and extends the scope. For RUE, it concentrates on speed, fuel consumption, tyre consumption, parts consumption and the **fixed** costs relating to utilisation and vehicle life. For RDWE, it concentrates on the initiations of surface distress modes, rutting progression, and maintenance effects, and enhances the estimate of environmental impacts. For the economic analysis, it ties cost data more closely to observed cost and price levels through data collection surveys.

With Level 2 calibrations, more detailed input data are also collected than with Level 1.

2.3.4 Level 3 - Adaptation

A Level 3 calibration is generally comprised of two components:

- **Improved data collection**
- **Fundamental research**

Some data items can be estimated with reasonable accuracy using short-term counts, for example the hourly distribution of traffic volume, but the reliability is greatly enhanced by collecting data over more sites over a longer period.

Fundamental research considers the relationships used in HDM. This consists of structured field surveys and experimental studies conducted under local conditions which lead to alternative relationships. For example, alternative functions may be developed for predicting fuel consumption or new pavement deterioration and maintenance effects functions for different pavement types. Such work requires a major commitment to good quality, well-structured field research and statistical analysis over a period of several years. Pavement deterioration research is a particularly long-term endeavour, typically requiring a minimum of 5 years.

2.4 Report scope

This report has been prepared as a guide to those responsible for preparing analyses with the HDM model. It mainly deals with the first two levels, that is **Application** and **Calibration** hereinafter referred to as Level 1 and Level 2 calibrations respectively which constitute the great majority of all uses.

Users interested in a Level 3 calibration are referred to the HDM-III background references (*Watanatada et al., (1987a)*; *(1987b)*; *(1987c)*; *Paterson, 1987*; and *Chesher and Harrison,* *1987*) detailing the original research. These contain material on experimental design, theory, model forms, analytical methods and empirical limitations.

3 Reliability concepts

3.1 Introduction

The objective of an HDM analysis is to model roads. This entails predicting the deterioration of the pavement under time and traffic, the road user effects, and the effects of maintenance on the pavement condition and rate of deterioration. As with any model, HDM is a representation of reality. How well the model predictions reflect reality is dependent upon a combination of the:

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

Since the underlying HDM relationships have proven to be robust and applicable in a number of countries, the reliability of most HDM analyses depends on the input data and the calibration factors.

This chapter addresses reliability issues in the context of HDM. It gives an overview of the commonly encountered problems and describes statistical methods to improve reliability.

3.2 Bias and precision

The only way of assessing the adequacy of HDM's predictions is by comparing the model predictions to known data. For example, one may have data on the current roughness of a number of pavements of known ages. By using HDM to predict the condition of pavements of the same age with the same attributes from when they were new, one could assess whether HDM was giving appropriate predictions.

There are two considerations when comparing predicted and observed data:

- **Bias**

  A systematic difference that arises between the observed and predicted values. For example, if the predictions are always 10 per cent lower than the observed data. The formal definition of bias is the difference between the mean predicted and mean observed values.

- **Precision**

  A measure of how closely the observed and predicted data are to each other[1]. It is represented by the reciprocal of the variances ($\sigma_{obs}^2/\sigma_{pre}^2$); that is, it is reflected by the **scatter** when plotting the observed versus predicted data. Precision is influenced the inherent stochastic variations of most natural processes, measurement and observational errors, and unexplained factors omitted from relationships in the model.

Figure 3.1 illustrates both of these concepts for four scenarios:

[1] Precision here should not be confused with precision of a measurement which refers to the closeness of repeated measurements to each other, for example, the weight is 10 kg ± 0.1 kg.

- **Low Bias - High Precision** (see Figure 3.1A)
- **Low Bias - Low Precision** (see Figure 3.1B)
- **High Bias - High Precision** (see Figure 3.1C)
- **High Bias - Low Precision** (see Figure 3.1D)

In Figure 3.1 the shaded ellipse represents observed data which has been plotted against predicted data. The solid line at 45° is the line of equality, where the observed and predicted are equal.

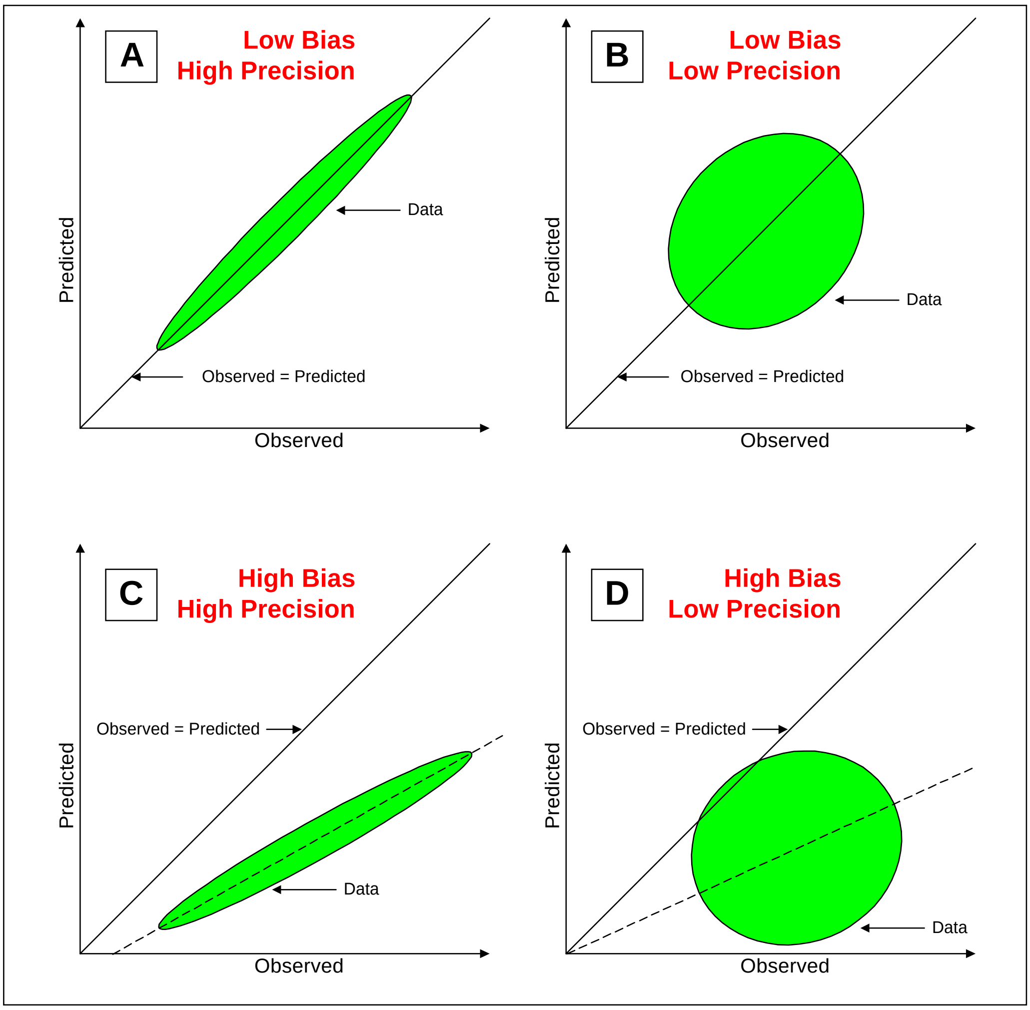


Figure 3.1 Example of bias and precision

When there is low bias, the data will fall around, or close to, the line of equality. As illustrated in the Figure 3.1-A (low bias and high precision), when there is high precision there is little scatter in the data. However, a reduction in precision sees an increase in the scatter and, thus, the standard deviation (Figure 3.1-B (low bias and low precision)).

When there is high precision, the change from low to high bias sees the slope of the observed data systematically different to the line of equality. This is illustrated in Figure 3.1-C (high bias and high precision). This figure also shows what happens when there additionally is a systematic difference, the slope of the observed versus predicted line does not pass through the origin.

The most difficult situation is illustrated in Figure 3.1-D (high bias and low precision). In this instance it is often difficult to verify the reasonableness of the model since the differences could be equally due to the poor precision as to the bias.

Box 3.1	Important considerations

- Calibration needs to extend over the full range of values likely to be encountered
- The sample size must be sufficient to detect the nature of the bias and the level of precision
- A high r^2 (correlation) does not always mean high accuracy, there can still be significant bias

3.3 Correction factors

Bias arises because of systematic differences in the observed versus predicted values. Correction factors are used to correct for the bias. As shown in Figure 3.2, there are two types of calibration factors: **rotation** and **translation**. Either or both of these may be present.

In the simplest case, the bias is established as the ratio of the mean observed to mean predicted to the mean observed. Thus, the correction factor is:

$$\text{CFrot} = \frac{\text{Mean Observed}}{\text{Mean Predicted}} \qquad \ldots(3.1)$$

This is referred to as the rotation correction factor because, as illustrated in Figure 3.2-A, the predictions are **rotated** down to where they correspond to the observed data.

The translation factor is used when there is a constant difference between the observed and predicted values across all conditions (see Figure 3.2-B). An example of this is where the vehicle operating costs are overestimated due do overheads being improperly included. In this instance the correction factor is:

$$\text{CFtrans} = \text{Mean Observed} - \text{Mean Predicted} \qquad \ldots(3.2)$$

Figure 3.2-C also shows the third, and common, scenario where there is a combination of rotation and translation.

3.4 Input data accuracy

Bias and precision also come into play in establishing the input data for HDM. This is because when quantifying input data, one obtains a sample of measurements whose properties are assumed to be the same as that of the underlying population. The bias is often expressed as:

$$\text{Bias} = \frac{\text{Mean Sampled}}{\text{Mean Population}} \qquad \ldots(3.3)$$

This is analogous to the rotation correction factor presented in Section 3.3.

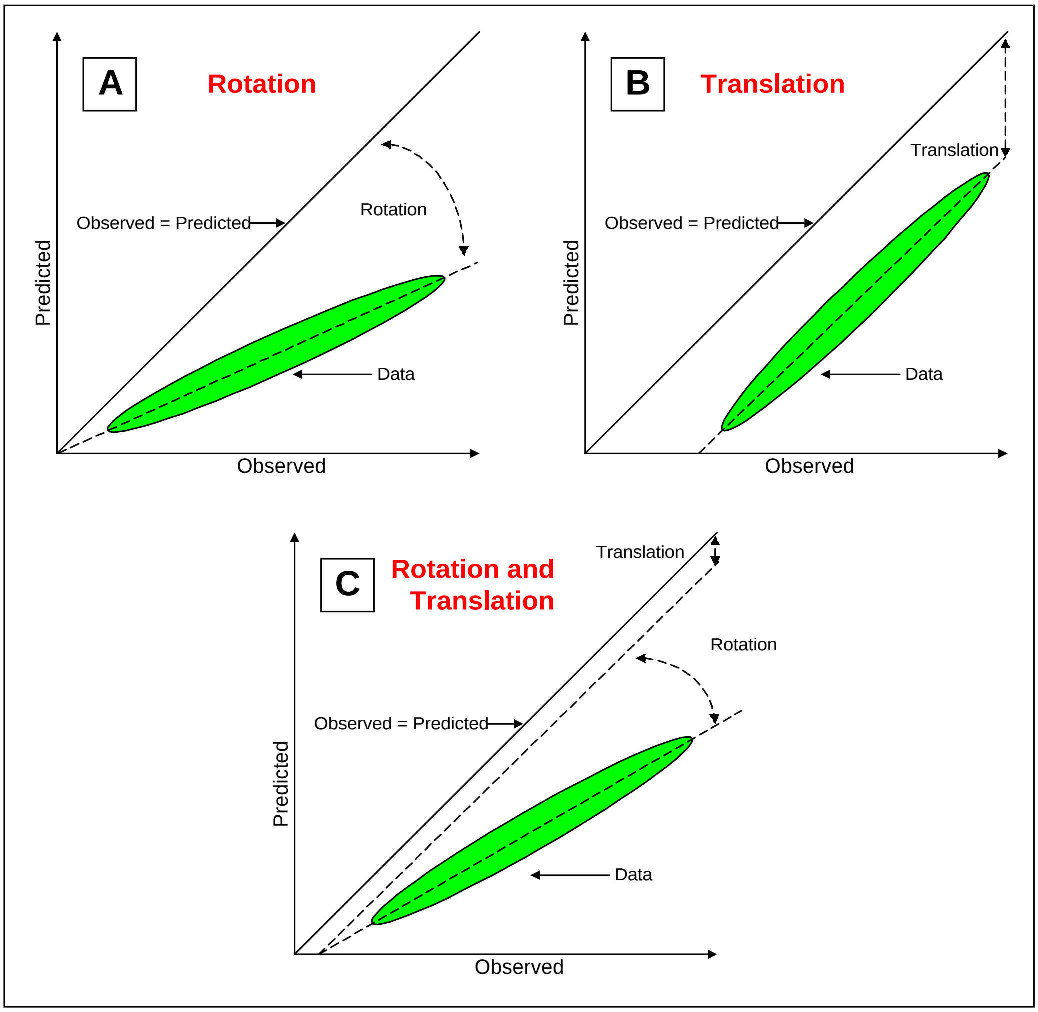


Figure 3.2 Correction factors

As illustrated in Figure 3.3, when the mean of the sample is different to the mean of the population, the sample is biased. If there is the same mean but different variances, there is a problem with the precision (see Figure 3.4).

In quantifying input data the objective is to ensure that the distribution of the sampled data is similar as that for the population. This is achieved by using standard statistical sampling techniques that ensure that there is a sufficient sample size to limit random sampling errors.

Appendix H describes the method for establishing the required sample sizes for different levels of confidence.

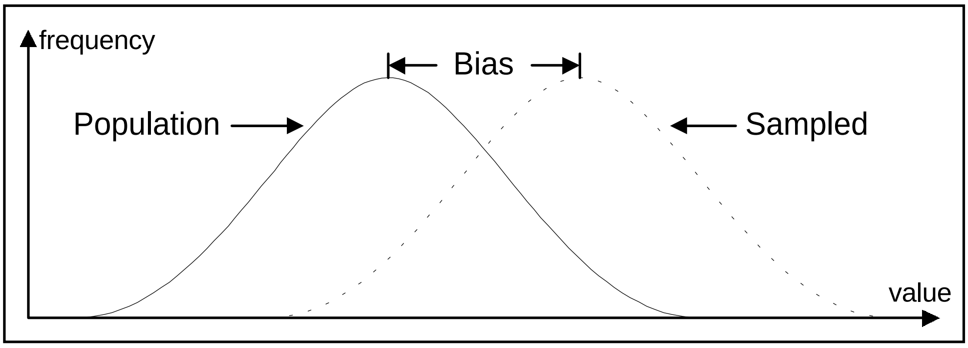


Figure 3.3 Bias in sampled data

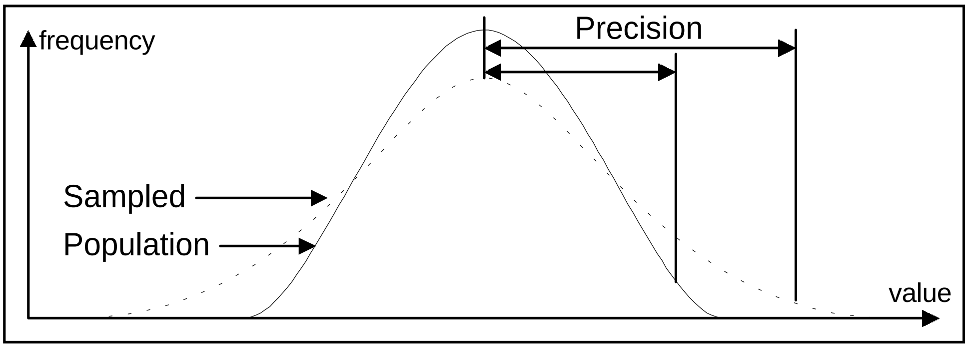


Figure 3.4 Precision in sampled data

Box 3.2	Bias and precision

The principal objective is to minimise the bias.

There is always a cost/resource trade-off when it comes to precision. High precision is costly and resource intensive whereas lower cost techniques usually result in lower precision.

If the bias is low, one can usually accept lower precision data since the mean observed and predicted will be similar (see Figure 3.1-B). If the bias is high, it is important to have high precision data in order to ascertain the nature of the bias (see Figure 3.1-C). High bias and low precision data (see Figure 3.1-D) makes it impossible to accurately apply the model.

3.5 Limits on bias

It is important that the bias in the model predictions be limited so as to ensure that the results are accurate. Since each of the many data items and model parameters may have their own biases, and the processes being modelled by HDM are complex, it is extremely difficult to determine the net bias effects of multiple parameters.

The goal of calibration is defined here with respect to the impact of an individual parameter, but very approximate in respect of the impact of all parameters. The goals of calibration, excluding the fundamental lack of fit of the model, are to reduce any bias of the predictions by the model to acceptable levels. Level 2, being of a higher standard, is intended to produce less residual bias in the model's predictions than a Level 1 calibration.

A Level 3 calibration aims to improve both precision and bias by reconfiguring one or more relationships within the model. It thus requires a comprehensive scope and database in order to adequately detect and determine the many factors, effects and interactions.

3.6 Assessing the reliability of HDM predictions

There are a number of techniques that can be used to assess the reliability of HDM predictions. The most appropriate one depends upon the available data and what the objective is. Specific guidance is given in later chapters.

3.6.1 Pavement performance: simulation of the past

One of the easiest methods for assessing the overall reliability of RDWE predictions is to simulate the past condition of the road. This can be done in a Level 1 or Level 2 calibration and is always a good check on the model.

In Bangladesh the evolution of the network over the previous 15 years was simulated (*Transroute, 1992*). It was assumed that the initial condition of the network was equivalent to the current condition and that the traffic growth rates over the period were equivalent to the forecast traffic in the next 10 years. The simulation was tested using two premises, the:

- Predicted network roughness distribution was similar to the current roughness distribution.
- Average annual expenditure was close to what was observed.

Using the above criteria and appropriate maintenance standards it was found that "the simulation [gave] a fairly good picture".

In Nepal, *NDLI (1993)* analysed a specific section of road that was opened in 1970. Estimates were made of the opening year traffic and HDM was run with a range of assumptions about pavement strength and initial roughness. The predicted conditions were then compared to the measured roughness and surface distress surveys assuming minimum maintenance.

The predicted roughness was dependent upon the assumed initial roughness and on the pavement strength. Discussions with local engineers indicated that the road had a smooth surface and using 3.5 IRI m/km in the modelling resulted in a predicted mean roughness only slightly higher than the measured roughness. HDM under-predicted patching but this was ascribed to the fact that HDM only patched potholes whereas local practices included badly cracked areas and local depressions. Overall, the model was considered to give reasonable predictions.

In terms of roughness progression, one method for testing the model predictions has been through the use of a "slice-in-time" analysis. This technique uses network roughness data to investigate the roughness-time and/or roughness-pavement strength relationship. Given the variations present in network roughness data it is unlikely that this method will yield useful results. *NDLI (1992)* applied this technique without success to Thailand. Several factors contributed to the failure of the analysis:

- The ages of pavements were difficult to accurately estimate which meant that the traffic loadings were inaccurate.
- It was difficult to estimate the post-construction roughness.
- The pavement strength was estimated from construction records. However, the variations in roughness along a section of road indicated that there were local variations in strength which were not reflected in the construction records.
- Variations in traffic growth over the life of the road were not accurately captured.

3.6.2 Pavement performance: controlled studies

The only way of completely calibrating the HDM-III pavement deterioration model is by conducting a study into the rate of pavement deterioration. Those considering such an undertaking should consult the material in *Hodges et al. (1975)*, *GEIPOT (1982)* and *Paterson (1987)*.

It is important that the experiment be designed to gather data on the following items:

- **The effect of traffic on pavement deterioration**
- **The effect of the environment on pavement deterioration**
- **Deterioration rates by pavement type**
- **Pavement strength effects**
- **Surface distresses**

The sites selected for the study should cover the full range of pavement types and strengths within the country. For each pavement type, pavements covering the complete range of strengths should be included. If there are major differences in the climate over the country, the experiment should be designed to account for this.

The data to be collected will depend upon the objectives of the study. For an adequate calibration the following items need to be collected as a minimum:

- **Roughness**
- **FWD/Benkelman beam deflection**
- **Cracking**
- **Rut depth**

In selecting the sections it should be appreciated that if the pavements have been properly designed for the traffic level it should be difficult to observe traffic loading effects as these have been catered for in the design. One should therefore try and use pavements under-designed for their traffic levels.

It is important that the test sections be continually monitored and that all the data items are collected at the same time. In a number of studies this was not done and it created problems with the subsequent analysis.

It is recommended that a vehicle mounted roughness meter (RTRRMS) **not** be used in measuring roughness. This is because the measurement errors with such instruments are of the same or greater magnitude to the incremental changes in roughness over time. These meters are also prone to calibration problems that may lead to additional errors. The roughness should thus be measured using one of the direct profiling techniques used for roughness meter calibration (for example, Dipstick, Walking Profilometer). This will ensure that the data are of the greatest possible accuracy.

It is important to monitor crack initiation and progression. Because of this, the test sections should not have any maintenance done to them over the course of the study. This is often difficult to arrange but is very important, particularly insofar as monitoring crack progression is concerned.

3.6.3 Road User Effects: controlled survey

Controlled surveys for RUE often concentrate on fuel consumption, as this is the easiest component to measure. Tyre wear can be investigated but it is often difficult to execute such a study effectively[1].

du Plessis and Meadows (1990) tried to investigate maintenance costs through a controlled survey. Three rental cars were driven over two pre-selected routes, one paved and one gravel, for a period of about three months during which they covered 40,000 km. Thorough maintenance checks were effected every two weeks and fuel consumption and tyre wear were monitored closely. The results showed that road roughness markedly affected both maintenance and tyre wear, but not fuel consumption. The latter was ascribed to the unavailability of speed and acceleration data. The maintenance and tyre results were used to check the validity of the HDM predictions.

3.6.4 Road User Effects: tariff survey

One of the easiest ways of evaluating the accuracy of HDM RUE predictions is by comparing the predictions to market tariffs. These can be done in Level 1 or Level 2 calibrations.

The underlying hypothesis is that tariffs reflect the total VOC of a vehicle and will therefore give a reasonable estimate not only of the magnitude of VOC but also of the marginal costs under different operating conditions. Generally, freight operations are more independent than passenger operations since the latter often have price regulations/subsidies.

In collecting data on tariffs for such purposes, the following should be considered:

- The use of freight tariff data for model calibration is most appropriate when there is a competitive market, that is, where there is a minimum of government control on operation. When regulation or monopolies affect the supply of transport services, the data must be carefully scrutinised.
- It is better to gather data from a single company operating over a range of operating conditions than from many companies. As described in *Chesher and Harrison (1987)*, the variation in costs between companies is often greater than the variations due to operating conditions.
- The data should be standardised in such a fashion that effects such as load levels are accounted for.
- The HDM predictions should be made using financial instead of economic costs.
- The predicted costs should be lower than the tariff due to the need for the owner to make a profit on the trip.

Hine and Chilver (1991) give a good example and discussion on using freight tariffs for VOC model calibration. Using comprehensive data collected in Pakistan, a comparison was made

[1] One major problem lies in being able to accurately establish the amount of tyre wear. Since this is a small amount weighing the tyre before and after a set of measurements often does this. However as described in *Transit (1997)*, even when using the most controlled conditions significant errors may arise.

between observed costs and the VOC predictions from the RTIM2, HDM-III Brazil and HDM-III India VOC models with observed costs.

There are two components which are reflected in the tariff, the:

- **Non-productive cost** (for example, loading/unloading, repairs, looking for work)
- **Per-km distance cost**

Thus, the cost can be expressed as:

$$\text{COST} = \text{a0}\ \text{DIST} + \text{a1} \qquad \ldots(3.4)$$

where:

COST	is the total cost of the trip
DIST	is the trip distance
a0	is the marginal cost due distance travelled
a1	is the fixed cost

The tariff rate can then be expressed as:

$$\text{RATE} = \text{a0} + \frac{\text{a1}}{\text{DIST}} \qquad \ldots(3.5)$$

where:

RATE	is the cost per km

NDLI (1997) used medium truck tariffs to check VOC predictions in India. Data were collected from nine operators in the form of the tariffs charged for a standard load to travel various distances. The data were quite variable, reflecting the different charges by operators for trips of the same length. A regression analysis of the data gave the following equations:

$$\text{COST} = 6.62\ \text{DIST} + 562 \qquad \ldots(3.6)$$

$$\text{RATE} = 6.62 + \frac{562}{\text{DIST}} \qquad \ldots(3.7)$$

The analysis indicated that the marginal **financial** cost for truck travel was 6.6 Rupees/km. The model was run using the financial unit costs instead of the economic costs. The predicted total cost was 6.2 Rupees/km, a difference of 6 per cent. To evaluate the predicted costs against the raw tariff data the total trip cost was calculated using the same constant cost of 562 Rupees/trip as from the tariff data. The results are plotted in Figure 3.5 against the original tariff data (*NDLI, 1997*). It was concluded that the model was giving a reasonable representation of the observed costs.

For passenger cars, taxi rates are often a ready source of data for evaluating the model predictions. The rates need to be adjusted for the value of driver time.

3.6.5 Road User Effects: fleet survey

Fleet surveys are often a good source of data, particularly when there are operators available with good records. For example, *Hine and Chilver (1991)* used driver logbooks to investigate truck costs in Pakistan while *du Plessis et al. (1991)* used bus company records in South Africa.

Transit (1995) gives a good discussion of potential sources of data for a study in New Zealand. It was found that there were a number of public and private organisations with data covering a range of vehicles and operating conditions. There were two commercial companies managing and recording vehicle operating cost data for their clients, and one of these was used to investigate maintenance costs. It was also found that many private operators had good databases with the costs recorded to a very disaggregate level.

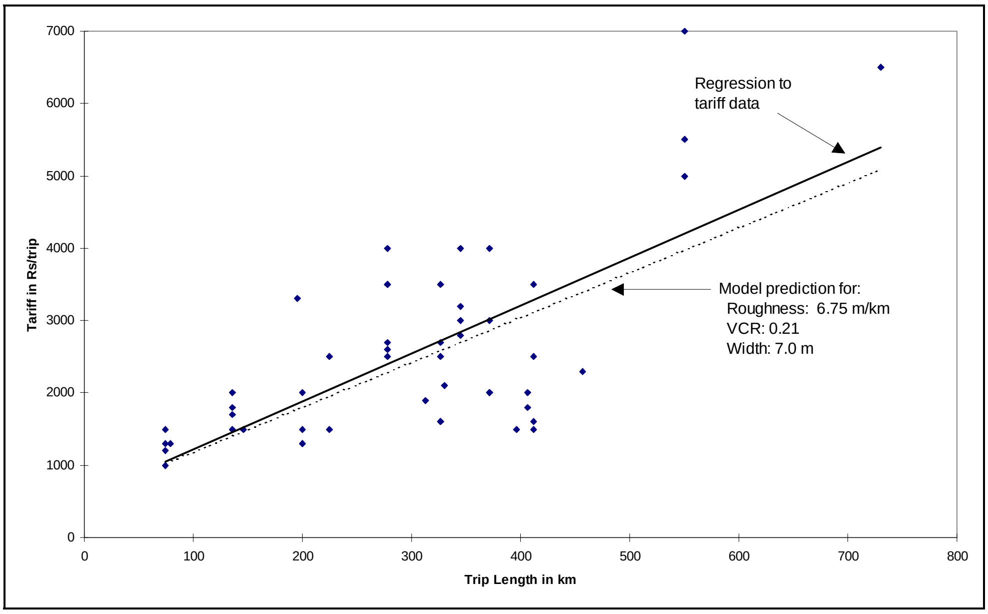


Figure 3.5 Comparison of predicted and tariff survey trip costs in Gujarat

du Plessis and Schutte (1991) contain a number of papers describing several studies conducted by the CSIR into South Africa that involved fleet surveys. These provide good guidance on how to approach a fleet study and analyse the data.

du Plessis et al. (1991) studied a single bus company since this eliminated any inter-company effects, although it was noted that the seven local depots had some latitude with their maintenance practices to reflect local conditions. The company operated on a range of terrain and road types. The data were grouped into depots in flat, rolling and mountainous terrain and average attributes were established for each depot (for example, roughness, and operation on different road types).

It was noted that "any direct attempt to relate costs to roughness may yield misleading results because of other factors such as bus age and maintenance policy that may play a role". The study was therefore initiated at each depot where the actual policies were investigated. This also allowed for discussions with the local maintenance managers that provided valuable insights into local conditions that were influencing the costs. For example, it was found that the tyre consumption was influenced by the angularity of gravel through casing penetrations.

The data were used to develop relationships that were compared to HDM. The results broadly confirmed the validity of the HDM predictions.

Findlayson and du Plessis (1991) conducted a similar study into trucks that developed maintenance, tyre and depreciation relationships. These also gave similar trends to HDM.

4 Sensitivity of HDM

4.1 Introduction

It is important for users to be aware of the general level of sensitivity of the model to each parameter[1] so that appropriate emphasis can be given to important parameters and less emphasis to second or third order effects. The influences of individual parameters differ according to the particular parameter, the particular result being considered, and the values assigned to other parameters in the particular analysis. The sensitivity of results to variations in a parameter therefore varies somewhat under different circumstances.

Sensitivity analyses were conducted with the HDM RUE[2] and RDWE sub-models so as to determine the levels of sensitivity and to rank them[3]. Sensitivity was quantified by the **impact elasticity**, which is simply the ratio of the percentage change in a specific result to the percentage change of the input parameter, holding all other parameters constant at a mean value.

For example, if a 10 per cent increase in traffic loading causes a 2.9 per cent increase in roughness developed after 15 years, the impact elasticity term of traffic loading for that roughness result is 0.29. If there were a 2.9 per cent decrease, the value would be -0.29.

As described by *Mrawira et al. (1998)*, there are different approaches which can be used for undertaking sensitivity analyses. The approach used here is the traditional **ceteris paribus** method: changing a single factor while holding all others constant. The alternative approach, using factoral experiments which combine all the levels of one factor with all levels of all other factors, were not used due to the large number of combinations to consider. Thus, the analysis here does not consider factor interactions. *Mrawira et al. (1998)* describes the results of using a factor approach for an HDM sensitivity analysis.

On the basis of the analyses, four classes of model sensitivity have been established as a function of the impact elasticity. The higher the elasticity, the more sensitive the model predictions. These classes are listed in Table 4.1. Throughout the remainder of this report the terms S-I to S-IV will be used to refer to the various sensitivity classes.

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1 In order to run HDM it is necessary to supply the basic input data and values for the model coefficients; for simplicity both of these are termed **parameters**.

2 As described in *Bennett (1999)*, the analyses were done using a stand-alone version of the HDM-4 RUE sub-model developed for this purpose.

3 The authors gratefully acknowledge the assistance of Rodrigo Archondo-Callao of the World Bank who performed the RUE sensitivity analysis for HDM-III.

Table 4.1
HDM sensitivity classes

Impact	Sensitivity class	Impact elasticity
High	S-I	> 0.50
Moderate	S-II	0.20 - 0.50
Low	S-III	0.05 - 0.20
Negligible	S-IV	< 0.05

Practitioners, as a guide to where their efforts should be directed, should use the results of these sensitivity analyses. Those data items or model coefficients with moderate to high impacts (S-I and S-II) should receive the most attention. The low to negligible impact (S-III and S-IV) items should receive attention only if time or resources permit. One usually assumes the default HDM values for S-III and S-IV items since these will generally give adequate results.

4.2 Road User Effects

RUE are comprised of the VOC, travel time, vehicle emissions (noxious gases and noise), safety, energy use along with developmental effects. This release of the report only considers VOC.

As described in Chapter 2, the HDM RUE sub-model predicts the amount of resources consumed; for example the amount of fuel and tyres. These are multiplied by the unit costs of the resources to obtain the total cost. There are a number of different components modelled and, as shown in Figure 4.1 and Figure 4.2, they are influenced by different factors. These figures also highlight the increased sophistication of RUE modelling in HDM-4 over HDM-III.

RUE are mainly influenced by vehicle speeds and roughness. The influence of these factors varies depending upon the RUE component. For example, fuel consumption is very sensitive to speed but relatively insensitive to roughness. By comparison, parts consumption is insensitive to speed and heavily influenced by roughness.

Analysts need to be aware of these differences since they can have a marked bearing on the data collected in a study. If the study is principally dealing with capacity improvements that will influence speeds, then factors such as roughness will be of lesser importance. However, road maintenance studies that will lead to major changes in roughness will need to have good data on roughness.

When considering RUE there are two situations that may be of interest, namely the:

- **Magnitude of the total RUE**
- **Effect of operating conditions on RUE**

The former is important in situations such as when there are different route lengths. The effects of operating conditions on RUE are important when you are comparing changes to road condition.

As described in *Bennett (1999)*, sensitivity analyses were conducted which considered both of these issues. The first set of analyses considered the sensitivity of the total VOC on a per-km to changes in the input data. The second considered the effects of roughness changes. For the latter, the sensitivity of the difference in the total VOC between 3 and 5 IRI m/km to changes in the input data was tested.

In order to cover the range of conditions encountered in different studies, the analyses were done with unit cost data adapted from actual HDM studies from Australia, India, and Thailand. These three data sets represent countries with markedly different levels of economic development. Using these different studies meant that the relative contribution of each VOC component was different thereby influencing the sensitivities. This ensured that the results are broadly representative of the full range of conditions where the model will be applied.

The individual data items were varied under a range of operating conditions and their impact elasticities (see Table 4.1) were determined. On the basis of the results of these analyses, the variables were assigned to the different sensitivity classes shown in Table 4.2 for HDM-4. For reference purposes, the results from a similar analysis with HDM-III are given in Table 4.3.

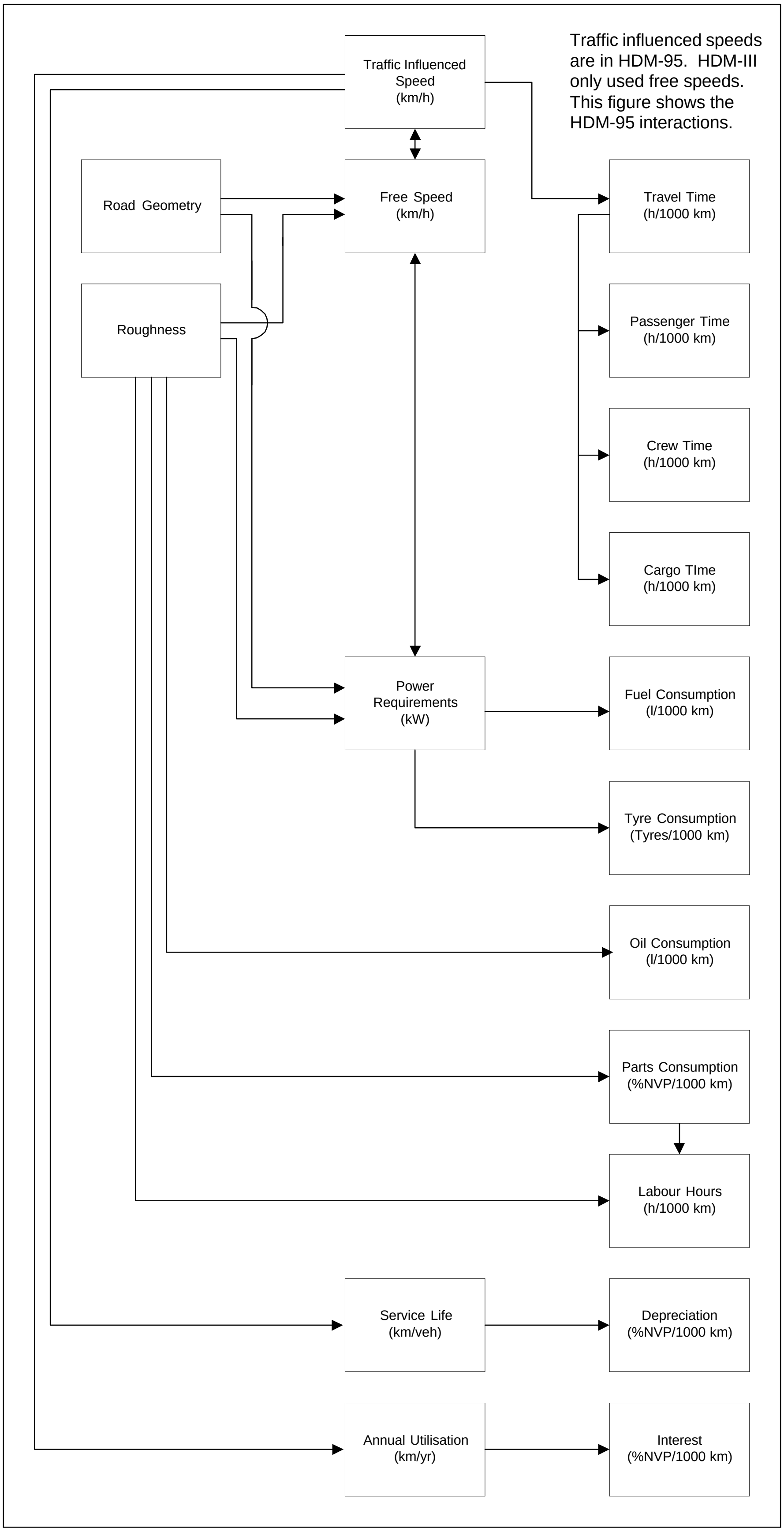


Figure 4.1 HDM-III/95 Road User Effects components and their interactions

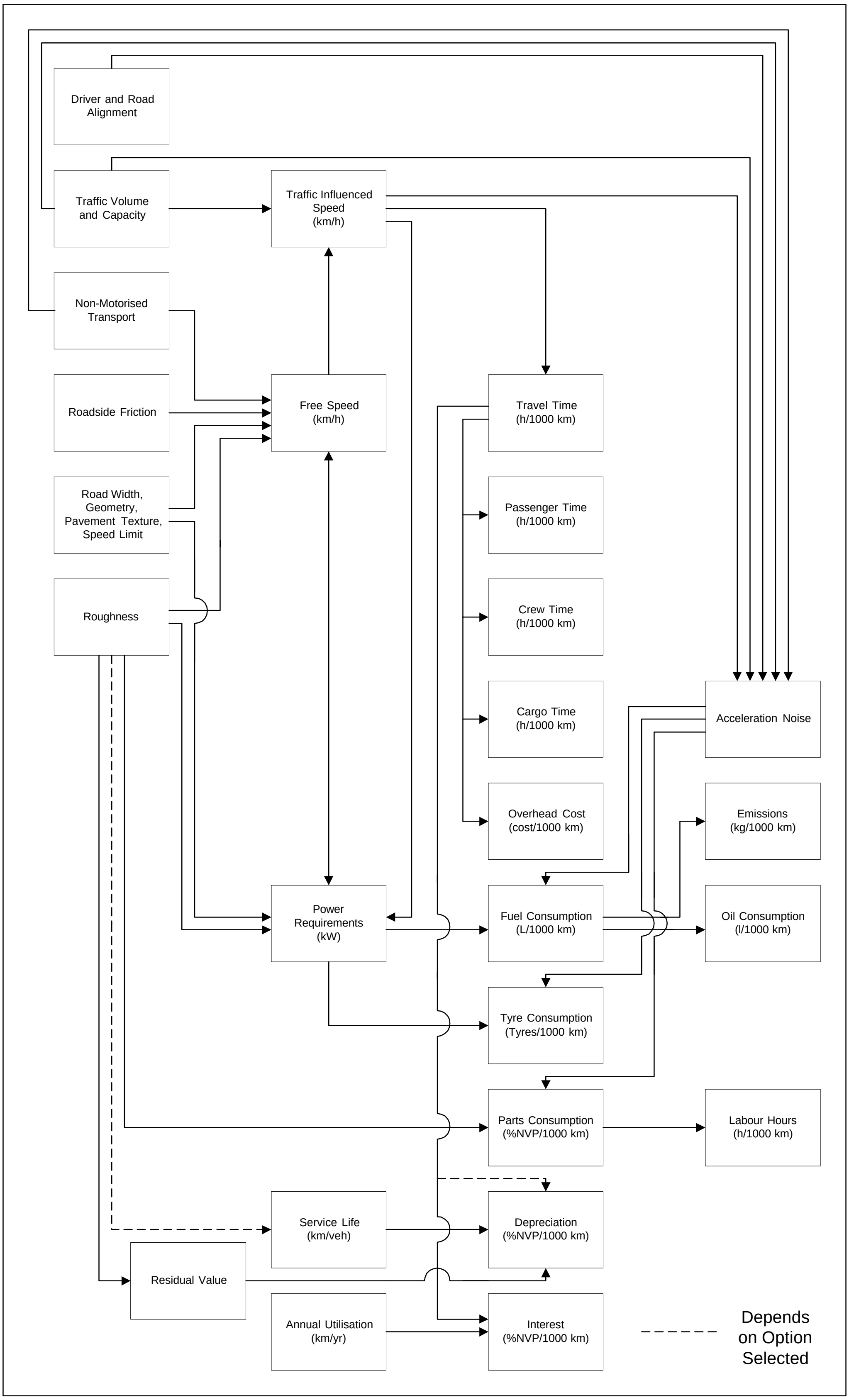


Figure 4.2 HDM-4 Road User Effects components and their interactions

Table 4.2
Sensitivity classes for HDM-4 RUE model[1]

Sensitivity class[2]	Impact elasticity	Variables or data important for total VOC[3]	Variables or data important for VOC savings[4]
S-I	> 0.50	Replacement vehicle price Parts model exponent kp	Replacement vehicle price Parts model exponent kp
S-II	0.20 - 0.50	Roughness in IRI Parts model a1	Roughness in IRI Parts model a1
S-III	0.05 - 0.20	Aerodynamic drag coefficient Aerodynamic drag multiplier Cdmult Annual utilisation Base fuel-power factor ξb Cost of fuel Cost of maintenance labour Cost of passenger work time Desired speed Driveline efficiency Driving power Engine accessories power factor Fuel ξ Interest rate Labour hours calibration factor Mass density of air Maximum average rectified velocity NMT friction factor XNMT Number of passengers Number of wheels Parts model a0 Posted speed limit Projected frontal area Rolling resistance CR1 Rolling resistance CR2 a0 Rolling resistance factor CR2 Roughness-speed a0 Side friction factor XFRI Speed bias correction factor Speed limit enforcement factor Vehicle mass Vehicle service life Wheel diameter	Aerodynamic drag coefficient Aerodynamic drag multiplier Cdmult Annual utilisation Base fuel-power factor ξb Cost of fuel Cost of maintenance labour Cost of passenger work time Cost of tyre Driveline efficiency Driving power Engine accessories power factor Engine idle speed Fuel ξ Interest rate Labour hours calibration factor Mass density of air Maximum average rectified velocity Number of passengers Number of wheels Parts model a0 Posted speed limit Projected frontal area Rolling resistance CR1 Rolling resistance CR2 a0 Rolling resistance factor CR2 Roughness-speed a0 Speed β Speed limit enforcement factor Vehicle mass Vehicle service life Volume of wearable rubber Wheel diameter
S-IV	<0.05	All others	All others

Notes:

1 The variables listed here are defined in *Bennett and Greenwood (1999).*

2 This is the highest class found over all common applications.

3 These are the variables important in determining the total RUE.

4 These are the variables important in determining the effect of roughness on RUE.

4.3 Sensitivity classes

4.3.1 High impacts, Class S-I (> 0.5)

For both sets of analyses the results indicate that the only high impact parameters are the replacement vehicle price and the parts model exponent kp. The former is used to calculate parts, depreciation and interest costs. The latter governs the magnitude of the parts consumption and the impact of vehicle age on parts consumption.

Table 4.3
Sensitivity classes for HDM-III RUE model[1]

Sensitivity class[2]	Impact elasticity	Variables or data important for total VOC[3]	Variables or data important for VOC savings[4]
S-I	> 0.50	kp - parts model exponent New vehicle price	kp - parts model exponent New vehicle price CSPQI - parts model roughness term C0SP - parts model constant term
S-II	0.20 - 0.50	Roughness E0 - speed bias correction Average service live Average annual utilisation Vehicle weight	E0 - speed bias correction ARVMAX - max. rectified velocity CLPC - labour model exponent
S-III	0.05 - 0.20	Aerodynamic drag coefficient Beta - speed exponent BW - speed width effect Calibrated Engine Speed CLPC – labour model exponent C0SP - parts model constant term CSPQI - parts model roughness term Crew/Cargo/Passenger cost Desired speed Driving power Energy efficiency factors Fuel cost Hourly utilisation ratio Interest rate Projected frontal area	Beta - speed exponent Vehicle age in km C0LH - labour model constant term Labour cost Hourly utilisation ratio BW - speed width effects Number of tyres per vehicle New tyre cost Lubricants cost Crew/Cargo/Passenger cost Vehicle weight Number of passengers
S-IV	<0.05	All others	All others

Notes:

1 The variables listed here are defined in *Watanatada et al. (1987a)*.

2 This is the highest class found over all common applications.

3 These are the variables important in determining the total VOC.

4 These are the variables important in determining the effect of roughness on VOC.

4.3.2 Moderate impacts, Class S-II (0.2 - 0.5)

The moderate impact parameters are the roughness and the parts consumption model parameter, a1, that governs the roughness effects.

4.3.3 Low impacts, Class S-III (0.05-0.2)

A large number of parameters fall into the low impact category. These cover a range of attributes and include most of the unit costs. The only major differences between the two analyses, total costs versus roughness effects, is in the importance of the side friction factor (XFRI) and non-motorised factor (XNMT) in the total cost analysis. This is because of the importance of the speeds in the total costs whereas the roughness-speed effects were important with the roughness costs.

4.3.4 Negligible impacts, Class S-IV (< 0.05)

By far the majority of the HDM-4 data items have a negligible impact on the results. Under some circumstances this changes; for example, gradient was S-III when the grades were significant, but for most HDM-4 analyses the default values for these should be adopted.

4.4 Road deterioration and works effects

The RDWE sub-model predicts the deterioration of the pavement over time and under traffic. When maintenance is applied, the pavement condition is improved and the volumes of material applied are multiplied by their unit costs to establish the cost of treatment.

In common with the RUE sub-model, there are a series of interdependencies in the modelling of RDWE. These are illustrated in Figure 4.3. It will be noted that surface distresses have separate initiation and progression models. This is illustrated in Figure 4.4.

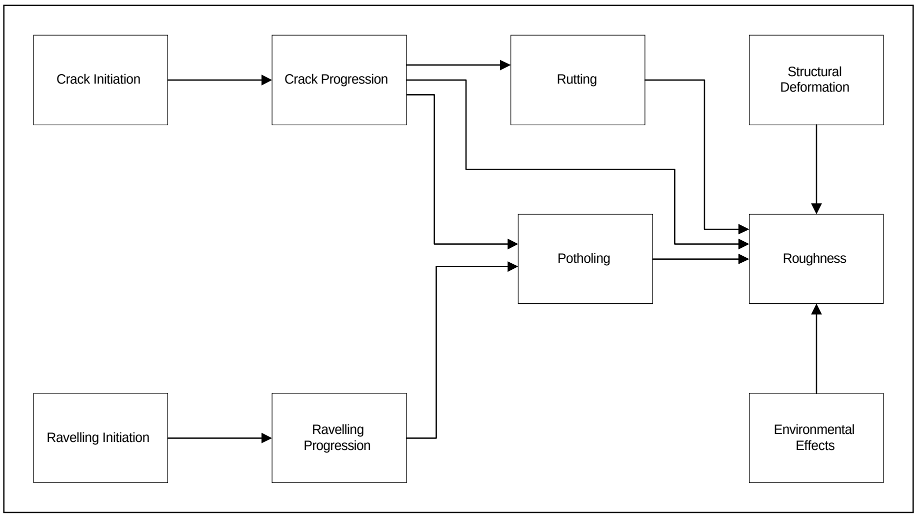


Figure 4.3 RDWE distress interactions

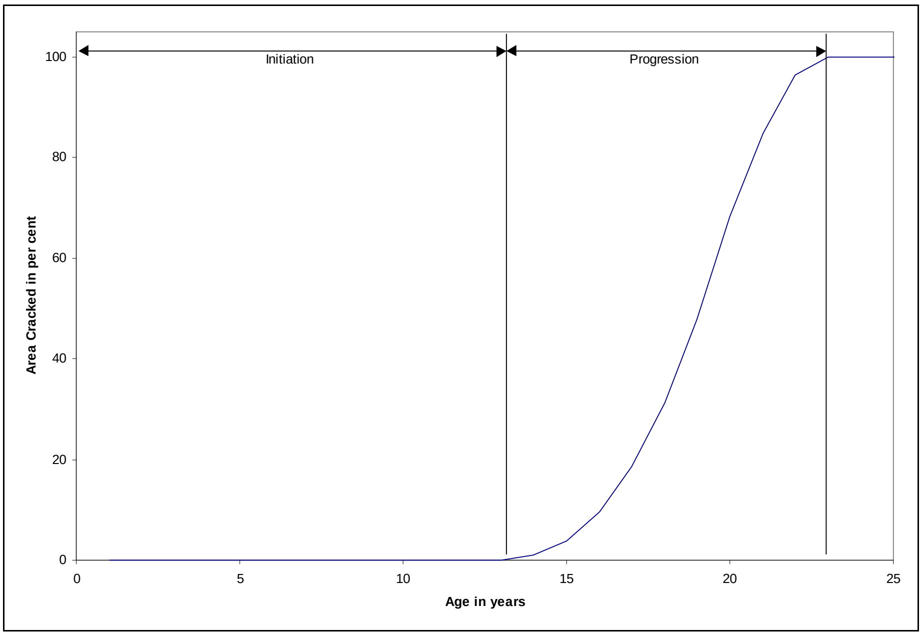


Figure 4.4 Surface distress initiation and progression

The sensitivity of road deterioration and maintenance predictions to variations in individual input parameters is summarised in the extreme right-hand column (see Table 4.4). The generalised ranking is made in the same four ranges of input elasticity as are used for RUE effects. For the RDWE, however, they are based on the impacts on physical conditions as well as on costs and economic returns because they influence the performance of the pavement, the timing of maintenance, and the economic returns and likely priority of the various treatments.

4.4.1 High impacts, Class S-I (> 0.5)

The most sensitive inputs include pavement structural variables, traffic and roughness, which are all measured data items. Variations in the structural variables, Modified Structural Number or deflection, and annual traffic loading (in million ESA), affect most major results including periodic maintenance and rehabilitation alternatives and the economic returns. They are sensitive to this high degree only when the pavement structural adequacy (PSA – see Section 4.2) is low or moderate compared with the loading (under other conditions when the PSA is high to moderate the impacts are moderate, that is, Class S-II). Variation of the traffic volume, which determines the number of users deriving benefits, has strong influence on the economic returns but low influences (S-II or S-III) on physical impacts. Variation of the pavement roughness, which affects the unit savings gained by each user, has high impacts on all economic results and on rehabilitation needs, but little on surface distress (S-III).

4.4.2 Moderate impacts, Class S-II (0.2 - 0.5)

Variations in pavement and surfacing ages have moderate impacts on the needs and timing of periodic maintenance, and low impact on economic returns. Variation in the amounts of all and wide cracking have moderate impacts on the economic returns for maintenance, strong impacts on maintenance needs, but low impacts on economic returns for rehabilitation. The calibration factors for adjusting predictions of environment-roughness effect, roughness progression, cracking initiation and cracking progression have mainly moderate impacts (some high and some low) on maintenance and rehabilitation needs and economic returns. Variations in the overlay thickness and the unit costs of all treatment types have moderate impacts on economic returns.

4.4.3 Low impacts, Class S-III (0.05-0.2)

Variations in the amount of potholing and the volume of heavy vehicles (as distinct from the loading) have high direct impacts on the amount of potholing and the economics of patching, but generally low impacts on maintenance and rehabilitation needs. Note, however, that the presence or not of potholing has moderate impacts in the absence of a patching alternative. Variations in the mean rut depth and standard deviation, and the rut depth progression adjustment factor, have either low or negligible impacts on cracking maintenance intervention, roughness intervention, and economic returns. Reseal thickness has low impacts on economic returns of maintenance.

4.4.4 Negligible impacts, Class S-IV (< 0.05)

Variations in the subgrade compaction and rainfall parameters have low impacts on rut depth progression and negligible impacts overall on interventions and economic returns. Other rainfall effects are implicit in the pavement strength (Structural Number or deflection) and are not linked to the rainfall parameter. Variations in the area of ravelling and the ravelling progression factor affect potholing but overall have negligible impacts on cracking or roughness intervention and economic returns. Variations in pavement deflection have negligible impacts when the Modified Structural Number is also provided.

Table 4.4
Sensitivity classes for RDWE variables

Sensitivity class	Impact elasticity	Parameter	Outcomes most impacted		
			Pavement performance	Resurfacing and surface distress	Economic return on maintenance
S-I	> 0.50	Structural number [1/]	•	•	•
		Modified structural number[1/]	•	•	•
		Traffic volume			•
		Deflection[3/]	•	•	•
		Roughness	•		•
S-II	0.20-0.50	Annual loading	•	•	•
		Age		•	•
		All cracking area		•	•
		Wide cracking area		•	•
		Roughness-environment factor	•		•
		Cracking initiation factor	•	•	•
		Cracking progression factor		•	
S-III	0.05-0.20	Subgrade CBR (with SN)	•		
		Surface thickness (with SN)		•	•
		Heavy axles volume		•	•
		Potholing area	•	•	
		Rut depth mean	•		
		Rut depth standard deviation	•		
		Rut depth progression factor	•		
		Roughness general factor	•		•
S-IV	< 0.05	Deflection (with SNC)		•	
		Subgrade compaction	•		•
		Rainfall (with Kge)	•		
		Ravelling area		•	
		Ravelling factor		•	

Notes:

1 Only one of the structural parameters is required, the other two are optional. When the structural capacity is high relative to the traffic loading these parameters are in class S-II.

4.5 Economic models

To be included in a subsequent edition of this document

5 Adapting data to the model

5.1 Introduction

Data represent the particular information that describes the case being analysed and are the specific inputs to the model that are required of the user. Examples include traffic volume and composition, road geometry, pavement type and condition, unit costs, etc. These data items are thus fundamentally different from the internal parameters and coefficients of the underlying simulation model which need calibration; an error in a data item means that the model is analysing a different case from the one intended, much like entering a wrong destination in a flight instruction.

For example, the accuracy of input data can have substantial impact on the timing of future interventions, sometimes more important than the deterioration rate. This is because HDM uses incremental models and the existing condition is the start point for the modelling. This is illustrated in Figure 5.1 which shows for the same intervention criteria a difference in the initial condition has markedly different times for future interventions.

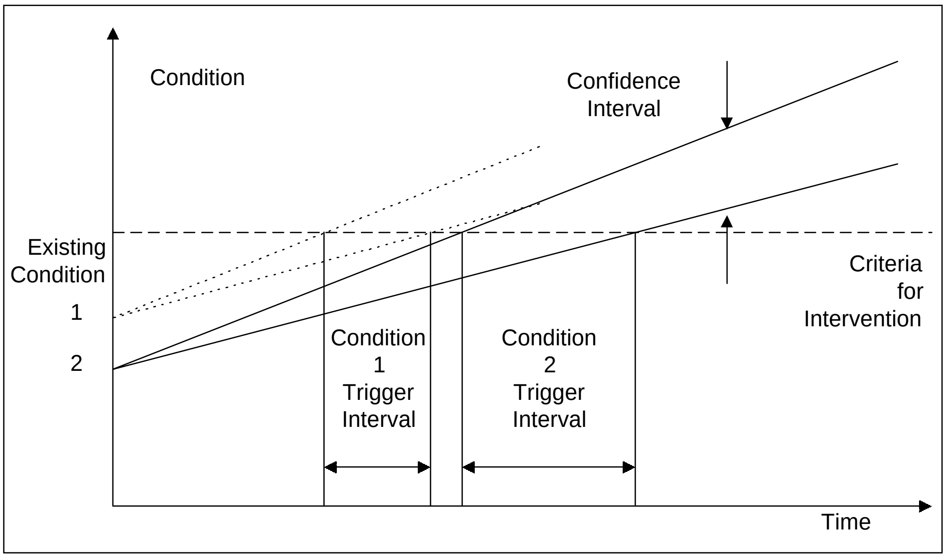


Figure 5.1 Effect of existing condition on triggering maintenance

Just as information can be either imprecise or very precise, so the HDM model can be operated with either very simple information or much more detailed information. However, by the nature of simulation models and software, the input parameters of the model are fixed and explicit. So this chapter describes how the user adapts various levels of locally available data (whether simple or complex) to the required input parameters, and later reconverts the results into local formats, where desired.

Finally, we look at how the user handles the related issues of accuracy and approximation. These often have significant cost implications for the operation of a road management system but also have an impact on the reliability of results and decisions.

5.2 Concept of Information Quality Level (IQL)

An item of information can be presented in either simple or detailed terms. Viewed through a lens, the image of an object from a distance or great height will be seen as an outline and in general features. Close-up or at low heights, the amount of detail seen increases and other features or **attributes** of the object can be identified. The object, or information, is the same but the quality of information has been enhanced. In some instances the general outline or overall situation is the quality of information which is required; that is, the high-level or macro-level information, whereas in other instances the greater detail (micro-level) is what is required.

The concept of Information Quality Levels (IQL), defined by *Paterson and Scullion (1990)*, allows us to structure road management information in ways that suit the needs of different levels of decision making and the variety of effort and sophistication of methods for collecting and processing data. In the IQL concept, very detailed information at a low level (low-level data) can be condensed or aggregated into progressively fewer items at successively higher levels of IQL (high-level data) as shown in Figure 5.2.

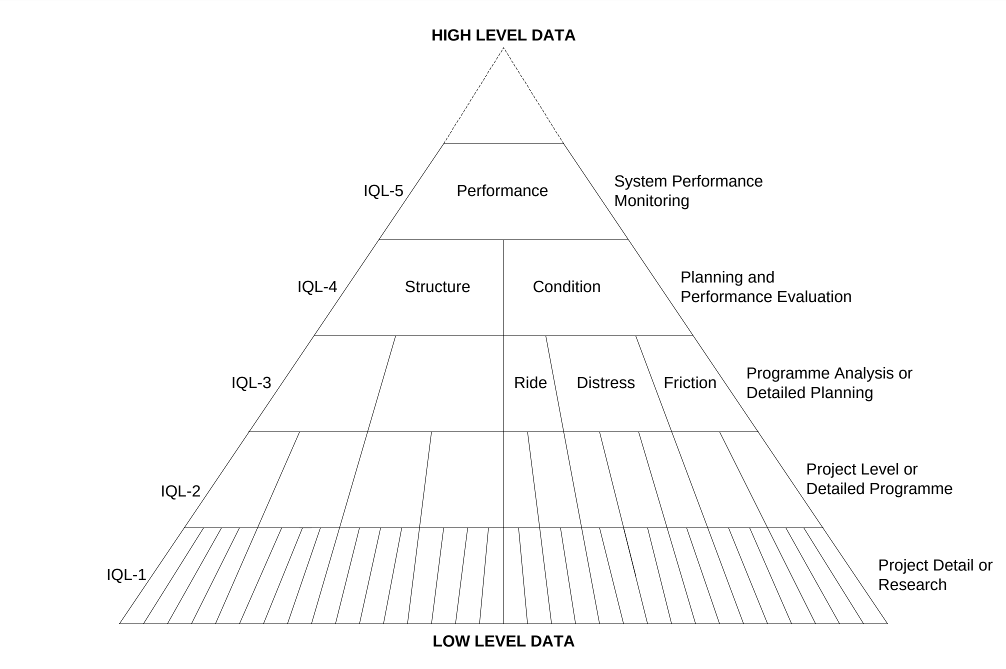


Figure 5.2 Information Quality Level concept

In road management, five levels have been identified for general use, as defined in Table 5.1:

- **IQL-1**

  Represents the following fundamental type data:

  - **Research**
  - **Laboratory**
  - **Theoretical**
  - **Electronic**

  where many attributes may be measured or identified.

- **IQL-2**

  Represents a level of detail typical of many engineering analyses for a project-level decision.

- **IQL-3**

  A simple level of detail (simpler than IQL 1 or IQL 2), typically two or three attributes, which might be used for large production uses like network-level survey or where simpler data collection methods are appropriate.

- **IQL-4**

  A summary or key attribute which has use in planning, senior management reports, or alternatively in low effort data collection.

- **IQL-5**

  Represents a top level such as key performance indicators, which typically might combine key attributes from several pieces of information. Still higher levels can be defined when necessary.

Table 5.1
Classification of Information Quality Level and detail

Level	Amount of detail
1	Most comprehensive level of detail, such as would be used as a reference benchmark for other measurement methods and in fundamental research. Would also be used in detailed field investigations for an in-depth diagnosis of problems, and for high-class project design. Normally used at project-level in special cases, and unlikely to be used for network monitoring. Requires high level of staff skills and institutional resources to support and utilise collection methods.
2	A level of detail sufficient for comprehensive programming models and for standard design methods. For planning, would be used only on sample coverage. Sufficient to distinguish the performance and economic returns of different technical options with practical differences in dimensions or materials. Standard acquisition methods for project-level data collection. Would usually require automated acquisition methods for network surveys and use for network-level programming. Requires reliable institutional support and resources.
3	Sufficient detail for planning models and standard programming models for full network coverage. For project design, would suit elementary methods such as catalogue-type with meagre data needs, and low-volume road/bridge design methods. Able to be collected in network surveys by semi-automated methods or combined automated and manual methods.
4	The basic summary statistics of inventory, performance and utilisation, of interest to providers and users. Suitable for the simplest planning and programming models, but for projects is suitable only for standardised designs of very low-volume roads. The simplest, most basic collection methods, either entirely manual or entirely semi-automated, provide direct but approximate measures, and suit small or resource-poor agencies. Alternatively, the statistics may be computed from more detailed data.

Table 5.3 serves to introduce the concept. At IQL-1, pavement condition is described by twenty or more attributes. At IQL-2, these would be reduced to 6-10 attributes, one or two for each mode of distress. At IQL-3, this reduces to 2-3, namely roughness, surface distress, and texture or skid resistance. At IQL-4, this reduces to one attribute, Pavement condition (or state or quality) which may have been measured by class values (**good**, **fair**, **poor**) or by an index (for example, 0-10). An IQL-5 indicator would combine pavement quality with other measures such as structural adequacy, safety aspects, and traffic congestion; that is representing a higher order information such as road condition.

From these definitions three observations arise:

- It can be observed that as the decision-level rises, so the IQL that is appropriate also rises. Information at IQL-4 or IQL-5 is appropriate for performance indicators and road statistics that are of interest to senior management and the public because they tend to be, or should be, easily understood without much technical background. At project-level, however, the appropriate IQL depends much more on the standard of the project and the resources of the agency:

  - For a rural road or a small local agency, IQL-3 is usually sufficient, being simple but effective for the purpose
  - For most agencies and main roads IQL-2 is typical, but
  - For expressways or a high-powered well-funded agency IQL-1 may be used in some instances.

  The criterion to use in selecting the appropriate IQL is to ask 'is the decision likely to be altered by having more detailed information?'

- The second observation is that primary data collection at a low-level or detailed IQL typically costs more and involves more complex or sophisticated equipment than collection of higher IQL data. Thus, the IQL for primary data collection which is appropriate to a given agency and situation depends on:

  - Financial and physical resources
  - Skills
  - Cost
  - Speed or productivity
  - Degree of automation
  - Complexity

  all summed up in the need for the method to be sustainable for the intended purpose, such as the regular operation of a road management system.

- A third observation is that a higher level IQL represents an aggregation or transformation of the lower level IQL. When there is a specific rule or formula for conversion, from say IQL-2 into IQL-3, then the information is reproducible and reliable. Thus, when the appropriate IQL is chosen, the data can be re-used through transformation to the higher IQLs as the decision-making moves up the project cycle, this avoids the need for repeating surveys and saves cost.

5.3 Relating the local IQL to the HDM model

The HDM model, in both versions HDM-III and HDM-4, operates internally at a level that is primarily IQL-2. This fairly detailed level of modelling was necessitated by the demands to make the model as universally applicable as possible and that could only be achieved by adopting fundamental, mechanistic and structured empirical formulations that would operate as close to first principles as would be practical. The downside of this fairly detailed level is that many users of HDM-III over the past decade felt the need to collect all the input data at that same IQL-2 level, or rejected the model as too complex, when this was not in fact necessary. In HDM-4, some of the more common data input simplifications have been provided in the form of buttons for built-in conversions, which provide for example an IQL-3 approximation to the IQL-2 internal inputs, so it can be used at either level.

Local data can be adapted for use with HDM by determining a conversion for transforming the data into the HDM parameters, and vice-versa. The basic approach for adapting local data to HDM is first to transform the local data (from whichever IQL they may have) into IQL2 input parameters, and later to transform outputs from IQL-2 to the user's desired output form, typically IQL-3 or IQL-4. Additional data collection would only be required when essential information items were missing. The approach is summarised as follows, and illustrated in Figure 5.3.

1 The local data items are sorted into groups that relate to the HDM input parameter groups (these groups are addressed in the following four sections).

2 Identify and record the IQL of each local data group (for information purposes), note that where the IQL of the data appears to be mixed it would be worthwhile to consider a separate review to normalise them around the most appropriate IQL.

3 If the local data is IQL-1, then the items will need to be combined or aggregated into the HDM inputs, usually this is a process of selection rather than conversion since the HDM inputs are directly measurable parameters such as area. These transformations are external to the model.

4 For local data of IQL-2, there may be a need for conversion of measurement units to the international units used in HDM, this is external to the model.

5 For local data of IQL-3, the information is less detailed than the HDM inputs; that is, one attribute needs to be subdivided into two or more attributes. Thus, it is only possible to estimate what the other attribute values might be and the answer is not unique. The approximations will use an average or mid-point value, assuming that values above and below will compensate each other and the final result will be reasonably accurate. The conversion can be made outside HDM. However, for those items that have an internal transformation built into HDM-4, the conversion will need to be checked against real local data to ensure that it is reasonable and to adjust the conversion as needed.

6 For local data of IQL-4, the information is even less detailed than HDM and the approximation is somewhat greater, but the procedures for adaptation are similar to those for IQL-3. This is unimportant if the data item is in Sensitivity classes S-III or S-IV, but deserves review if the data item is S-I or S-II, in which case consideration should be given to changing the data collection to a more detailed IQL.

7 For adaptation of the output data to local items and terminology, the reverse process may need to be applied. The same conversion formula should be used (in reverse) to ensure that the model's results will be reflected correctly in the local terminology. Adaptation is generally less of a problem for output data since there are fewer variations in the IQL-3 and IQL-4 definitions.

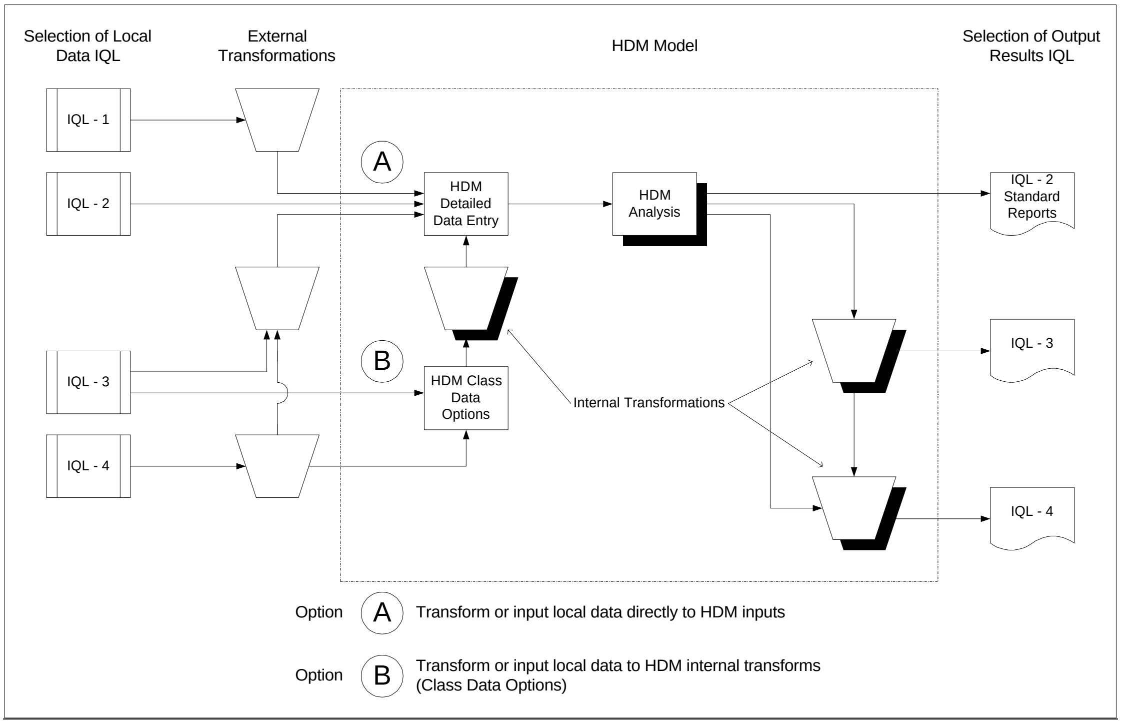


Figure 5.3 Data conversion process

5.4 Adaptation of local condition data units

When condition data have been collected using different units to those in HDM it is necessary to develop a transfer function to convert these data. This function can either be a mathematical relationship or a table.

The function is developed by conducting parallel surveys with both the local and HDM measures on a common sample of road sections. A total of 15-20 sections for paved roads and 10-15 sections for unpaved roads constitute an adequate sample size for most conversions. The total length of sections should be at least 20 km for each pavement type (that is, bituminous paved, rigid paved, unpaved).

An example of the conversion of units from Niger is shown in tabular form in *(Paterson, 1986*) and in graphical form in Figure 5.4. The data were based on a study of many sections with a total length of 120 km of paved roads. On each section the values of the appropriate HDM parameter and the local condition score were recorded. The rating for surface distress (Enduit) included ravelling, potholing and bleeding collectively. This was matched with representative areas of ravelling and potholing as shown in Table 5.2. The rating for cracking (Fissuration) was done from a moving vehicle so it was only related to wide cracks since narrow cracks cannot be observed in this manner. The local Deformation score included depressions, average rut depth and the subjective rating of roughness so these were broken into the individual components.

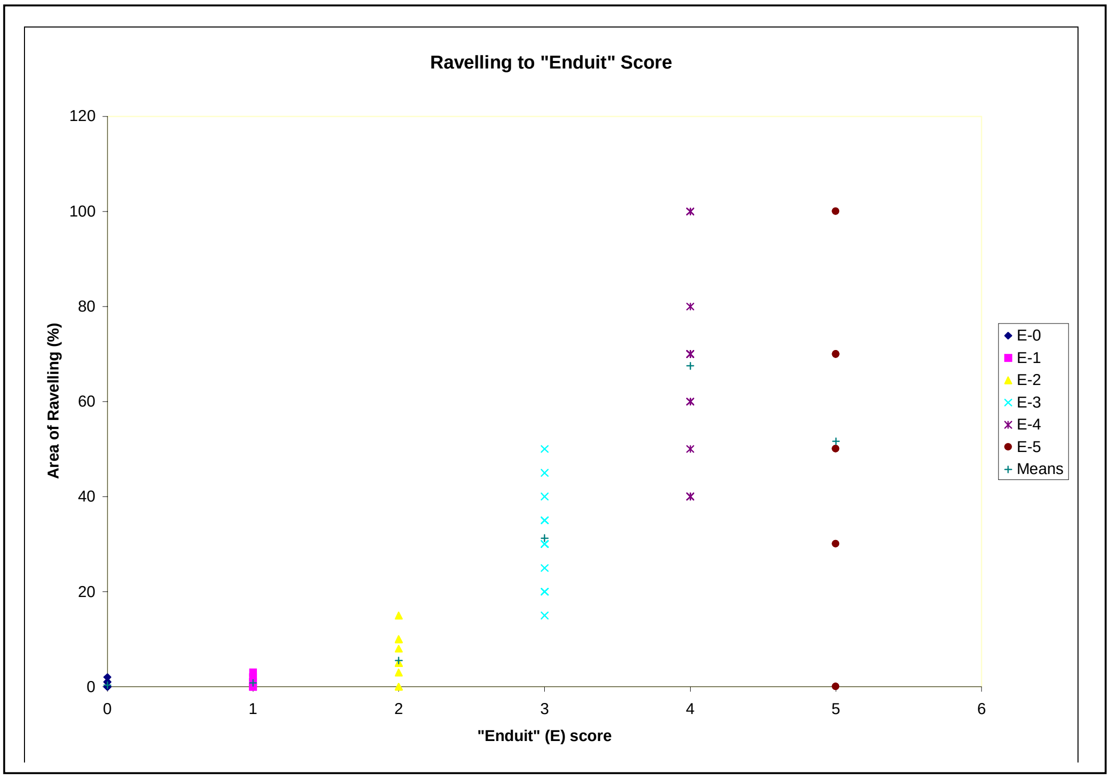


Figure 5.4 Graphical form of conversion units - Niger investigation

Table 5.2
Conversions between condition units and HDM-III input parameters - example from Niger office of road management

Distress mode	Symbol	Approximate conversion between units					
Surface distress (Enduit)	E	0	1	2	3	4	5
Area of ravelling (%)	ARAV	0	0	5	30	70	50
Area of potholing (%)[1]	APOT	0	0	0	0	0.08	1.0
Cracking (Fissuration)	F	0	1	2	3		
Area wide cracking (%)[2]	ACRW	0	5	25	50		
Deformation (deformation)	D	0	1	2	3		
Area of depressions (%)	ADEP	0	5	10	30		
Roughness (m/km)	IRI	2	3	4	5		
Rut depth[3]	RDM	-	-1	3	5		

Source: *Paterson (1986)*

Notes:

1 The incidence of potholes is recorded only as 4 or 5.

2 Predominantly only wide cracks are visible from a moving vehicles. If the cracks are widely spaced, these values should be reduced applying a factor of 0.4.

3 The rut depths were generally indiscernible (< 5 mm), even for high classes of deformation.

5.5 Transforming road input data

In Table 5.3 the four groups of input data for the road infrastructure:

- **Road geometry**
- **Pavement condition**
- **Pavement structure**
- **Environment**

are presented at four levels of IQL:

- **IQL-2** - the actual level of direct inputs into HDM-III and HDM-4
- **IQL-2B** - a special level of simplified inputs for HDM-4
- **IQL-3** - class-type information for the key attributes within each data group
- **IQL-4** - group-level information

The more detailed level, IQL-1, for each parameter is not shown for reasons of clarity but comprise the fundamental parameters that describe the detailed attributes of an IQL-II data parameter.

Table 5.3 shows the recommended grouping and aggregation of data items. Details of definitions, classification ranges and conversion relationships are given in Appendix F .

5.5.1 Road geometry

HDM-4 requires four primary parameters defining rise and fall and horizontal curvature, plus five that define the speed regime on the road. Continuous devices would measure the IQL-2 but simpler methods of estimation that are suited to visual means are available to estimate those for IQL-2B. A more approximate method in IQL-3 is to classify the vertical alignment into say 4 classes (for example, flat, rolling, moderate, steep) and the horizontal alignment into say 4 classes (for example, straight, fairly straight, curvy, and winding). The most aggregate at IQL-4 combines these into say 6-8 combinations of vertical and horizontal alignment.

5.5.2 Pavement condition

There are 12 data inputs for pavement condition at the full IQL-2, which are typically measured. Those can be simplified and approximated reasonably well by six indices, one for each key mode of distress (roughness, cracking, deformation, disintegration, texture and friction), which are typically estimated by a score or class. A higher level method at IQL-3 simplifies this to three (riding quality class, surface distress index and friction class) which can all be estimated by a trained observer (or alternatively, may be generated from the more detailed measures). At IQL-4, for performance indicators, these are combined into one pavement condition rating, which can be by class values (for example, **good**, **fair**, and **poor**) or an index (for example, pavement quality index).

5.5.3 Pavement structure

There are numerous parameters to be specified for HDM-4 at IQL-2, for example, 15 for bituminous pavements, 3 for concrete pavements, and 14 for unpaved roads. These can be simplified to 8, 2 and 9 respectively for a simpler, class-type method of estimating the inputs at IQL-2B. At IQL-3, these can be reduced to 3 (structural adequacy, construction quality and previous intervention) for bituminous pavements, 1 (structural adequacy) for concrete pavements, and 3 (gravel standard, earth passability and load rating) for unpaved roads. As performance indicators at IQL-4, these can be summarised to one for bituminous and concrete pavements, and one for unpaved roads.

Table 5.3
IQL examples for road data

<table>
<tr><th rowspan="2"></th><th colspan="4">Information Quality Level</th></tr>
<tr><th>IQL-2</th><th>IQL-2B</th><th>IQL-3</th><th>IQL-4</th></tr>
<tr><td rowspan="9">Road geometry</td><td>Rise and fall (m/km)</td><td>Average absolute gradient (%)</td><td rowspan="2">Gradient class (4 classes)</td><td rowspan="4">Geometry class (6-8 classes)</td></tr>
<tr><td>Number of rises and falls (no./km)</td><td>No. of gradient changes (no./km)</td></tr>
<tr><td>Average horizontal curvature (deg/km)</td><td rowspan="2">No. of curves by speed class (class-freq/km)</td><td rowspan="2">Curvature class (4 classes)</td></tr>
<tr><td>Super-elevation</td></tr>
<tr><td>Speed limit (km/h)</td><td rowspan="2">Desired speed (km/h)</td><td rowspan="5">Speed environment (6 classes)</td><td rowspan="5">n/a</td></tr>
<tr><td>Speed limit enforcement factor</td></tr>
<tr><td>Roadside friction (factor)</td><td rowspan="3">Speed reduction factors</td></tr>
<tr><td>Non-motorised transport speed reduction (factor)</td></tr>
<tr><td>Motorised transport speed reduction (factor)</td></tr>
<tr><td rowspan="12">Pavement condition</td><td>Lane roughness (m/km IRI)</td><td>Roughness (6 ranges)</td><td>Ride quality (class)</td><td rowspan="12">Pavement condition (class)</td></tr>
<tr><td>All cracks area (% area)</td><td rowspan="3">Cracking (score, or Universal Cracking Index, UCI)</td><td rowspan="9">Surface Distress Index (SDI)</td></tr>
<tr><td>Wide cracks area (% area)</td></tr>
<tr><td>Transverse thermal cracks (no./km)</td></tr>
<tr><td>Ravelled area (% area)</td><td rowspan="4">Disintegration (score)</td></tr>
<tr><td>Potholes number (units/lane-km)</td></tr>
<tr><td>Edge-break area (m2/km)</td></tr>
<tr><td>Patched area (% area)</td></tr>
<tr><td>Rut depth mean (mm)</td><td rowspan="2">Deformation (score)</td></tr>
<tr><td>Rut depth standard deviation (mm)</td></tr>
<tr><td>Macro-texture depth (mm)</td><td>Surface texture (class)</td><td rowspan="2">Surface friction (class)</td></tr>
<tr><td>Skid resistance (SF50)</td><td>Friction (class)</td></tr>
<tr><td>Pavement structure</td><td colspan="2">Pavement type (class)</td><td>Pavement type (class)</td><td>Pavement class (class)</td></tr>
</table>

… Continued

<table>
<tr><th rowspan="2"></th><th colspan="4">Information Quality Level</th></tr>
<tr><th>IQL-2</th><th>IQL-2B</th><th>IQL-3</th><th>IQL-4</th></tr>
<tr><td rowspan="15">Bituminous</td><td>Pavement Structural Number (adjusted SNP)</td><td rowspan="2">Deflection (mm BB)</td><td rowspan="5">Pavement structural adequacy (index)</td><td rowspan="18">Remaining service life (yrs)</td></tr>
<tr><td>Deflection (mm BB)</td></tr>
<tr><td>Thickness of surfacing (mm)</td><td>Surface thickness (class)</td></tr>
<tr><td>Thickness of base (mm)</td><td>Pavement depth (class)</td></tr>
<tr><td>Shoulder effect (factor)</td><td>Shoulder type</td></tr>
<tr><td>Construction defects surfacing (index)</td><td rowspan="3">Construction quality (index)</td><td rowspan="5">Construction quality (class)</td></tr>
<tr><td>Construction defects base (index)</td></tr>
<tr><td>Relative compaction (%)</td></tr>
<tr><td>Drainage (index)</td><td rowspan="2">Pavement environment (index)</td></tr>
<tr><td>Pavement environment (index)</td></tr>
<tr><td>Pavement age (yr)</td><td rowspan="2">Surfacing age (range)</td><td rowspan="5">Previous intervention (class)</td></tr>
<tr><td>Surfacing age (yr)</td></tr>
<tr><td>Previous all cracking area (%)</td><td rowspan="3">Previous condition (class)</td></tr>
<tr><td>Previous wide cracking area (%)</td></tr>
<tr><td>Previous thermal cracking (number)</td></tr>
<tr><td rowspan="3">Concrete</td><td>Thickness of slab (mm)</td><td>Slab thickness (class)</td><td rowspan="3">Pavement Structural Adequacy (index)</td></tr>
<tr><td>Modulus of rupture (MPa)</td><td rowspan="2">Material strength (class)</td></tr>
<tr><td>Reinforcing steel (%)</td></tr>
</table>

… Continued

	Information Quality Level			
	IQL-2	**IQL-2B**	**IQL-3**	**IQL-4**
Unpaved	*Surfacing Material:*		Gravel standard (class)	Durability standard (class)
	Material Type	Material Type (class)		
	Max. Particle Size (mm)	Material Size (class)		
	Material passing 2.0 mm (%)	Material Gradation (class)		
	Material passing 0.425 mm (%)			
	Material passing 0.075 mm (%)			
	Plasticity Index	Plasticity (class)		
	Subgrade Material:		Earth passability (class)	
	Material Type	Material Type (class)		
	Max. Particle Size (mm)	Material Size (class)		
	Material passing 2.0 mm (%)	Gradation (class)		
	Material passing 0.425 mm (%)			
	Material passing 0.075 mm (%)			
	Plasticity Index	Plasticity (class)		
	Structure:		Load rating (class)	
	Gravel depth (mm)	Surface depth (class)		
	Cross-section (class)			
Environment	Rainfall monthly mean (mm)	Rainfall class	Climate classification (class)	Climate classification (class)
	Dry season duration (fraction)			
	Moisture classification (class)	Climate (class)		
	Temperature classification (class)			
	Temperature mean annual (deg C)	Cold climate classification (class)		
	Temperature range (deg C)			
	Time above freezing (days)			
	Freezing Index (degC-days)			
	De-icing salt use (class)			
	Studded tire use (%)			
	Snow-driving time (%)			
	Wet-road driving time (%)			
	Air density (kg/m2)			

5.5.4 Environment

There are 13 parameters defining aspects of environment for the various pavement types at IQL-2. These can be approximated and estimated by three class parameters at IQL-2B:

- **Rainfall class**
- **Climate class**
- **Cold climate classification**

At IQL-3 and IQL-4, these can all be summarised by one climate classification with several classes.

5.6 Transforming traffic input data

5.6.1 Traffic volume

Full inputs at IQL-2 involve an AADT-adjusted daily volume for each vehicle class adopted in the fleet classification (see Table 5.4). These could be approximated at IQL-2B by applying appropriate, perhaps estimated, percentages for each class to the AADT. An IQL-3 method would provide only two parameters; for example, the AADT volume and the percentage heavy vehicles. At IQL-4, the volume would be grouped by class for example, preferably ranging by factors of three (for example, 30, 100, 300, 1000, 3000, 10,000, etc.) because these also approximate to key decision thresholds for capacity.

5.6.2 Traffic flow

Full inputs at IQL-2 range from 14 to 20 measured parameters, depending on the fineness of the flow-bands chosen. These could be simplified to seven class parameters that could be estimated at IQL-2B. At IQL-3, these would be summarised by two parameters (volume-capacity ratio and flow-type class). An IQL-4 measure could be a congestion classification, or a performance indicator such as veh-hours delay per day.

Table 5.4
IQL examples for traffic data

<table>
<tr><th rowspan="2"></th><th colspan="4">Information Quality Level</th></tr>
<tr><th>IQL-2</th><th>IQL-2B</th><th>IQL-3</th><th>IQL-4</th></tr>
<tr><td rowspan="7">Traffic volume</td><td></td><td>Volume (AADT veh/day)</td><td>Volume (AADT veh/day)</td><td rowspan="7">Traffic class (class)</td></tr>
<tr><td>Volume – veh. class 1 (veh/day)</td><td>Percent veh. class 1</td><td rowspan="2"></td></tr>
<tr><td>Volume – veh. class n (veh/day)</td><td>Percent veh. class n</td></tr>
<tr><td>Volume – veh. class 5 (veh/day)</td><td>Percent veh. class 5</td><td rowspan="2">Heavy traffic (% AADT)</td></tr>
<tr><td>Volume – veh. class n (veh/day)</td><td>Percent veh. class n</td></tr>
<tr><td>Volume growth (%/yr) – class 1</td><td rowspan="2">Volume growth (%/yr.) (all classes)</td><td rowspan="2">Volume growth (% AADT/yr)</td></tr>
<tr><td>Volume growth (%/yr) – class n</td></tr>
<tr><td rowspan="11">Traffic flow</td><td>Ultimate capacity (pcse/la/h)</td><td>Ultimate capacity (class)</td><td rowspan="5">Volume-capacity ratio</td><td rowspan="11">Congestion class (class)</td></tr>
<tr><td>Free-flow capacity (%)</td><td>Free-flow capacity (class)</td></tr>
<tr><td>Nominal capacity (fraction)</td><td>Nominal capacity (class)</td></tr>
<tr><td>Jam speed at capacity (km/h)</td><td>Jam speed at capacity (class)</td></tr>
<tr><td>Acceleration noise max. (m/s^2)</td><td>Acceleration noise max. (class)</td></tr>
<tr><td>Flow-frequency periods (no.)</td><td rowspan="5">Flow type (3 classes)</td><td rowspan="6">Flow type (3 classes)</td></tr>
<tr><td>Duration of F-F period 1 (h)</td></tr>
<tr><td>Duration of F-F period n (h)</td></tr>
<tr><td>Percent AADT in F-F period 1</td></tr>
<tr><td>Percent AADT in F-F period n</td></tr>
<tr><td>Intersection type (class)</td><td>Intersection type (class)</td></tr>
<tr><td>Safety</td><td>To be completed</td><td></td><td></td><td></td></tr>
<tr><td>Emissions</td><td>To be completed</td><td></td><td></td><td></td></tr>
</table>

5.6.3 Traffic safety

To be included in a subsequent edition of this document

5.6.4 Vehicle emissions

To be included in a subsequent edition of this document

5.7 Transforming output data and producing performance Indicators

For data groups that are also used as output data, the parameters defined for IQL-4 are generally suitable for use as performance indicators, such as the physical condition and functional condition of the assets; for example, pavement condition, traffic congestion, etc. Other indicators that are important are those relating to the effectiveness of the work alternatives being evaluated, for example, NPV per unit cost, etc.

5.8 Data accuracy

The accuracy of measurement of any of the parameters mentioned in this chapter would be a function of the data collection method and the quality control applied. In general, for the multiple analyses that are typically done with HDM for road management, the control of bias (that is, the reproducibility of the measurement by different instruments or at different times) is more important than the precision (the repeatability of the measurement between successive runs). Thus, calibration of the equipment being used and independent verification of non-equipment-based methods is vital for controlling the potential for bias in the input data.

Using the sensitivity function in HDM can test the impact of a bias error in the input data. An approximate estimate can be gained from the sensitivity classification given earlier in this book.

When an IQL-3 or IQL-4 method is used to estimate the HDM input parameters, average values need to be estimated for the various parameters that have been combined into the simplified ones that are collected. If all the values range widely it is unlikely that there will be much bias. However, it is useful to conduct a verification exercise that compares a substantial sample of the simple measures with the more detailed, formal parameters measured on the same road section or in same traffic streams.

5.9 Checklist for data adaptation

The steps for choosing an appropriate IQL for local data and establishing a means for adapting those data to the HDM model inputs include the following:

- Choose or verify that the local IQL is appropriate to the needed decision level and available collection resources
- Sort the local data into a format suitable for transformation to the HDM inputs (IQL-2)
- Determine a suitable transformation between the local data and HDM-input data, using:
  - desk-top estimates, or
  - field conversion sites, or
  - determining conversion relationships

- Apply transformation using conversion relationships

6 RUE model calibration

6.1 Introduction

The RUE model calibration focuses on ensuring that the key RUE model parameters and calibration factors are appropriate for the conditions under which the model is to be applied. As described in Chapter 2, there are three levels of calibration that entail different levels of resources and time. These are:

- **Level 1 - Basic application** (see Section 6.3)

  Determines the values of required basic input parameters, adopts many default values, and calibrates the most sensitive parameters with best estimates, desk studies or minimal field surveys.

- **Level 2 - Calibration** (see Section 6.4)

  Requires measurement of additional input parameters and moderate field surveys to calibrate key predictive relationships to local conditions. This level may entail slight modification of the model source code.

- **Level 3 - Adaptation** (see Section 6.5)

  Undertakes major field surveys and controlled experiments to enhance the existing predictive relationships or to develop new and locally specific relationships for substitution in the source code of the model.

Figure 6.1 shows the recommended priorities for the RUE calibration. This shows the data which are required, of first and second priority, and which should have the defaults assumed for.

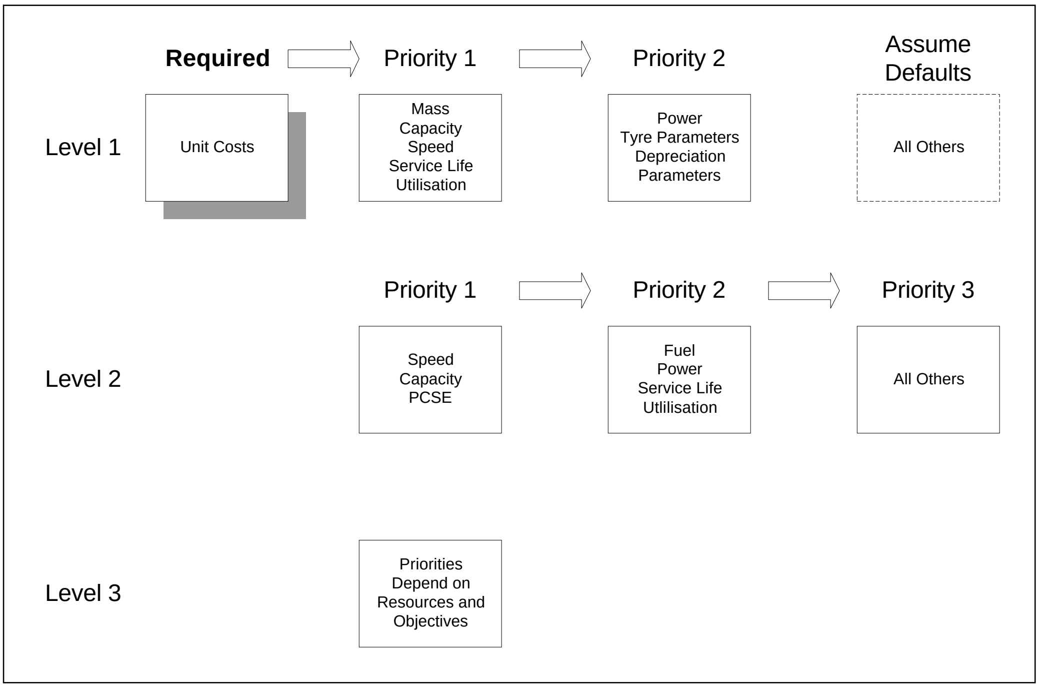


Figure 6.1 Calibration priorities

This chapter commences with a discussion of selecting representative vehicles. This is then followed by procedures for calibrating the model parameters to each of the three levels.

6.2 Selecting representative vehicles

As it is not possible to model the operating costs of individual vehicles, the analyst must resort to the use of representative vehicles. These are vehicles whose characteristics can be considered to be representative of all vehicles within a certain class. Depending upon the nature of the study, any number of representative vehicles may be adopted. The number will be influenced by factors such as the composition of traffic, functional differences between different types of vehicles, the objectives of the study, and the availability and quality of data. Table 6.1 lists vehicles adopted in HDM-III studies from a number of different countries *Bennett and Greenwood (1999)*.

HDM-4 allows an unlimited number of representative vehicles, and using a large number of representative vehicles to describe the vehicle fleet may seem to be advantageous. However, the intrinsic difficulties associated with accurately describing the characteristics of the representative vehicles and the continually changing composition of the fleet means that there will always be errors, irrespective of the number of vehicles selected. It is therefore recommended that for most studies a minimum of seven vehicles be used:

- **Motorcycle**
- **Small passenger car**
- **Utility/light commercial vehicle**
- **Light truck**
- **Medium/heavy truck**
- **Minibus**
- **Heavy bus**

If adequate data are available, the medium and heavy trucks should be further disaggregated since these vehicles tend to have the widest variations in the RUE and the greatest impact on pavement performance.

6.3 Level 1 - Basic application

6.3.1 Mass and vehicle damage factor: S-II

Mass

The vehicle mass influences the vehicle speeds, fuel and tyre consumption and, through the associated heavy vehicle damage factor, has a major impact on the rate of pavement deterioration.

The influence of mass on RUE is not a major issue for pavements in flat terrain. However, the presence of gradients will result in a major increase in fuel consumption and this increase is proportional to the vehicle mass.

Table 6.1
Representative vehicles adopted in different HDM-III studies

Country	MC	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Barbados		•	•		•	•		•	•				•	
Botswana		•			•	•		•	•	•	•		•	
Canada		•	•	•	•		•	•	•	•			•	
Ethiopia		•			•	•		•	•	•	•		•	LGV = 4WD
India		•	•		•		•	•	•	•			•	
India		•	•					•	•				•	
India		•	•		•		•	•	•	•			•	
India		•	•				•	•	•				•	
Indonesia			•		•		•	•	•		•		•	
Indonesia	•	•			•		•	•	•		•		•	
Lesotho		•			•			•	•		•	•	•	
Malaysia		•					•	•	•	•			•	
Myanmar					•	•	•	•					•	
Myanmar		•			•		•	•	•		•		•	Private/Gov't HT
Nepal		•			•			•					•	
Nepal			•		•			•					•	Empty/Full MT
New Zealand			•		•			•	•					
New Zealand		•	•		•	•	•	•	•	•			•	
Romania		•					•	•	•	•			•	
Tanzania		•			•			•	•				•	
Thailand		•					•	•	•		•		•	
Trinidad		•			•		•	•	•		•	•		
Uganda		•			•	•		•	•		•		•	LDV = 4WD
Uganda		•			•			•		•			•	
South Africa		•			•			•	•				•	

Source: *Bennett and Greenwood (1999)*

It is impossible to obtain a reliable estimate of vehicle mass without conducting a field survey, which is a Level 2 calibration activity. This is particularly so in countries with poor axle load enforcement practices: vehicles will often be overloaded well beyond the manufacturer's rated GVW.

If it is impossible to measure weights, a Level 1 calibration of the average mass should be estimated from the manufacturer's tare and rated gross vehicle weights. The analyst must estimate the percentage of vehicles that are travelling empty, half-full, full and overloaded. The average mass is then calculated as:

$$M = \frac{[P_e\, TARE + P_h(0.5\, TARE + 0.5\, GVW) + P_f(GVW) + P_o(zo\, GVW)]}{100} \qquad \ldots(6.1)$$

where:

M	is the average vehicle mass (km)
TARE	is the vehicle tare (empty) mass (kg)
GVW	is the manufacturer's gross vehicle mass (kg)
P*i*	is the percentage of vehicles empty, half-full, full and overloaded (%)
zo	is the overloaded weight relative to GVW (as a decimal)

With this approach, for container trucks standard container weights of 2 t and 4 t can be assumed.

Vehicle Damage Factor

The vehicle damage factor (VDF) is a measure of the damage caused to the pavement by a heavy vehicle. It is a function of the axle configuration and its mass. The VDF is calculated using the equation (*Watanatada et al., 1987a*):

$$VDFVEH_k = \sum_{i=1}^{n} \left(\frac{AX_i}{SX_i} \right)^4 \qquad \ldots(6.2)$$

$$VDF = \frac{\sum_{k=1}^{z} VDFVEH_k}{z} \qquad \ldots(6.3)$$

where:

$VDFVEH_k$	is the vehicle damage factor for vehicle *k* (ESA/vehicle)
VDF	is the vehicle damage factor for a stream of vehicles (ESA/vehicle)
AX_i	is the load on axle *i* (tonnes)
SX_i	is the standard axle load for the axle group *j* (tonnes)
n	is the number of axles on the vehicle

z	is the number of vehicles in the stream

It is common to divide the stream into similar vehicle class, for example, **medium**, **heavy** and **articulated** trucks, and calculate a VDF for each class.

The standard axle loads (SX_i for different configurations are (*Watanatada et al., 1987c*):

6.60 tonne	single wheel, single axle
8.16 tonne	dual wheel, single axle
9.00 tonne	dual wheel, per tandem axle
15.1 tonne	dual wheel, per tandem axle group
10.0 tonne	dual wheel, per triple axle
22.9 tonne	dual wheel, per triple axle group

There are several points to recognise with regard to the VDF:

- Because of the 4^{th} power for the exponent in the VDFVEH equation (see Equation 6.2 above), the VDF must be calculated as the average VDF per vehicle as opposed to the VDF for a vehicle with an average mass. The average VDF is always higher than the VDF of the average load.
- The VDF is the sum of all axle load factors for all axle groups of a vehicle, **not** the damage factor of the average axle load.
- The VDF must represent the average VDF of all vehicles in the class in the traffic, inclusive of empty, partially-laden and fully or over-laden vehicles.

Since the calculations use the 4^{th} power rule, some vehicles with high axle loads may have very high values for their VDFVEH. Care should be taken about including these when doing surveys, particularly with small samples, since they may distort the results. High VDFs may also be rendered invalid if there are changes in policy; for example, axle loads enforcement.

For a Level 1 calibration the VDF is estimated using the assumed percentages of vehicles empty, half-full, full and overloaded. A sensitivity analysis should be conducted with HDM to test the sensitivity of the results to the estimated values.

It must be emphasised that the Level 1 approach of estimating the vehicle mass and VDF will yield only the coarsest values. It is only by conducting a field survey that an accurate estimate of these parameters can be obtained. These surveys are discussed in Appendix E .

6.3.2 Capacity and speed flow data

Introduction

HDM-4 and HDM-95 uses the speed-flow model proposed by *Hoban (1987)* and shown in Figure 6.2. This requires the user to provide five key parameters:

- **Qult -** the ultimate capacity of the road
- **Qnom -** the nominal capacity of where all vehicles are travelling at the same speed
- **Qo -** the flow where interactions commence
- **Snom -** the speed at nominal capacity
- **Sult -** the speed at ultimate capacity

Table 6.2 gives the default HDM parameter values. In the discussion that follows the term **capacity** is applied to the ultimate capacity, unless otherwise noted.

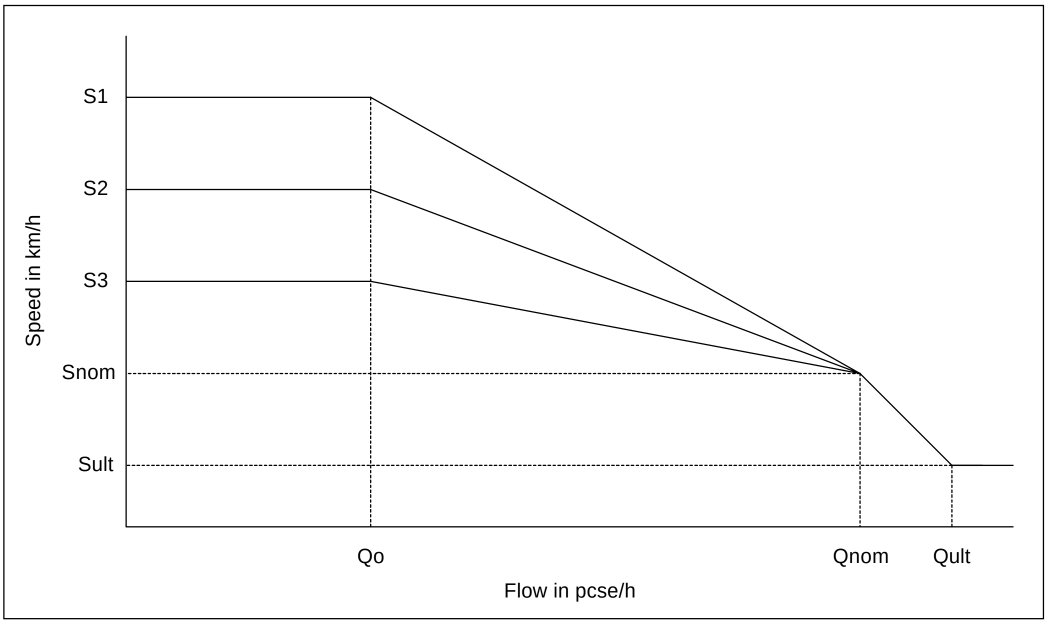


Figure 6.2 HDM speed-flow model

Table 6.2
Default HDM speed-flow model parameters

Road type	Width (m)	Qo/Qult	Qnom/ Qult	Qult (PCSE/h)	Sult (km/h)
Single lane road	< 4	0.0	0.70	600	10
Intermediate road	4 to 5.5	0.0	0.70	1800	20
Two lane road	5.5 to 9	0.1	0.90	2800	25
Wide two lane road	9 to 12	0.2	0.90	3200	30
Four lane road	>12	0.4	0.95	8000	40

Source: *Hoban et al. (1994)*

Estimating capacity

The capacity of a facility is defined as the maximum hourly flow rate at which vehicles can reasonably be expected to traverse a point or uniform section of road under prevailing road, traffic and control conditions. While often considered to be a single fixed value, there are always inherent variations due to factors such as driver behaviour, vehicle performance and prevailing conditions. One should therefore calculate an **average** capacity that will encompass these variations.

It is beyond the scope of this report to address capacity issues in detail and readers should refer to reports such as the Highway Capacity Manual (HCM) for a fuller discussion (*TRB, 1997*). *McLean (1989)* gives a good discussion of capacity issues and provides insight into the background behind the two-lane highway values adopted in the HCM.

For a Level 1 calibration the values in Table 6.2 should be adopted as defaults. However, it must be appreciated that these are **ideal** capacities and they must therefore be adjusted to reflect typical operating conditions. For example, the HCM reduces the ideal capacity to account for different directional splits (for example, 60/40 instead of the ideal 50/50) and the presence of no-passing zones. These reductions can be quite significant, reducing the ideal capacity by 20 per cent or more.

Table 6.3 to Table 6.5 give capacity reduction factors from the Highway Capacity Manual (*TRB, 1997*) for two-lane highways. These factors should be applied in a Level 1 calibration, albeit with caution as they may not be entirely appropriate for conditions outside of those from which they were developed. The ideal capacity is multiplied by these factors to reduce it to a value that reflects actual operating conditions. Multi-lane factors are given in *TRB (1997)*.

Table 6.3
Two-lane highway capacity reduction factors - no passing zones

Terrain	Reduction Factor by Percentage No Passing Zones					
	0	20	40	60	80	100
Flat	1.00	1.00	1.00	1.00	1.00	1.00
Rolling	0.97	0.94	0.92	0.91	0.90	0.90
Mountainous	0.91	0.87	0.84	0.82	0.80	0.78

Table 6.4
Two-lane highway capacity reduction factors - directional split

Reduction factor by directional split					
100/0	80/10	80/20	70/30	60/40	50/50
0.71	0.75	0.83	0.89	0.94	1.00

Table 6.5
Two-lane highway capacity reduction factors - lane and shoulder width

Usable shoulder width (m)	Capacity reduction factor by lane width (m)			
	3.6	3.3	3.0	2.7
≥ 1.8	1.00	0.94	0.87	0.76
≥ 1.2	0.97	0.92	0.85	0.74
≥ 0.6	0.93	0.88	0.81	0.70
0	0.88	0.82	0.75	0.66

Note: Where shoulder width is different on each side use the average width.

6.3.3 Average service life: S-II

A vehicle, or any physical property, has three measures of its life, namely the:

- **Service life -** is the period over which the vehicle is operated
- **Physical life -** is the period which the vehicle exists (even if it is not being used)
- **Economic life -** is the period which the vehicle is economically profitable to operate

With HDM, the service life is of interest. It is used by HDM in calculating the depreciation costs of vehicles that can have a significant impact on the RUE.

In HDM-4 the user needs to define the expected service life in kilometres for a vehicle operating on a smooth pavement. This value is then used to determine the effect of roughness on service life when using the **Optimal Life** technique. This expected service life is the distance at which it becomes appropriate to scrap the vehicle.

There are a number of different techniques available for calculating the service life and an overview of these may be found in *Winfrey (1969)*. For a Level 1 calibration the ages of a sample of vehicles should be obtained, either from a small survey or by sampling advertisements of vehicles for sale. *Daniels (1974)* indicates that the service life will be double the mean age. This was also found to be the case in New Zealand (*Bennett, 1985*) where several different techniques were tested for estimating the service life.

6.3.4 Vehicle utilisation: S-II/S-III

Annual utilisation - number of kilometres driven

Annual kilometreage; that is, the number of kilometres driven per year - data are used in calculating the parts consumption and the interest costs.

In order to determine the annual kilometreage it is necessary to have information detailing the ages of vehicles and the distances that they have travelled. The utilisation of a vehicle generally varies with age. In several studies older vehicles have been found to have lower utilisation than newer ones (*Daniels, 1974*; *Bennett, 1985*). It is therefore important that any data collected not be biased in favour of vehicles of a given age.

For a Level 1 calibration a suitable data source for utilisation is advertisements of used vehicles for sale. Newspapers and other similar sources provide data on the year and total kilometreage of a sample of used vehicles. Dividing the kilometreage by the age gives the average kilometreage of the vehicle over its life. Problems may be encountered with older vehicles wherein the odometer may have cycled beyond 100,000 km (or miles) or the ages may be in error. However, if a sufficiently large sample of data is obtained these problems will be minimised.

Annual hourly utilisation - the number hours per year

There are three definitions for the hourly utilisation:

- **HAV -** the number of hours the vehicle is **available** per year

  This is the number of hours per year (8760), less the time allowed for crew rest, time lost loading, unloading, refuelling, finding cargo, repairs, etc.

- **HRD -** the numbers of hours driven

  This is the hours that the vehicle is operated. It can be calculated from the annual kilometreage divided by the average annual speed.

- **HWK -** the number of hours **worked**

  This is similar to the hours driven (HRD), except it includes the time spent loading, unloading and refuelling.

The HDM-III adjusted utilisation model was based on the hours drove. HDM-4 is based on the hours-worked approach.

Using a standard working week, a vehicle is typically available for approximately 1800 hours per year. However, since there are substantial periods of time when the vehicle is not in use, for example due to loading/unloading, the driving time would often be less than 50 per cent of this value. In the Brazil study, for example, the vehicles were available 839 - 2414 h, but only driven 652 - 1863 h (*Watanatada et al., 1987c*). Trucks and buses had the highest utilisations; utilities the lowest.

As discussed in *Bennett (1995)*, it is important that the value adopted for hours driven be consistent with the annual utilisation and the average speed. If not, the predicted costs could be distorted. To this end it is recommended that in the absence of more detailed data, the hours driven be calculated using the following equation:

$$HRD = \frac{AKM}{S0} \qquad \ldots(6.4)$$

where:

AKM	is the average annual utilisation in km
S0	is the average operating speed in km/h

To calculate hours worked it is necessary to have sufficient data to identify the times spent "undertaking the essential tasks of making a complete round trip, in normal circumstances. Time spent idle, where the crew is eating, sleeping or otherwise resting should not be included. Time in repair should also, in general, be excluded because it is not part of a regular trip. Driving, loading and unloading should be included together with refuelling" *(Hine, 1996*).

One may also include the administration time that a driver had to spend finding loads or the time that must be spent waiting with the vehicle to move it up the queue, if the driver can't do anything else. It should be appreciated that in some circumstances not all loaded time will be working time, for example if the driver stops the vehicle to sleep while it is loaded.

Working time is established by conducting small surveys covering vehicle activities over several days. The work of *Hine (1996)* in Pakistan gives valuable insight into issues arising from these surveys and special considerations that must be made in approaching the issue.

Percentage private use

The percentage private use can only be established through a small survey of users. This will differentiate between those on work trips and private trips. The resulting values are used to calculate the value for travel time.

6.3.5 Speed prediction model parameters: S-II/S-III

The HDM speed prediction model is mechanistic, being based on physical and kinematic principles, as well as behavioural constraints. Consequently, the basic physical model is highly transferable and the focus of Level 1 calibration should be on the behavioural constraints, defined by:

- **VDESIR -** the desired speed of travel

  This can be expected to differ considerably between countries, and even regions within the same country. VDESIR represents the maximum speed of travel adopted by the driver of a vehicle when no other physical constraints, such as gradient, curvature, roughness or congestion, govern the travel speed. The value of VDESIR is influenced by factors such as speed limits and enforcement, road safety, cultural and behavioural attitudes.

- **β -** the 'draw down'

  Indicates how far from the constraining speeds the predicted speed will be. β is the **Weibull Shape Parameter**. As described in *Watanatada et al. (1987c)*, it is functionally related to the dispersion of the underlying distribution of the constraining speeds.

This assumes that the physical performance of the vehicle has been properly calibrated by identifying valid representative values of other important vehicle characteristics, namely:

- **Vehicle mass**
- **Used driving power**
- **Braking power**

For calibration purposes it is important to understand the role of β. The HDM speed model predicts that speeds are the probabilistic minimum of five constraining speeds based on:

- **Power**
- **Braking**
- **Curvature**
- **Roughness**
- **Desired speed**

As shown in Figure 6.3 (*Bennett and Greenwood, 1999*), when β approaches zero, the mean speed for a given road section would be equal to the minimum of the five constraining speeds.

The greater the value for β, the further away the predicted mean speed will be from the constraining speeds. Thus, if the desired speed is 100 km/h, with a value of β = 0, the predicted speed would be 100 km/h. However, if a non-zero value for β is used, the predicted speed would be less than 100 km/h.

In calibrating the model, *Watanatada et al. (1987c)* found value of β=0.24-0.31 for Brazil. A similar calibration from India found β=0.59-0.68. *Watanatada et al. (1987c)* considered that the higher values for β reflected the more congested traffic conditions in India. It was recommended that "For environments, such as India, which have congested rural roads and low level of traffic discipline, somewhat higher values of the β parameter (than Brazil) may be used". They note that the quantification of β would require a major field study, that is, a Level 2 calibration, so it is discussed later in Section 6.4.1.

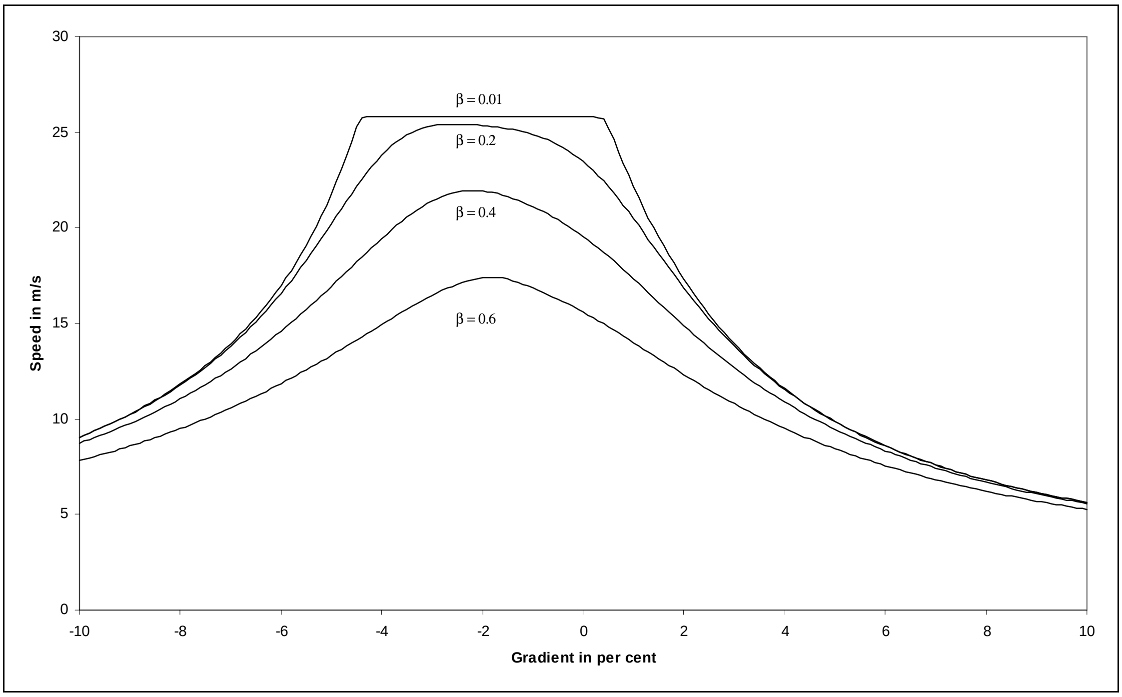


Figure 6.3 Effect of β on predicted speed

In undertaking a calibration of VDESIR, it is therefore suggested that this be considered in tandem with the value of β. If the default values of β result in values for VDESIR which are markedly higher than the predicted speed, unless there is heterogeneous traffic such as in India, the value for β should be increased from the default.

For a Level 1 calibration when the value of travel timesaving is not required, subjective estimates of the average free speeds of each vehicle class are adequate. Ideally, these would be for several road sections with different conditions, but one will suffice.

The HDM model should be run on a road section with **average** characteristics reflecting the conditions for the average speed estimates. The predicted free speed should be compared to the average speed estimate and the value for VDESIR used in the modelling adjusted by the ratio of the predicted speed to the estimated speed. After several runs of the model the predicted and estimated speeds should be the same. If a number of road sections are to be used in the analysis VDESIR should be iteratively established using this method for each section and then these values averaged[1]. As described above, it may also be necessary to adjust the value for β to ensure a reasonable value for VDESIR.

6.3.6 Vehicle driving power: S-III

The driving power only has a significant effect on speeds when the gradient is positive and higher than about 4 per cent for light vehicles and 2-3 per cent for heavy vehicles. For a Level 1 calibration the used power can be estimated from the vehicle attributes using the following equations:

Diesel vehicles

$$\text{HPDRIVE} = 0.70\ \text{HPRATED} \qquad \ldots(6.5)$$

where:

HPDRIVE is the used driving power (kW[2])

HPRATED is the SAE maximum rated engine power (kW)

This equation is from *Watanatada et al. (1987c)* and so applies to older technology vehicles (that is, pre-1985). *Bennett (1994)* indicated that for modern vehicles the factor should be 0.75. His data showed that larger vehicles tended to use more power and the following alternative equation was developed relating power to mass:

$$\text{HPDRIVE} = \left(8.5 \times 10^{-6}\ \text{M} + 0.53\right)\text{HPRATED} \qquad \ldots(6.6)$$

Petrol vehicles

$$\text{HPDRIVE} = 1.8\ \text{HPRATED}^{-0.3} \qquad \text{Pre-1985}$$

$$\text{HPDRIVE} = 2.0\ \text{HPRATED}^{-0.3} \qquad \text{Post-1985}$$

The modern technology petrol equation is an update of the old technology equation from *Watanatada et al. (1987c)* using data from *Bennett (1994)*. For use in HDM-III the values will need to be divided by the factor of 0.736 to convert to MPH.

The SAE rated engine power is generally available from motor publications and manufacturers. A weighted-average should be calculated based on the frequency of different vehicle types, and thus engine powers, in the vehicle fleet.

In applying these equations it should be noted that there is evidence that there is a relationship between the gradient and power usage, with vehicles having higher power usage on higher grades (*Bennett, 1994*). It may therefore be necessary to increase the driving power from those predicted using the above equations for vehicles operating in hilly conditions.

1 If there are marked differences in the values of VDESIR it may be appropriate to eliminate the greatest outliers - as well as reviewing the estimated speeds.

2 Engine power conversions: 1 BHP = 0.746 kW; 1 MPH = 0.736 kW; 1 BHP = 0.987 MPH

6.3.7 Tyre type, wheel diameter and number of wheels: S-IV

The tyre type and the number of wheels are used in HDM-4 for establishing the rolling resistance. Bias ply tyres have greater rolling resistance than radial tyres, and the resistance increases with an increasing wheel diameter and number of wheels.

Tyre sizes have a standard typology. The two most common types are shown in Figure 6.4 along with a description of what each term means. The top typology is common with truck tyres and is based on the nominal section width being expressed in inches. The second is used with for all vehicles and has the nominal section width in mm along with the aspect ratio[1]. For clarity, the section width is separated from the aspect ratio by a slash (/)[2]. The discussion that follows is based on the second definition, that is, a metric nominal section width. The imperial section width can be converted from inches to mm using the factor 1” = 25.4 mm.

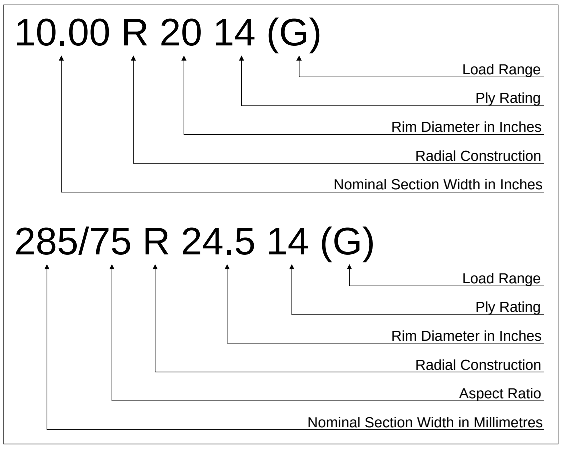


Figure 6.4 Standard tyre typology

The ply rating and load range usually only apply to heavy vehicle tyres so the standard typology reduces to:

$$\text{xxx/yyRzz} \qquad \ldots(6.7)$$

where:

xxx	is the nominal width of the tyre (mm)
yy	is the aspect ratio
zz	is the rim size (inches)

For light vehicles the aspect ratio of 82 is often omitted so they are specified, for example, as 175/R13 instead of 175/82R13. The value of 82 can be assumed for heavy vehicles when omitted.

1 The aspect ratio is the tyre’s section height, which is the distance from the bead to the centre of tread, to the section width. An aspect ratio of 65 means that the tyre’s section height is 65% of the tyre’s section width.

2 A less common typology uses the nominal width in inches and the aspect ratio separated by the slash, for example, 14/80R20. Because of the much lower magnitude of the nominal width, it is readily apparent when this case arises that the data should be converted to mm.

The tyre type can be established from manufacturer's specifications since motorists tend to purchase the same replacement tyres as new vehicles. Discussing with tyre retailers can check this information. Alternatively, a small study can be conducted which records the tyres on a sample of vehicles, for example in a parking lot or at a truck stop.

For Level 1 calibration the wheel diameter can be estimated from the tyre typology using the following equation (*Greenwood, 1997*):

$$\mathrm{DIAM} = 25.4\ \mathrm{zz} + 2\ \mathrm{xxx}\ \frac{\mathrm{yy}}{100} \qquad \ldots(6.8)$$

The number of wheels can be estimated from the manufacturer's specifications or from a small study.

6.3.8 Volume of wearable rubber: S-III

The volume of wearable rubber is used in the tyre consumption modelling and dictates the tyre life. The costs are directly proportional to the volume of wearable rubber. For a Level 1 calibration it is estimated from the tyre size using the following procedure (*Bennett, 1998*).

With reference to Figure 6.5, the tread cross-section is treated as a trapezoid with the upper and lower bases representing the upper and lower parts of the tread (the lower part being in contact with the road).

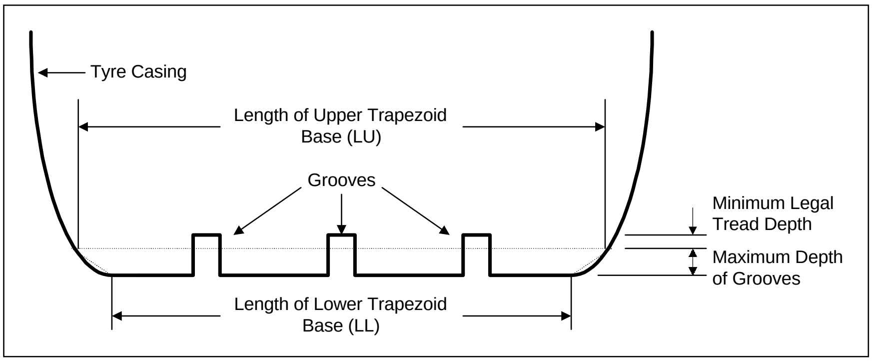


Figure 6.5 Factors influencing volume of wearable rubber calculation

The area or wearable rubber is calculated as:

$$\mathrm{AREA} = \frac{\mathrm{LL} + \mathrm{LU}}{2}\,(\mathrm{DE} - \mathrm{DEmin})\ \mathrm{ARUB} \qquad \ldots(6.9)$$

where:

AREA	is the area of the tread cross-section (mm^2)
LL	is the length of the lower trapezoid base; that is, at road contact (mm)
LU	is the length of the upper trapezoid base (mm)
DE	is the depth of the grooves (mm)
DEmin	is the minimum depth of grooves (mm)
ARUB	is the area of rubber versus grooves (decimal)

The volume of wearable rubber is calculated from the tread area as:

$$VOL = \frac{PI\ DIAM\ AREA}{1,000,000} \qquad \ldots(6.10)$$

where:

VOL is the volume of wearable rubber (dm^3)

DIAM is the tyre diameter (mm)

To calculate the tyre volume it is therefore necessary to predict the tyre diameter, the tread depth, the tread width and the area of rubber versus grooves.

The tread width as a function of the nominal section width is predicted as (*Bennett, 1998*):

$$LL = a0\ xxx - a1 \qquad \ldots(6.11)$$

where:

LL Tread width corresponding to the lower length of the trapezoid base

a0, a1 Equation coefficients (see Table 6.6)

It was found that the coefficients for Equation 6.11 above varied by vehicle class so separate equations were developed for light and heavy vehicles. The coefficients are given in Table 6.6.

The tread width LL corresponds to the lower length of the trapezoid base in Figure 6.5. For the upper width (UL) *Bennett (1998)* recommended an increase of 10 mm for heavy vehicles; 6 mm for light vehicles and 4 mm for motorcycles. Table 6.6 gives these recommended values by vehicle class.

Table 6.6
Wearable rubber model parameters

Vehicle class	Tread width model parameters		Tread depth (DE)	Width of UL over LL	Area of rubber (ARUB)
	a0	a1	(mm)	(mm)	(decimal)
Motorcycles	1.05	-52.5	5	4	0.90
Passenger cars	1.05	-52.5	8	6	0.85
Utilities	1.05	-52.5	8	6	0.85
4WD	1.05	-66.7	9	6	0.85
Light trucks	1.05	-66.7	11	10	0.80
Medium trucks	1.05	-66.7	15	10	0.70
Heavy trucks	1.05	-66.7	17	10	0.70
Articulated trucks	1.05	-66.7	17	10	0.70
Light buses	1.05	-66.7	11	10	0.80
Medium buses	1.05	-66.7	15	10	0.70
Heavy buses	1.05	-66.7	15	10	0.70
Heavy truck driven axles	1.05	-66.7	26	10	0.70
Heavy truck non-driven axles	1.05	-66.7	15	10	0.70
Heavy truck trailers	1.05	-66.7	12	10	0.70
Super single tyres	0.76	-3.8	16	10	0.70

Source: *Bennett (1998)*

Tread depths are available from manufacturer's specifications, or can be easily measured from a sample of tyres, or the values given in Table 6.6 can be used. This table also contains the values for ARUB: the tread area as a decimal (*Bennett, 1998*). The latter can be easily measured by recording the groove widths and frequency of grooves on a tyre.

The volume of rubber is calculated as follows:

1. Establish the tyre size for the representative vehicle (for example, 175SR13).
2. Calculate the tyre diameter from the tyre typology using Equation 6.8 above.
3. Calculate the tread width LL from the tyre typology using Equation 6.11 above.
4. Establish the value for UL relative to LL from Table 6.6 (for example, for passenger cars UL = LL + 6).
5. Establish the value for ARUB from Table 6.6 (for example, for passenger cars ARUB = 0.85).
6. Establish the tread depth DE from a survey or from Table 6.6.
7. Establish the legal minimum tread depth DEmin or, if there is no enforcement, assume a value of 0 mm.

8 Substitute the values for ARUB, DE, DEmin, LL and LU into Equation 6.9 above to establish the tyre area.

9 Substitute the tyre area and diameter into Equation 6.10 above to establish the volume of wearable rubber.

6.3.9 Depreciation parameters

HDM-4 contains two methods for depreciation predictions:

- **Constant life**
- **Optimal life**

With the constant life method, the depreciation is calculated as the replacement value less residual value[1] divided by the vehicle life. When less than 50 per cent of the trips are for private use, this life is adjusted by the number of hours worked. The calibration of this method is therefore achieved using the calibrated values for annual utilisation, service life and hours worked.

For the optimal life calibration, the user needs to establish the average service life of the vehicle at an average roughness. Using the HDM Tools software (See Appendix K), the service life as a function of roughness is predicted. A regression equation is then fitted to these data and used in HDM-4. Details of how this is done are given in Appendix K .

6.3.10 Aerodynamic drag coefficient and projected frontal area: S-III

The aerodynamic drag coefficient and projected frontal area should be considered in tandem since their product is used to calculate the aerodynamic resistance. Aerodynamic drag coefficients are difficult to obtain since the values reported in the literature are highly dependent upon the testing conditions. Aerodynamic drag coefficients are also influenced by the operating conditions; the value changes as a function of the wind angle (*Biggs, 1987*). The default values given in HDM-4 should thus be used unless more appropriate values are readily available.

The projected frontal area is obtained by subtracting the area under the vehicle from the product of the maximum height with the maximum width. For light vehicles manufacturer's specifications may be used to determine the appropriate measurements. For trucks and buses it is recommended that the data be gathered in a small field survey due to the wide variety of body configurations and loading practices.

In HDM-4 there is an additional value termed the **CD Multiplier** (CDmult). This is used to reflect the effect of wind on the aerodynamic drag coefficient (CD) with *Bennett and Greenwood (1999)* discussing its quantification. Its calculation is done using the **HDM Tools** software that is described in Appendix K . The CD Multiplier does not need to be requantified unless there is a significant change in the value for CD.

6.3.11 Braking power: S-IV

In HDM-III the braking power was used to establish the speed on downgrades. HDM-4 differentiates between speeds on short and long grades (*NDLI, 1995a*). On short grades, the gradient has no effect on speed. When the gradient exceeds a critical length the speed is governed by the braking power so the vehicle slows down to maintain control.

[1] The residual value is assumed to be 1% of the replacement vehicle price.

For a Level 1 calibration it can be assumed that the default HDM values are adequate, although if the analyses include steep gradients it may be desirable to calculate new values. This can be done using the following equations:

$$HPBRAKE = 10.3\ GVW \qquad \text{HDM-III} \qquad \ldots(6.12)$$

$$HPBRAKE = 9.3\ GVW + 13 \qquad \text{HDM-4} \qquad \ldots(6.13)$$

where:

HPBRAKE is the braking power in Kw

The HDM-III equation is from *Watanatada et al. (1987c)*; the HDM-4 is from an analysis of the default parameter values from *NDLI (1995a)*. For use in HDM-III the values will need to be divided by the factor of 0.736 to convert from kW to MPH.

6.3.12 Engine speed

Engine speed is used in HDM-4 to predict the fuel required to maintain engine operation. The engine speed depends on the road speed, the gear selected and the differential ratio.

To calibrate the engine speed a simulation model has been developed. *Bennett and Greenwood (1999)* give a description of its principles and operation. Appendix K describes the model's operation. This simulation model should be used for a Level 1 calibration of the engine speed model.

Using manufacturer's specifications, a set of gear and differential ratios for a selection of representative vehicles should be established. These are used as input to the simulation model that applies a set of rules for driver behaviour to estimate the engine speed at a range of road speeds. The model output is a set of engine versus road speeds for each set of vehicle characteristics supplied. These data should then be analysed using regression techniques to develop the necessary set of model parameters for HDM-4. Appendix K illustrates the use of this model.

6.4 Level 2 - Calibration of primary relationships

6.4.1 Speed prediction

Measurement of desired speed (VDESIR)

A Level 2 calibration calls for VDESIR to be measured. This should ideally be done on a number of sections of roads that are straight, level or with a minor downgrade, and with a low roughness. The free speed should be measured using either manual or automated methods.

It is important that a sufficient number of vehicles be observed to minimise statistical error. For the same level of accuracy, the sample size is proportional to the standard deviation of speeds. Thus, developed countries with homogeneous traffic will require smaller sample sizes than underdeveloped countries with heterogeneous traffic. Appendix I presents a technique for estimating sample sizes. With indirect measurements, the sample size should be sufficient to ensure that there is 90 per cent confidence that the estimate is $\pm$ 2.5 km/h. With speeds, one can assume the coefficient of variation is approximately 0.12 for estimating the sample size.

Having established the mean operating speed, one then uses this in conjunction with HDM-4 to determine the appropriate value for VDESIR. HDM-4 is run iteratively using the same road section parameters as prevailed in the speed survey varying VDESIR until the predicted and observed speeds agree.

It should be recognised that the desired speed is a function of bendiness. This effect was observed in several studies when vehicles travelling on roads with severe alignments had lower desired speeds (*McLean, 1991*). *NDLI (1995a)* proposed a negative exponential model should be used to reduce the desired speed as a function of bendiness within HDM-4. However, on testing it was found that this model gave unreasonable predictions at extreme levels of bendiness so it was not included in HDM-4.

If HDM will be applied at different levels of bendiness, it would be prudent to establish separate values of VDESIR to apply at these different levels.

Width

The HDM-4 desired speed-width model is illustrated in Figure 6.6. It is based on the work of *Hoban et al. (1994)* and *Yuli (1996)*. The underlying assumption is that there is critical width below which speeds will be unaffected by width (CW1). Between this minimum speed (VDESMIN) and the desired speed on two-lane highways, there is a linear increase in the speed. On roads wider than two-lanes there is a continued increase in speed, but at a much lower rate.

The minimum desired speed, in HDM-4, is assumed to be 75 per cent that of the two-lane highway desired speed. Conducting free speed studies on narrow and two-lane roads can check this value. It is important that the roads studied have similar levels of roughness and, if possible, be on flat, tangent sections. By holding all other factors constant the differences in speeds will be due to width. A similar approach is used when roads are wider than two lanes to obtain the speed increase slope.

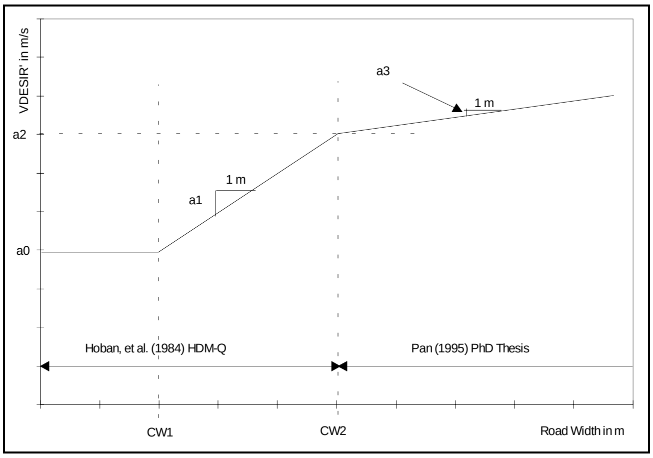


Figure 6.6 HDM-4 desired speed - width model

Roughness

The speed-roughness model is calibrated through the maximum rectified velocity.

Data for calibrating the maximum rectified velocity are best obtained from a before and after study on a pavement selected for maintenance which will reduce the roughness. If there are no changes to the width or alignment, any increases in speed after the maintenance is attributable to the roughness reduction. Section 7.5 describes special considerations in undertaking such a study, particularly the need for **control** sections to check the roughness meter calibration.

Table 6.7 shows the change in speeds before and after overlays from such a study in India (*NDLI, 1997*). In all cases there was an increase in speed accompanying the decrease in roughness.

Calibration of the model is done by running HDM-4 at each of the roughnesses in the before and after study and recording the predicted speeds. The other speed model parameters should reflect the conditions of the speed survey sites.

The speed-roughness slope (km/h/IRI) should be calculated and compared to that from the studies. The ratio of the means is used to modify the maximum average rectified velocity used as input to HDM-4. Since the speed-roughness model is non-linear, the process should be repeated at least two times to ensure that consistent results are obtained.

Table 6.7
Data on speed-roughness study

Vehicle	Site	Dir.	Speed in km/h			Roughness in IRI m/km			Speed/ Roughness Slope (km/h/IRI)	
			Before	After	Change	Before	After	Change	By site	Mean
PC	1	1	61.1	65.4	4.3	5.7	4.0	1.7	2.5	
	1	2	61.0	66.5	5.5	7.9	4.3	3.6	1.5	
	2	1	66.0	72.5	6.5	7.4	3.7	3.7	1.8	
	2	2	59.7	70.0	10.3	6.7	3.7	3.0	3.4	2.3
MCV	1	1	48.3	52.3	4.0	5.7	4.0	1.7	2.4	
	1	2	51.4	54.8	3.4	7.9	4.3	3.6	0.9	
	2	1	50.6	55.8	5.2	7.4	3.7	3.7	1.4	
	2	2	53.1	54.8	1.7	6.7	3.7	3.0	0.6	1.3
BUS	1	1	57.0	57.3	0.3	5.7	4.0	1.7	0.2	
	1	2	55.0	56.7	1.7	7.9	4.3	3.6	0.5	
	2	1	57.2	60.6	3.4	7.4	3.7	3.7	0.9	
	2	2	56.7	57.1	0.4	6.7	3.7	3.0	0.1	0.4
MC	1	1	44.3	49.0	4.7	5.7	4.0	1.7	2.8	
	1	2	44.1	49.0	4.9	7.9	4.3	3.6	1.4	
	2	1	44.0	50.1	6.1	7.4	3.7	3.7	1.6	
	2	2	43.1	48.9	5.8	6.7	3.7	3.0	1.9	1.9

Source: *NDLI (1997)*

Curve speed

In order to calibrate the curve speed model it is necessary to undertake a series of measurements on roads with different radius of curvature. *HTC (1999)* and *Bennett (1994)* describe in detail how to undertake such studies. Data are collected on speeds at the midpoints of curves over a range of curve radii and analysed to develop a non-linear model relating curve speed to the radius of curvature. Figure 6.7 is an example of the data for passenger cars from Thailand (*HTC, 1999*).

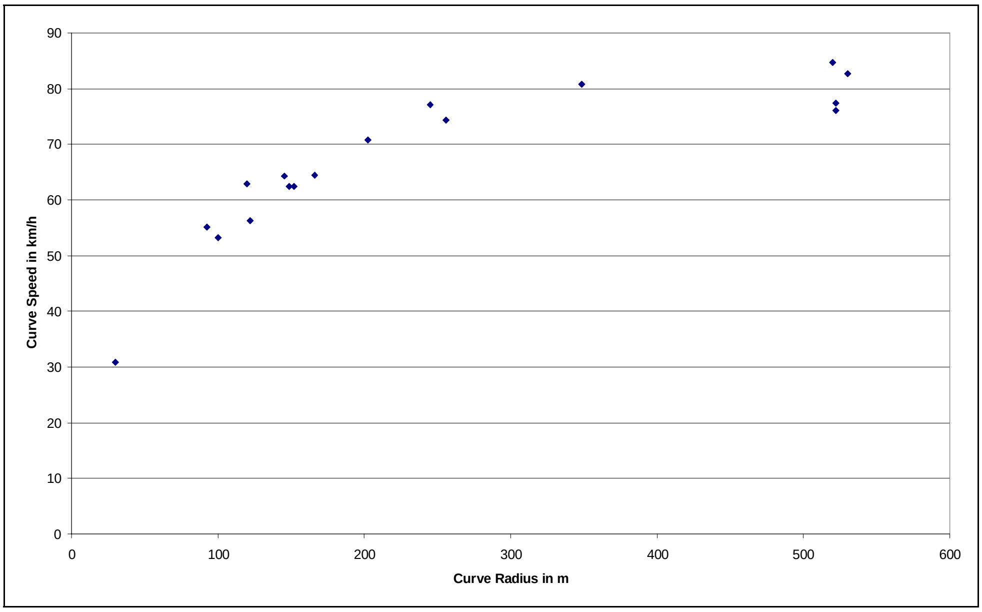


Figure 6.7 Effect of curve radius on speed from Thailand

The data are fitted to the following model:

$$VCURVE = a0 \times R^{a1} \quad \text{…(6.14)}$$

where:

VCURVE	is the limiting speed due to curvature (m/s)
R	is the radius of curvature (m)
a0, a1	are model parameters

HTC (1999) discusses in detail the analytical requirements for developing such a model.

Speed limit enforcement factor

The speed limit enforcement factor is the speed by which traffic travels above the posted speed limit under **ideal** conditions. It can be estimated by conducting a speed survey on a tangent road and comparing the mean speeds to the mean posted speeds. The default value is 1.1 that indicates that traffic will travel up to 10 per cent above the posted speed limit.

6.4.2 Capacity and speed flow

Capacity

As described earlier with Figure 6.2, HDM requires the nominal and ultimate capacities, and a Level 2 calibration undertakes field trials to establish these capacities.

As shown in Figure 6.8, there are several different methods available for estimating capacity (*Minderhoud et al., 1997*). These authors make an important observation: "Attempts to determine the capacity of a road by existing methods will generally result in a capacity value estimate, but the validity of this value is hard to investigate because of the lack of a reference capacity value, which is supposed to be absolutely valid. A clear, reliable method does not appear to be available at this time."

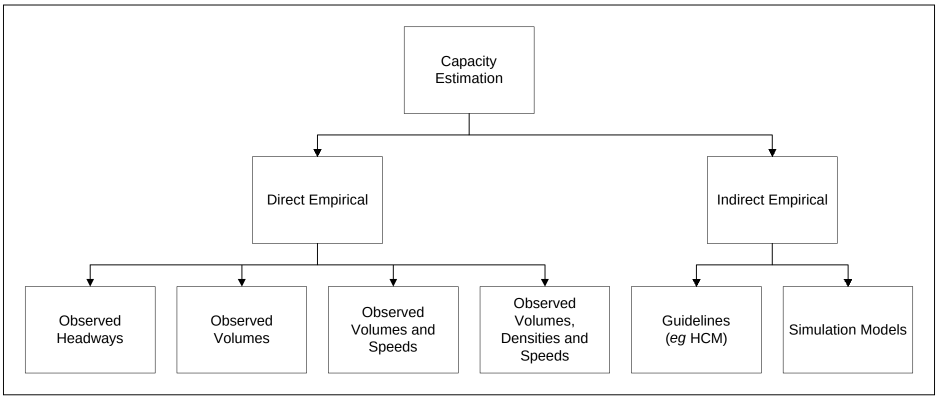
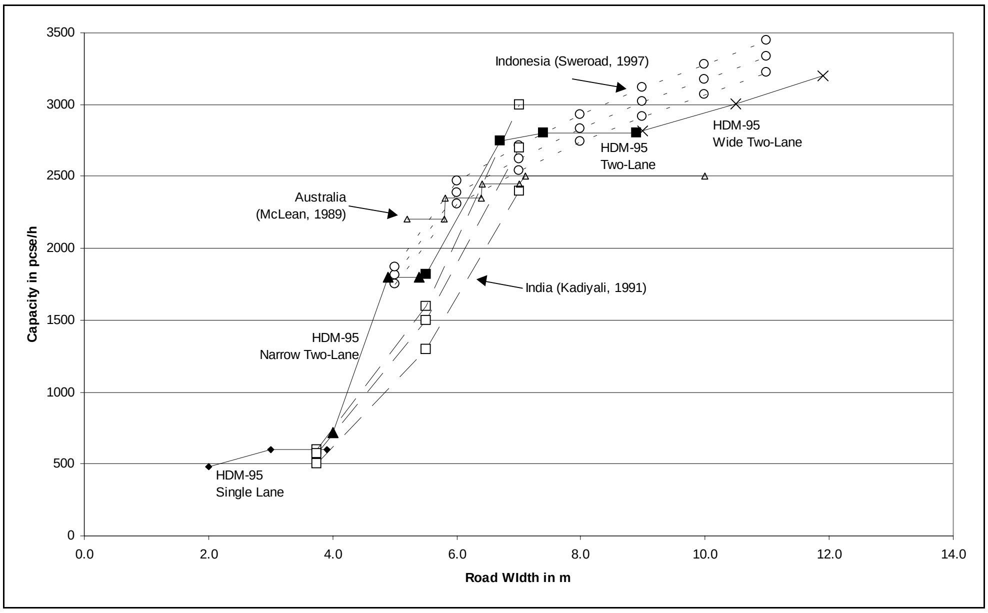


Figure 6.8 Capacity estimation methods

Irrespective of what method is used, it is necessary to sample over sufficient time intervals to obtain a reliable estimate. One normally measures the flow in 15-minute periods and converts this to an hourly flow. In most situations capacity is a difficult condition to reliably encounter so one must often extrapolate data collected at lower flows to estimate the capacity.

The capacity estimates must consider the range of factors influencing capacity. In Section 6.3.2 a series of default capacity reduction factors were given. The capacity analyses should consider these to establish local values.

Width is a particularly important factor. The HDM approach results in a step-function of capacity versus width. However, as shown in Figure 6.9, other studies have treated capacity as a continuous function of width. This has the advantage of avoiding discontinuities that can arise at the boundaries of step functions. Figure 6.9 also shows the different capacities found in different countries, and highlights the importance of local calibration.



Source: *Sweroad (1997)* and *Kadiyali (1991)*

Figure 6.9 Effect of width on capacity

From a practical perspective, it is important to focus on the pavement widths that are most relevant. Many applications consider incremental widening in the range of 6-8 m and in this band minor width changes can have a marked impact on capacity. If pavements are less than 6 m wide, they often deal with a major widening of several metres that will have a substantial increase in capacity, analogous to the step functions in HDM. Conversely, small changes in width below 6 m have little impact on capacity. One should therefore establish realistic capacity estimates that will reflect the operational impacts expected.

In the context of HDM speed modelling, both the nominal and ultimate capacities are of interest. As described in *HTC (1999)*, the ultimate capacity prevails over short time periods so is best represented by the maximum flow observed on a road. The nominal capacity is the maximum flow rate that can reasonably be expected to be maintained so it should be based on a longer period. In Thailand *HTC (1999)* used 5 minute intervals for the ultimate capacity and 10-15 minute intervals for the nominal capacity.

Given the limited data available for most Level 2 calibrations, there are two techniques that should be considered for estimating the capacity:

- **Headways**
- **Observed flows**

Headways are the time difference, in seconds, between successive vehicles (usually measured from rear bumper to rear bumper although if using axle detectors from front axle to front axle). The capacity of the road is defined as:

$$Qult = \frac{3600}{hc} \qquad \ldots(6.15)$$

where:

hc is the mean headway of constrained vehicles (s)

The method is based on the theory that at high flows there are still two populations of vehicles:

- **Constrained** (followers)
- **Unconstrained** (leaders)

The distribution of following headways is expected to be the same as for constrained drivers in any stationary traffic stream.

Although relatively easy to collect, it is considered that this method tends to overestimate capacity due to the assumption that the distribution of constrained drivers can be compared at capacity with that below capacity.

With observed flows, data are collected on roads that will reach capacity at some point during the study period. The road capacity is taken to be the maximum flow, or the mean of several very high flows, observed during the analysis period. This is illustrated in Figure 6.10.

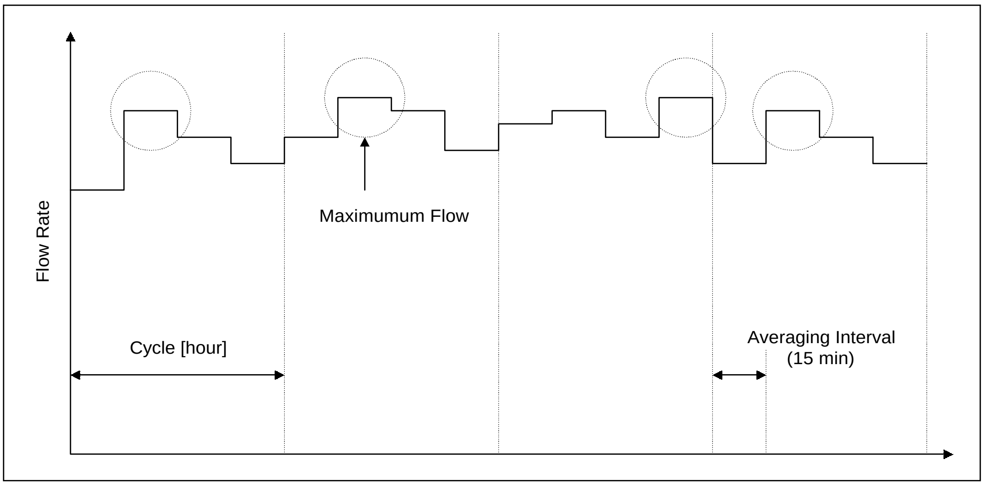


Figure 6.10 Estimating capacity from observed flows

Figure 6.11 is an example of speed-flow study results from Thailand (*HTC, 1999*). This figure shows the capacity of 2121 PCSE/h. This was based on 5-minute peak flows measured using data loggers.

The disadvantage of this approach is that the roads may not always reach capacity. However, if the sites are carefully selected this situation can be minimised. One way of ensuring high flows is to plan the data collection around special events, such as holidays when traffic flows can be expected to be high.

Speed at nominal capacity

HTC (1999) estimated the speed at nominal capacity by fitting a regression function to speed-flow data for flows beyond the breakpoint flow (Qo). The nominal capacities were then substituted into these equations to establish the corresponding speeds. The results were fairly consistent between different vehicle classes and suggested that the HDM approach of assuming 85 per cent of the free speed was too high and it should be 77-82 per cent.

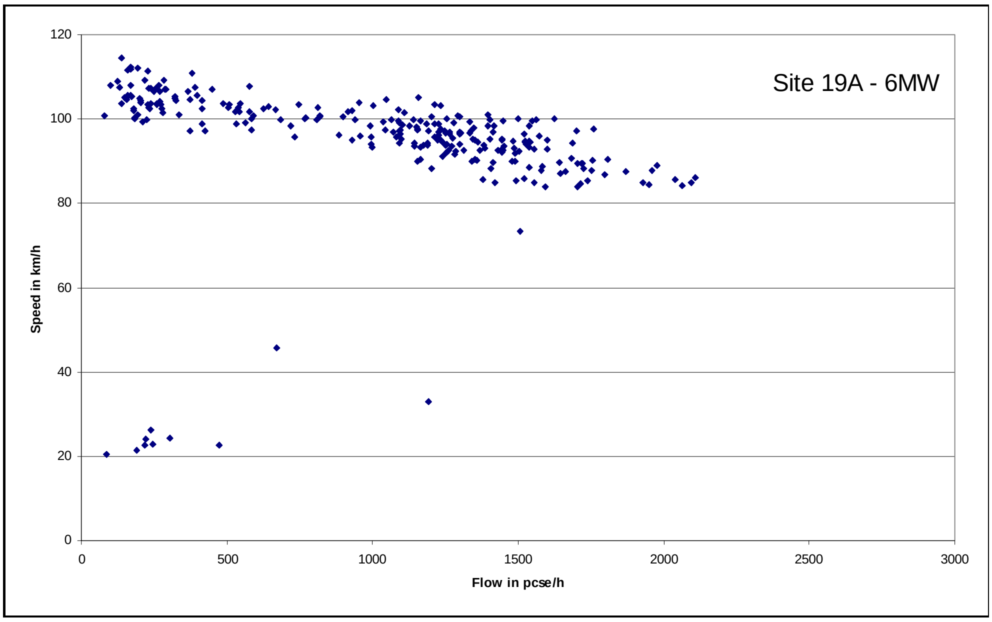


Figure 6.11 Thailand speed-flow results for 6-lane motorway

Ultimate capacity

As noted by *HTC (1999):* “The speed at ultimate capacity is notoriously difficult to estimate. This is because the flows become unstable in that region when the demand exceeds capacity there is a breakdown in the flow that leads to markedly lower speeds.” Measurements of speeds at flows such as shown in Figure 6.11 should be used to determine an appropriate speed.

Low flow period

There are two techniques that can be used to investigate the low flow period (*HTC, 1999*):

- Regression approach

  Linear regressions can be performed for a number of flow ranges:

  <100; <200, <300, <400, <500, etc.

  If there are no traffic interactions one would not expect to find that the resulting equation was statistically valid[1]. The upper limit of the last range before a statistically valid equation was taken to be the breakpoint where the low-flow period ends.

- Maximum speed approach

  For each site, average speeds at different flows are established. The flow where the maximum speed was observed was taken to be the breakpoint. To account for variability in the data it is common to average the flows in bands of, say, 100 PCSE/h.

HTC (1999) found that the regression method did not give valid results at all sites whereas the maximum speed approach did. However, the regression method was considered to give more reliable results.

[1] There are a variety of methods for assessing the validity of a regression equation. The easiest is to ensure that the 't' statistics of the regression coefficients are significant at 95% confidence. *HTC (1999)* found that this yielded the best results.

6.4.3 Passenger car space equivalencies: S-II/S-III

Traffic streams are comprised of a range of vehicles, from passenger cars to heavy trucks. For the purposes of capacity it is necessary to convert these into a homogeneous traffic stream. This is done in HDM-4 through the use of Passenger Car Space Equivalents (PCSE). The PCSE differs from the tradition passenger car unit/equivalency (PCU/PCE) in that it is based on the area occupied by the vehicle. The PCU is based on the area occupied as well as the vehicle's performance. Since HDM explicitly models performance, the use of PCE/PCE would lead to double counting. For this reason, PCSE values are always lower than PCU values.

Hoban et al. (1994) describes the recommended PCSE values for HDM. This work builds upon the earlier work of *Hoban (1987)*. The PCSE were established based on the assumption that each vehicle has a typical length as well as typical leading and following headways[1]. Using an assumed speed of 72 km/h, *Hoban et al. (1994)* calculated the basic PCSE values shown in Table 6.8. These basic values only accounted for the longitudinal space occupied by vehicles. Additionally, larger vehicles tend to impact adjacent lanes, with this "adjacent lane" effect being greater for larger vehicles and for narrower roads (*Hoban et al., 1994*). This led to the PCSE values varying by width as shown in Table 6.8.

Table 6.8
PCSE values by vehicle class

						Recommended values (includes "basic" plus adjacent lane effects)		
Vehicle class	**Avg. length (m)**	**Time headway (s)**	**Space headway (m**	**Total space (m)**	**Basic PCSE**	**2-lane 4-lane**	**Narrow 2-lane**	**1-lane**
Car	4.0	1.6	32	36.0	1.0	1.0	1.0	1.0
Pickup	4.5	1.8	36	40.5	1.1	1.0	1.0	1.0
Heavy Bus	14.0	2.2	44	58.0	1.6	1.8	2.0	2.2
Light Truck	5.0	2.0	40	45.0	1.3	1.3	1.4	1.5
Medium Truck	7.0	2.2	44	51.0	1.4	1.5	1.6	1.8
Heavy Truck	9.0	2.4	48	57.0	1.6	1.8	2.0	2.4
Truck and Trailer	15.0	2.5	50	65.0	1.8	2.2	2.6	3.0

Notes:

1 Basic data from *Hoban et al. (1994)*

2 Time headway calculated from space headway using 72 km/h speed

3 Truck and Trailer average length increased to 15 from 11 given in *Hoban et al. (1994)* based on difference between total space and space headway

1 The PCSE is analogous to truck equivalencies calculated using the headway method. This defines the equivalency of a truck as, the ratio of the average headway for trucks in the stream (in s), to the average headway for cars in the stream (in s). See *McLean (1989)* for a detailed discussion of equivalency factors and their determination.

The PCSE values in Table 6.8 were calculated by firstly establishing the total space occupied by passenger cars (default = 36.0 m). The **Basic PCSE** values for the other vehicle classes were then determined from the ratio of their total space to the passenger car space. This basic value was then subjectively adjusted for width effects to obtain the recommended values in the final three columns.

Local calibration of the PCSE values can be done by adjusting the basic assumptions in Table 6.8 to reflect local vehicles. The easiest data to obtain is that on vehicle lengths. Field studies could focus on establishing the average headways for vehicles thereby giving the total space. Axle detectors are quite useful in this type of work since the time between the first axles of successive vehicles divided by the velocity (in m/s) gives the total space occupied.

6.4.4 Fuel consumption

Overview

The HDM-III and HDM-4 fuel consumption models are based on mechanistic principles. HDM-III was developed using a regression-based approach while HDM-4 is more of a **pure** mechanistic formulation that more effectively reflects the different components affecting fuel consumption. To calibrate the fuel consumption model parameters it is either necessary to have a series of raw fuel consumption measurements or, alternatively, existing fuel consumption models, such as those developed using multiple linear regression techniques. Each of these will be discussed individually. The discussion will focus on the HDM-4 fuel model, with some reference to HDM-III.

HDM-4 fuel consumption model

The HDM-4 fuel consumption model predicts the fuel consumption as a function of the power as follows:

$$IFC = MAX\,(\alpha,\ \xi\ Ptot) \qquad \ldots(6.16)$$

which can be expressed as:

$$IFC = MAX\,(\alpha,\ \xi\ Ptr + \xi Pengaccs) \qquad \ldots(6.17)$$

where:

IFC	is the instantaneous fuel consumption (ml/s)
α	is the idle fuel consumption (ml/s)
ξ	is the fuel-to-power efficiency factor (ml/kW.s)
Ptot	is the total power requirements (kW)
Ptr	is the total tractive power requirements (kW)
Pengaccs	is the total engine and accessories power (kW)

The fuel model calibration therefore focuses on the parameters α and ξ. It is assumed that the models used to predict the total power have been correctly calibrated elsewhere. The equations used to predict the total power are as follows:

$$Ftr = Fa + Fg + Fr \qquad \ldots(6.18)$$

$$Fa = R1\ v^2 \qquad \text{…(6.19)}$$

$$Fg = R2\ GR \qquad \text{…(6.20)}$$

$$Fr = R3 + R4\ v^2 \qquad \text{…(6.21)}$$

The variables R1 to R4 are simplifications of the variables in the actual HDM-4 equations:

$$R1 = 0.5\,RHO\ CD\ CDMULT\ AF \qquad \text{…(6.22)}$$

$$R2 = M\ g \qquad \text{…(6.23)}$$

$$R3 = FCLIM\ CR2\,(b11\ Nw + CR1\ b12\ M) \qquad \text{…(6.24)}$$

$$R4 = FCLIM\ CR2\ CR1\ b13 \qquad \text{…(6.25)}$$

The total forces are:

$$Ftr = R3 + (R1 + R4)\,v^2 + R2\ GR \qquad \text{…(6.26)}$$

$$Ptr = Ftr\,\frac{v}{1000} \qquad \text{…(6.27)}$$

$$Ptr = \frac{\left[(R3 + R2\ GR)\ v + (R1 + R4)\ v^3\right]}{1000} \qquad \text{…(6.28)}$$

where:

v	is the vehicle speed (m/s)
RHO	is the mass density of air (km/m^3)
CD	is the aerodynamic drag coefficient
CDMULT	is the aerodynamic drag coefficient multiplier
AF	is the projected frontal area
CR1 and CR2	are rolling resistance model parameters
b11 to b13	are rolling resistance model parameters
GR	is the gradient (%)
FLCIM	is a climatic adjustment factor

Calibration using raw fuel consumption data

This is the best way of calibrating the model, although the data are seldom readily available. There have been numerous studies conducted into fuel consumption and these are readily found in the literature. *Yuli (1996)* gives a good description of calibrating the HDM-III fuel model, and the conversion of the results to the HDM-4 fuel model. There are two parameters to be calibrated:

- **Base engine efficiency (ξb)**
- **Change in engine efficiency at higher power**

The calibration procedure is as follows:

1 Undertake observations with the engine idling. These give the idle fuel rate.

2 Undertake a series of on-road measurements travelling at different speeds. This will result in a set of data that are pairs of speed and fuel. Ensure that the fuel is expressed in terms of ml/s. If, for example, it was measured in terms of ml/km it is converted to ml/s by multiplying by the factor v/1000.

3 It is necessary to calculate the power required by the vehicle at each speed. Using the HDM-4 mechanistic model formulation calculate the total tractive power requirements (Ptr) for the speeds where fuel measurements are available. This is done using the equation given above for Ptr.

4 Using the default HDM-4 parameters, calculate the engine and accessories power Pengaccs.

5 The tractive power is corrected for driveline efficiency using the default HDM-4 parameter to give the following equation for total power:

$$Ptot = \frac{Ptr}{edt} + Pengaccs \qquad \dots(6.29)$$

6 Divide the fuel by the total power (Ptot). This gives the power-to-fuel conversion factor ξ for that power.

7 Undertake a regression analysis and fit the model parameters ξb and ehp in the model:

$$\xi = \xi b\left(1 + ehp\,\frac{Ptot}{Pmax}\right) \qquad \dots(6.30)$$

where:

Pmax is the maximum rated power for the vehicle.

8 If the analysis does not allow for the simultaneous estimation of ξb and ehp, assume the HDM-4 default relationship for ehp.

Calibration from existing relationships

There two ways by which existing relationships can be used to develop the calibrated parameters for α and ξ:

- **Generating data**
- **Via the model coefficients**

Generating data

1 Use the existing models to generate a series of fuel consumption predictions covering the full range of operating conditions likely to be encountered, for example: speeds, roughness, gradients.

2 Make an assessment of the range of conditions typically encountered and weight the data set accordingly. For example:

70% operate on flat terrain between 50 and 80 km/h;

20% on rolling terrain between 40 and 70 km/h;

10% in mountainous terrain between 20 and 50 km/h.

3 Create a data set that contains data reflecting the assessments from (2). This will ensure that the results have the appropriate weightings.

4 From this data set, follow the instructions given above for calibration with raw fuel data.

Model coefficients

As described in *NDLI (1997)*, a commonly used model form that has been fitted to fuel consumption data is:

$$SFC = A0 + \frac{A1}{S} + \left(A2\ S^2\right) + (A3\ \ IRI) + (A4\ \ RS) + (A5\ \ FL) \qquad \ldots(6.31)$$

where:

SFC	is the specific fuel consumption (l/1000 km)
A0 to A5	are regression coefficients
S	is the speed (km/h)
RS	is the rise (m/km)
FL	is the fall (m/km)

It can be shown that this model is compatible with the HDM-4 model. Ignoring the fall term, the following are the terms that correspond between the two models:

Regression	HDM-4
A0 + A3 IRI	(ξ/edt) R3
A1	3600 ξ Pengaccs
A2	(ξ/edt) (R1 + R4)/(3.6^2)
A4	(ξ/edt) R2

The terms A0 and A3 are thus mainly related to rolling resistance; A1 to the fuel required to operate the engine; A2 to aerodynamic resistance, and A4 to the gradient resistance.

Calibration is done as follows:

1 Calculate the SFC for a range of conditions similar to those described above under **Generating Data**.

2 The idle fuel consumption α is given by A1/3600, or ξ Pengaccs.

3 Subtract A1/S from the total SFC to obtain the fuel consumption due to forces opposing motion. Convert this to the IFC in ml/s by dividing SFC by the speed.

4 Calculate the total tractive power using the HDM equations with the vehicle characteristics and divide by the driveline efficiency.

5 Calculate ξ by dividing IFC by the tractive power, corrected for driveline efficiency.

In applying this method in India, *NDLI (1997)* found some inconsistencies between the results from Indian equations and the default values in HDM-4. For some vehicles the values were outside of the range of values considered viable, while for others they agreed well. This method should therefore be applied prudently.

6.4.5 Acceleration noise

Overview

Acceleration noise is used in HDM-4 to predict the effects of speed variations due to congestion or road conditions on vehicle operating costs. It is a relatively simple parameter to measure and can be done through a small controlled experiment. Once the appropriate equipment has been obtained, the work should only take a few days.

Figure 6.12 is an example of the acceleration profile of a vehicle on a motorway (*NDLI, 1995a*). It can be observed that the accelerations before entering the motorway were much greater than once on the motorway. If a vehicle has the same start and end speeds over a section, the mean acceleration will be zero. HDM-4 therefore uses the standard deviation of acceleration, called the **acceleration noise** to represent the magnitude of accelerations; the higher the acceleration noise, the greater the magnitude of the accelerations.

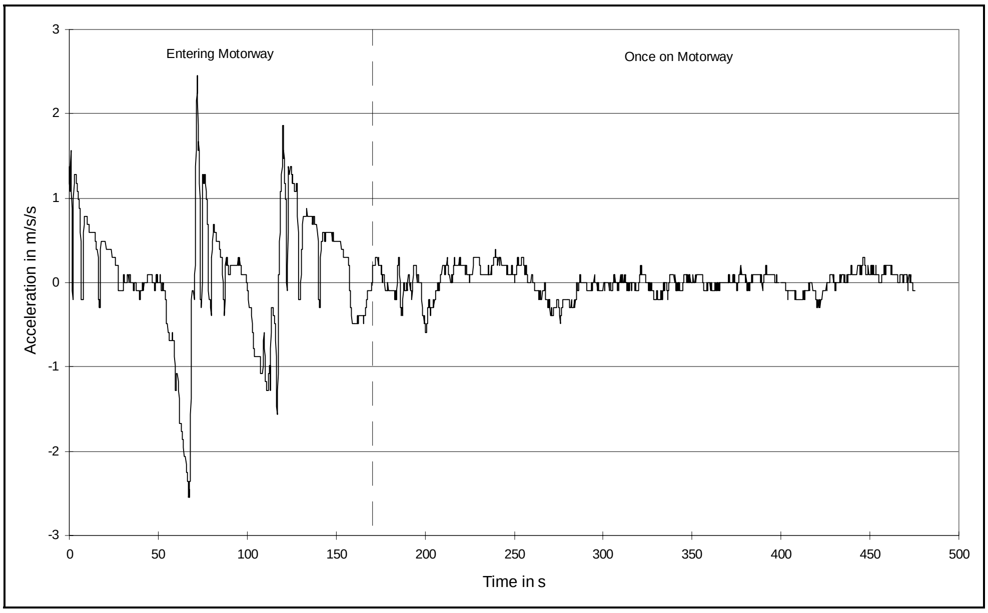


Figure 6.12 Example of acceleration data on motorway

In HDM-4 the acceleration noise is a function of:

- **Traffic interactions** (that is, the volume-to-capacity ratio)
- **Driver** (σadr)
- **Roughness** ($\sigma airi$)
- **Road alignment** (σaal)
- **Non-motorised traffic -** NMT ($\sigma anmt$)
- **Side friction** (σasf)

The total acceleration noise is calculated as:

$$\sigma a = \sqrt{\sigma at^2 + \sigma an^2} \qquad \ldots(6.32)$$

where:

σa	is the acceleration noise (m/s^2)
σat	is the traffic noise (m/s^2)
σan	is the natural noise (m/s^2)

The natural noise is calculated as:

$$\sigma an = \sqrt{MAX((\sigma adr^2 + \sigma aal^2), \sigma asf^2, \sigma anmt^2, \sigma airi^2)} \qquad \ldots(6.33)$$

HDM-4 uses a sigmoidal function for the traffic interactions and a linear function for the others. The acceleration noises are combined to obtain the total acceleration noise for a given road section.

A Level 2 calibration of any of the model consists of making measurements at two extremes and assuming that the existing model applies. Level 3 calibration sees the model formulations requantified.

It should be noted that it is not necessary to calibrate all components of the model. If one is looking only at the effects of capacity improvements, the traffic interaction model should be calibrated and possibly the NMT/side-friction model. If roughness improvements are of interest then only the roughness model needs to be calibrated.

Measurement of acceleration noise

To measure acceleration noise it is necessary to have a vehicle equipped with some form of a data logger. This data logger must measure speed or acceleration at 1 s intervals. Direct speed measurement is preferable to an accelerometer as with accelerometers it is necessary to identify flat sections of road to eliminate gradient effects.

There are a variety of devices available, ranging from low to high cost. A suitable device usually incorporates a dedicated data logger which will monitor the speed or acceleration on an instantaneous basis. Speed based systems are easier to use than accelerometers since the data can usually be directly analysed to establish the acceleration noise.

One such device with purpose-built features, ROMDAS, measures the speed of the vehicle and contains data processing routines designed for calibrating the HDM-4 acceleration noise model. Further information is available through the HTC Infrastructure Management web site given in the About This Manual section of this document.

Irrespective of which component(s) of the acceleration noise model are being investigated, the basic approach is the same:

- Outfit a vehicle with the measurement equipment. The vehicle should be as representative of the vehicle class as possible. Ideally, the measurements should be made with several vehicles and several drivers, but *NDLI (1995a)* found that the results were broadly similar so this is not critical.
- A test route should be established which would allow for the two critical points to be monitored:
  - low impact
  - high impact

  For example, if roughness is of interest the measurements should be made on a low and high roughness road, for traffic impacts on non-congested and highly congested roads. It is vital that all the other road attributes (for example, width, alignment, traffic composition) be similar. That way, any differences in acceleration noise can be attributed to the parameter of interest.
- It is important that the route be long enough for the process to stabilise. Accelerations are assumed to be random and if too short a route is used the process will not stabilise. Figure 6.13 shows the effect of sampling interval on data based on an analysis of the *NDLI (1995a)* measurements (*Greenwood, 1998*). On the basis of this it is recommended that a minimum sample of 5 minutes be collected in a single run for each attribute of interest.

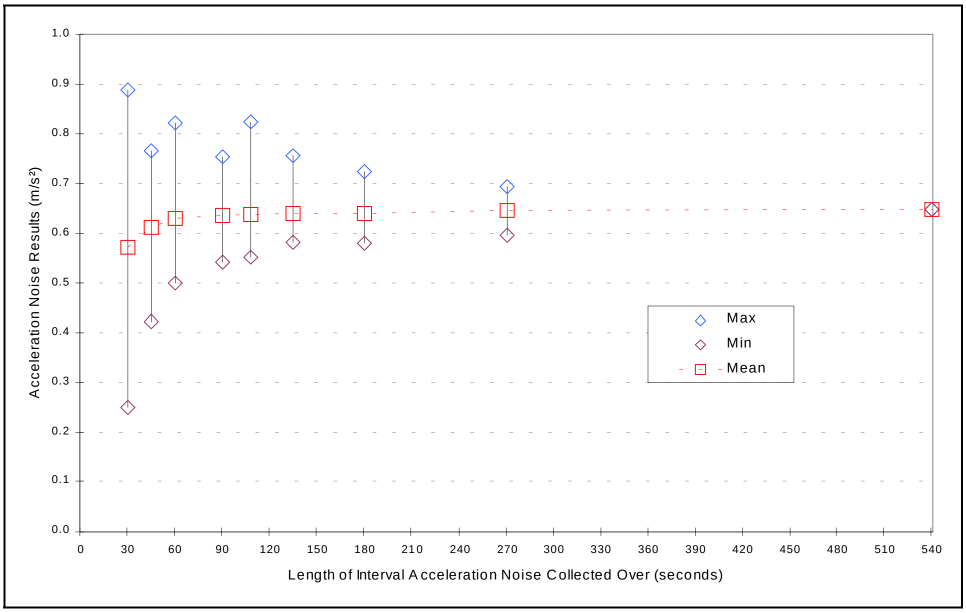


Figure 6.13 Variation of acceleration noise with sample length

Driver noise calibration

This must always be calibrated. The driver noise is the noise that arises when a vehicle is operated under normal operating conditions without traffic, roughness, alignment, NMT or side friction effects.

The measurements should be made on as good a standard road as is available for the type of road of interest. HDM allows data to be input for different road classes so measurements can be made, for example, on two-lane and multi-lane highways.

If no other measurements are made (for example, alignment) it is assumed that the natural noise is entirely due to the driver: σan = σadr.

Traffic noise calibration

A Level 2 calibration of the traffic noise model entails taking measurements under severely congested conditions. This gives the maximum noise (σamax). Since the measurements also include the natural noise, the maximum traffic noise (σatmax) is calculated as:

$$\sigma atmax = \sqrt{\sigma amax^2 - \sigma an^2} \qquad \ldots(6.34)$$

The value for σatmax is used in conjunction with a sigmoidal relationship to predict the traffic noise at intermediate volume-to-capacity ratios.

Roughness, road alignment, NMT and side friction

All of these assume a linear relationship between severity and acceleration noise. Measurements are taken at the extreme conditions, for example a roughness of 20 IRI m/km, and the noise is linearly interpolated from a value of 0 to this extreme condition. The measurements are done as follows. The example applies to roughness, but can be done the same way for the other factors.

1 Identify road sections with low and high roughnesses which are otherwise identical (that is, alignment, traffic, roadside friction etc.).

2 Measure the acceleration noise on the low and high roughness sections in the absence of any traffic interactions.

3 Establish the difference between these values. This represents the maximum change due to roughness.

6.4.6 Parts and labour costs

A Level 2 calibration can focus on the magnitude of costs as well as the effects of roughness.

The magnitude of the costs is done by surveying a sample of users and establishing an average cost and then adjusting the HDM parts and labour model predictions so that the predicted cost is the same as the surveyed cost at the average conditions. *Kampsax (1986)* use this approach in Pakistan for HDM-III. A National Trucking Study had established a total maintenance and repair cost of 0.35 Rs/km for medium trucks. Using an average age of 300,000 km and an estimated average roughness of 6 IRI m/km, the parts cost was calculated using the default HDM-III parameters and the rotation factor was reduced to 32.5 per cent of the default value to obtain the same costs.

Opus (1999) presents the results of a study using accelerometers to investigate the effects of roughness on parts consumption. This work offers great scope for a relatively rapid and low-cost method for verifying the shape of the roughness model (that is, linear) and the roughnesses below which parts consumption will not be influenced.

Accelerometers (10 g strain gauge with 0-120 Hz frequency response and low signal to noise ratio) were fitted to vehicles. For cars, an accelerometer was fixed to the rear axle, as close to the wheel assembly as possible. Another was firmly fixed to the boot floor, as close as possible to being directly over the axle accelerometer. For trucks, an accelerometer was fitted to the rear axle and another immediately above on the cargo tray. By mounting the axle and body accelerometers in close proximity to each other, the degree to which roughness induced axle vibrations were transmitted to the vehicle body could be determined.

The vehicles were driven at different speeds over sites with different roughnesses and the accelerations were recorded. Figure 6.14 is an example of the body accelerations versus roughness for medium trucks[1]. It can be seen that the body accelerations increase with increasing roughness and this can be used to calibrate the roughness effects in the HDM parts consumption model. It will also be noted that at lower roughnesses the accelerations do not have much of an impact. This can be used to confirm the roughness level below which there is no impact on parts consumption.

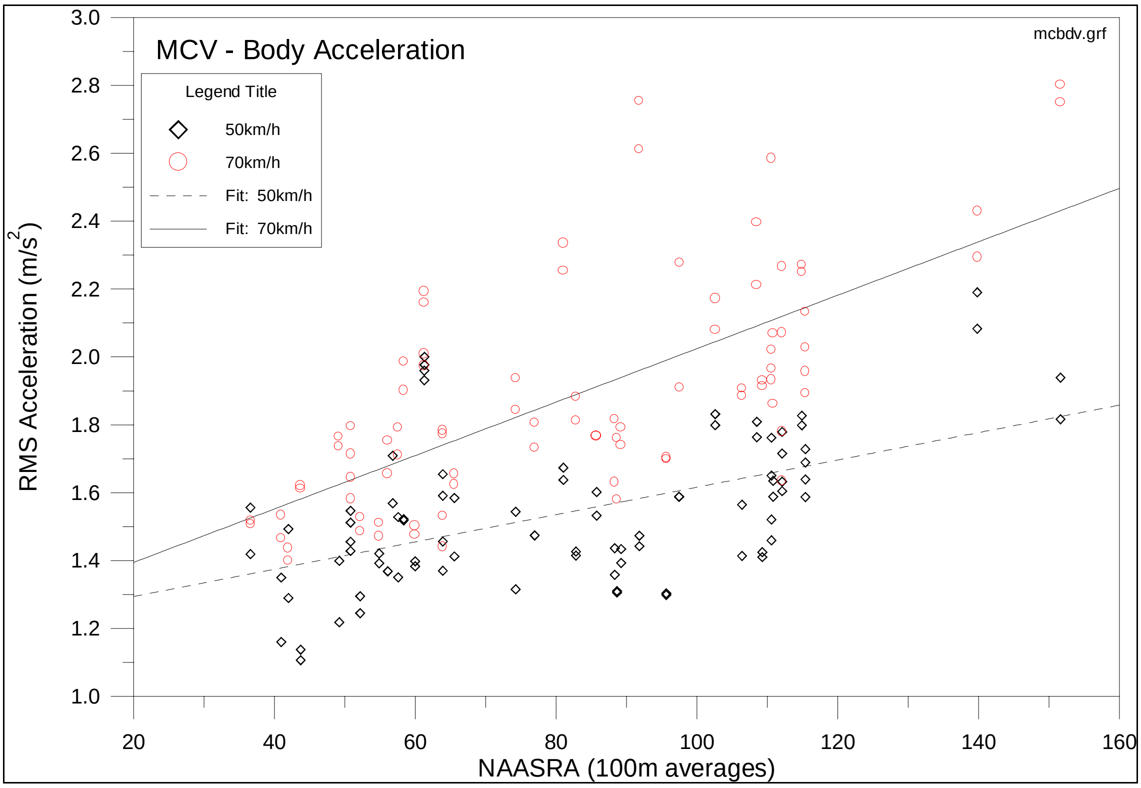


Figure 6.14 Effect of roughness on body acceleration (Opus (1999))

One finding of this research which is not considered in HDM is the speed-roughness dependence. The results clearly show that the higher the speed, the greater the impact of roughness on body accelerations.

6.4.7 Used power

For a Level 2 calibration the driving power is established by measuring speeds on gradients. As described in *Bennett (1994)*, there are two approaches that can be used:

- **Crawl speeds**
- **Speed differences** (or the **spatial method**)

[1] The roughness units in this figure are NAASRA counts/km. A conversion to IRI is 1 IRI = 26.4 NAASRA counts/km.

Crawl speeds

On a long, steep upgrade vehicles will travel at a constant **crawl** speed where all the forces are in balance. Since there is no acceleration, the observed speed can be substituted into the following equation to calculate the used power:

$$\frac{Pu}{M'} = a\,v + \frac{0.5\ RHO\ CD\ CDmult\ AF + R3}{M'}\,v^3 + \frac{R4}{M'}\,v + \frac{M\ g\ GR}{100\ M'}\,v \qquad \ldots(6.35)$$

where:

Pu	is used power (W)
v	is the vehicle speed (m/s)
M’	is the effective mass (km)
RHO	is the mass density of air (km/m^3)
CD	is the aerodynamic drag coefficient
CDmult	is the aerodynamic drag coefficient multiplier
AF	is the projected frontal area
R3 and R4	are rolling resistance components (Equations 6.24 above and 6.25 above)
GR	is the gradient (%)
g	is the acceleration due to gravity (m/s^2)

Speed differences

When the gradients are insufficient for vehicles to reach their crawl speeds the speed reduction between two points on the grade can be used to establish the driving power. Called the speed difference, or **spatial method**, this uses the following equation to establish the used power:

$$\frac{\overline{Pu}}{M'} = \frac{v_1^2 - v_o^2}{2\ T} + \frac{0.5\ RHO\ CD\ CDmult\ AF + R3}{M'}\left(\frac{SL}{T}\right)^3 + \frac{R4}{M'}\,\frac{SL}{T} + \frac{M\ g}{M'}\,\frac{HC}{T} \qquad \ldots(6.36)$$

where:

$\overline{Pu}$	is the mean power used over the section
v_1	is the speed at the exit to a section (m/s)
v_o	is the speed at the entry to a section (m/s)
SL	is the section length (m)
T	is the time taken to travel the section length (s)
HC	is the height change over the section (m)

It should be noted that this method gives the average power used over the section, as opposed to the crawl speed method which gives the maximum used power.

Bennett (1994) compared these two methods and concluded that both will give similar results, with the spatial method giving the best results when measurements are made as far along the grade as possible. When using the spatial method there is almost a linear relationship between the predicted power used and the distance measured. Thus, any errors in measured distances will lead to a corresponding over or under estimation of the used power.

Used power will likely be proportional to the gradient, with vehicles using more power on steeper gradients (*Bennett, 1994*). However, HDM only allows for a single value to be input so either the average or a value based on the most typical gradient should be adopted.

6.4.8 Braking power

For a Level 2 calibration the used power can be calculated from speed in the same manner as the driving power. Either the **crawl speed** or **speed difference** approach can be used (*Bennett, 1994*). This entails measuring the speed of the vehicle and then calculating the used braking power (which is expressed as -Pu).

6.4.9 Service life

The Level 2 method of estimating the service life is through the use of a survival curve. This is a curve showing the proportion of the original number of vehicles remaining in service of a given age. The curves for each original year of registration are unique although a reasonable representation may be obtained by evaluating the age structure of the vehicles in use at a particular date. By comparing this age structure with the number of units originally put into service each year, a quasi-survival curve may be prepared from which the average service life may be calculated. Appendix I illustrates how the survival curve is derived and how the average service life is calculated from the curve.

In order to employ a survival curve it is necessary to have an age distribution of the existing vehicle fleet and records of the number of vehicles originally registered. In certain circumstances the registration data may not be available or non-applicable. A survival curve can still be constructed by obtaining two age distributions one year apart. This will show the number of vehicles retired for each age group from which a survival curve can be calculated.

These techniques will give the average service life for the fleet as a whole, or a sub-section of the fleet operating in a certain area. It is necessary to estimate the traffic-weighted roughness for this area so as to adjust the value during the Optimal Life calculations to the roughness on a smooth pavement. This is done by through calculating the product of the roughness on major links and the traffic volume.

6.4.10 Utilisation

For a Level 2 calibration a field survey is required. This entails stopping a sample of vehicles and recording their odometer readings or, for commercial vehicles, recording data from the driver’s logbook . In some countries the data are available from fleet management companies. The data should be grouped by age with the average annual kilometreage of all vehicles of a given age being calculated. If a survival curve is available, the average lifetime kilometreage can be calculated by multiplying the average annual kilometreage for each year by the survival ratio. The total of these products represents the average lifetime kilometreage. Appendix I illustrates this technique using data from *Bennett (1985)*. The average annual kilometreage is then calculated as the average lifetime kilometreage divided by the average service life.

If a survival curve is not available, a reasonable estimate of the average annual kilometreage will be obtained by calculating the average kilometreage of all the survey data. If the sample is unbiased, this should yield a similar value to that from the survival curve technique. *Bennett (1985)* used this approach and it yielded an average of 10,800 km/year that compared with 9,850 km/year using the survival curve approach with the same data. The differences are probably largely due to the biasing of the *Bennett (1985)* data towards newer vehicles that had higher average utilisations.

6.4.11 Oil consumption

Oil consumption is broken into two components:

- Oil loss due to contamination, and
- Oil loss due to operation.

Oil loss due to contamination is a function of the distance between oil changes. Oil loss due to operation is a function of the fuel consumption.

The contamination losses can be calibrated by establishing the typical distances between oil changes. These may be based on the manufacturer’s specifications but most likely are different. The losses are calculated as:

$$\mathrm{OILCONT} = \frac{\mathrm{OILCAP}}{\mathrm{DISTCHG}} \qquad \ldots(6.37)$$

where:

OILCONT	is the contamination loss (l/1000 km)
OILCAP	is the engine oil capacity (l)
DISTCHG	is the distance between oil changes (km)

Oil loss due to operation is more difficult to measure and, as it likely has limited impacts on the total costs, should not usually be calibrated. If calibration is desired it should be done as follows for a sample of vehicles:

1. Change the oil and replace with a known quantity of oil.
2. Operate the vehicles for at least 5-10,000 km, keeping exact records of the amount of fuel used and, if necessary, additional oil added.
3. At the end of the period drain the oil completely and measure the amount remaining. The difference between this volume and the initial volume is the oil lost due to operation. Dividing this by the total fuel consumption gives the operation loss rate.

6.5 Level 3 - Adaptation of model

6.5.1 Acceleration noise

Overview

Compared with a Level 2 calibration, a Level 3 calibration of acceleration noise can address:

- Quantifying parameters for additional vehicle classes or road types; or,
- Developing alternative relationships

Expanding the data to additional vehicle classes would see experiments done in a similar manner to Level 2, but with a larger matrix of vehicles, drivers, and road conditions. To develop alternative relationships would address the key assumptions in the traffic noise and roughness/road alignment/NMT/side friction relationships.

Traffic noise calibration

The traffic noise model is a sigmoidal function that gives different predictions depending upon the volume-to-capacity ratio (VCR) and the volume where speeds begin to be affected by traffic interactions (Qo). The latter integrates the traffic noise model with the HDM-4 speed flow model so that when VCR < Qo there is only natural noise; above that the combination of natural and traffic noise. The model form in HDM-4 is:

$$\sigma at = \sigma atmax \frac{1.04}{1 + e^{(a0 + a1\,VCR)}} \qquad \ldots(6.38)$$

The effort should focus on calibrating the parameters a0 and a1 and then developing a new model form around the calibrated parameters. This will entail taking measurements of σat at different levels of VCR and then fitting a curve to the data. Because of the difficulties in estimating VCR, it is recommended that surveys be conducted at VCR levels of 1, 0.5 and 0.3. This will provide an indication of the trend in the data. Additional measurements can be then be made to supplement the data as necessary.

Roughness, road alignment, NMT and side friction

The assumption is that there is a linear relationship between these factors and acceleration noise. Collecting data over a range of road conditions would test the validity of this assumption. It is important that all the other factors be held constant so that the only factor influencing the results is the factor of interest.

6.5.2 Speed β

The quantification of the speed β requires a major field study. A series of speed data are required covering the full range of conditions under which the constraining speed models will be applied.

This data set should then be analysed using the same statistical methods as those described in *Watanatada et al. (1987c)*. The analysis should consider the same issues, such as logarithmic transformations, and requantify the model from first principles.

6.5.3 Rolling resistance

To be included in a subsequent edition of this document

6.5.4 Idle fuel consumption

To be included in a subsequent edition of this document

6.5.5 Fuel efficiency

To be included in a subsequent edition of this document

6.5.6 Critical gradient length

To be included in a subsequent edition of this document

6.5.7 Tyre stiffness

To be included in a subsequent edition of this document

6.5.8 Engine accessory power

To be included in a subsequent edition of this document

6.5.9 Engine drag power

To be included in a subsequent edition of this document

6.5.10 Drivetrain efficiency

To be included in a subsequent edition of this document

6.5.11 Tyre wear parameters

To be included in a subsequent edition of this document

6.5.12 Parts consumption

To be included in a subsequent edition of this document

6.5.13 Labour hours

To be included in a subsequent edition of this document

7 RDWE calibration

7.1 Bituminous paved road deterioration

The HDM flexible pavement deterioration and works effects (RDWE) model (as implemented in HDM-III) has six deterioration adjustment factors. Table 7.1 shows the impact elasticity class for various applications combined with the typical range of values of the factors to give a potential net impact. Six criteria representing different applications were used in the analysis and these are listed in the footnote to Table 7.1.

Table 7.1
Ranking of impacts of road deterioration factors

Deterioration factor	Impact class for given criteria[1]						Impact Elast-icity	Typical values of Factor	Net Impact (%)	Sensi-tivity class
	1	2	3	4	5	6				
Roughness-age-environment	D	D	B	C	B	B	0.20	0.2 - 5.0	10	High
Cracking initiation	A	C	B	B	C	B	0.25	0.5 - 2.0	6	
Cracking progression	A	C	C	C	C	C	0.22	0.5 - 2.0	6	
Rut depth progression	D	A	B	D	C	C	0.10	0.5 - 2.0	3	Low
Roughness progression general	D	D	B	D	C	B	0.09	0.8 - 1.2	1	
Potholing progression	D	D	C	B	C	D	0.03	0.3 - 3.0	2	
Ravelling initiation	D	D	D	C	D	D	0.01	0.2 - 3.0	1	

Note:

1 Criteria
1 = Amount of cracking
2 = Rut depth
3 = Roughness
4 = EIRR for patching
5 = EIRR for reseal
6 = EIRR for overlay

Impact sensitivity
A = S-I
B = S-II
C = S-III
D = S-IV

The roughness-environment factor is clearly the most important, due to the wider range of its values, followed by the cracking initiation and progression factors. The general roughness progression factor has low priority, despite its moderate sensitivity, because its range is small based on many inter-country validation studies. As shown in the table, for convenience, these adjustments can be grouped into just two classes:

- **High impact**
- **Low impact**

The following sections describe calibrating these factors for the three levels of calibration.

In HDM-4, the flexible pavement deterioration and works effects models have been generalised to allow more specific adaptations to local materials, and adding a few improved relationships. The changes include an increase in the number of calibration factors available for adjusting the bituminous RDWE model from 6 to 20.

Consequently, while this chapter discusses mainly the determination of calibration adjustments for the HDM-III RDWE model, there are only relatively brief and incomplete references to specific HDM-4 calibration factors. Cross-references are given to HDM-4 model descriptions published in Volume 4 of this document series, Analytical Framework and Model Descriptions.

In most cases, the guidelines provided may be directly transposed to apply to the HDM-4 model, although such approach requires a good knowledge of the specific relationships. The HDM-4 aspects of these guidelines will be published in a future update to this Guide.

7.2 Level 1 - Basic application

A Level 1 calibration thus adjusts the top three deterioration factors from Table 7.1 and retains the default values of unity for the remaining factors in most instances.

7.2.1 Roughness-age-environment adjustment factor: S-I

This factor, which determines the amount of roughness progression occurring annually on a non-structural time-dependent basis, is related to the pavement environment and is effectively an input data parameter rather than a calibration adjustment. The factor adjusts the environment coefficient, *m*, which has a base value of 0.023 in the model, representing 2.3 per cent annual change independent of traffic, that is:

$$\Delta R_{te} = K_{ge}\ 0.023\, R_t \qquad \ldots(7.1)$$

where:

K_{ge} is the roughness age-environment calibration factor (HDM-III)

ΔR_{te} is the change in the roughness component due to environment in the 1-year analysis time increment

R_t is the roughness at the beginning of the year *t*

In HDM-4, the calibration factor K_{gm} is equivalent to the HDM-III factor K_{ge}. The following discussion uses K_{ge}.

Method 1

For a Level 1 Calibration, the values are established based on the general environmental conditions and the road construction, drainage standard. This is done as follows:

Step 1 Identify the environment applicable to the immediate vicinity of the road project in terms of the classifications provided in Table 7.2.

Step 2 Select the appropriate value of *m* from Table 7.3 according to the environmental classification.

Step 3 Determine the effective *m*-value, m_{eff}, by multiplying *m* by a factor k_m according to the standard of road construction and drainage Table 7.4, as follows:

$$m_{eff} = m\ k_m \qquad \text{…(7.2)}$$

Step 4 Calculate K_{ge} from m_{eff} as follows:

$$K_{ge} = \frac{m_{eff}}{0.023} \qquad \text{…(7.3)}$$

Table 7.2
Classification of road environment

Temperature classification	Description	Typical temperature range (°C)	
Tropical	Warm temperatures in small range	20 to 35	
Subtropical - hot	High day cool night temperatures, hot-cold seasons	-5 to 45	
Subtropical - 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	
Moisture classification	**Description**	**Typical moisture index**	**Typical annual precipitation (mm)**
Arid	Very low rainfall, high evaporation	-100 to -61	< 300
Semi-arid	Low rainfall	-60 to -21	300 to 800
Subhumid	Moderate rainfall, or strongly seasonal rainfall	-20 to 19	800 to 1600
Humid	Moderate warm season rainfall	20 to 100	1500 to 3000
Perhumid	High rainfall, or very many wet-surface days	> 100	> 2400

Table 7.3
Recommended values of environmental coefficient, m

Moisture classification	Temperature classification			
	Tropical	Subtropical non-freezing	Temperate - shallow freeze	Temperate - extended freeze
Arid	0.005	0.010	0.025	0.040
Semi-arid	0.010	0.016	0.035	0.060
Subhumid	0.020	0.025	0.060	0.100
Humid	0.025	0.030	0.100	0.200
Perhumid	0.030	0.040		

Table 7.4
Modifying factor of environmental coefficient for road construction and drainage effects

Construction and drainage	Non-freezing environments	Freezing environments
High standard materials and drainage; for example, motorways, raised formation, free-draining or non-frost-susceptible materials, special drainage facilities.	0.6	0.5
Material quality to normal engineering standards; drainage and formation adequate for local moisture conditions, and moderately maintained.	1.0	1.0
Variable material quality in pavement, including moisture or frost-susceptible materials; drainage inadequate or poorly maintained, or formation height near water table.	1.3	1.5
Swelling soil subgrade without remedial treatment	1.3 - 2.0	1.2 - 1.6

Method 2

The values for K_{ge} estimated using this approach could be enhanced with limited field data, as follows:

A small number (two to five) of strong old pavements should be selected in one environment for which accurate roughness, traffic loading and age data are already available, and for which:

- Pavement structural adequacy: PSA > 1; and,
- Pavement surfacing age > 10 years.

The observed estimates of *m* are then calculated for the selected pavements (following the method in Section 7.3.1), averaged and compared with the values selected from the tables. If the values are similar, no changes should be made to the originally chosen values. If the differences are consistently different, such as mostly higher or lower, a partial correction may be made (say about one half of the difference in the means). But care should be taken not to attach too much significance to these few results until a more thorough application of a Level 2 calibration can be made.

7.2.2 Cracking initiation adjustment factor: S-I

Cracking initiation is predicted in terms of the time to the first visible crack, and when the surfacing age first exceeds this time, cracking is deemed to begin. The adjustment factor is a simple multiplier of the time to first crack, that is:

$$TY_{cra} = K_{ci} \ TY \text{ (Predictive relationship for relevant surfacing type)} \qquad \ldots(7.4)$$

where:

TY_{cra} is the time to first visible crack in years

The predictive relationships for each surfacing type take account of the interactive fatigue effects of pavement strength and traffic loading, and the durability effects of ageing. What the relationships could not do without requiring significantly more complex data entry was to define how satisfactory were the material design, manufacture and construction quality, or define the oxidising power of the environment, except in average terms. The calibration adjustment therefore compensates for these in specific situations. Cracking behaviour of bituminous materials tends to be variable, even under fairly identical situations, so it can be dangerous to make general adjustments based on only one or two cases.

The Level 1 calibration attempts to estimate the durability properties by considering the actual or approximate behaviour of lightly-trafficked or strong pavements. It assumes that the predicted structural effects are correct. Thus, it is necessary to estimate the quality of each bituminous surfacing type and likely adjustment required.

Step 1 Evaluate the quality of the available refined bitumens:

- **High quality (HB)**

  Refined by international oil companies for specific uses in the highway industry, from selected crude sources with low wax content.

- **Low quality (LB)**

  Produced by local refineries or from high-wax crude sources, with poor oxidation resistance.

Step 2 Evaluate the likely oxidation by the atmosphere on exposed road surfaces given the local climate:

- **Highly oxidising (HO)**

  Low cloud cover, high incidence of sunshine, high altitudes, depleted ozone area.

- **Moderately oxidising (MO)**

  Mixed conditions of sunshine hours and cloud cover, low to medium altitudes.

- **Low oxidising (LO)**

  Frequent cloud cover, low altitudes, cool subhumid or high rainfall climate.

Step 3 Evaluate the construction quality:

- **High (HC)**

  Careful binder temperature control, adequate binder content, low air voids in asphalt mixtures (or good compaction), high standard of mixing plant and compaction equipment, use of medium or soft bitumens (for example, 80/100 penetration or higher).

- **Fair (FC)**

  Moderate or variable adherence to the qualities above.

- **Low (LC)**

  Frequent over-heating of binder, low binder content, high air voids in asphalt mixtures (or poor compaction), extensive use of hard bitumens (40/50 or 60/70 penetration).

Step 4 Select adjustment factor based on binder quality, oxidising climate and construction quality from Table 7.5.

Table 7.5
Level 1 adjustment factor for cracking initiation

Construction quality	Bitumen quality	Oxidising climate		
		High	Medium	Low
High	High	1.0	1.2	1.5
	Low	0.8	1.0	1.1
Medium	High	0.8	1.0	1.1
	Low	0.6	0.8	0.9
Low	High	0.6	0.8	0.9
	Low	0.4	0.6	0.7

7.2.3 Cracking progression adjustment factor: S-I

The rate of cracking progression in the analysis year is a function of the area of cracking, surfacing type and other factors. The adjustment factor multiplies the amount of increase in area of cracking, so factor values greater than 1 accelerate the progression of cracking.

For a Level 1 calibration, it is recommended that the factor be taken as the inverse of the cracking initiation factor, that is:

$$K_{cp} = \frac{1}{K_{ci}} \qquad \ldots(7.5)$$

7.2.4 Rut depth progression adjustment factor: S-I

In HDM-III, the predicted increase in rut depth in an analysis year is a strong function of the existing:

- **Rut depth** (-)
- **Traffic loading** (+)
- **Pavement strength** (-)

and slightly sensitive to the amounts of:

- **Cracking** and **rainfall** (+)

and whether the surfacing is an:

- **Overlay** (-)

Progression at the early stages is a strong positive function of relative compaction, which is the average ratio of layer density to reference compaction standards. After the first 1-3 years, the rate of progression slows down and only accelerates in the presence of cracking and rain. The adjustment factor is a direct multiplier of the predicted increase so a higher factor accelerates rut depth progression.

If the predicted rut depth is low when the pavement is young (< 5 years) but the observed rut depth is much higher, then the adjustments should first be made through decreasing the value for the relative compaction.

The HDM-III prediction model is a good representation of the effects of structural deformation and densification through the pavement and subgrade. For badly cracked pavements in wet climates, the model may underestimate the rate of progression. The model does **not** represent the:

- **Plastic deformation -** which can occur in thick bituminous surfacings under high temperatures and heavy loading, nor
- **Abrasion -** that occurs under studded tyres used in some freezing climates

In HDM-4, the rut depth progression model has been decomposed to separately represent the four components of rutting:

- **Initial densification**
- **Structural deformation**
- **Plastic deformation**
- **Surface wear due to studded tyres**

Each component may be independently calibrated using separate K adjustment factors (see Chapter C2, Section 10 of Analytical Framework and Model Descriptions).

The procedures given in this edition of this Guide relate primarily to calibrating the prediction of rut depths in HDM-III. Further guidance on calibrating the HDM-4 rut depth component models will be given in a future edition of this Guide. In the meantime, the procedures given in this Guide for calibrating rut depth progression will assist in planning an HDM-4 rut depth model calibration study.

In most cases for a Level 1 calibration of HDM-III or HDM-4, adjustment is not considered necessary. Economic results in the model are barely sensitive to rut depth and the small benefits derive from reduction of the rut depth variation that is associated with rut depth. However, if one of the exceptional situations mentioned above is significant and dominant, the user should consider some adjustment, as follows:

1. **High temperature -** plastic flow deformation in asphalt

   Effects are visible as wavey surface on upgrades or at intersections or as ridge alongside rut, formed by lateral displacement.

   - Used pavements:

     K_{rp} = 1.5 – 4; or,

   - New pavements:

     K_{rp} = 1.2 – 3.

2. **Abrasive wear -** from studded tyre use

   Effects are visible as pock-marked surface wear in a rut without side ridges.

   - All pavements:

     K_{rp} = 1.2 – 2

3. **Cracked pavement -** clayey or silty base materials, and high to moderate rainfall:

- All pavements:

  $K_{rp} = 1.5(1.1 – 2)$

As the available adjustment is approximate for traffic volume-related distress (cases (1) and (2)), a wide degree of latitude is provided and iterative adjustments may be needed until the results meet a test of reasonableness (if the user has some data for verification). If calibration of this mode of distress is important to the user, a Level 2 or 3 calibration is advisable.

7.2.5 General roughness progression factor: S-III

The structure and coefficient values of the roughness progression prediction have proved to be very reliable throughout many countries and climates, so no adjustment is considered necessary in this factor in a Level 1 calibration.

7.2.6 Ravelling initiation factor: S-III

As the adjustment factor for ravelling initiation has low impact on most applications, it is usually reasonable to retain the default value of 1 for it. However, if ravelling and potholing are extensive in practice, the user may wish to make adjustment.

The ravelling initiation factor in HDM applies only to surface treatments. Its value is likely to be similar to the cracking initiation factor when the ravelling is caused by oxidation and hardening of the bituminous binder. A high incidence of hydrophilic mineral aggregates, siliceous minerals such as quartzite or chert, may cause a high incidence of stripping and thus early ravelling. Poor quality construction practices, such as contaminated stone, poor compaction or wet weather, have a strong influence but are allowed for separately in the input variable Construction Quality (HDM-III) or Construction Defects Indicator for Surfacing (CDS) in HDM-4. The HDM-III factor K_{rv} is equivalent to the HDM-4 factor K_{vi} for ravelling initiation.

Typical values may vary widely (but have low importance in the model), for example:

1 **Oxidation-sensitive binder or climate**

$K_{rv} = K_{vi} = K_{ci}$

2 **Siliceous aggregates and moderate-high rainfall**

$K_{rv} = K_{vi} = 0.5\ (0.1 – 0.8)$

3 **Binders modified with adhesive agent or anti-oxidant**

$K_{rv} = K_{vi} = 1.3\ (1.0 – 1.6)$

7.2.7 Potholing progression adjustment factor: S-IV

As the adjustment factor for potholing progression has low impacts on most maintenance alternatives except patching and extremely low maintenance, it is reasonable in most cases to adopt the default value of 1. However, if surface disintegration is dominant in practice, if patching alternatives are expected to dominate, and if there is evidence from preliminary runs of the model that the predicted area of potholing is very different from observed data, the user may wish to make adjustment.

The progression of potholing is highly variable and unpredictable. It depends not only on factors such as the amount of potholing and cracking which are in the model, but also on factors that are difficult to quantify, such as how sensitive the base material is to disintegration and to moisture, which are not in the model. Potholing area is typically small (from <0.01 to 1.0 %) (1 % is about 50-100 potholes per 100 m) and has a fairly strong influence on roughness inside the model, so the user is advised to be careful not to exaggerate the effects.

The adjustment factor directly multiplies the incremental area of potholing of the analysis year. Adjustments may vary widely, for example, K_{ph} = 0.3 - 3.0.

7.3 Level 2 - Calibration of primary relationships

7.3.1 Roughness-age-environment deterioration factor: S-I

Pavement environment coefficient (m)

The pavement environment coefficient, *m*, can be estimated directly from samples of pavements selected in several different environmental zones. The calibration segments should be selected by the following criteria:

Field sampling

About five pavement segments are selected in each of 2-4 climatic zones as follows:

1 Select 2-4 climatic zones representative of the study area in which each climate zone is typical of one (or, at most, two adjacent zones) of the moisture-temperature zones defined in Table 7.2, for example, semi-arid/subtropical, or humid-perhumid/tropical.

2 Select about 5 (between 3 and 10) pavement segments in each zone as follows:

(a) Segment

Select segments about 5 km long (minimum 3 km, maximum 10 km), which may comprise non-contiguous sections of a minimum 1 km length on the same road and reasonably homogeneous in the basic data (type 1) in **Field measurements** below.

(b) Pavement

Select pavement (rehabilitation) age preferably > 10 year (minimum 6 year.)

Note: AGE3 is the age since the last overlay, reconstruction or construction whichever is the most recent, but excluding surface treatments.

Field measurements

For HDM-4 roughness model relationships, refer to Chapter C2, Section 11 of Analytical Framework and Model Descriptions.

Collect the following data:

1 **For all sections**

Pavement age (AGE3), cumulative traffic loading (NE ESA), roughness (m/km IRI), pavement structural number (SNP), pavement type, and drainage environment type.

2 **If pavement surface distress exceeds average PSD 2.0** - then also collect:

RDS, ACRX, APAT, APOT.

3 **Determine average values of each parameter** - for each calibration segment.

Evaluation

The parameter *m* is estimated directly from the summary model for roughness progression (*Paterson and Attoh-Okine, 1992*), assuming the rest of the summary predictive model is correct, as follows:

1 Estimate original roughness, RI_0 (m/km IRI) for each pavement type within either the climate zone or the study zone, whichever is considered most practicable. This is best estimated from pavements of the given type less than 3 year old with mean PSD < 1.0, but also may be made from other estimates (this estimate does not need to be restricted to the same pavements as selected for the *m*-calibration).

2 Estimate *m* from the summary model rewritten as follows depending on which data are available, and using a spreadsheet or the worksheet provided in HDM Tools:

$$m = \frac{\left\{ \ln[RI_t] - \ln\left[RI_0 + 263\ NE(1+SNP)^{-5}\right] \right\}}{AGE3} \qquad \text{…(7.6)}$$

$$m = \frac{\left\{ \ln[1.02\ RI_t - (0.143\ RDS + 0.0068\ ACRX + 0.056\ APAT)] - \ln\left[RI_0 + 263\ NE(1+SNP)^{-5}\right] \right\}}{AGE3}$$

…(7.7)

Adjustment factor K_{ge} (HDM-III) or K_{gm} (HDM-4)

Once the observed values of *m* have been determined for the representative climate zones, the factors Kge or K_{gm} will normally be set to 1.0 and adjusted only for the Road Construction and Drainage quality factor, k_m, according to Table 7.4. In order to calibrate those k_m factor values as well as *m*, the calibration study sampling should include construction and drainage type (in accordance with Table 7.4) as a selection parameter in addition to climate zone in Field Sampling (1) above.

If the observed values of *m* differ from the standard values (Table 7.3) by more than half of the difference between *m*-values for adjacent environmental zones, the assumptions and work of the calibration study should be carefully reviewed to identify possible errors before adopting the observed values.

7.3.2 Crack initiation adjustment factor: (S-II or S-I)

Cracking initiation is predicted as the surfacing age when fatigue cracking becomes visible, with a minimum area of 0.5 per cent of the carriageway area (or about 1.8 m^2 of 100 m lane length). The Level 2 calibration takes account of both the **implicit** and **explicit** predictive parameters through direct field observations. *Implicit parameters* include the material design and climate-related ageing which both affect fatigue behaviour - cracking predictions are therefore calibrated specific to the surfacing type and the climate.

Major surfacing types are represented by separate equations, but in some instances the user may also wish to define subtypes where the behaviour is expected to be significantly different. For example, to distinguish asphalt concrete and gap-graded or hot-rolled asphalt within Asphalt Mixture; or double chip seal, cape seal, and penetration macadam within Surface Treatment. **Explicit** parameters include pavement structural parameters (such as structural number, deflection, surfacing thickness, etc.) and traffic loading parameters (for example, annual loading (YE4), etc.) - these are measured on the sample pavements.

Field sampling

First, stratify the sample by:

1 **Surfacing type and subtype** (material type); and

2 **Climate -** based on oxidising potential (see Section 7.2.2).

This stratification needs to be made realistically into the minimum number of **surfacing-climate** groups that is considered necessary and practical to be applied (typically 1-4). Note that original surfacings need to be classed as separate groups from second and subsequent surfacings (for example, overlays and reseals).

Second, in each **surfacing-climate** group, identify a minimum of 15 pavement sections (of 300 m lane length) with **Low** surface distress (less than 5 per cent area of cracking), selected from a wide range of annual traffic loading (ESA/lane/year) representing the range found on the network. The requirement for **low** surface distress is ideal (since it relates to the initiation state). It should not include sections with nil cracking unless:

The surfacing age is Medium *(6-15 yrs)* or Old *(> 15 yrs)* and they comprise less than 20 per cent of the sample; and sections of Medium distress (5-30 per cent cracking) may be included where necessary, but with adjustment to the observed surfacing age (for example, reduced by one year for each 10 per cent of cracked area).

For a high precision calibration (S-I), to be applied when cracking is considered the primary intervention parameter or when higher precision is generally preferred, the size of the sample should be increased to a minimum of 30 pavement sections in each surfacing-climate group.

Field measurements

On each identified cracking calibration pavement section, the following data should be determined and recorded:

1 **Surfacing age** (years)

2 **Percentage area of all cracking** (more than 1 mm width)

3 **Percentage area of wide cracking** (more than 3 mm width or spalled narrow cracks)

4 **The explicit independent parameters relevant to each surface type relationship**

Evaluation

For cracking model relationships, refer to Chapter C2, Section 5.1 of Analytical Framework and Model Descriptions.

For each pavement section data record, the predicted all cracking initiation age should be calculated by the model relationship using a spreadsheet or the HDM Tools software (see Appendix K). The calibration adjustment factors and prediction errors should be determined separately for each surfacing-climate group from the mean predicted (mean PTCI) and observed initiation (mean OTCI) ages as follows:

$$K_{ci} = \frac{\text{mean OTCI}}{\text{mean PTCI}} \qquad \text{…(7.8)}$$

$$RMSE = SQRT\left\{mean\left[(OTCI_j - PTCI_j)^2\right]_{j=1,n}\right\} \qquad \text{…(7.9)}$$

7.3.3 Cracking progression factor: S-III

Cracking progression is predicted as a function of the area cracked and surfacing type in a sigmoidal function. The variance of progression data is typically very high so the required sample size is similar to cracking initiation though the sensitivity is lower. In general, a calibration derived from the initiation adjustment factor (similar to Level 1) is considered adequate without further field data collection, that is:

$$K_{cp} = \frac{1}{K_{ci}} \qquad \text{...(7.10)}$$

Alternatively, the following explicit field sampling based approach may be adopted:

Field sampling

First, stratify the sample by **surfacing type**. In most cases, further stratification by surfacing material subgroup and climate type (as for cracking initiation) is unlikely to add reliable differences, but may be done optionally.

Second, in each surfacing group, identify a minimum of 15 pavement sections (of 300 m lane length) with either **medium** surface distress (5-30 per cent area of cracking), or **high** surface distress (>30 per cent area of cracking).

Field measurements

On each identified cracking calibration pavement section, the following data should be determined and recorded:

1 **Surfacing age** (years)

2 **Percentage area of all cracking** (more than 1 mm width)

3 **Percentage area of wide cracking** (more than 3 mm width or spalled narrow cracks)

4 **Values of the explicit independent parameters required for predicting cracking initiation and progression**

Evaluation

For cracking model relationships, refer to Chapter C2, Section 5.1 of Analytical Framework and Model Descriptions.

For each pavement section data record, determine:

1 The predicted cracking initiation age, calculated by the model relationship using a spreadsheet or the HDM Tools software and adjusted by the calibration adjustment factors determined under Section 7.3.2

2 The estimated age since cracking initiation, calculated by subtracting the predicted cracking initiation age from the observed surfacing age

3 Fit a sigmoidal curve to the observed cracking area versus the estimated age since initiation data, and determine the estimated age at 30 per cent cracking area (ET30) by interpolation or extrapolation

4 Calculate the predicted age at 30 per cent cracking area (PT30) using the HDM-4 equation with coefficients appropriate for the pavement and surface type

5 Calculate the adjustment factor from the mean values of ET30 and PT30 across all calibration sections separately for each surface type.

$$K_{cp} = \frac{\text{mean PT30}}{\text{mean ET30}} \qquad \ldots(7.11)$$

7.3.4 Rut depth progression factor: S-II/S-III

The progression of mean rut depth is predicted by a single relationship for all flexible pavement types. For HDM-III, calibration is achieved just by comparing the observed and predicted mean rut depths for a range of pavements. For HDM-4, the prediction is determined from four components that can be calibrated separately. A Level 2 calibration focuses on the plastic deformation component and can be applied for either version of HDM as follows:

Field sampling

Identify a minimum of:

- **20 pavement sections -** (200 m lane-length) if the flexible pavements are predominantly of the thin surfacing type (less than 50 mm bituminous material), or
- **30 sections -** (about half thin and half thick) if thick asphalt pavements are common (more than 50 mm thickness of asphaltic layers).

At least 50 per cent of the sample should have medium to high mean rut depth (greater than 6 mm) and the range should include the highest level of rutting prevailing on the road network. In high standard road networks this sampling requirement may be difficult to meet, in which case the calibration of rut depth prediction is unlikely to be important.

Field measurements

On each identified section, the following data should be measured or determined:

1. **Mean and standard deviation of rut depth values -** measured in each wheelpath at 10 m intervals under a 2 m straight-edge (or computed from automated non-contact survey data for a 2 m straight-edge simulation); and
2. **The IQL-2 explanatory parameters -** required by the HDM-4 predictive relationships, for example, SNP, YE4, COMP, ACX, HS etc.

Evaluation

For rut depth model relationships, refer to Chapter C2, Section 10 of Analytical Framework and Model Descriptions.

For each calibration section *j*, compute the predicted mean rut depth (PRDMj) and the predicted standard deviation of rut depth. Calculate the adjustment factor for mean rut depth progression, by geometric means or from log values (LORDMj and LPRDMj) as follows:

$$\begin{aligned} K_{rp} &= \text{Geometric Mean [ORDMj] / Geometric Mean [PRDMj] or} \\ K_{rp} &= \text{[Sum (log ORDMj)] / [Sum(log PRDMj)]} \end{aligned} \qquad \ldots(7.12)$$

Alternatively, determine K_{rp} and the prediction error by linear regression of LORDMj v. LPRDMj.

7.3.5 General roughness progression factor: S-III

Since the roughness progression relationship predicts the change in roughness under the influence of several factors including distress, a Level 2 calibration of Kgp requires a time-series of at least four years of reliable roughness data on a wide range of pavement segments. If such historical data are already available the following guidance can be used to determine what adjustment should be made to Kgp if any. Once again, the user is advised that it is unusual for this general factor to need adjustment and it is more likely that one of the internal factors need correction; for example, the environmental coefficient (m or kge), structural parameters (SNP or YE4), or the patching and potholing coefficients. If adjustment is found to be necessary, then plans should be made to conduct the more rigorous analysis described for a Level 3 adaptation (see Section 7.4)

Step 1 Select 20 or more pavement segments (minimum 10 per pavement type), 1 km length, uniformly distributed in a matrix of age group (**young**, **medium**, **old**), and annual traffic loading (**light**, **medium**, **heavy**), for each pavement type (AMGB, STGB, etc. as appropriate)

Step 2 Using IQL-2 data from at least three consecutive applicable surveys spanning 4-5 years, process the data to determine:

The mean and incremental values of each condition parameter over the duration of the observed period (ACRX, ARAV, APOT, APAT, RDM, RDS, RI) and the mean values of the pavement and traffic parameters.

The incremental values should be determined preferably by linear regression between the first and last applicable survey (or alternatively simplified by the difference between the averages of the first and last pairs of values, adjusted to the full applicable period by extrapolation), and the mean values by arithmetic averaging, for example:

$$\Delta ORI_t = (EORI_{tn} - EORI_{t1}) \qquad \ldots(7.13)$$

where:

$EORI_{tn}$ is the regression estimate of observed roughness RI at time *n*, and *tn* is the last applicable time and *t1* is the first applicable survey time; or

$$\Delta ORI_t = \left\{ \frac{[AVG(ORI3, ORI4) - AVG(ORI1, ORI2)\ ](OT4 - OT1)}{[AVG(OT3, OT4) - AVG(OT1, OT2)]} \right\} \qquad \ldots(7.14)$$

where:

ORI4 is the observed roughness RI at the t=4th (or last) observation, and

OT4 is the corresponding time in years of the t=4 observation.

$$MORI = \frac{SUM(ORI1 : ORIn)}{n} \qquad \ldots(7.15)$$

where:

the notation is similar to the above for RI.

The incremental and mean values of the other parameters would be determined similarly.

Step 3 The unadjusted predicted value of incremental roughness (ΔPRI_t) should be calculated for each calibration segment using the primary prediction relationship (refer to Chapter C2, Section 11 of Analytical Framework and Model Descriptions) using a spreadsheet or the HDM Tools software, for example:

$$\Delta PRI_t = f\{MORI, SNP, YE4, \Delta OCX_t, \Delta ORDS_t\ \Delta OPOT, m, MOT, etc.\} \quad \ldots(7.16)$$

Step 4 Calculate the residual errors by differencing the observed and predicted values of incremental roughness for each calibration section *j*, for example:

$$RESRIj = \Delta(PRIj_t - \Delta ORIj_t,) \quad \ldots(7.17)$$

Determine correlation and slope (b) without intercept between RESRI and MORI. If the correlation and the determination of 'b' are significant, then determine the adjustment factor, K_{gp}, as follows:

$$K_{gp} = 1 + b \quad \ldots(7.18)$$

Check for other influences by examining the bivariate correlations of RESRIj with other key explanatory variables from the equation in Step 3; for example, MOCX, SNP, MORD, etc.

7.3.6 Ravelling initiation factor: S-IV

Ravelling initiation is predicted as the surfacing age when erosion of surface material becomes evident on at least 0.5 per cent of the carriageway area. Since this factor has relatively low impact on results except when disintegration and potholing are prevalent, the relationship would only be calibrated at Level 2:

1 As part of a comprehensive calibration study, or

2 If surface treatment is a prevalent pavement surface type; or

3 If ravelling is the prevalent distress for a surfacing material not specifically represented by the surface treatment relationship.

Since ravelling is evidence of a shortcoming in material design, specifications or construction, and is a function of binder oxidation, ravelling distress may be concentrated in certain districts, climatic zones or in projects of a certain period. If this tendency is evident, the field sampling should group similar problem areas together so those effects can be quantified, and keep other problem-free areas in a separate group.

In the HDM-III ravelling initiation model, this disparity of performance is captured by the Construction Quality parameter, CQ, which is set to CQ=0 for normal performance and CQ=1 for premature failures. In the HDM-4 ravelling initiation model, construction defects in the surfacing are captured by the parameter CDS (refer to Chapter C2, Section 4 of Analytical Framework and Model Descriptions).

Calibration may be performed on the combined groups if the CQ or CDS values have been assigned, or may be performed on the separate groups if specific, fractional values of CQ or CDS are to be determined by the calibration.

Field sampling

The sampling should include about 15 pavements in each stratum as follows:

- Samples should be stratified by:
  - surface type and material type; and

- problem and non-problem areas, defined as described above, where problem areas have been assigned a non-zero CQ value, or where separate adjustment factors are to be determined for each area.

- In each group, select a minimum of 15 pavement calibration sections, of 300 m lane length, either with:
  - low positive incidence of ravelling (0 < area < 10 %), or
  - nil incidence of ravelling if the surfacing age is six years or older (provided that this group does not exceed 20 per cent of the total sample).

Field measurements

On each calibration section, the following data should be measured or recorded, in accordance with the HDM-4 relationships:

- **Surface type**
- **Material type**
- **Surfacing age**
- **Annual vehicle axle passes** (YAX)
- **Assessed construction defects indicator for bituminous surfacings** (CDS)
- **Assessed ravelling retardation factor** (RRF)

Evaluation

For ravelling initiation model relationships, refer to Chapter C2, Section 6.1 of Analytical Framework and Model Descriptions.

For each calibration section, the observed time to ravelling initiation (OTRVj) for the groups should be set as shown below:

- **Low area of ravelling**

  Should be set equal to 10 per cent less than the surfacing age, that is, 0.9 AGES.

- **Nil ravelling** (OTRV)

  Should be set to 20 per cent more than the surfacing age, that is, 1.20 AGESj.

The predicted time to ravelling (PTRVj) should be determined for each section by applying the appropriate HDM-4 relationship and coefficients, using a spreadsheet or the worksheet in HDM Tools. The calibration factor for ravelling initiation is determined for each strata group (*g*) as follows:

$$K_{vi}(g) = \frac{\text{mean}[\text{OTRVj}(g)]}{\text{mean}[\text{PTRVPj}(g)]} \qquad \ldots(7.19)$$

HDM-4 also provides for a separate calibration factor for ravelling progression K_{vp}. It may be derived in a similar way to the initiation factor, or else set to equal the inverse of the initiation factor.

7.3.7 Potholing adjustment factor: S-III

In HDM-III, the rate of potholing progression is adjusted by the factor K_{ph}, but potholing initiation and the contribution of potholing to roughness are fixed. As mentioned in Section

7.2.7, potholing progression is highly variable and generally difficult to predict. A local calibration is particularly valuable when potholing is highly prevalent and severe in substantial portions of the road network, because potholing progression is dependent on the sensitivity of the base material to disintegration and moisture and is a function of construction and maintenance practices. Otherwise, if potholing is rare or quickly repaired by routine maintenance, a detailed calibration of potholing progression is not worthwhile and may be omitted.

In HDM-4, the conceptual approach to modelling of initiation and progression of potholes has been expanded from that of HDM-III, and allows for the separate internal adjustment of contributions to potholing from structural cracking, ravelling and enlargement of potholes (refer to Chapter C2, Section 7 of Analytical Framework and Model Descriptions).

The following procedure relates only to calibrating the prediction of potholes in HDM-III. Adjustment of any of the HDM-4 coefficients should not be attempted in a Level 2 calibration.

Field sampling

Select about 10 pavement sections of 100 lane-m area with incidence of potholing ranging from 10 to 500 pothole units (10 litre volume, about 0.1 m^2 area), that is, from 1 m^2 or 0.3 per cent, up to about 50 m^2 or 15 per cent of carriageway area.

Field measurements

On each calibration section, measure or estimate the IQL-2 parameters used in the HDM-III predictive relationship, that is, pavement surface thickness, annual average heavy axles, surfacing age (AGES), area of open potholes and estimation of number of pothole units.

Evaluation

For each calibration section, estimate the time for initiation of cracking (PTCI), the time for initiation of potholing (PTPI), and the time for progression of potholing to X units (PTPX) up to 500 potholing units. Compute the observed and predicted potholing times as follows:

$$OTPXj = AGES - PTPIj$$

$$PTPXj = PTPXj - PTPIj$$

Determine the potholing adjustment factor, either by linear regression of OTPXj against PTPXj, or as follows:

$$K_{ph} = \frac{mean(OTPXj)}{mean(PTPXj)}$$

7.4 Level 3 - Adaptation of model

A Level 3 study adopts a fundamental approach to the development of predictive models, for the purposes of:

1 Improving the accuracy of specific predictions;

2 Determining coefficients or new relationships for new distress types, specific materials or pavements not directly covered by the model; or

3 Enhancing the capability of the model to assess the influence of additional factors in existing relationships.

The Level 3 approach uses the framework of the model forms in the HDM-4 RDWE as the starting point for identifying either a new set of coefficients or also some modifications to the set of explanatory parameters in individual predictive relationships. In this way, the user is guided to make maximum utilisation of best practice and existing knowledge. However, users are not constrained by the existing model forms and may evaluate other potential model forms as they wish.

7.4.1 General design and methods of experimental study

The keys to successful development of predictive models that are robust and reliable lie in the adoption of formal experimental design principles and advanced statistical techniques. The strength of the HDM road deterioration and maintenance effects models and their ability to be transferable to many different countries and conditions is due to several fundamental characteristics as follows:

- **Factorial design**

  The main factorial matrix should comprise the primary explanatory variables and cover a range of values as close as possible to the range over which they will be used. The HDM-III and HDM-4 relationships, and the sensitivity classes assigned in Chapter 4, are the starting point to identify which parameters are primary and which secondary. When important interactions are to be determined, especially between parameters that may exhibit collinearity, there must be sufficient range for the cross-effects to be estimated. But to estimate the effect statistically it is necessary to have data with **high**, **normal** and **low** traffic loading on a given SNP, and **strong**, **normal** and **weak** pavements under a given loading YE4. It is not necessary to fill all cells of a factorial for all primary parameters; a partial factorial can be used to reduce the number of sections to be found. A summary of factorial designs for road deterioration studies is given in *Paterson (1987)* and *GEIPOT (1982)*, and design principles can be found in relevant textbooks.

- **Cross-sectional and longitudinal analyses**

  A cross-sectional analysis, or **slice-in-time** across pavements at differing stages of the life-cycle and differing factor values will only be valid when all relevant factors are combined in the correct model form. The experience of the HDMS study shows that complex interactions and even basic pavement performance models can only be achieved by a combination of longitudinal analysis, or **time-series**, of data from pavements monitored continuously over a period of four or more years, and cross-sectional analysis. The impact is illustrated in Figure 7.1, and pertinent examples are found in the discussion of roughness progression and rut depth progression models in *Paterson (1987)* - (Chapters 7 and 8).

- **Discrete events and analysis of censored data**

  For discrete events such as the initiation of cracking, ravelling or potholing, studies which are limited to only pavements showing the initial stages of distress can risk being biased if the set excludes pavements that out-perform the norm or that have premature failures. The application of maximum likelihood statistical estimation (MLE) enabled the HDM-III analyses to determine the concurrent effects of ageing and fatigue, and the MLE techniques have become readily accessible now in currently available advanced statistical software.

- **Analysis of residuals**

  Analysis of the residual errors of a statistical model should always be used to determine whether they correlate with any of the possible explanatory variables and to reveal any bias trends that may remain in the model. This can lead to the testing of further model formulations and improving their explanatory power.

- **Model form**

  The formulation of predictive models was derived in the first instance from:

  - The received knowledge of mechanistic and empirical research,
  - Selecting and clustering explanatory parameters on the basis of a mechanistic model where this was available, and
  - Adding other parameters in the forms in which they were expected to interact, for example, in multiplicative, additive, or power functions.

  Advanced statistical methods were used to test the validity of the relationships, derive the observed coefficients and determine the real significance of individual parameters or parameter clusters. Thus they represent **structured empirical** models, combining mechanistic and logical theory with empirical estimation. The resulting model forms are therefore sound starting points for further research. It is strongly advised that Level 3 adaptation **always** begin with a statistical re-estimation of the coefficients of the HDM relationship directly from the local data.

7.4.2 Distress initiation models

The field sampling procedure outlined in the Level 2 section (sections 7.3.2, 7.3.6 and 7.3.7) should be applied with two modifications, that is:

1. The sample sizes should be doubled or tripled to a minimum of 30 per surfacing type-material type combination; and
2. The sections selected for modelling of distress initiation should be monitored over a 4-6 year period to derive distress progression data on the same sections, so that the direct interactions between initiation and progression behaviour can be identified and characterised.

Two sets of models should be evaluated:

1. The structured model defined for HDM (that is, the HDM-4 road deterioration model for bituminous pavements described in Chapter C2 of Analytical Framework and Model Descriptions), specifying the same model parameters and estimating the coefficients from the local data; and
2. Variations on the structured model, including for example the mechanistic (strain-based) relationships tested in (*Paterson, 1987* - chapters 5 and 6) and others.

Some tips from previous research:

1. Structural and traffic loading variables must always be clustered interactively and not additively.

2 In relationships which use a log transform to represent an interaction by an additive function, care must be taken that the rate of trafficking (for example, ESA per year) preserves dimensional consistency in both the dependent and independent parts of the relationship

7.4.3 Distress progression models

To be included in a subsequent edition of this document

7.4.4 Rut depth progression models

To be included in a subsequent edition of this document

7.4.5 Roughness progression models

To be included in a subsequent edition of this document

7.5 Works effects on bituminous paved roads: S-II

In modelling the life cycle pavement deterioration it is vital that the effect of works on pavement condition be adequately modelled. These effects may vary between pavement types and construction practices in different countries. This is illustrated in Figure 7.1, which compares the effects of overlays on roughness compares models from HDM-III (regular paver), Thailand, Indonesia, and Sweden for an overlay thickness of 50 mm (*NDLI, 1995b*).

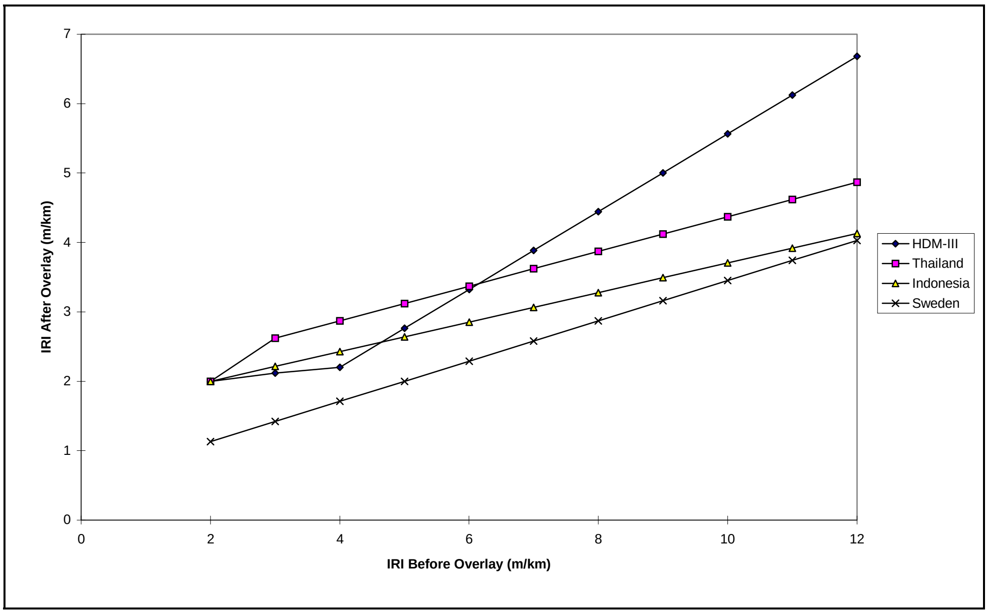


Figure 7.1 Effect of overlays on roughness (NDLI (1995b))

The effect of overlays on roughness is a critical parameter as it dictates the future deterioration rate and thus, maintenance activities. It is therefore an S-I parameter.

In HDM-III the overlay-roughness effects were either predicted using an internal relationship or the roughness after maintenance was specified on the C-Series data cards. Unfortunately, the relationship was **hard coded** in the model and could not be adapted without modifying the source code. In HDM-4 the coefficients of the relationship can be specified.

7.5.1 Level 1

A Level 1 calibration applies a constant value for the roughness after an overlay that is applied to all pavements receiving a particular treatment. The default values in HDM-4 may be used. However, since this is an S-II or S-I sensitivity, it is valuable to use some simple field measurement. A number of pavements which have recently been overlaid and for which the current roughness is available should be identified. On the basis of their roughness levels an average roughness after overlay is established. If different overlay thicknesses are used the sample can be stratified by overlay thickness and different values determined as a function of thickness.

7.5.2 Level 2

A Level 2 calibration conducts a small study into the effects of overlay on roughness and develops a local relationship for the effects of overlay on roughness. A series of sections scheduled for overlays should be selected. There should be at least five sections, with a minimum length of 500 m each. The roughness should be measured on each section. If a manual method is used (for example, Dipstick, Walking Profilometer) only a single run is required in each wheelpath; if using a vehicle a minimum of five runs is recommended.

In addition to measuring the roughnesses of the sections selected for maintenance, measurements should also be made on untreated sections on the same road near the maintenance sections. These provide **control** sections: since no maintenance is scheduled for them, and the time between surveys is short, it can be expected that there will be little, if any change in the roughness. They can therefore be used to correct the data for any roughness measurement instrument drift over time.

The before and after roughness data should be analysed to establish a general relationship between these roughnesses. If different overlay thicknesses are available the relationship should take this into account.

If possible, speed measurements should be made at the same time, as this will allow for the effects of roughness on speed to be established as well.

7.6 Concrete pavement deterioration

To be included in a subsequent edition of this document

7.7 Concrete pavement works effects

To be included in a subsequent edition of this document

7.8 Unpaved roads deterioration and works effects

To be included in a subsequent edition of this document

8 Summary and conclusions

To be included in a subsequent edition of this document

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Appendix A HDM data items and calibration factors

This appendix presents data items, model parameters and their calibration levels.

Table A.1
HDM data and model parameters and calibration levels

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
RUE	Aerodynamic drag coefficient		6.3.10	•		
	Aerodynamic drag coefficient multiplier		6.3.10	•		
	Average annual utilisation	km/year	6.3.4	•		
	Average service life	years	6.3.3	•		
	Base number of retreads – NR0			•		
	Desired speed	m/s	6.3.5	•		
	Engine speed – a0		6.3.12	•		
	Engine speed – a1		6.3.12	•		
	Engine speed – a2		6.3.12	•		
	Engine speed – a3		6.3.12	•		
	Engine speed - Idle		6.3.12	•		
	Equivalent standard axles	ESA/veh	6.3.1	•		
	Hours driven	h/yr		•		
	Number of axles		6.3.7	•		
	Number of wheels		6.3.7	•		
	Operating weight	t	6.3.1	•		

…Continued

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
RUE *cont.*	Optimal life depreciation parameters		6.3.9	•		
	Percentage of private use	%		•		
	Power - braking	kW	6.3.11	•		
	Power - driving	kW	6.3.6	•		
	Power - rated	kW	6.3.6	•		
	Projected frontal area	m^2	6.3.10	•		
	Travel on wet roads	%		•		
	Travel on snow covered roads	%		•		
	Tyre type		6.3.7	•		
	Utilisation method		6.3.4	•		
	Volume of wearable rubber	dm^3	6.3.8	•		
	Wheel diameter	m	6.3.7	•		
	Acceleration noise - max. alignment noise		6.4.5		•	
	Acceleration noise - max. driver noise		6.4.5		•	
	Acceleration noise - max. NMT noise		6.4.5		•	
	Acceleration noise - max. noise	m/s^2	6.4.5		•	
	Acceleration noise - max. roughness noise		6.4.5		•	
	Acceleration noise - traffic noise – a1		6.4.5		•	

…Continued

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
RUE *cont.*	Acceleration noise -traffic noise – a0		6.4.5		•	
	Fuel - fuel to power efficiency factor	ml/kW/s	6.4.4		•	
	Fuel - ldle fuel rate	ml/s	6.4.4		•	
	Labour model rotation – K0lh		6.4.6		•	
	Labour model translation – K1lh		6.4.6		•	
	Oil loss due to contamination	l/1000 km	6.4.11		•	
	Oil loss due to operation		6.4.11		•	
	Parts model rotation - K0pc		6.4.6		•	
	Parts model translation - K1pc		6.4.6		•	
	Speed flow - low flow interaction level - Qo	pcse/h	6.4.2		•	
	Speed flow - nominal capacity - Qnom	pcse/h	6.4.2		•	
	Speed flow - speed at nominal Capacity	km/h	6.4.2		•	
	Speed flow -speed at ultimate Capacity	km/h	6.4.2		•	
	Speed flow - ultimate capacity - Qult	pcse/h	6.4.2		•	
	Speed flow - vehicle equivalency factors	pcse/veh	6.4.3		•	
	Speed limit enforcement factor		6.4.1		•	
	Speed roughness - a0		6.4.1		•	

…Continued

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
RUE *cont.*	Speed roughness - maximum rectified velocity	mm/s	6.4.1		•	
	Speed width critical width factor - CW1		6.4.1		•	
	Speed width critical width factor - CW2		6.4.1		•	
	Speed width desired speed-width factor - a1		6.4.1		•	
	Speed width desired speed-width factor - a2		6.4.1		•	
	Drivetrain efficiency					•
	Labour model constant					•
	Labour model parts coefficient					•
	Parts model - congestion factor					•
	Parts model - constant					•
	Parts model - minimum roughness					•
	Parts model - roughness					•
	Parts model age term - kp					•
	Parts model - roughness exponent					•
	Parts - roughness shape					•
	Power - engine accessory power - Kpea					•
	Power - engine power - a0					•
	Power - engine power - a1					•

…Continued

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
RUE *cont.*	Power due to engine drag	%				•
	Rolling resistance - a0					•
	Rolling resistance - a1					•
	Rolling resistance - a2					•
	Rolling resistance factor - Kcr2					•
	Speed curve - a0					•
	Speed curve - a1					•
	Speed gradient - critical gradient length - a0					•
	Speed gradient - critical gradient length - a1					•
	Speed gradient - critical gradient length - a2					•
	Speed parameter - Beta					•
	Tyre model constant - C0TC	dm^3				•
	Tyre stiffness - a0					•
	Tyre stiffness - a1					•
	Tyre stiffness - a2					•
	Tyre stiffness factor - Kcs					•
	Tyre wear coefficient - CTCTE	dm^3/J-M				•

…Continued

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
RDWE	Altitude	m		•		
	Area potholed	%	Appendix D	•		
	Area with **all** cracking	%	Appendix D	•		
	Area with **wide** cracking	%	Appendix D	•		
	Average rainfall	m/month	Appendix D	•		
	Base type		Appendix D	•		
	Benkelman beam deflection	mm	Appendix D	•		
	Carriageway width	m	Appendix D	•		
	Construction age	yr	Appendix D	•		
	Effective number of lanes		Appendix D	•		
	Horizontal curvature	deg/km	Appendix D	•		
	Mean rut depth	mm	Appendix D	•		
	Number of surface layers		Appendix D	•		
	Posted speed limit		Appendix D	•		
	Preventative treatment age	yr	Appendix D	•		
	Rise plus fall	m/m	Appendix D	•		
	Roughness	IRI	Appendix D	•		
	Roughness age term		7.2.1	•		

…Continued

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
RDWE *cont.*	Roughness progression		7.2.5	•		
	Sand patch texture depth	mm	Appendix D	•		
	Shoulder width	m	Appendix D	•		
	Structural number		Appendix D	•		
	Subgrade CBR	%	Appendix D	•		
	Superelevation	%	Appendix D	•		
	Surface type		Appendix D	•		
	Surfacing age	yr	Appendix D	•		
	Unit costs for construction and maintenance		Appendix H	•		
	Area of previous **all** cracks	%	Appendix D		•	
	Area of Previous **wide** cracks	%	Appendix D		•	
	Construction fault		Appendix D		•	
	Crack initiation factor		7.2.2	•	•	
	Crack progression factor		7.2.3	•	•	
	Cracking retardation time	yr	Appendix D		•	
	Maximum acceleration noise	m/s^2	6.4.2		•	
	Number of base layers		Appendix D		•	
	Pothole progression		7.2.7	•	•	

…Continued

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
RDWE *cont.*	Ravelling initiation		7.2.6	•	•	
	Ravelling progression		7.2.6	•	•	
	Ravelling retardation factor	%	Appendix D		•	
	Rut depth progression		7.2.4	•	•	
	Soil cement resilient modulus	GPa	Appendix D		•	
	Standard deviation of rut depth	mm	Appendix D		•	
	Thickness of base layers	mm	Appendix D		•	
	Thickness of surface layers	mm	Appendix D		•	
TRAFFIC	Average annual daily traffic	veh/day	Appendix E	•		
	Traffic growth rate	%/year	Appendix E	•		
	Hourly distribution of traffic		Appendix E	•		
UNIT COSTS	Cost of cargo	cost/h	Appendix F	•		
	Cost of crew	cost/h	Appendix F	•		
	Cost of fuel	cost/l	Appendix F	•		
	Cost of maintenance labour	cost/h	Appendix F	•		
	Cost of oil	cost/l	Appendix F	•		
	Cost of overhead	cost/yr	Appendix F	•		
	Cost of retreaded tyre	%	Appendix F	•		

…Continued

Sub-model	Description	Units[1]	Discussed in section or appendix	Calibration level where local values should be established in place of defaults[4]		
				Level 1	Level 2	Level 3
UNIT COSTS	Cost of travel time	cost/h	Appendix F	•		
	Cost of tyre	cost/tyre	Appendix F	•		
	Cost of vehicle	cost/vehicle	Appendix F	•		
	Interest rate	%	Appendix F	•		
ECONOMIC	Discount rate	%	Appendix H	•		
	Analysis period	yr	Appendix H	•		

Appendix B Parameter values used in HDM studies

This appendix presents parameter values used in different HDM-III studies. It is from *Bennett (1995)*.

Table B.1
Vehicle tare weights

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
NDLI (1994a)	Barbados	0.60	0.70	1.10	0.70		1.40	3.50	9.20		2.70	4.80	7.27	
SWK (1993b)	Ethiopia	0.96			1.32	1.32		4.00	7.80	14.70	2.40		8.10	
GITEC (1992)	Guatemala		1.00		1.50				5.40	12.70		3.10	8.10	
RPT (1993)	India	0.60	1.20				2.20	8.10	10.00				4.00	
NDLI (1991)	Myanmar	1.00			1.60		1.60	2.00	4.20			3.00		
NDLI (1993)	Nepal	1.00			1.30			5.70				3.89		
Bennett (1989)	New Zealand	1.00	1.30		1.35	2.00	2.10	12.00	14.50	14.50		10.30	10.30	
Zukang et al. (1992)	P.R. Chna	1.00				1.80	1.60	4.10	8.40	7.20	1.40		7.20	
Kampsax - Beca (1990)	Papua New Guinea	0.90			1.40		2.60	5.50	10.00	15.30				
NDLI (1991)	Thailand	1.00			1.30			3.40	7.60		1.40	3.40	12.00	
NDLI (1994b)	Trinidad	1.00			1.50	1.50	3.00	6.00	10.00					
TRDF (1982)	USA	1.00	1.20	1.20	1.80	1.50	4.90	7.20	11.20					

Note: Tare weight in tonnes.

Table B.2 Vehicle operating weights

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	0.95	1.15	1.35	2.50		4.50	13.00	21.00	41.00			18.00	
Transroute (1992)	Bangladesh	1.10			2.10			10.40					9.20	
NDLI (1994a)	Barbados	0.75	1.00	1.30	1.00		2.05	7.00	16.60		4.70	6.80	9.67	
IBRD (1990)	Burundi	1.30			3.50	2.20		8.00	15.50	29.40	2.60			LGV is Jeep
GITEC (1992)	Guatemala		1.00		2.20				12.70	37.70		6.10	12.10	
SWK (1993b)	Ethiopia	1.40			2.30	2.30		8.40	14.80	29.30	4.40		16.50	
TSA (1995)	Hungary	1.16			1.16		7.00	7.00	7.00	39.70			13.40	
JBP (1990)	India	0.90	1.30		1.00	2.00		12.20	16.30	30.00			10.00	
RPT (1993)	India	0.90	1.50				6.00	16.30	26.00				10.00	
INDEC (1988)	Indonesia	1.60			2.20		5.50	9.50	14.50		5.50		10.80	
Sammour (1992)	Jordan			1.00				35.00					11.00	
World Bank (1995)	Jordan			1.00			35.00	45.00		50.00			11.00	
NDLI (1991)	Myanmar	1.20			2.10		2.90	5.00	8.20			7.90		
NDLI (1993)	Nepal	1.20			1.60			15.70					10.00	
Louis Berger (1990)	Nigeria		1.10			1.70		9.30	33.00				11.50	
Bennett (1989)	NZ	1.80	2.50		2.55	3.50	5.60	33.20	44.00					
Kampsax - Beca (1990)	Papua New Guinea	1.40			2.20		4.30	10.60	16.60	29.70				
Zukang et al. (1992)	P.R. Chna	1.4				3.00	2.80	6.50	13.40	23.20	1.70		7.50	
CESTRIN (1994)	Romania	1.36					6.27	10.40	18.60	39.70			12.10	
Kampsax - Beca (1990)	Papua New Guinea		1.40		1.70		3.70	8.60	14.6	26.00				
World Bank (1995)	Russia	0.85			2.00		5.00	15.00	20.0	35.00			11.00	
Estudio (1993)	Spain	1.10			1.90			12.70		29.70			11.10	
TSPC (1992)	Sri Lanka	1.10			2.20		4.80	8.30	14.00	15.00		4.80	8.20	
NDLI (1991)	Thailand	1.20			1.60			5.70	14.40		2.00	4.50	15.00	
NDLI (1994b)	Trinidad	1.20			2.50	2.30	5.00	12.50	25.00					
RPT (1990)	Uganda	1.45				2.00	2.45	10.60	26.00	39.60	2.55		12.50	
TRDF (1982)	USA	1.10	1.40	1.40	2.00	1.70	7.80	16.30	25.50					

Note: Operating weight in tonnes. It is defined as the sum of the tare weight plus load.

Table B.3
Projected frontal area

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	1.80	2.00	2.20	2.72		4.00	5.00	6.50	8.00			6.50	
Transroute (1992)	Bangladesh	1.80			2.72			5.20					5.80	
NDLI (1994a)	Barbados	1.80		2.20	2.72		3.25	5.20	5.20		3.25	3.25	6.30	
IBRD (1990)	Burundi	1.80			2.72	1.80		5.20	5.75	5.75	2.72			
GITEC (1992)	Guatemala		2.08		2.72				5.20	5.75		3.25	6.30	
TSA (1995)	Hungary	1.80			2.72		3.25	5.20	5.20	5.75			6.30	
JBP (1990)	India	1.90	2.00		2.72	4.42		5.20	5.20	7.22			6.30	
INDEC (1988)	Indonesia	1.80			2.72		3.25	5.20	5.20		3.25		6.30	
World Bank (1995)	Jordan			2.20			3.25	5.20		5.75			6.30	
NDLI (1991)	Myanmar	2.05			2.75		2.75	4.70	6.60			5.20		
NDLI (1993)	Nepal	2.05			2.75			8.00					5.37	
Louis Berger (1990)	Nigeria		2.08			2.72		5.20	5.75				6.30	
Bennett (1989)	NZ	2.72	2.72		2.72	2.72	3.25	5.75	5.75	2.08		6.30	6.30	
Zukang et al. (1992)	P.R. Chna	1.86				3.42	2.76	6.06	5.81	5.62	3.30		7.68	
CESTRIN (1994)	Romania	1.80					3.25	5.20	5.20	5.75			6.30	
TSPC (1992)	Sri Lanka	1.80			2.70		4.00	5.20	5.20	5.75		4.50	6.00	
NDLI (1991)	Thailand	2.05			2.75			4.70	6.60		3.25	7.25	8.50	
NDLI (1994b)	Trinidad	1.80			2.72	2.72	3.25	5.20	5.20					
TRDF (1982)	USA	1.92	2.05	2.05	2.62	2.87	3.60	5.32	8.90					

Note: Projected frontal area in m^2.

Table B.4
Aerodynamic drag coefficient

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	0.42	0.44	0.46	0.52		0.60	0.64	0.64	0.72			0.64	
Transroute (1992)	Bangladesh	0.45			0.55			0.85					0.70	
NDLI (1994a)	Barbados	0.45	0.46	0.45	0.46		0.70	0.85	0.85		0.70		0.65	
IBRD (1990)	Burundi	0.45			0.46	0.45		0.85	0.63	0.63	0.46			
GITEC (1992)	Guatemala		0.50		0.46				0.85	0.63		0.70	0.65	
TSA (1995)	Hungary	0.35			0.46		0.70	0.85	0.85	0.63			0.65	
JBP (1990)	India	0.45	0.45		0.46	0.52	0.58	0.85	0.85	0.58			0.65	
INDEC (1988)	Indonesia	0.40			0.46		0.70	0.85	0.85		0.70		0.80	
World Bank (1995)	Jordan			0.45			0.70	0.85		0.63			0.65	
NDLI (1991)	Myanmar	0.35			0.40		0.40	0.70	0.85			0.70		
NDLI (1993)	Nepal	0.35			0.40			0.85					0.65	
Louis Berger (1990)	Nigeria		0.50			0.46		0.85	0.63				0.65	
Bennett (1989)	NZ													
Zukang et al. (1992)	P.R. Chna	0.35				0.60	0.60	0.75	0.80	0.85	0.45		0.55	
CESTRIN (1994)	Romania	0.45					0.70	0.85	0.85	0.63			0.65	
TSPC (1992)	Sri Lanka	0.45			0.55		0.65	0.85	0.85	0.85		0.60	0.70	
NDLI (1991)	Thailand	0.35			0.40			0.70	0.70		0.70	0.70	0.70	
NDLI (1994b)	Trinidad	0.45			0.46	0.46	0.70	0.85						

Table B.5
Elasticity of vehicle utilisation

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	0.25	0.24	0.24	0.27		0.25	0.32	0.34	0.46			0.30	
NDLI (1994c)	Barbados	0.60	0.80		0.85	0.80	0.85	0.85	0.85			0.85	0.75	
SWK (1993a)	Botswana	0.10			0.15	0.20		0.35	0.40	0.50	0.35		0.40	
IBRD (1990)	Burundi	0.00			0.80	0.00		0.85	0.85	0.85	0.80			
GITEC (1992)	Guatemala		0.60		0.80				0.85	0.85		0.85	0.75	
SWK (1993b)	Ethiopia	0.10			0.20	0.20		0.20	0.30	0.40	0.20		0.30	
JBP (1990)	India	0.60	0.60		0.90	0.80		0.90	0.90	0.90			0.90	
INDEC (1988)	Indonesia	0.50			0.75		0.45	0.45	0.45		0.65		0.65	Sumatra
MRCU (1992)	Nepal	0.00			1.00			1.00					1.00	
Louis Berger (1990)	Nigeria		0.00			0.40		0.35	0.35				0.40	
CESTRIN (1994)	Romania	0.00					0.31	0.42	0.45	0.50			0.80	
TSPC (1992)	Sri Lanka	0.50			0.50		0.50	0.50	0.50	0.50		0.60	0.60	
Arup (1992)	Tanzania													
NDLI (1991)	Thailand	0.10			0.17			0.21	0.36		0.15	0.17	0.23	
NDLI (1994b)	Trinidad	0.45			0.80		0.85	0.85	0.85					

Table B.6
Number of hours driven per year

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	400	500	500	600		600	700	800	1500			700	
Transroute (1992)	Bangladesh	500			1100			1250					1750	
NDLI (1994a)	Barbados	500		3000	1400		1600	1200	1200		1200	3500	3400	
SWK (1993a)	Botswana	188			279	390		695	833	1040	732		805	
IBRD (1990)	Burundi	0			900	0		800	800	1800	1800			
GITEC (1992)	Guatemala		730		1095				1460	1825		1825	1825	
SWK (1993b)	Ethiopia	250			450	450		450	650	850	450		650	
TSA (1995)	Hungary	285			285		875	875	875	1500			1750	
JBP (1990)	India	417	417		500	1250	1429	1429	1700	1900			2286	
INDEC (1988)	Indonesia	450			1555		1820	1820	1830		3150		3150	Sumatra
Sammour (1992)	Jordan			4000				4000					4000	
World Bank (1995)	Jordan			2000			3000	3000		3000			2000	
NDLI (1991)	Myanmar				800		800	1440	1680			2400		
Louis Berger (1990)	Nigeria		500			1500		800	1500				2033	
Kampsax - Beca (1990)	Papua New Guinea	500			1200		2200	2200	2400	2400				
CESTRIN (1994)	Romania	400					740	1030	1140	1550			1580	
World Bank (1995)	Russia	230			825		825	825	1300	2400			2400	
TSPC (1992)	Sri Lanka	270			500		950	1100	800	700		950	1250	
NDLI (1991)	Thailand	354			462			615	1308		523	615	1077	
NDLI (1994b)	Trinidad	380			875	1250	875	875	980					
RPT (1990)	Uganda	579			824		1000	1200	1200	1450	1333		1765	
SWK (1993c)	Uganda	2000			2000			2000		2000			2000	

Table B.7
Vehicle service life

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	14	12	15	13		14	16	14	10			14	
Transroute (1992)	Bangladesh	10			8			10					8	
NDLI (1994a)	Barbados	6	6	5	6		4	10	12		7	12	12	
SWK (1993a)	Botswana	8			8	8		7.5	7.5	6	4		8	
IBRD (1990)	Burundi	8			5	10		10	10	10	3			
GITEC (1992)	Guatemala		10		10				10	10		10	10	
TSA (1995)	Hungary	13			13		10	10	10	13			10	
SWK (1993b)	Ethiopia	12			8	8		15	12	10	12		14	
JBP (1990)	India	15	15		12.5	8		15	8	8			10	
RPT (1993)	India	12	12				12	12	12				12	
INDEC (1988)	Indonesia	8			4		10	13	13		6		6	Sumatra
Sammour (1992)	Jordan			12				12					10	
World Bank (1995)	Jordan			12			12	12		12			10	
NDLI (1993)	Nepal	15			10			12					12	
Louis Berger (1990)	Nigeria		4			5		9	9				7	
CESTRIN (1994)	Romania	15					15	15	15	15			15	
Kampsax - Beca (1990)	Papua New Guinea	6			4		4	4	4	4				
World Bank (1995)	Russia	8			6		6	6	6	6			6	
Estudio (1993)	Spain	10			12			12		12			10	
TSPC (1992)	Sri Lanka	16			16		14	20	20	22		14	20	
NDLI (1991)	Thailand	10			8			12	12		8		12	
NDLI (1994b)	Trinidad	10			10	10	10	10	10					
RPT (1990)	Uganda	6			5		6	5	7	7	7		6	
SWK (1993c)	Uganda	6			6			5		7			6	

Notes: Service life in years.

Table B.8
Annual utilisation

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	15	17	17	20		21	23	31	116			50	
Transroute (1992)	Bangladesh	20			44			50					70	
NDLI (1994a)	Barbados	15		60	15		40	30	30		40	100	85	
SWK (1993a)	Botswana	18			24	30		50	60	75	60		66	
IBRD (1990)	Burundi	15			40	20		30	30	45	100			
NDLI (1994a)	Ethiopia	20			35	35		30	40	50	30		50	
GITEC (1992)	Guatemala		20		25				60	80		120	120	
TSA (1995)	Hungary	11.4			11.4		35	35	35	50			70	
RPT (1993)	India	20	16				40.2	50	80				80	
INDEC (1988)	Indonesia	23.7			70		80	80	75		125		150	Sumatra
Sammour (1992)	Jordan			50				50					50	
World Bank (1995)	Jordan			30			80	80		80			60	
NDLI (1991)	Myanmar	20			20		20	36	42			77		
NDLI (1993)	Nepal	20			37.5			60					43.5	
Louis Berger (1990)	Nigeria		50			54		50	77.6				122	
Kampsax - Beca (1990)	Papua New Guinea	15			30		40	50	70	85				
CESTRIN (1994)	Romania	15					42	55	58	66			70	
World Bank (1995)	Russia	7			18		24	24	40	70			80	
Estudio (1993)	Spain	15			30			50		80			60	
TSPC (1992)	Sri Lanka	15			26		40	45	35	30		45	65	
NDLI (1991)	Thailand	23			30			40	85		34		70	
NDLI (1994b)	Trinidad	17.3			35	35	35	35	35					
SWK (1993c)	Uganda	14			20			31		44			37.5	

Notes: Utilisation in '000 km/year.

Table B.9
Value for fuel consumption model adjustment factor - $\alpha 1$

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Transroute (1992)	Bangladesh	0.85			0.85			0.90					0.95	
NDL I (1994)	Barbados	1.00	1.00	1.00			1.00	1.00	1.00		1.00	1.00	1.00	
SWK (1993a)	Botswana	0.80			0.80	0.80		1.00	1.00	0.80	0.80		1.00	
IBRD (1990)	Burundi	1.00			1.00	1.00		1.00	1.00	1.00	1.00			
SWK (1993b)	Ethiopia	0.80			0.60	0.60		0.80	1.00	0.80	1.00		1.00	
GITEC (1992)	Guatemala		0.75		0.97				1.00	1.00		1.00	1.00	
TSA (1995)	Hungary	0.88			1.00		1.00	1.15	1.00	0.60			1.00	
JBP (1990)	India	0.70	1.00		1.00	0.60	0.70	0.76	0.60	0.60			0.87	
INDEC (1988)	Indonesia	0.90			0.90		0.90	0.90	0.90		0.90		0.90	
World Bank (1995)	Jordan			1.00			1.00	1.00		1.00			1.00	
Zukang et al. (1992)	P.R. Chna	1.00				0.59	0.59	1.00	0.77	0.98	0.86		1.00	
Kampsax - Beca (1990)	Papua New Guinea	0.90			0.95		0.95	0.95	0.95	0.95				
CESTRIN (1994)	Romania	1.00					1.00	1.00	1.00	1.00			1.00	
TSPC (1992)	Sri Lanka	0.85			0.85		0.90	0.90	0.90	0.90		0.90	0.90	
Arup (1992)	Tanzania	0.85				0.95		0.95	0.80				0.95	
NDLI (1994b)	Trinidad	0.90			1.00	1.00	1.00	1.00	1.00					

Table B.10
HDM-III fuel consumption model adjustment factor - $\alpha 2$

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Transroute (1992)	Bangladesh	1.16			1.16			1.15					1.15	
NDLI (1994a)	Barbados	1.16	1.16	1.16			1.15	1.15	1.15		1.15	1.15	1.15	
IBRD (1990)	Burundi	1.16			1.16	1.16		1.15	1.15	1.15	1.16			
GITEC (1992)	Guatemala		1.16		1.16				1.15	1.21		1.15	1.15	
JBP (1990)	India	1.15	1.20		1.20	1.15	1.15	1.15	1.15	1.15			1.15	
World Bank (1995)	Jordan			1.16			1.15	1.15		1.15			1.15	
MRCU (1992)	Nepal	1.50				1.50		1.5	1.5				1.5	
CESTRIN (1994)	Romania	1.16					1.15	1.15	1.15	1.15			1.15	
TSPC (1992)	Sri Lanka	1.16			1.16		1.15	1.15	1.15	1.15		1.15	1.15	
Arup (1992)	Tanzania					1.26		1.00	1.00				1.00	
NDLI (1991)	Thailand	1.15			1.15			1.15	1.15		1.15		1.15	
NDLI (1994b)	Trinidad	1.16			1.16	1.16	1.15	1.15	1.15					

Table B.11
Used driving power

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	45	55	70	67		75	110	170	290			210	
Transroute (1992)	Bangladesh	32			39			90					98	
NDLI (1994a)	Barbados	60	40	85	40		72	160	350		90	120	180	
SWK (1993a)	Botswana	51			47	51		90	130	220	51		90	
IBRD (1990)	Burundi	30			40	30		100	210	210	40			
SWK (1993b)	Ethiopia	30			60	50		80	80	210	80		150	
GITEC (1992)	Guatemala		70		85				130	230		100	170	
TSA (1995)	Hungary	60						100		210			100	
JBP (1990)	India	26	28.5		30	62			121	102			75	
RPT (1993)	India	43	39				92.5	69	135				65	
Sammour (1992)	Jordan			85				280					240	
World Bank (1995)	Jordan			85			60	100		210			100	
INDEC (1988)	Indonesia	50			50		75	115	120		50		75	
NDLI (1991)	Myanmar	49			45		45	57	184			199		
NDLI (1993)	Nepal	48			63			78					78	
Louis Berger (1990)	Nigeria		70			40		100	210				100	
Yuli (1996)	P.R. China	58					67.5	98.8	135.3				86.5	
Kampsax - Beca (1990)	Papua New Guinea	55			51		76	137	221	248				
CESTRIN (1994)	Romania	33					62	95	151	179			134	
Estudio (1993)	Spain	43			56			120		210			170	
TSPC (1992)	Sri Lanka	30			40		54	78	140	160		66	88	
Arup (1992)	Tanzania	30				60		120	250				180	
NDLI (1991)	Thailand	48			63			88	133		63		225	
NDLI (1994b)	Trinidad	54.6			64.2	48.3	77.4	90.0	205					

Notes: Used driving power in MPH.

Table B.12
Values for braking power

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Chamala (1993)	Australia	18	21	24	35		63	168	294	375			280	
Transroute (1992)	Bangladesh	23			35			180					160	
NDLI (1994a)	Barbados	17	30	27	30		100	250	250		100	100	160	
IBRD (1990)	Burundi	17			30	17		250	500	500	40			
GITEC (1992)	Guatemala		21		30				250	500		100	160	
TSA (1995)	Hungary	17						250		500			160	
JBP (1990)	India	13	19		15	84		177	236	336			145	
INDEC (1988)	Indonesia	25			35		110	250	270		110		180	
World Bank (1995)	Jordan			27			100	250		500			160	
NDLI (1991)	Myanmar	35			42		42	70	195			111		
NDLI (1993)	Nepal	36			43			171					129	
Louis Berger (1990)	Nigeria		17			30		160	300				160	
Yuli (1996)	P.R. China	32.5					96.6	237.3	355				175.3	
Kampsax - Beca (1990)	Papua New Guinea	27			46		80	198	311	558				
CESTRIN (1994)	Romania	20					90	150	260	260			170	
TSPC (1992)	Sri Lanka	22			39		62	180	350	500		82	165	
Arup (1992)	Tanzania	50				100		160	310				220	
NDLI (1991)	Thailand	36			43			112	294		36		175	
NDLI (1994b)	Trinidad	17			30	30	100	250	250					

Notes: Braking power in MPH.

Table B.13
Parts consumption model parameter - C0SP

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Transroute (1992)	Bangladesh	20.00			20.00			4.30					0.76	
GITEC (1992)	Guatemala		32.49		32.49				1.49	13.94		1.49	1.77	
TSA (1995)	Hungary	42.50						0.60		1.50			0.60	
JBP (1990)	India	15.20	2.20		2.20	0.70	0.70	0.70	5.90	8.50			1.00	
INDEC (1988)	Indonesia	25.04			25.04		1.08	1.08	4.71		1.08		1.34	
World Bank (1995)	Jordan			32.49			1.49	1.49		13.94			1.77	
Kampsax - Beca (1990)	Papua New Guinea	40.61			40.61		1.86	1.86	10.76					
Zukang et al. (1992)	P.R. Chna	12.95				1.87	1.87	1.87	5.52	5.52	12.95		1.87	
CESTRIN (1994)	Romania	32.49					1.49	1.49	8.61	13.94			1.77	
TSPC (1992)	Sri Lanka	25.00			6.30		5.50	4.50	6.00	6.50		1.50	0.90	
Arup (1992)	Tanzania	17.60				15.00		0.50	2.40				1.20	

Table B.14
HDM-III parts consumption model parameter - CSPQI

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
GITEC (1992)	Guatemala		13.70		13.70				251.8	15.65		251.8	1.77	
TSA (1995)	Hungary	13.70			13.70		251.8	251.8	251.8	15.65			3.55	
JBP (1990)	India	13.70	30.00		30.00		251.8		35.31	15.65			3.56	
INDEC (1988)	Indonesia	13.70			13.70		251.8	251.8	35.31		251.8		3.56	
World Bank (1995)	Jordan			13.70			251.8	251.8		15.65			3.56	
Zukang et al. (1992)	P.R. China	17.80				327.3	327.3	327.3	45.9	20.35	17.81		4.63	
CESTRIN (1994)	Romania	13.70					251.8	251.8	35.31	15.65			3.56	
TSPC (1992)	Sri Lanka	10.50			30.00		30.00	30.00	30.00	30.00		20.00	15.00	

Table B.15
Labour hours model parameter - C0LH

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Default		77.14			77.14		242.0	242.0	301.0		242.0		293.0	
Chamala (1993)	Australia	14.91	17.99	21.22	23.27		25.2	50.46	68.24	120.1			84.08	
Transroute (1992)	Bangladesh	77.14			77.14			300.0					200.0	
GITEC (1992)	Guatemala		77.14		77.14				242.0	652.5		242.0	293.4	
TSA (1995)	Hungary	77.14			77.17		242.0	242.0	242.0	652.5			193.4	
JBP (1990)	India	898.0	746.5		746.5	950.0		258.4	1000	1000			252.2	
INDEC (1988)	Indonesia	77.14			77.14		242.0	242.0	301.0		242.0		293.0	
World Bank (1995)	Jordan			77.14			242.0	242.0		652.5			293.4	
Kampsax - Beca (1990)	Papua New Guinea	68.65			68.65		215.4	215.4	301	242.0				
CESTRIN (1994)	Romania	77.14					242.0	242.0	301.5	652.5			293.4	
NDLI (1991)	Thailand	274.0			166.0			276.0	440.0		151.0		786.0	
TSPC (1992)	Sri Lanka	100.0			150.0		150.0	300.0	350.0	400.0		120.0	200.0	

Table B.16
Labour hours model parameter - CLHPC

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Transroute (1992)	Bangladesh	0.400			0.450			0.519					0.470	
GITEC (1992)	Guatemala		0.550		0.550				0.520	0.520		0.520	0.520	
TSA (1995)	Hungary	0.550			0.550		0.519	0.519	0.519	0.519			0.517	
JBP (1990)	India	0.547	0.700		0.700		0.520	0.520	0.439	0.397			0.820	
World Bank (1995)	Jordan			0.550			0.520	0.520		0.520			0.520	
CESTRIN (1994)	Romania	0.547					0.519	0.519	0.519	0.519			0.517	
TSPC (1992)	Sri Lanka	0.500			0.519		0.519	0.519	0.519	0.519		0.470	0.470	

Table B.17
Tyre consumption model parameter - C0TC

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
Transroute (1992)	Bangladesh							0.090					0.090	
NDLI (1994a)	Barbados						0.160	0.160	0.160		0.160	0.160	0.160	
TSA (1995)	Hungary						0.164	0.050	0.164	0.164			0.050	
GITEC (1992)	Guatemala								0.160	0.160		0.160	0.160	
JBP (1990)	India						0.160	0.160	0.190	0.080			0.160	
INDEC (1988)	Indonesia						0.164	0.164	0.164		0.164		0.164	
World Bank (1995)	Jordan						0.160	0.160		0.160			0.160	
Zukang et al. (1992)	P.R. Chna						0.068	0.068	0.134	0.134			0.068	
CESTRIN (1994)	Romania						0.164	0.164	0.164	0.164			0.164	
TSPC (1992)	Sri Lanka				0.164		0.164	0.164	0.164	0.164		0.164	0.164	
NDLI (1991)	Thailand							.1008	.0849				0.072	
NDLI (1994b)	Trinidad						0.160	0.160	0.160					

Table B.18
Tyre consumption model parameter - CTCTE (x 10^{-3})

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
NDLI (1994a)	Barbados						12.78		12.78		12.78	12.78	12.78	
GITEC (1992)	Guatemala								12.78	12.78		12.78	12.78	
TSA (1995)	Hungary						12.78	7.00	12.78	12.78			7.00	
JBP (1990)	India						12.78	12.78	9.00	9.00			12.78	
INDEC (1988)	Indonesia						12.78	12.78	12.78		12.78		12.78	
World Bank (1995)	Jordan						12.78	12.78		12.78			12.78	
Zukang et al. (1992)	P.R. Chna						5.30	5.30	10.40	10.40			5.30	
CESTRIN (1994)	Romania						12.78	12.78	12.78	12.78			12.78	
TSPC (1992)	Sri Lanka				12.78		12.78	12.78	12.78	12.78		12.78	12.78	
NDLI (1991)	Thailand							7.85	6.61				5.61	
NDLI (1994b)	Trinidad						12.78	12.78	12.78					

Table B.19
Volume of wearable rubber

Reference	Country	PC			LDV	LGV	LT	MT	HT	AT	LB	MB	HB	Comments
		S	M	L										
NDLI (1994a)	Barbados						4.30	7.60	7.30		4.30		6.85	
GITEC (1992)	Guatemala								7.60	8.39		4.30	6.85	
TSA (1995)	Hungary						4.30	15.00	7.60	15.0			15.0	
JBP (1990)	India						7.60	7.60	8.99	8.99			6.85	
INDEC (1988)	Indonesia						4.30	7.60	7.30		4.30		6.85	
World Bank (1995)	Jordan						4.30	7.60		8.39			6.85	
Zukang et al. (1992)	P.R. Chna						5.88	5.88	7.69	7.69			5.88	
CESTRIN (1994)	Romania						4.30	7.60	7.30	8.39			6.85	
TSPC (1992)	Sri Lanka				3.00		4.00	5.34	7.30	7.30		4.00	6.34	
NDLI (1991)	Thailand							12.16	14.30				13.80	
NDLI (1994b)	Trinidad						4.30	7.60	7.30					

Notes: Volume of wearable rubber in dm^3/tyre.

Table B.20 HDM-III RDWE deterioration factors used in different countries

Country	Pavement type	Pavement composition (thickness in mm)	Y/N	Deterioration factor							SN	CBR	Traffic AADT/ Million eqv std. axle	Rainfall m/month	Rough-ness m/km IRI	Mainten-ance policy	Analysis period Year	Reference
				Kci	Kcp	Kvi	Kpp	Krp	Kge	Kgp								
Bangladesh	Existing Pavement	Paved Road with Herring Bone Brick Subbase	N	0.50	2.00	1.00	10.00	1.00	1.30	1.10	NA	NA	NA	NA	NA	NA	NA	Transroute (1992)
Botswana	Existing Pavement	ST + Base, Crushed Rock/ Stab. Grav/ Natural Gravel	N	0.67	1.00	0.67	1.00	0.82-1.72	0.51	1.00	1.3-1.6	10-80	117-2438	NA	1786-3266 mm/km BI	NA	NA	SWK (1993a)
Brazil	Existing Pavement	Road Network Data	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	NA	NA	NA	NA	NA	NA	NA	Liautaud and Archondo-Callao (1994)
Chile	New Pavement	ST30 + Gran Base240	N	0.50	NA	1.00	1.95	0.70	NA	NA	13	Low	Med	NA	NA	NA	NA	Gaete et al. (1991)
	New Pavement	ST20 + Gran Base 290	N	NA	NA	NA	NA	1.16	1.74	NA	NA	25	Low	Heavy	NA	NA	NA	
	Overlay On Asphalt Pavement	Dense Surf 35-45 + Open Asph Base40-60+ Old Surf 60+ Gran155-170	N	0.67	0.67	NA	NA	0.80-1.06	0.43	3.00	NA	3-10	Medium	Dry	NA	NA	NA	
	Overlay On Asphalt Pavement	Dense Asph Surf 55-70+ Binder 70-100 + Dense Asph Base120-130 + Old Surf130 + Gran Base 130	N	0.58-0.67	NA	NA	NA	0.74-0.99	0.70	2.20	NA	4-13	Medium	Med	NA	NA	NA	
	Overlay On Asphalt Pavement	Dense Asph Surf60-100 + Dense/ Open + AsphBase115+ Conc185+Gran Base100-200	N	0.33-0.75	0.76	NA	NA	0.51-1.09	NA	NA	NA	8	Heavy	Med	NA	NA	NA	

…Continued

Country	Pavement type	Pavement composition (thickness in mm)	Y/N	Deterioration factor							SN	CBR	Traffic AADT/ Million eqv std. axle	Rainfall m/month	Rough-ness m/km IRI	Mainten-ance policy	Analysis period Year	Reference
				Kci	Kcp	Kvi	Kpp	Krp	Kge	Kgp								
	Overlay On Exist Pavement	Open Graded Cold Mix Surf55 + Old Surf90+ Gran Base 145	N	1.20	NA	1.20	NA	0.81	0.70	NA	NA	6	Medium	Med	NA	NA	NA	Gaete et al. (1991); Solminihac et al. (1989).
	New Pavement	Dense Asphalt Surf40 + Dense Asph Base95 + Gran. Base 320	N	0.83	NA	NA	0.51	NA	1.74	NA	NA	NA	Medium	Heavy	NA	NA	NA	
Guinea Bissau	Existing Pavement	Road Network Data	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	All	All	NA	NA	NA	NA	NA	Fonseca et al. (1991)
India	Reseal & Overlaid	Asph. Surfacing +Granular Base	N	1.00 1.00 1.00	1.25 1.50 2.00	0.70 0.60 0.50	1.25 1.50 2.00	1.25 1.50 1.75	1.00 1.50 2.00	1.00 1.15 1.30	NA	NA	NA	Low, Medium, High	NA	Patch + Routine	NA	CES (1989)
	Reseal & Overlaid	Asph. Surfacing +Granular Base	N	1.00	1.50	0.60	1.50	1.50	1.15	1.15	NA	NA	NA	NA	NA	NA	NA	JBP (1990)
	Reseal & Overlaid	Asph. Surfacing +Granular Base	N	1.50	1.50	1.00	1.50	1.50	1.00	1.50	2	5	1-8 msa/year	Medium	2500 mm/km BI	Patch + Routine	15	RITES (1994)
	Reseal & Overlaid	Asph. Surfacing +Granular Base	Y	1.00	1.00	1.00	1.00	1.00	0.70	1.00	2.0 , 3.0	NA	2 msa/year	Medium	2500 mm/km BI	Patch + Routine	20	NDLI (1994a)
Jordan	Existing	AC Over Granular Base	N	2.50	1.00	1.50	3.00	1.00	1.00	0.50	NA	13	NA	0.025 m/month	NA	NA	12	Sammour (1992)
	New	AC Over Granular Base	N	0.50	1.00	2.00	1.00	1.00	2.00	1.00	NA	15	2.5msa/year		NA	NA	NA	
Malaysia	Reseal & Overlaid	Existing & New Pavements	N	0.70-0.80	1.20-1.40	1.00	1.00	1.00	1.00-1.30	1.00	All	All	NA	NA	NA	Routine+ Patch+ Reseal & Overlay	NA	JKR (1991)
Morocco	Existing Pavement	Road Network Data	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	NA	NA	NA	NA	NA	NA	NA	CID (1993)

…Continued

Country	Pavement type	Pavement composition (thickness in mm)	Y/N	Deterioration factor							SN	CBR	Traffic AADT/ Million eqv std. axle	Rainfall m/month	Rough-ness m/km IRI	Mainten-ance policy	Analysis period Year	Reference
				Kci	Kcp	Kvi	Kpp	Krp	Kge	Kgp								
Mozambique	Existing Pavement	ST20 + Cement Stab. Base 150 + Select Soil 150	N	1.00	1.50	1.00	1.50	1.50	1.50	1.50	2.0-3.0	5	Low	Medium	4000 mm/km BI	Routine/NIL	20	RITES (1992)
Nepal	Existing Pavement	ST and AC over Granular Base	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	SNC= 3	NA	Low & Medium	Medium/ Heavy	10m/km IRI	Routine + Patch	NA	NDLI (1993)
New Zealand	Existing Pavement	Overlays & Reseals		NA	NA	NA	NA	NA	0.76	1.00	2-4	NA	500-6700, 0.01-0.09 msa/lane/ year	NA	NA	NA	7 years	Cenek and Patrick (1991)
Nigeria	Existing Pavement	Road Network Data	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	NA	NA	NA	NA	NA	NA	NA	Louis Berger (1990)
Pakistan	Existing Pavement	Road Network Data	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	NA	NA	NA	NA	NA	NA	NA	Riley et al. (1987)
Peru	Existing Pavement	Road Network Data	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	NA	NA	NA	NA	NA	NA	NA	Liautaud and Archondo-Callao (1994)
South Africa	New Pavement	Surface Treatment	N	1.00-1.50 (avg. 1.21)	0.10-0.30 (avg. 021)	NA	NA	1.50-1.75 (avg. 1.57)	0.392(1) 0.607(2) 0.886(3)	0.8-1.2(4) 0.6-1.4(5) 0.8-1.2(6)	NA	NA	NA	NA	NA	Reseal/ Overlay @ 8 y	20 (1975-1995)	Kannemeyer and Visser (1994)
	Existing Pavement	Overlays & Reseals	N	0.40-0.80 (avg. 0.63)	0.30-0.70 (avg. 0.21)	NA	NA	1.00	0.392(1) 0.607(2) 0.886(3)	0.8-1.2(4) 0.6-1.4(5) 0.8-1.2(6)	NA	NA	NA	NA	NA	Reseal/ Overlay @ 8 y	20 (1975-1995)	
Spain	Existing Pavement	AC and ST over Granular Base	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	3.1-4.7	NA	NA	0.045 m/ month	2253-3352 mm/km BI	NA	15 years	Estudio (1993)

…Continued

Country	Pavement type	Pavement composition (thickness in mm)	Y/N	Deterioration factor							SN	CBR	Traffic AADT/ Million eqv std. axle	Rainfall m/month	Rough-ness m/km IRI	Mainten-ance policy	Analysis period Year	Reference
				Kci	Kcp	Kvi	Kpp	Krp	Kge	Kgp								
Tanzania	Existing Pavement	SSD/DSD + Lime / Cement Stab. Base	N	0.50-1.30	0.50-1.65	1.00	1.00	1.50-2.10	0.20-0.30	0.60-1.30	SNC= 2.3-3.0	NA	ADT (1991)= 85-266	0.09 m/ month	2253-3352 mm/km BI	Reseals as per Actually Applied	Year off Constr.-1992	Arup (1993)
Tunisia	Existing Pavement	Road Network Data	Y	1.00	1.00	1.00	1.00	1.00	1.00	1.00	NA	NA	NA	NA	NA	NA	NA	Houcine (1982)
UK	Existing Pavement	AC60+Old Surf 50+ Cement Stab. Base 120		1.00	1.00	NA	NA	NA	NA	NA		80	5535-7057	0.03 m/month	1500 mm/km BI	Patch + Routine	10 (1974-1984)	Wyley et al. (1986)
	Reseal on AC	SD10+Old Surf 90 + Gran Base150		NA	NA	NA	NA	3.00	NA	NA	2.3	80	5535-7057	0.03 m/month	1500 mm/km BI	Cold Mix 20 @ 5Y + Patch + Routine	10 (1974-1984)	
	Surface Treatment	ST30 +Old Surf. 20+Granular Base 205		NA	NA	NA	NA	2.70	NA	NA	1.5	8	2892-3659	0.03 m/month	1500 mm/km BI	AC30@ 15/ 7Y +Patch +Routine	10 (1974-1984)	

Appendix C Road user data

The road user data were discussed in Chapter 6.

Appendix D Road and pavement data

D.1 Introduction

The road and pavement data required for HDM consists of general road data, pavement characteristics, pavement condition and the maintenance alternatives. Each of these data are discussed in the following sections after a description of how one classifies pavements.

D.2 Pavement classifications

The HDM pavement deterioration model contains different pavement deterioration models for different types of pavements. It is therefore necessary to establish a pavement classification system at the onset of the HDM-III analysis.

HDM-III considered the following pavement types:

- **Surface treatment**
- **Asphalt concrete**
- **Slurry seal on surface treatment**
- **Reseal on surface treatment**
- **Reseal on asphalt concrete**
- **Open graded cold mix surfacing**
- **Asphalt overlay or slurry seal on asphalt concrete**

For HDM-4 the pavement classification system is much more comprehensive to reflect the wider range of pavement types modelled.

Table D.1 lists the HDM-4 bituminous pavements classification system and Table D.2 defines the pavement codes (ISOHDM, 1997). In addition to these there are concrete, block and unsealed pavements in HDM-4.

Table D.1
HDM-4 bituminous pavement 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	CS	STGB
	DBSD		GM	
	SBSD	AB	AB	STAB
	SL	SB	CS	STSB
	PM		LS	
		AP	TNA	STAP
			FDA	

Source: *ISOHDM (1997)*

Note:

1 AM and ST surfacings on concrete pavements, that is, AMCP & STCP, are modelled in HDM-4 as concrete pavement types in the rigid pavements sub-model.

Table D.2
Definitions of HDM-4 pavement codes

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

The available HDM-4 models are based on different factors. Many are based on surface and base type, while some are based on surface material. Accordingly, the modelling is initially done in terms of surface material and base type, even though base materials can be specified. HDM-4 has default coefficients for the bituminous pavement types given in Table D.3.

Table D.3
Generic HDM-4 bituminous pavement types

Pavement type	Surface type	Base type	Description
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

While it is straightforward to classify new pavements as surface treatment or asphalt concrete, once the pavement has received a maintenance treatment, the classification becomes very complicated.

Without adequate maintenance records the only way of classifying a pavement into one of the post-maintenance categories is by conducting field surveys and gathering data on a sample of pavements.

D.3 General road data

HDM-III requires the user to specify the following pavement geometric characteristics:

- **Rise and fall** (m/km)
- **Horizontal curvature** (degrees/km)
- **Carriageway width** (m)
- **Shoulder width** (m)
- **Superelevation** (m/m)

These data are usually straightforward to obtain. The first two data are available from topographic maps, highway information sheets, or route surveys. For the carriageway and shoulder widths a sample of data should be used to estimate average or median values. These are often a function of traffic volume so it may be useful to stratify the data as a function of volume. The superelevation can either be supplied, or it can be calculated in HDM-III as a function of curvature using the following equation developed from the Brazil data (*Watanatada et al., 1987a*):

$$e = a0\,C \qquad \ldots(D.1)$$

where:

e is the superelevation (m/m)

C is the curvature (degrees/km)

a0 is a constant (0.012 for paved roads; 0.017 for unpaved)

Another value termed **the effective number of lanes** is used in HDM-III. This is used to reflect the practice of traffic on narrow roads to travel towards the middle of the pavement, thereby increasing pavement deterioration. *Watanatada et al. (1987a)* give the following recommended values:

Width (m)	Number of effective lanes
< 4.5	1.0
4.5 – 6.0	1.5
6.0 – 8.0	2.0
8.0 – 11.0	3.0
> 11.0	4.0

It should be noted that in places where there is poor lane discipline where vehicles do not tend to drive in set wheelpaths, such as India, it may be appropriate to increase the number of effective lanes above these recommended values due to the reduced levels of effective traffic loading.

D.4 Pavement characteristics

D.4.1 Strength: modified structural number and deflection

HDM-III requires the pavement strength to be calculated in terms of the modified structural number (SNC) and Benkelman beam deflection. In HDM-4 the deflection from a falling weight deflectometer (FWD) can be supplied.

Ideally, data on both these characteristics should be supplied to HDM, however the SNC is often difficult to accurately quantify.

The SNC is the structural number of the pavement increased to reflect the contribution of the subgrade. When the pavement layer history is available, it is possible to calculate the SNC from the thicknesses of the individual layers using appropriate strength coefficients from *Watanatada et al. (1987a)*. It is also necessary to supply an estimate of the **in situ** subgrade CBR for calculating the subgrade contribution. However, it should be noted that these coefficients are based on laboratory tests of materials whereas the SNC should reflect their actual performance in the field.

In instances where the pavement layer history is unavailable, the SNC can be calculated from the Benkelman beam deflection using the following equations (*Watanatada et al., 1987a*):

$$SNC = 3.2\,DEF^{-0.63} \qquad \text{If base not cemented} \qquad \ldots(D.2)$$

$$SNC = 2.2\,DEF^{-0.63} \qquad \text{If base cemented} \qquad \ldots(D.3)$$

Watanatada et al. (1987a) provide similar equations for calculating the deflection from the SNC. *Paterson (1987)* who used an orthogonal regression to ensure that the equations give consistent predictions when going to/from either attribute developed these equations. Appendix J describes orthogonal regression.

NDLI (1995b) describe several techniques for estimating SNC from FWD deflection and these have been adopted for HDM-4.

If the existing pavements are strong, SNC of 5.0 or greater, there is not a great need to accurately specify this parameter. This is because there are limited differences in the predictions of the HDM-III pavement deterioration model with strong pavements. Thus, it is only when there are weak pavements that the SNC is a critical parameter.

D.4.2 Pavement history: age of surfacing and layers

For each pavement type it is necessary to supply the pavement history in terms of the age of the last surfacing, the number of layers and their thicknesses.

Many organisations have maintenance histories so these data can often be used to provide the data. Where these are unavailable, it is often necessary to undertake field surveys.

Pavement age can be estimated from discussions with maintenance staff. Another good source is local residents adjacent to the roads, particularly those in the hospitality industry. They often recall when a road was improved due to the increased level of business associated with the works.

The pavement layer information is more difficult. When there are set standards it is often possible to assume a standard surface thickness based on an estimated average age and the number of years between typical maintenance treatments.

D.4.3 Drainage and environmental factors

The HDM-III pavement deterioration model is predicated on the assumption that there is good drainage of the pavements. Since in practice this is not always the case, it is necessary to adjust the model to reflect poor drainage. Similarly, the model must be adjusted to reflect different environmental conditions. The techniques for doing this are discussed in Section 4.1.6 of *Watanatada et al. (1987a)*.

D.5 Pavement condition

D.5.1 Roughness

The pavement roughness is a very important parameter. It is not only a measure of the condition of the pavement but it also has a major impact on the RUE.

When measuring sections of road, roughness is usually measured using a response-type road roughness measuring system (RTRRMS) or a non-contact profilometer (laser/accelerometer). For short sections or studies manual methods such as a rod-and-level survey, dipstick or walking profilometer may be used.

RTRRMS consist of an instrument mounted on the floor of a vehicle that is connected to the rear axle. The instrument records the displacement of the vehicle body relative to the axle and usually expresses it in terms of a vertical displacement per unit distance travelled, such as mm/km. The TRRL Bump Integrator is probably the most widely used RTRRMS.

Profilometers usually employ sensors to measure the elevation of the vehicle to the pavement. Accelerometers are double integrated to get the movement of the vehicle through space. The difference between these values gives the road profile elevation. These data are processed to obtain the roughness.

Since the response of a vehicle to road roughness will depend upon the characteristics of the suspension system, tyres and other factors, it is necessary to calibrate the vehicle against a standard reference roughness. The International Roughness Index (IRI) was developed by the World Bank (*Sayers et al., 1986*) and is based on a complex simulation of a vehicle suspension. HDM-III used QI but since then IRI has become the international standard and all roughnesses should be expressed in terms of IRI m/km. The conversion between QI and IRI is 1 QI = 13 IRI m/km.

In order to calculate the IRI it is necessary to obtain an accurate profile of the pavement surface on a number of test sections. From this profile, one calculates the IRI. The vehicle is then operated over these sections and a regression equation is developed between the roughness meter readings and the reference roughness. This constitutes roughness meter calibration.

Once the vehicle has been calibrated, it is necessary to measure the roughness of the pavements. There are several points that should be considered in undertaking these measurements:

- **Sampling interval**

  It is best to record the roughness at the shortest practical interval - if possible, every 100 m. When longer lengths such as 1 km are used, short sections of poor pavements will be disguised by other sections in better condition.

- **Measurement speed**

  The measurements should be made at a single standard speed. Where data loggers are available and roughnesses are low, this can be upwards of 80 km/hr. If different speeds are to be used, it is necessary to have calibration equations for each speed.

- **Vehicle load**

  The vehicle should be calibrated and operated at standard loads. In addition, the tyre pressures should also be standard.

D.5.2 Surface distress

HDM calls for the pavement condition to be specified in terms of the areas of all cracks, wide (> 3 mm) cracks, ravelled, and potholed. It also requires the mean and standard deviation of rut depth. It is important that the values input for these characteristics are based at least somewhat on field surveys.

As with roughness, it is recommended that these parameters are measured at short intervals of 200 m. If longer intervals such as 1 km are used, it will often be found that there is at least some cracking within the interval, even if it confined to a short section. In the HDM-III analysis the entire section will be treated as cracked which will lead to markedly different results than if the analysis were conducted, for example, with 80 per cent not cracked and only 20 per cent cracked.

D.5.3 Rutting

To be included in a subsequent edition of this document

D.5.4 Potholes

To be included in a subsequent edition of this document

D.6 Maintenance alternatives

Table D.4 illustrates the range of maintenance alternatives that were adopted in a study in India (*NDLI, 1997*). There are several points to note with this table.

- A range of treatments from overlays to reconstruction were adopted. This allowed for the analysis to establish the mix of treatments given the available budget.
- In keeping with the pavement design standards, the treatments are a function of increasing traffic loading.
- For new construction, there is a constant SNC for a given level of traffic. This reflects pavement design standards wherein there is a given strength required for each level of traffic loading. If the strengths were not constant, there would be different performances between the pavements being tested which could distort the results.

Table D.4
Example of maintenance treatments

Cumulative loading (ESA x 10^6)	Pavement structure	Existing pavement				New construction				
		Recon-struction	Rehab-ilitation	Medium overlay	Thin overlay	CBR < 3	CBR 3-6	CBR 7-11	CBR 12-19	CBR ≥ 20
<25	Asphalt (BC)	70	70	40	45	30	30	30	30	30
	Asphalt (DBM)	0	0	0	0	40	40	40	40	40
	Asphalt Levelling Course	0	0	30	20	0	0	0	0	0
	Base	200	200	0	0	200	200	200	200	200
	Sub-Base	200	0	0	0	250	300	240	150	100
	Granular Levelling Course	75	50	0	0	0	0	0	0	0
	Select Subgrade	0	0	0	0	375	100	0	0	0
	Total	545	320	70	65	895	670	510	420	370
	SN (Existing) or SNC (New)	3.0	1.9	0.9	0.4	4.5	4.5	4.5	4.5	4.5
25 to 80	Asphalt (BC)	30	30	40	40	30	30	30	30	30
	Asphalt (DBM)	40	40	70	20	70	70	70	70	70
	Asphalt Levelling Course	0	0	30	20	0	0	0	0	0
	Base	200	200	0	0	225	225	225	225	225
	Sub-Base	200	0	0	0	300	300	300	350	275
	Granular Levelling Course	75	50	0	0	0	0	0	0	0
	Select Subgrade	0	0	0	0	550	350	175	0	0
	Total	545	320	140	80	1175	975	800	675	600
	SN (Existing) or SNC (New)	3.2	2.1	1.1	0.6	6.0	6.0	6.0	6.1	5.9
> 80	Asphalt (BC)	40	40	40	40	30	30	30	30	30
	Asphalt (DBM)	60	60	100	50	70	70	70	70	70
	Asphalt Levelling Course	0	0	30	20	0	0	0	0	0
	Base	200	200	0	0	250	250	250	250	250
	Sub-Base	200	0	0	0	300	300	300	300	325
	Granular Levelling Course	75	50	0	0	0	0	0	0	0
	Select Subgrade	0	0	0	0	625	400	225	100	0
	Total	575	350	170	110	1275	1050	875	750	675
	SN (Existing) or SNC (New)	3.5	2.4	1.4	0.9	6.9	6.4	6.4	6.4	6.4

Source: *NDLI (1997)*

D.7 Homogeneous sections

All economic analyses use **homogeneous** sections. These are sections of the network where the road characteristics are consistent. There are two approaches to defining sections:

- **Fixed**
- **Dynamic**

Fixed sections are defined at a regular interval, for example between km stones. Dynamic sections are defined on the basis of the data attributes.

When characterising the condition of a road, each measurement has its own appropriate sampling interval:

- **Roughness -** may be measured at intervals of, say, 100 m,
- **Deflection -** at intervals of 50 m,
- **Condition to a longer section -** such as 1000 m.

When using fixed links it is necessary to aggregate the data over the arbitrary link length, even if the interval is not the best for the data items being measured.

Dynamic links avoid this by creating links based on the condition (or any other parameter such as traffic) of the pavement. This leads to a much more realistic representation of the network since the treatments one applies will be based on the pavement condition. The process by which these links are created is termed automatic or **dynamic sectioning**.

Dynamic sectioning is achieved by defining the allowable variation in an attribute, for example roughness ± 1.0 IRI m/km; width ± 0.5 m. The condition data are then analysed and when the condition deviates from these variations a new section is created. This is illustrated in Figure D.1 for a hypothetical road section.

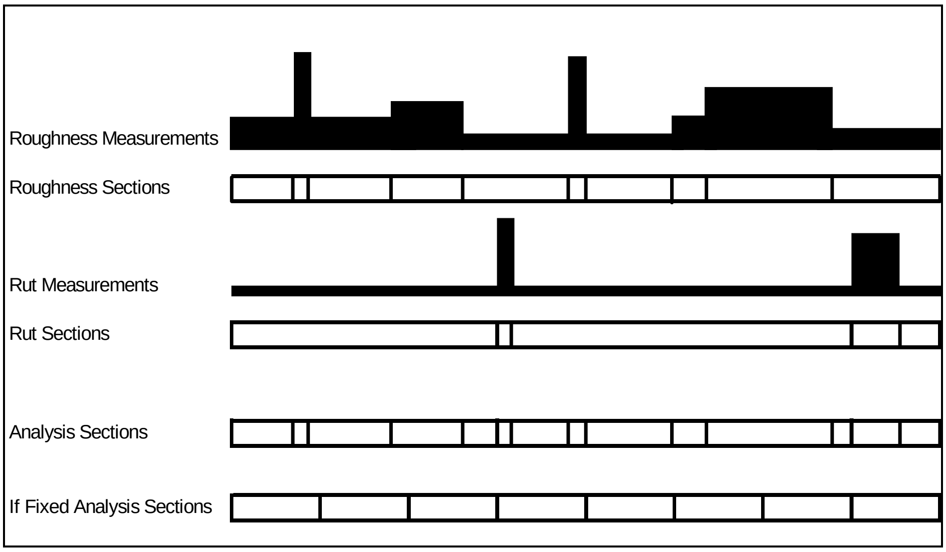

Figure D.1 Fixed versus dynamic sections

It is not unusual for fixed section systems to lead to inappropriate treatments due to the averaging of characteristics over the section disguising short, poor sections. This is illustrated in Figure D.1 where the fixed sections would result in fundamentally different treatments being applied to the dynamic analysis sections.

One can establish homogeneous sections based on a range of attributes. Attributes commonly used to establish homogeneous sections include:

- **Roughness**
- **Strength**
- **Width**
- **Traffic volume**
- **Environment**

The sectioning process can be visualised as **driving** along a road. Whenever an attribute changed by a certain amount a new section would be created. For example, a:

- **Roughness -** change of more than *x* IRI m/km from the mean roughness of the section to that point
- **Strength -** change of more than *y* from the mean strength of the section to that point
- **Width -** change of more than *z* m from the mean width of the section to that point
- **Change in the AADT on the section**
- **Change from urban to rural, or vice versa**
- **Change from normal to black cotton soil, or vice versa**

Having established the homogeneous sections it is necessary to prepare a single data file for use in the HDM modelling. This is done using **dynamic segmentation** and is illustrated in Figure D.2. Depending upon the data attribute, one will take the average, weighted average, maximum, minimum, or dominant feature.

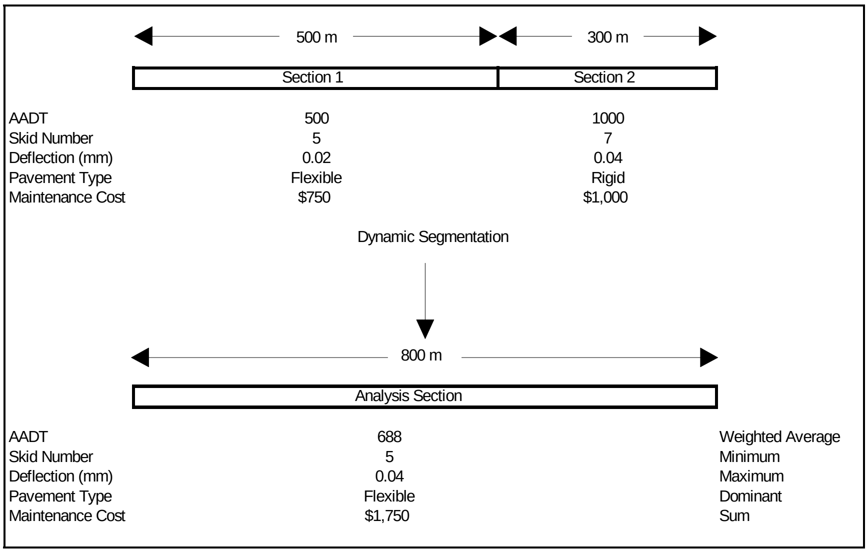

Figure D.2 Example of dynamic segmentation

Appendix E Traffic data

E.1 Introduction

Traffic volume is a critical input data item for all HDM analyses. Since the total transport costs are dominated by road user costs which are proportional to the traffic volume. In addition, the commercial vehicle volumes influence the rate of pavement deterioration. This appendix describes traffic volumes, growth rates and other associated issues.

E.2 Traffic volume and composition

The traffic flow is the number of vehicles passing a point in time:

$$q = \frac{n}{t} \quad \text{...(E.1)}$$

where:

q	is the traffic flow in vehicles during time *t*
n	is the number of vehicles
t	is the time interval *t*

Annual traffic figures are expressed in veh/day. There are two important measures, depending on how the data are sampled:

- **Average Annual Daily Traffic (AADT)**
- **Passenger Car Units (PCU)**

Average Annual Daily Traffic (AADT) is the average number of vehicles on a road over the year:

$$AADT = \frac{\text{Annual Traffic}}{365} \quad \text{...(E.2)}$$

This can only be calculated at locations where there is continuous traffic counting.

Since it is more common to conduct short-term traffic counts, most data are termed Average Daily Traffic (ADT). This is dependent upon the length of the traffic count and it is important to specify what it refers to; for example:

- **5 - day ADT** - is used for urban streets
- **7 - day ADT** - is used for rural highways

The base traffic data supplied to HDM must be an AADT. ADT are converted to AADT using correction factors. These are often based on the time of year when the traffic is counted. If, for example, the count is made in winter or during the monsoon season the factor usually increases the ADT to convert it to an AADT. Conversely, summer or dry season counts are often reduced to make them an AADT. Table E.1 shows seasonal correction factors from

India (NDLI, 1997). The data in this table indicate, for example, that an ADT from a count made at Location 1 in August would be multiplied by 1.13 to convert it to an AADT.

Hourly traffic flows are expressed in veh/h. This is the most common measure to use in traffic evaluations. HDM uses hourly flows in its congestion analyses.

The traffic stream is comprised of a number of different vehicle types. It is therefore common to convert these to more homogeneous measures.

Table E.1
Example of seasonal correction factors from India

Month	Seasonal correction factor by month		
	Location 1	Location 2	Location 3
January	0.94	0.95	0.91
February	1.00	0.97	0.98
March	0.97	0.90	0.87
April	0.99	1.00	0.97
May	0.96	0.97	0.94
June	1.02	1.17	1.08
July	1.11	1.14	1.14
August	1.13	1.17	1.09
September	1.09	1.14	1.19
October	0.99	1.01	0.99
November	0.98	1.03	1.06
December	0.86	0.84	0.90

Source: *NDLI (1997)*

Passenger Car Units (PCU) is a measure that converts all vehicles to equivalent passenger cars. These are most commonly applied on rural road appraisals where the larger vehicles have a significant negative impact on traffic flow. The values for PCU conversions vary between countries and depend upon factors such as the vehicle size and power to weight ratio. Recently, the measure Bicycle Space Equivalent (BSE) has been proposed as a measure for use in countries such as China and India where there are high levels of non-motorised traffic (*Yuli, 1996).*

In HDM, traffic is expressed in terms of Passenger Car Space Equivalents (PCSE). The PCSE differs from the PCU in that it is based on the area occupied by the vehicle[1]. A discussion of PCSE and how they are calculated was given in Section 6.4.3.

It is important to consider the traffic composition as well as the total volume since different vehicle classes have different operating costs and impacts on pavements. The traffic stream is modelled in HDM using representative vehicles and so the default HDM representative vehicles should be used as a guide when considering the traffic survey classes. Issues such as

1 PCSE are primarily based on length. However, as described in Section 6.4.3, *Hoban et al. (1994)* recommend increased values of PCSE for narrower roads to reflect the greater effects of the vehicles on the total road space under these conditions.

the likely operating cost differences between vehicle classes, occupancies, loading levels, etc. all should be assessed. As a minimum, the following are the recommended vehicle classes for HDM analyses:

- **Non-motorised traffic**
- **Motorcycles**
- **Passenger cars**
- **Light trucks**
- **Medium/heavy trucks**
- **Mini-buses**
- **Heavy buses**

E.3 Hourly distribution of traffic volume

Over the space of a year most roads undergo different hourly flows. Some roads that are congested during peak hours have greatly reduced flows at night-time. Other roads, such as interurban routes in India, experience major flows at night when trucks prefer to travel.

These variations are considered in HDM through the provision of an hourly distribution of traffic volume. This consists of the number of hours per year that the flow is at different levels. As shown in Figure E.1, the distributions are markedly different between countries and so should be established based on local data. They will also vary between road classes.

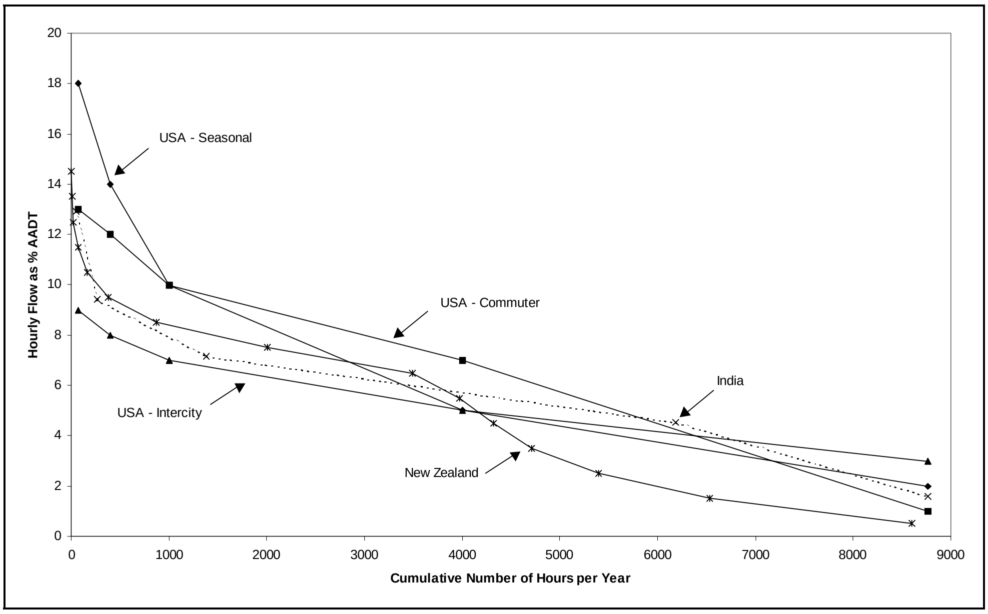

Figure E.1 Comparison of traffic flow distributions

For HDM it is necessary to establish the number of hours per year the traffic is at different flow levels. It is common to adopt at least five levels. These should consist of a **low** and **high** flow level, as well as an appropriate number of intermediate levels. When there are non-motorised traffic (NMT), it is necessary to account for these in the flow bands. Analysing the NMT traffic associated with the same flow bands as the motorised traffic does this.

Table E.2 is an example of how the hourly distribution is established (*NDLI, 1997*). Short-term traffic counts were expanded to cover a full year (8760 h/year) and broken down into five flow bands. After reviewing the distribution five flow-bands were selected and the number of hours in each band was established along with the mean flow.

Table E.2
Example of hourly distributions from India

Motorised traffic					Non-motorised traffic
Flow band	Hourly flow as percentage of AADT		Length of year at that hourly flow		Mean hourly flow as
	Bin range	Mean	Hours	Days	Percentage of NMT AADT
1	0 - 2.5	1.59	2575	107.3	0.79
2	2.5 - 6.5	4.53	4805	200.2	4.20
3	6.5 - 8.5	7.14	1114	46.4	8.88
4	8.5 - 10.5	9.41	216	9.0	12.17
5	$\geq$ 10.5	12.92	50	2.1	35.04

Source: *NDLI (1997)*

When establishing these distributions a check should be made that:

$$365\ \text{AADT} = \sum_{i=1}^{f} \text{HRYR}_i\ \text{HRVOL}_I \qquad \ldots(\text{E.3})$$

where:

$HRYR_i$ is the number of hours per year for flow band I

$HRVOL_i$ is the hourly volume in veh/h

f is the number of flow bands

The values for $HRVOL_i$ should be adjusted to ensure this check is made.

E.4 Traffic growth rates

Establishing the traffic growth rate for an HDM analysis is a very important task. This is because it will not only influence the total RUE, but also will influence capacity improvements and other intervention alternatives.

Traffic grows over time, although the growth can be quite sporadic. Figure E.2 shows the results from traffic counts taken on a road in India over a 10-year period. The data are presented from April and October, which is before and after the monsoon period. It will be noted that the post-monsoon flows are always lower than the pre-monsoon flows. This is due to the major reduction in agricultural activities after the monsoon. While there is a continual growth, the October data from 1992 and 1994 is out of context with the rest of the historical data. This could be due to some unusual event at the site, for example a temporary road closure or a local catastrophe. If such drops are not recorded elsewhere, they should probably be ignored.

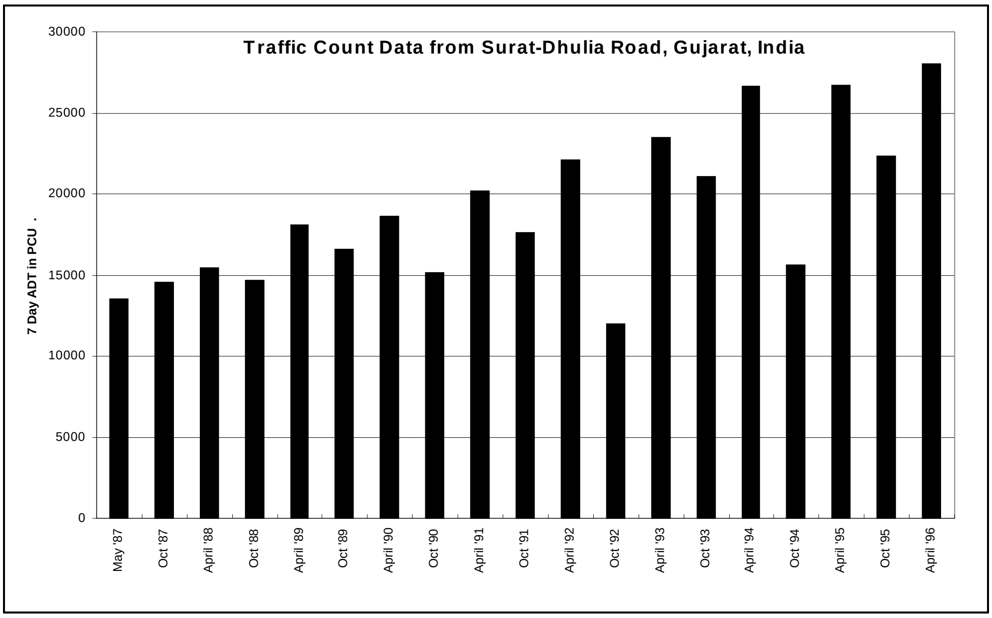

Figure E.2 Example of variation in traffic over time

The two usual ways of expressing growth are as a **geometric** or as an **arithmetic** growth rate. These are calculated as:

- **Geometric**

$$AADT_{year} = AADT_{base}\left(1+\frac{GROWTH}{100}\right)^{(YEAR-1)} \qquad \ldots(E.4)$$

- **Arithmetic**

$$AADT_{year} = AADT_{base}\left(1+\frac{GROWTH}{100}\left(YEAR-1\right)\right) \qquad \ldots(E.5)$$

where:

$AADT_{year}$	is the traffic volume in the analysis year
$AADT_{base}$	is the traffic volume in year 1
YEAR	is the year of analysis
GROWTH	is the traffic growth rate (%)

HDM uses a geometric growth rate.

The traffic growth rate is usually calculated using **historical traffic trends**, such as the data illustrated in Figure E.2, **economic trends**, such as growth in GDP, **vehicle ownership trends**, or a combination.

Using historical traffic trends is the least accurate method for predicting growth rates. This is because many developing countries are experiencing rapid motorisation and economic growth which will alter the historical trends.

Calculating growth from economic trends is often a more sound method than historical data since there are strong correlations between traffic volume and economic activity, something that a straight historical model will not capture. The underlying approach is to collect data on traffic growth rates and then to do a statistical analysis that fits a model to the data. The variables often used are income, population, fuel use, industrial production, agricultural production, or any other measure of the economy. These lead to elasticities which express the change in traffic as a function of the economic measures. Typical equations would be:

$$\mathrm{GROWTH} = \mathrm{dGNP}\ \mathrm{EGNP} + \mathrm{dPOP}\ \mathrm{EPOP} \qquad \ldots(E.6)$$

$$\mathrm{GROWTH} = \mathrm{dGNP}\ \mathrm{EFUEL} \qquad \ldots(E.7)$$

where:

dGNP	is the forecast change in the GNP (%)
dPOP	is the forecast change in population (%)
EGNP	is the GNP elasticity
EPOP	is the population elasticity
EFUEL	is the fuel use elasticity

The use of fuel is preferred by some since it gives a good overall view of the historical growth in traffic. By comparing the changes in fuel use with changes in GDP one calculates the effect of GDP changes on fuel use. Coupling this with the forecast GDP gives the total forecast traffic growth. This method has advantages in that it implicitly considers the population growth.

As an example of the elasticities, in India *CES (1991)* gave values for the GNP elasticity of:

- **1.75 and 1.0 for freight vehicles**
- **1.0 and 0.5 for passenger vehicles**

in the periods 1994-2005 and after 2005 respectively. Values of 1.43 and 2.71 were used for the population elasticity for freight and passengers respectively. In Nepal (*NDLI, 1993*) adopted a value of 1.5 for the elasticity of fuel. The use of declining elasticities with time is common as this reduces the impacts of future uncertainties on the predictions.

One important feature of traffic growth is that it is different depending upon the level of a country's development. Developed countries, with high levels of vehicle densities have markedly different car ownership growth rates than developing countries that are building up their densities. The model that best reflects this is a sigmoidal ('S' shaped) model. This is discussed in *Button and Ngoe (1991)* who present a generalised model with data covering a number of developing countries to use with the model.

WARNING

> **When forecasting traffic, always consider whether or not the predictions are reasonable. It is easy to adopt what seems to be a relatively low growth rate, such as 5 per cent, but this may mean that after a period of time your facility is at capacity. For this reason it is often prudent to adopt several different growth rates based on short, medium, and long-term considerations, with the rates declining in future years.**

There are four principal categories of future traffic which are forecast:

- **Normal traffic**

  The future traffic that can be expected assuming the current trends (for example, historical patterns) remains steady.

- **Diverted traffic**

  The traffic that can be expected to divert to the road because of the improvement.

- **Generated traffic**

  Traffic that would not have existed but is expected because of reduced travel times or diversion from other modes.

- **Induced traffic**

  Traffic expected because of the new development created by better access (for example, building a road into a new area will open it up for development).

E.5 Traffic volume and classification surveys

The purpose of these surveys is to collect data on the number and types of vehicles passing a point on a link (link counts) or making specified movements at a junction (turning counts). The occupancy of the vehicles may also be recorded to provide data on the volumes of people using the road space. Volume and classification surveys are carried out either by **manual** or **automated** traffic counts.

E.5.1 Manual counting

Manual surveys are conducted with a surveyor standing by the road, counting and classifying the vehicles as they pass, dividing the survey into fixed time periods (usually 15 minutes). It is normal for the surveyor to record only one direction of flow.

For relatively low flows, count marks on a suitable form are adequate, but re-settable hand counters mounted on boards are required for heavier flows. Another method is to use a hand held data logger or personal computer for this purpose.

E.5.2 Moving vehicle surveys

As described in *TRL (1988)*, a general indication of the traffic flows can be obtained through a moving vehicle survey. As a vehicle drives along a section of road the number of vehicles met and the number of vehicles overtaking the survey vehicle, and overtaken by the survey vehicle, are recorded. The flow is calculated as:

$$q = \frac{x + y}{t} \qquad \text{…(E.8)}$$

where:

q is the total flow in both directions in time *t*

x is the number of vehicles travelling in the opposite direction

y is the number of vehicles that overtake the survey vehicle less the number overtaken by the survey vehicle

This expression assumes that the flows in each direction are balanced. To apply this equation one first converts the flows to veh/h for by-hour-of-day. Using data on the variation of flow by-hour-of-day, the one-hour flow is converted to an ADT. This is then factored up to an AADT using the procedure described earlier.

In the Tamil Nadu Project Co-ordinating Consultancy Study, *Riley (1998)* adjusted the hourly flow from the moving survey using factors based on 7-day counts that were done using manual methods. Figure E.3 shows the excellent agreement between 7-day count data factored from the moving survey and those using data from the actual 7-day count stations.

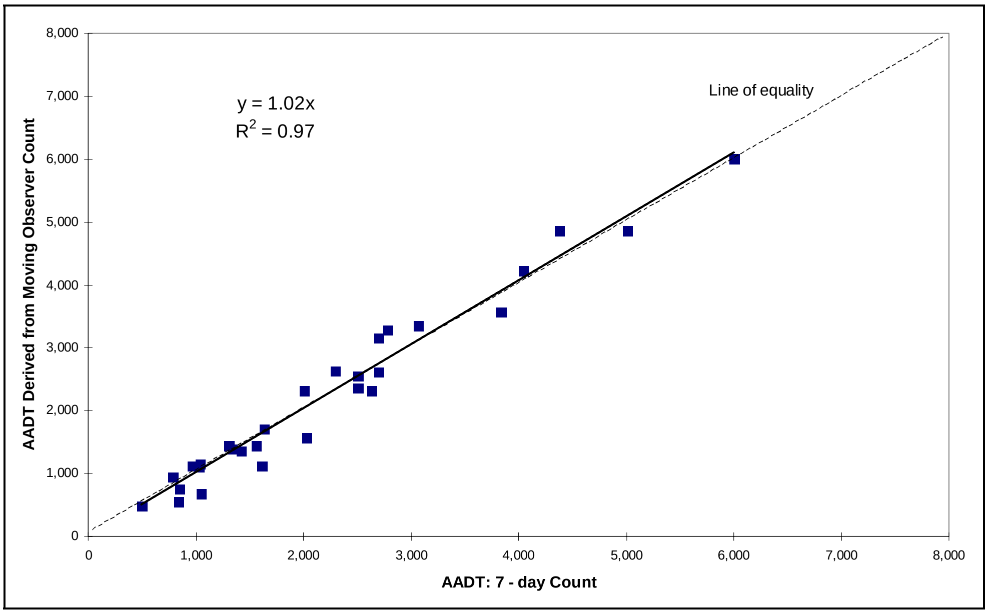

Figure E.3 Comparison of moving car and 7-day survey counts from India

E.5.3 Automatic counting

Automatic traffic counter equipment consists of detector(s) to detect the vehicle, and a counter to record the information. There are a number of different types of **detectors** used:

- **Pneumatic tubes**

  When a wheel crosses the rubber tube and air impulse is sent along the tube which activates an air switch. They are very common and relatively reliable over several days and simple to operate. Their disadvantages are that the tube in time wears and breaks and they are unsuitable where there is heavy breaking or turning. They are mainly confined to short-term count stations.

- **Inductive loops**

  A wire loop carrying a pre-determined frequency signal is embedded in the pavement. A vehicle passes over the loop and changes the inductance and thus the frequency, thereby detecting the vehicle. It is almost universally used for permanent count stations as it is very reliable, relatively inexpensive and largely unaffected by the weather. It is used at almost all traffic signals.

- **Positive contact**

  The detector is activated by the weight of the vehicle causing two thin metal strips to touch and so to complete an electrical circuit. Provides an excellent signal but fairly short life so limited to special counting and research.

- **Video**

  Analyses a video recording to estimate counts and speeds.

- **Other types:**

  - **Photoelectric**

    Uses infrared to detect the interruption of light. Not reliable.

  - **Pressure sensitive**

    A pressure sensitive switch is embedded in the roadway surface.

  - **Magnetic**

    Senses the change in the earth’s magnetic field.

  - **Radar**

    Uses radar signal.

  - **Ultrasonic**

    Transmits an ultrasonic signal.

  - **Tribo-electric cable**

    Consists of a two-core cable that creates a charge when it comes into contact with a wheel.

  - **Piezo-electric film**

    Creates a voltage when comes into contact with a wheel.

Counters work on similar principles. The detector transmits a signal to the counter. Most counters record the time when the axle (or vehicle) is detected. The data are used to establish the vehicle class and, optionally, to calculate the speed. With axle detectors, two detectors are required to establish these accurately.

As an example of how this is done, consider a two-axle vehicle which is detected by two detectors at a distance ‘D’ metres apart. The following is how the speeds and axle lengths are calculated:

$$v_1 = \frac{D}{t_{21} - t_{11}} \qquad \text{…(E.9)}$$

$$v_2 = \frac{D}{t_{22} - t_{12}} \qquad \text{…(E.10)}$$

$$SPACING_1 = v_1 \, (t_{12} - t_{11}) \qquad \text{…(E.11)}$$

$$SPACING_2 = v_2\,(t_{22} - t_{21}) \qquad \text{...(E.12)}$$

where:

D	is the distance between the detectors (m)
t_{ij}	is the time of detection, in s, at detector *i* and axle *j*
v_i	is the velocity of axle *I*
$SPACING_i$	is the spacing, in m, of axle *I*

The values for v_1 and v_2 represent the velocity of axle 1 and the velocity of axle 2 (in m/s). The spacings are the distances between axle 1 and axle 2 (in m), based on these velocities. These values are usually very similar, with the differences due to timing errors in the detectors. It is common to average the values or else to adopt only one.

It must be appreciated that there are errors associated with the measurement of all automated traffic counters and even when installed perfectly there will still be errors due to the rounding off of the timing data. For example, if the recorder is accurate to a millisecond, the time will fall within the range of 0.001 $\pm$ 0.0005 secs. While this may seem to be quite small, if the detectors are placed close together (D in the above equations), it can result in a sizeable error when measuring speeds. As described in *Bennett and Dunn (1992),* on the basis of this it is recommended that detectors be spaced at 5 m intervals.

The axle spacing data allows a vehicle to be classified into a number of different classes. It needs to be appreciated that all automatic classifiers follow certain logic in assigning the vehicles to specific classes. These are usually based on the common vehicle types in the country of development so may not be appropriate to every country. Unfortunately, suppliers of traffic counters are often reticent to modify their systems for local conditions so the output often needs to be scrutinised carefully.

There are a number of caveats to the use of automated counters, the:

- Count sites should not be located in places where they are likely to be interfered with (for example, near schools).
- Road at the site should be straight and level.
- Detector must be at right angles to the traffic flow; and if two detectors are used they must be parallel.
- Detector must be firmly fixed to the road surface.
- Detector or connections must not cross the footpath.
- Counter must be securely locked to a permanent object such as a telephone pole.

If only a single detector is used the counter will only record the total number of axles and the counter assumes that two axles equals one vehicle. This overestimates the total volume and a correction factor must be applied, calculated from manual classified counts as follows:

$$\text{Correction Factor} = \frac{\text{Number of Vehicles}}{0.5\ \text{Number of Axles on Vehicles}} \qquad \text{...(E.13)}$$

A correction factor is calculated for a specific site, but various sites can be averaged to produce factors for an area, or different classifications of roads.

E.6 Vehicle loading

Vehicle loading, or axle load, surveys are conducted using either static/low speed equipment or high-speed weigh-in-motion (WIM) equipment. *TRL (1978)* gives a good discussion of how to conduct static/low speed surveys.

The success of the survey, and the ease that it can be carried out, depends largely on the choice of site. The site must be selected so that the traffic can be sampled easily and safely. It should ideally be on a clear stretch of road with good visibility as it is important to give traffic ample times to slow down and stop. It is often useful to survey at the crest of a hill where vehicles will be travelling slowly due to the gradient. If both directions are being sampled, it is not necessary that the measurements be made exactly opposite to each other. The site should allow for vehicles to be safely weighed off the carriageway, and queue if necessary. An ideal site is one where there are slip roads, although it is more common to use the shoulders or a level area adjacent to the road.

The general area must be level and firm, with no **high spots** or risk of subsidence during the survey. Depending upon the length of the survey and the type of equipment, a concrete pit may be used for holding the scales. Alternatively, ramps may be used to ensure a smooth transition of the vehicle onto the scales. Care should also be taken to ensure that vehicles inadvertently driving over them couldn't damage the cables. It is useful to have the approach to the scales clearly defined using flags or rocks to guide the vehicles to the scales. If only measuring on a single side of the vehicle it is best to position the scale on the driver's side as this facilitates positioning the vehicle.

Before the survey commences the scales should be calibrated using the manufacturer's recommended procedure. It is important to ensure that there are spare batteries, cables, etc. available so that the survey will not be interrupted.

Experience has shown that it is best to monitor both sides of a road, as there are usually differences in the axle loading. The sampling of vehicles should be made on a systematic basis as opposed to random basis. Thus, every n^{th} vehicle is weighed as opposed to selecting vehicles when the scales are free (that is, a random approach). Since the heaviest vehicles tend to travel the slowest, they have a higher likelihood of being sampled with a random approach, thereby overestimating the axle loads. The sampling rate should be based upon the traffic volume, assuming a measurement rate of 60 to 90 veh/h. The achievable measurement rate is dependent upon both the equipment and the experience of the crew.

For each vehicle both the axle configuration and the axle loads should be recorded, as these are required for calculating the vehicle damage factor (see Section 6.3.1). It is also useful at the same time to collect additional data from the driver, such as the commodity carried, as these data can be used in determining the road user effects.

With the advent of relatively inexpensive WIM technology, this is becoming more widely used for collecting vehicle weight data. With most WIM technology, particularly portable equipment, there is a trade-off between accuracy and sampling; one samples the entire traffic stream but with less accuracy than one gets by weighing individual vehicles from the traffic stream using static techniques. This reduction in accuracy arises because of the effects of vehicle dynamics.

As with all measurements, WIM equipment is subject to random and systematic errors. *Slavik (1998)* notes that calibration will limit the systematic errors, however, most procedures will not always eliminate the random error. On-site calibration, which involves stopping and statically weighing trucks and comparing their static loads to the dynamic loads, will consider both systematic and random errors. But there has often been uncertainty as to how many vehicles one needs to weigh to achieve suitable confidence in the data from the WIM scale. *Slavik (1998)* addresses this issue and shows that the sample size is much smaller for estimating the average load as opposed to the number of equivalent standard axles. Figure E.4 gives the 90% confidence intervals for both of these based on a sample of 218 axles (*Slavik, 1998*).

The *Slavik (1998)* approach takes into consideration the properties of the WIM equipment used, the condition of the road surface, the composition and loading of truck traffic which makes the procedure site specific. The raw axle loads are multiplied by a correction factor to convert them to adjusted axle loads:

$$a_i = k \; r_i \qquad \text{…(E.14)}$$

where:

a_i	is the adjusted axle load *I*
r_i	is the raw axle load *I*
k	is the correction factor

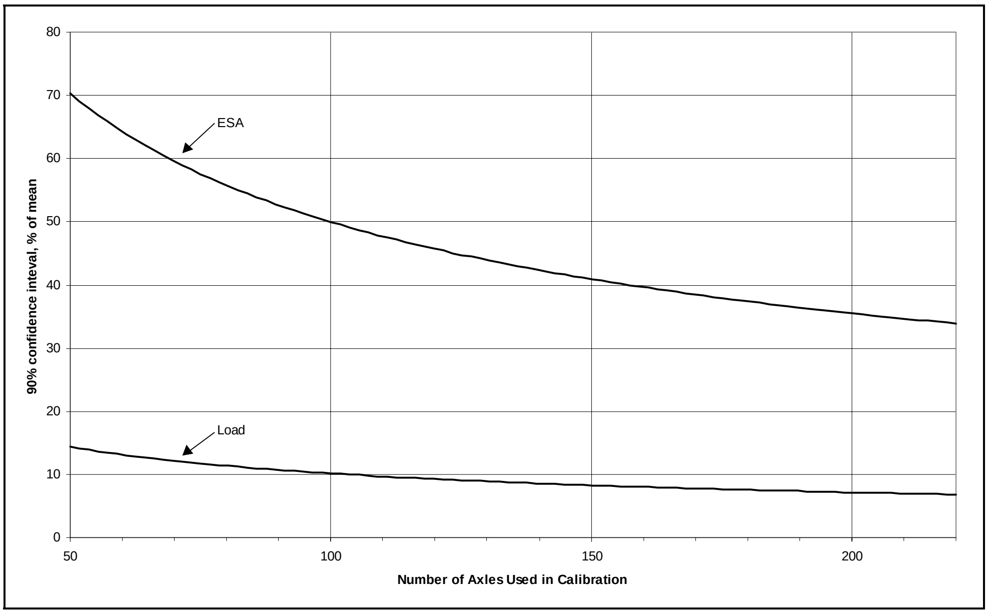

Figure E.4 90% confidence intervals for WIM equipment

This suppresses the systematic error but affects the distribution of axle loads. The distribution is therefore corrected by converting the adjusted axle loads into corrected axle loads using the equation:

$$c_i = \overline{a_i} + f\left(ai - \overline{a_i}\right) \qquad \text{…(E.15)}$$

where:

$\overline{a_i}$	is the mean adjusted axle load
f	is a correction factor

The values for *k* and *f* are established from the on-site calibration data using the following equations. Their derivation is described in *Slavik (1998)*.

$$k = 0.5\left[\frac{\overline{s_i} - \sigma s_i / \sqrt{n}}{\overline{a_i}} + \frac{\overline{s_i} + \sigma s_i / \sqrt{n}}{\overline{a_i}}\right] \qquad \text{...(E.16)}$$

$$f = \sqrt{1 - Ve_i / Va_i} \qquad \text{...(E.17)}$$

where:

$\overline{s_i}$	is the mean static axle load
σs_i	is the sample standard deviation of the static axle load
n	is the sample size
Va_i	is the variance of the adjusted axle loads
Ve_i	is the variance of the difference between the adjusted and static axle loads

Appendix F Unit cost data

F.1 Introduction

The HDM road model operates by predicting the amount of resources consumed and multiplying them by the unit costs. It is therefore necessary to supply unit cost data for RUE and works effects (WE). These costs can be supplied as financial and/or economic costs, which are defined below.

The RUE unit costs can be established from various sources that are listed in Table F.1. In establishing these unit costs, they should reflect the cost over the life of the project, normally 10 - 20 years. For most items the current prices can be used as the basis since inflation can be expected to influence the various components similarly so that they maintain the same cost in relation to one another. An exception to this is often fuel and lubricants that are heavily influenced by world supply and demand. As described later, it is therefore better to estimate these based on the long-term average future oil prices[1].

Table F.1
Sources for unit cost data

Data source	Data item
Vehicle dealers and distributors	Replacement vehicle price, finance charges
Motor vehicle magazines/trade publications	Replacement vehicle price
Oil companies	Fuel prices, discounts, regional variations
Customs and Excise Department	Fuel prices, tax structures
Tyre companies	Tyre prices, discounts, regional variations
Insurance companies	Insurance and accident costs
Trade unions	Wages and earnings
Licensing authorities	Annual license and registration fees

For works effects, unit costs are best established from recent tender prices or contracts.

F.2 Economic and financial costs

HDM allows for two types of costs; economic and financial, although the economic costs are those generally used in analyses[2]. The financial costs are the market costs. The economic costs are the market costs net of (excluding) taxes and subsidies (for example, diesel is subsidised in many countries). In establishing the economic costs **shadow pricing** is often used. This sees the price of the commodity adjusted to reflect the real scarcity value of the resources in the commodity. *Daniels (1974)* discusses the use of shadow pricing in a number of different transport studies in developing countries and highlights some of the difficulties

1 For example, *Hoff & Overgaard (1994)* indicate that in early 1994 the prevailing price for a barrel of oil (158.6 l) was $USD 15 whereas the long-term average price was expected to be $USD 20.

2 HDM-III also included foreign exchange costs.

encountered in using shadow pricing. While shadow pricing is desirable, for most studies it is sufficient to quantify the economic costs, as the market costs net of taxes.

To illustrate how this is done, consider Figure F.1. This shows the economic and financial cost components of petrol and a medium truck from Thailand. Petrol had a series of taxes and levies added to the cost of production and supply. The taxes with the truck had two components, the:

1 **Import duty on the CKD chassis**

2 **Sales tax on the completed unit**

Since these were levied at different stages of production one cannot just deduct them directly from the financial cost.

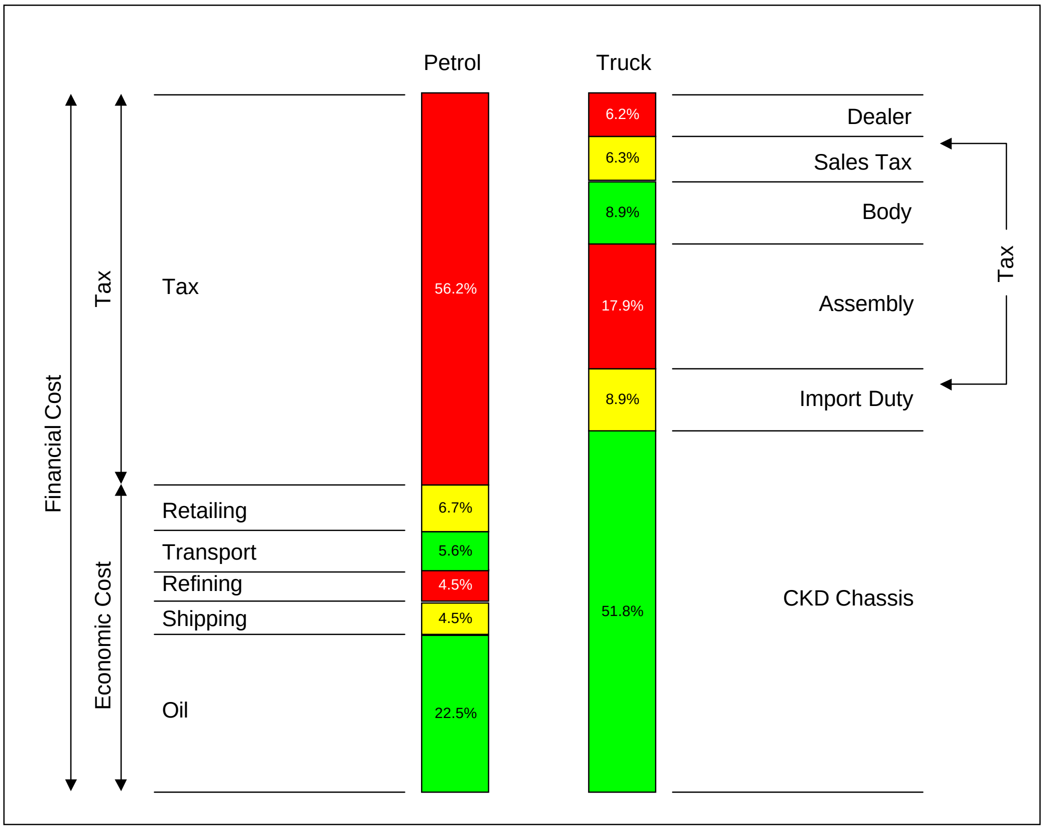

Figure F.1 Breakdown of 1992 Thailand petrol and medium truck prices

When deducing the taxes from the market cost, it is important to consider not only the direct but also the indirect taxes. To illustrate this, consider the cost of petrol from Thailand shown in Figure F.1.

The indirect tax component of this price amounted to 56.2 per cent. The cost of production and supply 43.8 per cent, however, it is necessary to deduct the direct tax component from this remaining cost. This would entail assessing profit margins and the direct taxes on these profits. The foreign exchange component would consist of the refinery cost less production costs and margins along with a component of distribution costs. For simplicity, most analyses would ignore the direct taxes and just use the total indirect taxes.

Hoff & Overgaard (1992) as well as *Hoff & Overgaard (1994), Kampsax (1992), NDLI (1997), Kinhill (1998)* used an alternative approach to that outlined above. Instead of starting with the financial cost of fuel and deducting the taxes, this method begins with the cost of a barrel of oil and increases the costs to reflect the refining, transport and marketing costs. It has the advantage that it will identify any hidden taxes that may be missed using the above approach. These hidden taxes may arise in situations such as Nepal where the Indian government taxes fuel exports to Nepal and these taxes are later refunded as a credit to the Nepal government. Since these taxes are external to Nepal several projects missed removing them when calculating the economic cost of fuel. An example of this approach from *Hoff & Overgaard (1994)* is given in Table F.2.

Table F.2
Example of calculating economic cost of fuel

Component	Petrol ($)	Diesel ($)
Crude oil price	20.00	20.00
Refining margin	6.20	4.80
Transport and distribution	3.00	3.00
Cost per barrel	29.00	27.80
Cost per litre	0.184	0.175

Source: *Hoff & Overgaard (1994)*

Another argument in favour of using the price of a barrel of oil for calculating fuel prices is that oil exporting countries often price fuel below the prevailing world price for oil. As noted by *Hoff & Overgaard (1994)*, changes in fuel consumption as a result of road projects affect the quantity of fuel available for export and the potential export as opposed to the market price is therefore the appropriate value of fuel to use.

F.3 Road user effect unit costs

F.3.1 New vehicle price: S-I

Because of high sensitivity, the new vehicle price should always be quantified as accurately as practical. The new vehicle price is used in calculating the parts, depreciation and interest costs. It is the singularly most important unit cost, and probably data item, in any HDM analysis.

The values should be those for the vehicle less tyres, since tyres have their costs calculated separately. If the tyre cost was used there will be double counting in the analysis[1].

Within a representative vehicle class there will be a range of different vehicle types, each with different vehicle prices. The best method for calculating the representative vehicle price is by using recent sales data to determine the number of different models of vehicles sold. Weighting these frequencies by the costs of the individual models will give the weighted-average cost. Since most commercial vehicles are sold in a cab and chassis configuration it is necessary to increase this cost to reflect the additional costs of bodywork. The same often applies to heavy buses as well.

As discussed earlier, it is necessary to convert the vehicle prices from financial costs to economic costs. If there are different import duties and taxes on individual vehicles, for example due to their engine capacity or country of origin, it is necessary to convert the prices

[1] It should be noted that in HDM-III this would lead to an underestimation of the interest costs.

from financial to economic before calculating the weighted-average. If there are uniform taxes for all vehicles within a class, the weighted-average financial cost can be directly converted to the weighted-average economic cost with a single factor.

In many countries vehicles are completely rebuilt at a certain kilometreage. This sees major components replaced or refurbished. If the cost of rebuilding is included in the new vehicle price the depreciation costs will be higher over the life of the vehicle. It is better to have two different vehicles, one for vehicles that have not been rebuilt and a second for those that have been rebuilt. This will reflect the different parts costs while not biasing the depreciation costs.

In HDM-III there was no mechanism to include a residual value for the vehicle in the depreciation cost. These can be quite high, particularly in countries where old vehicles are assigned to different activities, such as port work. To circumvent this problem, *Transroute (1992)* assumed that vehicles had a 30 per cent residual value. The residual value was discounted at the assumed accounting rate of interest and deducted from the replacement vehicle price. This was done to reflect the "cascading down of both types of vehicles in their later years to short distance, low annual mileage, low maintenance, long life work". Since this procedure served to reduce the maintenance and repair costs, the latter were adjusted through the calibration parameters. HDM-4 provides for a residual value in the analysis.

F.3.2 Fuel and lubricant costs: S-III/S-IV

In HDM-4 the fuel type is defined for each representative vehicle. If vehicles within the representative vehicle class have petrol or diesel, it is recommended that these be modelled as two different vehicles given the difference fuel consumption rates of these engine types.

When there are different types of the same fuel available, for example **regular**, **premium** and **unleaded** petrol, the fuel cost for each of these types should be weighted by the proportion of each fuel type sold, or an estimate of these proportions, to obtain a weighted-average cost.

In some countries there are major fluctuations in fuel prices by region due to transport and distribution costs. Another problem arises when there are distortions created by artificial exchange rates. This can see some vehicles, usually those owned by the government, obtaining fuel at a much lower cost than privately owned vehicles. Under these circumstances, for financial analyses it is necessary to define separate vehicle classes by tax regime or ownership, and to use different financial fuel costs for the vehicle classes.

HDM requires the cost per litre of engine oil. This should be established in the same manner as petrol. It should be noted that this is a minor cost component so does not warrant a large effort.

F.3.3 Tyre cost: S-III

The average tyre cost is the cost of the tyre - not the entire wheel. Tyres can have a wide range of costs and qualities. In some countries more expensive tyres are preferred because there is a net advantage due to their longer lives. Some users prefer radial to bias ply tyres; some tyres have inner tubes while many are tubeless. All of these need to be taken into account when establishing the average tyre cost.

The average tyre cost is multiplied by the rate of tyre consumption (in tyres/1000 km) to establish the cost of tyre wear. The cost should therefore exclude inner tube costs and be restricted to the tyre carcass. If inner tubes are a major cost item associated with road use, for example due to frequent damage and replacement, the tyre cost should be increased to reflect this. This is done as follows:

1. Establish the cost of inner tubes per 1000 km (ITC) for an **average** road.
2. Run HDM using **average** conditions and establish the tyre cost for each representative vehicle class in cost/1000 km (TYRE).
3. Multiply the average tyre cost by 100 ITC/TYRE to obtain the adjusted tyre cost. This adjusted tyre cost should be used in all further modelling.

The issue of radial versus bias tyres; high quality, long-life versus lower-quality, shorter life tyres is accounted for by calculating a weighted-average cost. The number of different tyre sizes sold for each vehicle within a representative vehicle class should be weighted by the average cost of the tyres to obtain a weighted-average cost. If there are significant differences in prices between countries of origin, the averaging should also reflect this. Contacting tyre retailers and asking them for an assessment of the percentages of sales of different tyre types can readily collect these data. For example, in Nepal *NDLI (1993)* recorded the tyre types on a sample of vehicles stopping at a customs checkpoint.

The HDM tyre model also requires the cost of retreads as a percentage of the new tyre price. This is used in the calculations to establish an average tyre cost for vehicles that use a mix of retreads and new tyres. There are two considerations here, the:

1. **Average value**
2. **Probability of a retreaded tyre being used**

The average value is calculated in the same manner as for a regular tyre. This value then needs to be reduced based on the probability of a retreaded tyre being purchased since HDM assumes an equal likelihood of a retread or a new tyre being on a vehicle. This is done as follows:

1. Interview tyre retailers, other industry sources, truck drivers, etc. and establish the percentage of tyres in use, which are retreads (PCTRET).
2. Express the average retread cost as a percentage of the new tyre cost (RETPCT).
3. Calculate the retread cost as:

   $$\mathrm{RREC} = \mathrm{PCTRET}\ \mathrm{RETPCT}$$

   This is the value used as input to HDM.

F.3.4 Passenger time, crew and cargo costs: S-II/S-III

The value of passenger time is probably the singularly most difficult cost to quantify. There have been a number of reports dedicated to this issue. *Symonds (1997)* gave a review of the more recent work with *Cox (1983)* giving a good historical view. *Symonds (1997)* also gave a good discussion of the issue of non-linearity of travel time savings[1].

[1] This relates to the question of to the appropriateness of treating a large number of small time savings as equal to a single large time saving of the same magnitude. There are several arguments in favour of this, but the most common is that since small time savings cannot be perceived, they should not be valued lower than large time savings - hence the non-linearity of the value of time savings. *Symonds (1997)* reviews the issue and concludes "… there are no compelling theoretical or empirical reasons to adopt non-linear assumptions on values of time savings, and there are strong practical reasons in favour of a constant [value]". They note that the exception to this is leisure time where there are different levels of disutility but that for practical reasons linear values should be used.

While some studies have simply ignored passenger time, this is not wise as it can bias the results in favour of projects with high transport costs. Conversely, an unrealistically high value for passenger time can bias projects in favour of those with large speed increases.

In quantifying the value it should be recognised that there are two sectors of the economy: the **formal** and the **informal** sectors. The formal sector is comprised of wage earners. The informal sector is those without salaries - such as many of the rural population in developing countries.

For simplicity, travel is usually separated into **work** and **leisure** or **non-work** travel. Work travel is comprised of those on business activities. Leisure, comprised of other activities (for example, social, visiting family, personal business, going to school, etc.) is often a significant component of travel in developing countries.

Savings in time when journeys are related to work clearly have a value; if less time is spent travelling more time in the working day can be used for economically productive purposes. Another way of looking at this is the employer pays the employee an hour’s wages for no return. The employer would be willing to pay equal to an hour’s wages to reduce travel time by one hour. It can be argued that due to overheads and **social charges** the employer would be prepared to pay even more, but the common practice in developing countries is to equate the value of work time to the earnings rate of the traveller. In developed countries, where there are often large social costs, this gross wage is increased by the employer’s on-costs.

The use of wage rates is complicated by the fact that official statistics on wages will probably underestimate the earnings of travellers. Wage statistics do not usually cover the earnings of the highest paid workers, and the wages of those travelling during working time may be higher than the average. Often, there are also regional variations in wages that make it impractical to adopt a national average. For example, in Thailand the 1995 average wage was Baht 9900/month in Bangkok versus 4500 in the north-eastern region of the country.

Those in the informal sector or travelling in leisure time are not considered to be productive in the same way as those travelling in work time. Ultimately, the value of non-working time should reflect Government policy. If the policy is to maximise GDP, ignoring leisure time preferences and increasing the welfare of passengers, then a zero value should be placed on non-work time. It must be recognised that assigning a zero value to the time for those in the informal sector will serve to bias the results in favour of those who contribute to the modern, cash economy.

There is evidence that the leisure time savings are valued, particularly since these travellers still prefer their trips to be faster than slower and are often willing to pay more for this to happen. How much a person is prepared to pay for a quicker trip is based upon their income and wealth. It is therefore common practice to assume a value of personal time related to the individual’s income. Various percentages have been assumed in different studies, usually in the range 20-50 per cent, but 20-25 per cent seems to be the most common.

Many who travel in personal time do not earn any income and so using this approach would have no value for their time. In affluent societies this would not be true, but it is argued by some that in some countries a zero value of time is appropriate.

In these instances the mean income is used to calculate the value of time. The alternative approach is to calculate the value of time based only on those working and then to apply the value to all travellers. This will yield a higher value of time than using the mean income of travellers.

To summarise, there are three sets of passenger time values to be considered:

1 **Employed, travelling in work time**,

2 **Employed, not travelling in work time**; and,

3 **Unemployed or in non-paid activities**.

There is evidence that travel time values are higher for traffic travelling under congested as opposed to free-flow conditions. *MVA et al. (1987)* suggested a factor of 1.4 for the situations they studied, and an even higher factor under more congested conditions.

When establishing the value of time, particularly for truck and bus operators, it is important to include extra income that may be obtained above the base salary, for example:

- **Daily allowances -** to cover food and rest,
- **Carrying passengers (trucks) or extra, non-reported passengers (buses)**; and,
- **Backhaul of goods by trucks -** where the operator instead of the owner keeps the income.

These can be significant, for example in India *NDLI (1997)* estimated that the total salaries for the truck driver and helper were Rs 4500 but that there was a further monthly income of Rs 3000 from collecting passengers and Rs 3000 from daily allowances. About 20-25 per cent of drivers managed to backhaul goods, earning a further Rs 7000 per month.

It is also common to differentiate between modes of travel. This, for example, sees different passenger time costs for passengers in private transport and those in buses or other public transport.

Table F.3 gives an example of calculating passenger time and crew costs, using basic data from India *NDLI (1997)* along with some assumed additional values. The notes at the bottom of the table detail how the individual values were calculated.

Table F.3 Example of calculating passenger time and crew costs

		Two-wheeler	Car	Medium truck	Heavy truck	Heavy bus
(1)	Income - Work Trips (Rs/h)	15	32	-	-	43
(2)	Income - Non-Work Trips (Rs/h)	16	47	-	-	31
(3)	Time Value - Non-Work Trips (Rs/h)	3.2	9.4	-	-	6.2
(4)	Time Value - Not Employed (Rs/h)	0.6	1.8	-	-	1.2
(5)	Persons per vehicle	1.5	3.5	2.0	2.0	32.0
(6)	Passengers per vehicle	1.5	3.3	-	-	30.0
(7)	Percentage in employment	75	75	-	-	50
(8)	Number employed	1.1	2.5	-	-	15.0
(9)	Number not employed	0.4	0.8	-	-	15.0
(10)	Percentage travelling in work time	36	19	-	-	34
(11)	Number travelling in work time	0.5	0.6	-	-	10.2
(12)	Number employed but not on work trip	0.6	1.9	-	-	4.8
(13)	**Passenger Time cost Rs/h**	9.7	38.5	0.0	0.0	486.4
(14)	Crew per vehicle	-	0.2	2.0	2.0	2.0
(15)	Cost of Driver (Rs/month)	-	600	3,000	3,000	3,000
(16)	Cost of Helper(Rs/month)	-	0	1,500	1,500	1,500
(17)	Additional Income—Allowances (Rs/month)	-	0	3,000	3,000	3,000
(18)	Additional Income—Phantom Passengers (Rs/month)	-	0	3,000	3,000	3,000
(19)	Additional Income—Phantom Backhauls (Rs/month)	-	0	1,500	1,500	0
(20)	Monthly Crew Time (Rs/month)	-	600	13,000	13,000	10,500
(21)	**Crew Time Cost Rs/h (200 h/month)**	0.0	3.0	65.0	65.0	52.5
(22)	**Cargo Cost (Rs/h)**	0.0	0.0	2.5	5.0	0.0
	Total Cost (Rs/h)	9.7	41.5	67.5	70.0	538.9

Notes:

(3) = (2) x 0.20. Assumed 20% for value of work time.

(4) = (3) x 0.20. Assumed 20% of value of non-work time.

(8) = (6) x (7)

(9) = (6) - (8)

(11) = (8) x (10)/100

(12) = (8) - (11)

(13) = (1) x (11) + (3) x (12) + (4) x (9)

For cars which had less than one crew, (15) = (14) x Rs 3,000

(19) is calculated from the extra revenue (here Rs 7,000/month) times the percentage of backhauls (here, approximately 20 per cent).

(20) = (15) + (16) + (17) + (18) + (19)

(21) = (20)/200. Assumed 200 working h/month

(23) = (13) + (21) + (22)

If goods spend less time in transit the amount of goods held in inventory may be able to be reduced. Some goods will not benefit from the time savings, for example, if they arrive before the business opens and thus cannot be unloaded. Thus, only a portion of the goods in transit should be included in the calculations. The value will depend upon the nature of operations in the country, or even in the area of the country, but values of 50-75 per cent are common.

The value of cargo time is calculated using the opportunity cost and the following equation:

$$CARGO = \frac{PCTCGT\ OPC\ VALCAR}{365 \times 24} \qquad \ldots(F.1)$$

where:

CARGO is the cargo cost in cost/h

PCTCGT is the fraction of vehicles whose cargo will benefit from time savings

OPC is the opportunity cost of the cargo as a decimal

VALCAR is the value of the cargo

F.3.5 Maintenance labour: S-III

The maintenance labour costs should reflect the costs of labour, tools and workshop overheads. It is therefore not appropriate to base the costs only on the prevailing wage rates. In some countries the wages will contain a number of transfer payments such as taxes, social security, etc. These need to be taken into consideration when calculating the average cost. For example, in *Transroute (1992)* assumed a 100 per cent overhead on the monthly wage and 250 h/month working time.

One of the simplest methods of estimating the average hourly labour costs is by conducting a small survey. A number of typical maintenance activities should be defined such as replacing the clutch, engine tuning, etc. Workshops should be surveyed to obtain their estimate of the time it would take to perform the repairs and the total costs for the repairs. This will give an hourly cost that includes overheads as well as labour and considers unproductive time. In developing countries where a mechanic also has semi-skilled and unskilled assistants this approach will implicitly include their costs as well[1]. The resulting value should be adjusted for taxes etc. to convert it to an economic cost.

F.3.6 Interest rate: S-III

HDM requires an annual interest rate for calculating the opportunity cost of vehicle ownership - also called the interest costs. For economic analyses, the interest rate should be the same as the discount rate. For financial analyses, the value selected should be the real long-term interest rate (that is, the market-borrowing rate less the underlying inflation rate). The value selected must be based on long-term considerations since interest costs can constitute a major component of the total vehicle operating costs.

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[1] Some analysts adopt the approach of calculating an average hourly rate based on a percentage of skilled/semi-skilled/unskilled time spent working on a vehicle. This method is generally not appropriate since in many countries the practice is to have more than one person working on the vehicle at a time. Thus, the costs should be accumulated rather than weighted.

F.3.7 Overhead/standing costs

The overhead/standing costs are those costs associated with vehicle ownership. They consist of costs due to licensing, insurance, and garaging. Overhead costs are often ignored in analyses due to their having a minor impact on the total costs as influenced by road conditions. Another view is that they should be entirely excluded since in an economic evaluation the marginal overhead is zero.

If overhead costs are to be included in the analysis, it is best to use the overhead costs to estimate an hourly value of vehicle time by dividing the total annual overhead costs by the number of hours per year the vehicle is used. This hourly cost can then be treated in an equivalent manner to travel time/crew costs.

F.4 Works effects unit costs

To be included in a subsequent edition of this document

Appendix G Economic data

To be included in a subsequent edition of this document

G.1 Introduction

G.2 Discount rate

G.3 Analysis period

Appendix H Determining sample sizes

The approach to use to determine the appropriate sample size depends upon the number of samples. With few data items one deals with small samples (*n* $\le$ 30), so the *t* Distribution should be used. With larger samples (*n* > 30), the normal distribution should be used. The basic approach adopted is the same with each distribution.

The standard error of the mean for a sample of data is given by:

$$E = \frac{s}{\sqrt{n}} \qquad \text{...(H.1)}$$

where:

s is the standard deviation of the sample

n is the number of observations

The true mean of the population is aid to lie between x $\pm$ E. Thus, the larger the sample size the smaller the value of E and the closer the sample mean will be to the population, thereby limiting the bias.

The necessary sample size for a given level of accuracy and confidence is given by:

$$e = K\frac{s}{\sqrt{n}} \qquad \text{...(H.2)}$$

where:

e is the error limit

K is the *t* statistic (*n* < 30) or the number of standard deviations about the mean of the normal distribution (*n* > 30)

Rewriting the above equation gives the following equation for determining the required sample sizes:

$$n = \frac{K^2 s^2}{e^2} \qquad \text{...(H.3)}$$

Hamilton (1990) gives the values in Table H.1 for 90 and 95 per cent confidence intervals using the *t* Distribution (*n* $\le$ 30). Table H.2 gives the values of *K* for the Normal Distribution (*n* > 30).

In order to use the above equations for determining the required sample size, it is necessary to know the standard deviation of the sample. Since this is not available until after the study has been completed, it appears impossible to estimate the required sample sizes before the study. However, the distributions of many data items are relatively stable between sites and between countries and have a consistent value for the coefficient of variation. This is defined as:

$$\mathrm{COV} = \frac{\sigma}{\mu} \qquad \ldots(H.4)$$

where:

COV is the coefficient of variation

σ is the standard deviation

μ is the mean

Table H.1
Critical 't' values (n < 30)

Number of samples	Critical t value by confidence interval (K)	
	90	**95**
3	2.920	4.303
4	2.353	3.182
5	2.132	2.776
6	2.015	2.571
7	1.943	2.447
8	1.895	2.365
9	1.860	2.306
10	1.833	2.262

Table H.2
Critical K values (n > 30)

Confidence level	K
68.3	1.00
90.0	1.65
95.0	1.96
95.5	2.00
99.0	2.58
99.7	3.00

Appendix I Survival curve analysis

This example of calculating the survival curve is based on the approach of *Zaniewski et al. (1982)* and uses data from *Bennett (1985)*.

Table I.1 presents the results of a survey of vehicle ages. A sample consisting of the number of vehicles at different ages was obtained and expanded to represent the entire population. This expanded sample is presented in column (2) of Table I.1 and column (3) contains the number of the vehicles of each age, which were originally registered.

Dividing the expanded sample in column (2) by the number of original registrations in column (3) gives a survival curve in column (4). Multiplying this column by 100 per cent gives the percentage of each age surviving.

Zaniewski et al. (1982) recommends modifying this survival curve to improve the predicted service life. In theory, the highest value in column (4) represents a year when all of the registered vehicles are still in the population. As such, it is the sampling factor for the survey. Dividing the survival curve in column (4) by this sampling factor gives the survival ratios in column (5). These ratios are used in place of the values in column (4) for the survival curve[1].

The sum of column (5) is the area under the survival curve and this corresponds to the average service life. The data in Table I.1 indicate that the average service life is 19 years.

In column (6) of Table I.1 the average annual kilometreage for vehicles in each age range is given. To determine the average lifetime kilometreage the product of the survival ratio (column (5)) and the annual utilisation is calculated. These products are given in column (7). The sum of the values in column (7) represents the average lifetime kilometreage - in this example 187,202 km. The average annual kilometreage is given by the average lifetime kilometreage divided by the average service life. Using the earlier derived service life of 19 years results in an average annual kilometreage of 9,850 km/yr.

[1] For those years preceding the sampling factor the survival ratio is set to 1.0.

Table I.1
Example of establishing survival curve and lifetime utilisation

Vehicle age	Number in expanded sample	Original registrations	Sample/original [(3)/(2)]	Survival ratio	Average annual utilisation (km/yr)	Survival Ratio x Av. annual util [(5)*(6)]
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0- 1	65131	67149	0.97	1.00	11500	11500
1- 2	59261	61824	0.96	1.00	11500	11500
2- 3	71789	73527	0.98	1.00	11500	11500
3- 4	81653	83612	0.98	1.00	11500	11500
4- 5	91854	99213	0.93	1.00	11250	11250
5- 6	99569	102626	0.97	1.00	11000	11000
6- 7	89524	90302	0.99	1.00	10750	10750
7- 8	65413	74626	0.88	0.88	10500	9284
8- 9	68118	70426	0.97	0.98	10250	10000
9-10	53328	56438	0.94	0.95	10000	9531
10-11	46308	48808	0.95	0.96	9750	9331
11-12	50636	56484	0.90	0.90	9500	8590
12-13	53721	63768	0.84	0.85	9250	7860
13-14	54516	67189	0.81	0.82	9000	7366
14-15	52632	65742	0.80	0.81	8750	7066
15-16	44482	58347	0.76	0.77	8500	6536
16-17	27301	42680	0.64	0.65	8250	5323
17-18	24714	37663	0.66	0.66	8000	5295
18-19	18188	33859	0.54	0.54	7750	4199
19-20	13388	27214	0.49	0.50	7500	3722
20-21	13190	30985	0.43	0.43	7250	3113
21-22	15208	41323	0.37	0.37	7000	2599
22-23	11130	38980	0.29	0.29	6750	1944
23-24	12516	46237	0.27	0.27	6700	1829
24-25	8812	36610	0.24	0.24	6500	1578
25-26	2958	25724	0.12	0.12	6500	754
26-27	5645	37095	0.15	0.15	6500	998
27-28	2884	26595	0.11	0.11	6500	711
28-29	1378	20575	0.07	0.07	6500	439
> 29	8087	393744	0.02	0.02	6500	135

Source: *Bennett (1985)*

Notes:

Sampling factor:

Appendix J Orthogonal regression

Linear regression analysis fits the following model to data:

$$y = a\,x + b + \varepsilon \qquad \text{…(J.1)}$$

where:

y	is the dependent variable
x	is the independent variable
ε	is the error term
a and b	are regression coefficients

The analysis is done in such a way that the error term is minimised.

There are instances where the dependent and independent variables are interchangeable, for example as in the HDM equations to predict modified structural number from the Benkelman Beam deflection and vice versa. This gives rise to two sets of equations:

$$y = a\,x + b + \varepsilon \qquad \text{…(J.2)}$$

$$x = c\,y + d + \varepsilon \qquad \text{…(J.3)}$$

The issue here is one of **orthogonal regression** where instead of minimising the sum of squares in the *x* or *y* plane the minimisation needs to be done in the orthogonal plane. To illustrate the importance of this, consider Table J.1. Columns 1 and 2 contain sample data. These data are plotted in Figure J.1 along with the linear regression equations.

Table J.1
Orthogonal regression illustrative data

Data		Standard regression			Orthogonal regression	
X	**Y**	**Y Predicted**	**X Predicted**	**Residual X**	**Y Predicted**	**Y Predicted**
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**
1.00	9.50	9.44	1.05	0.05	9.46	1.00
1.14	9.40	9.37	1.18	0.04	9.38	1.14
1.29	9.20	9.28	1.32	0.03	9.29	1.29
1.43	9.20	9.20	1.44	0.01	9.21	1.43
1.57	9.10	9.13	1.57	0.00	9.13	1.57
1.71	9.00	9.05	1.70	-0.01	9.05	1.71
1.86	9.05	8.97	1.84	-0.02	8.96	1.86
2.00	8.87	8.89	1.97	-0.03	8.88	2.00
2.14	8.70	8.81	2.10	-0.04	8.80	2.14
2.29	8.83	8.73	2.24	-0.05	8.71	2.29

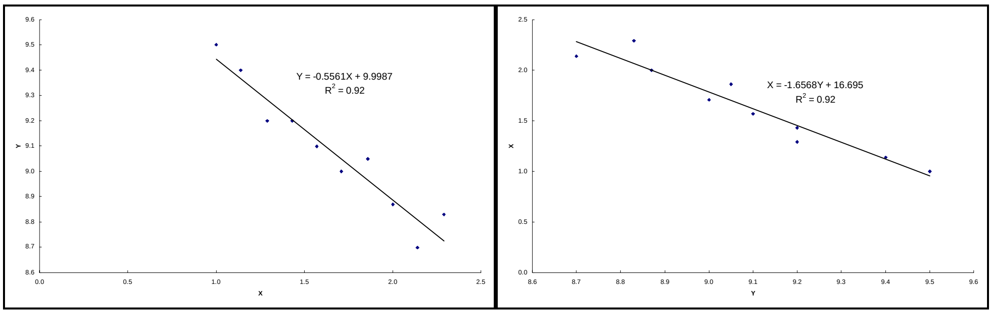


Figure J.1 Fitted regression lines

The two regression equations developed were:

$$y = -0.56\,x + 10.00 \qquad \text{…(J.4)}$$

$$x = -1.66\,y + 16.70 \qquad \text{…(J.5)}$$

Column 3 in Table J.1 shows the value of Y predicted using Equation J.2 above along with the original X data from Column 1. Using these values as input data for Equation J.3 above gives the value of X. As shown in Column 5, there is a difference between these predicted values of X and the original X values. This difference would not exist had an orthogonal regression been done.

Orthogonal regressions require sophisticated statistical techniques. However, when the residuals are similar in each direction the following simple solution may suffice.

$$y = a\,x + b \qquad \text{…(J.6)}$$

$$x = c\,y + d \qquad \text{…(J.7)}$$

so:

$$y = \frac{(x - d)}{c}$$

$$y = \left[\frac{\left(a + \frac{1}{c}\right)}{2}\right]x + \left[\frac{\left(b - \frac{d}{c}\right)}{2}\right] \qquad \text{…(J.8)}$$

and:

$$x = \frac{1}{\left[\frac{\left(a + \frac{1}{c}\right)}{2}\right]}y - \left[\frac{\left(b - \frac{d}{c}\right)}{\left(a + \frac{1}{c}\right)}\right] \qquad \text{…(J.9)}$$

Using this approach with Equations J.4 and J.5 above results in the following equations:

$$y = -0.58\,x + 10.04 \qquad \text{…(J.10)}$$

$$x = -1.72\,y + 17.31 \qquad \text{…(J.11)}$$

Column 6 in Table J.1 shows the predicted values of *y* using Equation J.4 above in conjunction with the data from Column 1. Substituting these values into Equation J.5 above gives the predicted values of *x* (Column 7). These values are the same as the original data in Column 1.

Appendix K HDM Tools user guide

K.1 Introduction

HDM Tools is a set of software applications designed to assist in the calibration of HDM-4. The software is available from the Internet at:

- http://www.opus.co.nz
- http://www.htc.co.nz

The HDM Tools applications have been designed to run completely independently of the HDM-4 software, although the data can be used to update the HDM-4 data files.

The applications fall into three groups:

- **RUE**
- **RDWE**
- **Analysis Tools**

Each are discussed separately below.

K.2 Road user effects

The current HDM Tools RUE applications are:

- **ACCFUEL -** Effect of acceleration on fuel and tyre consumption simulation;
- **CDMult -** Wind adjusted aerodynamic drag coefficient;
- **GEARSIM -** Engine speed simulation;
- **OPTIMAL -** Optimal life calibration; and,
- **WORKZONE -** Work zone effects.

These applications are run from a single launch button and output data that can be imported to the HDM Vehicle Fleet.

If the default HDM-4 vehicle parameters are used it is not necessary to run the HDM Tools applications. However, if the default values are changed then the guidelines in Table K.1 should be used to establish if the HDM Tools applications need to be re-run.

Table K.1
Criteria for re-running RUE HDM tools

Program	When should be run	Calibration level where should be run		
		Level 1	Level 2	Level 3
ACCFUEL	If there is a change of more than 5 per cent to the vehicle mass; 10 per cent to the frontal area and/or the aerodynamic drag coefficient	•	•	•
CDMult	If there is a change of more than 10 per cent to the frontal area and/or aerodynamic drag coefficient	•	•	•
GEARSIM	For Level 2 or 3 calibrations of engine speed		•	•
OPTIMAL	For all HDM applications using the optimal life for capital costs	•	•	•
WORKZONE	If work zone effects are of interest			

K.3 HDM Tools Launchpad

The HDM Tools RUE component was developed by Ian Greenwood[1] of Opus International consultants Ltd. Running under Windows 95, the software is installed as per any standard Windows application.

The applications are run from the HDM Tools **Launch pad**. Figure K.1 shows this Launch pad and the individual applications. These may be changed in the future as additional modules are added to the HDM Tools software. Selecting any button the Launch pad will start the individual application. The lauchpad remains open in the background unless it is exited.

1 Comments and suggestions on the software can be sent to: ian.greenwood@opus.co.nz.

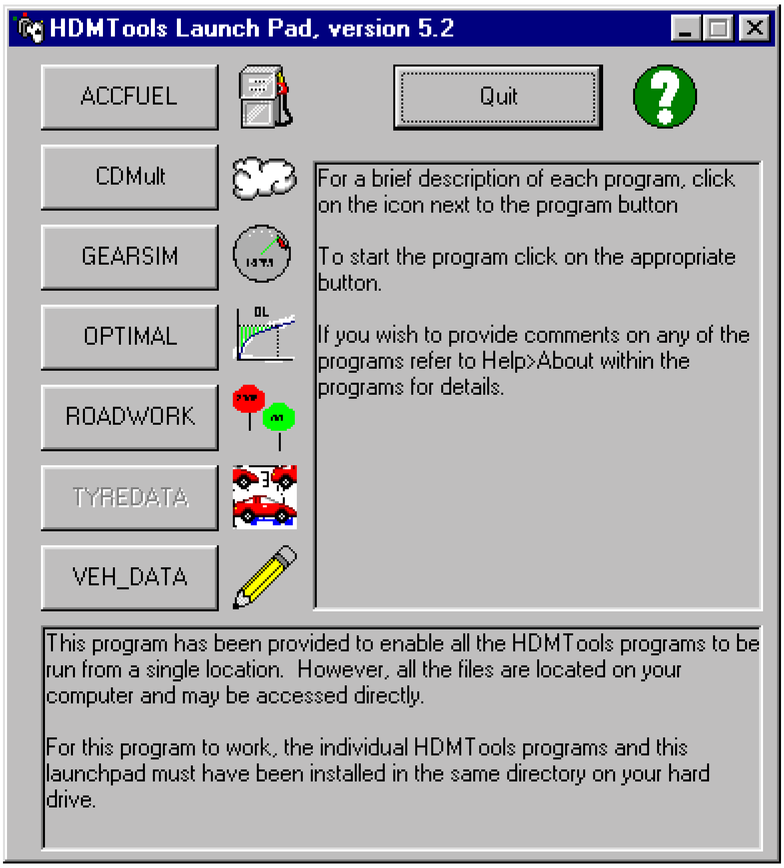


Figure K.1 HDM Tools Launch Pad

The use of these individual applications is discussed in the following sections.

K.3.1 Vehicle data

Since the HDM Tools software is run independently of HDM-4, HDM Tools comes with its own vehicle database. This is the same database used with the HDM-4 standalone RUE model.

The data can be exported from HDM-4 using the Vehicle Fleet (VF) export function and, when copied to the HDM Tools Launch Pad directory, these data will be used as input to HDM Tools. The user can alter these data, and Figure K.2 shows the data entry screen presented when the VEH_DATA button is selected from the Launch Pad.

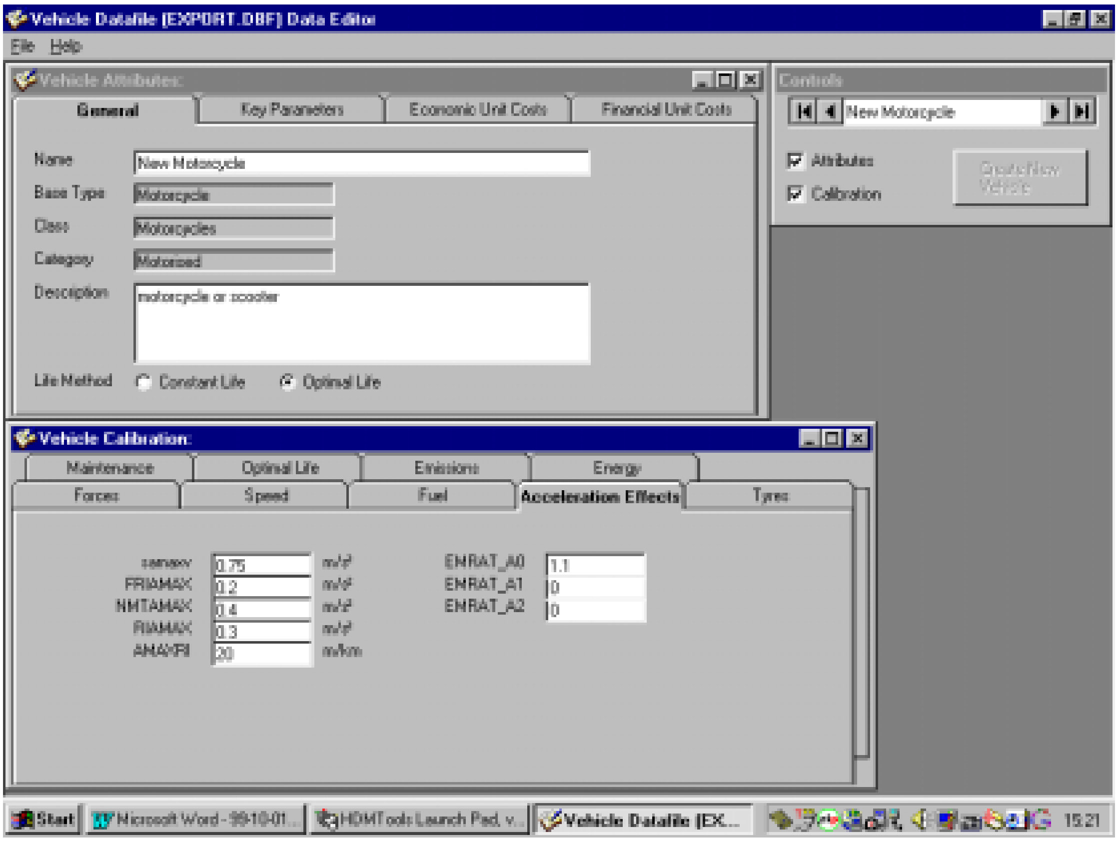


Figure K.2 Vehicle database entry screen

K.3.2 Fuel-acceleration simulation

The Fuel-Acceleration Simulation (ACCFUEL) Tool is used to generate data on the effects of accelerations on fuel and tyre consumption. As discussed in Section 6.4.5, these accelerations may arise due to traffic interactions (that is, congestion), the road alignment, roughness, and non-motorised traffic or side friction.

The acceleration is specified in HDM-4 as the **acceleration noise**. This is the standard deviation of acceleration. This parameter has been adopted since the mean acceleration over a homogeneous section will always be 0. However, the additional fuel and tyre consumption are proportional to the **magnitude** of the accelerations and the higher the acceleration noise, the higher the magnitude of the accelerations and thus, the higher the fuel and tyre consumption.

There are two modes of operation:

1 Simulation of a vehicle under a range of speed and acceleration noise levels; or,

2 Simulation of a vehicle under a user defined speed cycle.

The first option generates a large matrix of results that enable the HDM-4 software to increase the calculated steady speed fuel consumption and tyre wear values to account for acceleration effects. Simulating a vehicle travelling along a section of road with different initial speeds experiencing varying levels of acceleration does this. The additional fuel for each of these speeds and levels of acceleration noise is calculated and stored as a matrix. This matrix is read by HDM-4 and the additional fuel due to accelerations is applied in the analyses.

The second option enables for comparison of simulated versus observed fuel consumption data such that the input vehicle parameters may be adjusted to yield the best match. Here, the user defines a speed cycle and the software predicts the fuel consumption associated with travelling that cycle. Comparing this to known fuel consumption enables key parameters to be calibrated.

To run ACCFUEL the user supplies the number of vehicles to simulate and the minimum length of road that each vehicle should travel over. It is recommended that a minimum of 250 vehicles travelling 5-10 km be simulated. For each mean initial speed, vehicles are simulated travelling with acceleration noises from 0.05 to 1.00 in 0.05 m/s^2 increments. This results in a matrix as illustrated in Figure K.3 and Figure K.4 for fuel and tyres respectively. The fuel data are also presented as Table K.2.

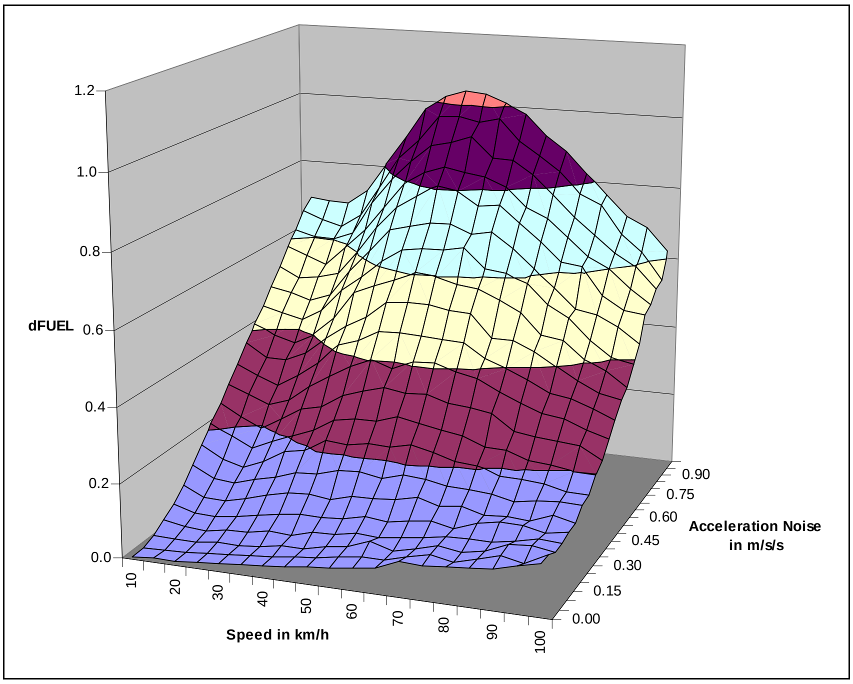


Figure K.3 Additional fuel due to accelerations - medium truck

While the values show consistent trends, the fuel results show the inherent variations that arise with Monte Carlo simulations. The tyre results do not have these irregularities due to the way in which the tyre consumption is calculated (*Bennett and Greenwood, 1999*). The following points will be noted in the figures:

- The highest additional fuel consumption arises in the area of the minimum fuel consumption. This is most noticeable with heavier vehicles.
- There is a significant increase in congestion effects with heavier vehicles over light vehicles for the same speed and acceleration noise. This is because of their higher mass and, thus, inertial resistance.
- There is only a very limited effect of congestion on motorcycles - speed has a much more significant impact. This is because motorcycles have such a low mass that the inertial effects on fuel are minimal.

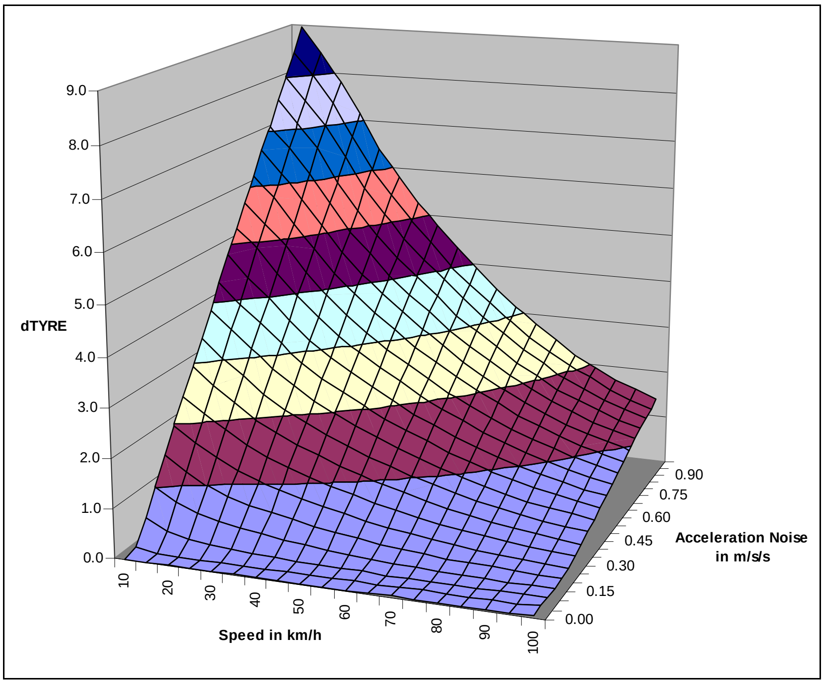


Figure K.4 Additional tyre consumption due to accelerations – medium truck

When generating a matrix there are two output files per vehicle, one for fuel consumption and one for tyre wear. Each file has the format of vehicle speed down the rows and acceleration noise across the columns. The name of the files is dfuel.*xx* and dtyre.*xx*, where *xx* is the vehicle number within the vehicle database.

If you run the **Combine files** option from the **File** menu, then the program will combine all the various files into two large files ready for inclusion into HDM-4. These files are called dfuel.out and dtyre.out.

To simulate a known speed profile, the user must supply a text file with the vehicle number to simulate followed by rows containing the time (seconds), velocity (m/s), speed (km/h), acceleration (m/s/s), distance (m). An example of this is:

3				
1	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0
4	0.0	0.1	0.0	0.0
5	0.4	1.6	0.4	0.5
6	1.1	4.0	0.7	1.6
7	1.7	6.0	0.5	3.3
8	1.9	7.0	0.3	5.2
9	2.0	7.1	0.0	7.2
10	2.2	8.0	0.2	9.4

This table is then used in the analysis and then a single output file is created with the same name as the input file but the extension **.OUT**. This contains all the input data plus two extra columns, the first being the instantaneous fuel consumption (ml/s) and the second the cumulative fuel consumption (ml).

Table K.2
Additional fuel due to accelerations – medium truck

Speed (km/h)	Additional fuel due to acceleration (dFUEL) by total acceleration noise																				
	0.00	**0.05**	**0.10**	**0.15**	**0.20**	**0.25**	**0.30**	**0.35**	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**	**0.65**	**0.70**	**0.75**	**0.80**	**0.85**	**0.90**	**0.95**	**1.00**
10	0.007	0.009	0.020	0.042	0.065	0.098	0.134	0.171	0.212	0.252	0.289	0.328	0.373	0.413	0.452	0.493	0.535	0.582	0.620	0.662	0.689
15	0.011	0.012	0.020	0.030	0.046	0.069	0.102	0.138	0.183	0.220	0.266	0.303	0.339	0.388	0.437	0.480	0.510	0.562	0.602	0.640	0.684
20	0.012	0.013	0.019	0.028	0.045	0.069	0.097	0.122	0.156	0.195	0.245	0.281	0.339	0.371	0.422	0.464	0.517	0.538	0.603	0.624	0.682
25	0.016	0.017	0.025	0.037	0.055	0.082	0.115	0.144	0.182	0.228	0.262	0.315	0.364	0.401	0.455	0.486	0.532	0.593	0.625	0.665	0.726
30	0.023	0.022	0.025	0.041	0.063	0.098	0.127	0.179	0.205	0.263	0.307	0.359	0.410	0.453	0.520	0.559	0.600	0.651	0.717	0.752	0.809
35	0.026	0.029	0.029	0.052	0.074	0.113	0.147	0.191	0.229	0.274	0.342	0.415	0.454	0.532	0.562	0.621	0.678	0.722	0.790	0.842	0.892
40	0.032	0.035	0.040	0.045	0.084	0.118	0.154	0.227	0.263	0.315	0.378	0.435	0.484	0.556	0.614	0.687	0.740	0.794	0.846	0.896	0.978
45	0.039	0.041	0.041	0.063	0.083	0.109	0.170	0.220	0.272	0.332	0.403	0.465	0.529	0.578	0.651	0.704	0.777	0.832	0.897	0.969	1.017
50	0.045	0.048	0.054	0.065	0.086	0.127	0.172	0.232	0.273	0.349	0.383	0.468	0.535	0.605	0.659	0.717	0.809	0.851	0.909	0.972	1.035
55	0.052	0.058	0.062	0.076	0.100	0.128	0.162	0.228	0.293	0.338	0.410	0.463	0.535	0.599	0.660	0.720	0.799	0.840	0.929	0.960	1.031
60	0.061	0.067	0.085	0.074	0.094	0.132	0.179	0.242	0.280	0.344	0.411	0.459	0.527	0.583	0.671	0.694	0.761	0.832	0.872	0.974	1.008
65	0.069	0.066	0.067	0.082	0.103	0.145	0.175	0.227	0.270	0.324	0.393	0.456	0.521	0.574	0.618	0.707	0.769	0.808	0.871	0.913	0.979
70	0.093	0.084	0.082	0.089	0.112	0.130	0.164	0.211	0.261	0.320	0.380	0.421	0.459	0.548	0.608	0.663	0.722	0.754	0.833	0.877	0.923
75	0.090	0.096	0.099	0.101	0.100	0.137	0.161	0.206	0.237	0.311	0.352	0.398	0.458	0.520	0.587	0.614	0.673	0.728	0.781	0.837	0.881
80	0.095	0.086	0.090	0.102	0.121	0.121	0.158	0.185	0.236	0.268	0.327	0.373	0.433	0.464	0.532	0.565	0.624	0.677	0.722	0.778	0.822
85	0.102	0.103	0.100	0.101	0.132	0.126	0.145	0.183	0.216	0.259	0.306	0.344	0.399	0.453	0.501	0.511	0.614	0.629	0.676	0.749	0.765
90	0.107	0.120	0.124	0.111	0.132	0.123	0.153	0.178	0.210	0.254	0.298	0.335	0.365	0.408	0.445	0.500	0.537	0.600	0.629	0.669	0.710
95	0.124	0.108	0.131	0.125	0.118	0.144	0.159	0.166	0.199	0.250	0.273	0.312	0.338	0.382	0.404	0.472	0.504	0.547	0.596	0.625	0.682
100	0.138	0.137	0.120	0.121	0.129	0.131	0.159	0.163	0.198	0.217	0.256	0.289	0.326	0.341	0.391	0.430	0.497	0.508	0.539	0.549	0.617

K.3.3 Optimal life calibration

A full description of the Optimal Life (OL) calibration is found in *Bennett and Greenwood (1999)*. The program is used to develop a table of the effects of roughness on the OL for each representative vehicle. In HDM-4 the standard equation for applying the OL method is:

$$\text{LIFEKMPCT} = \text{MIN}\left[100, \frac{100}{1 + \text{EXP}\left(a0\,\text{RI}^{a1}\right)}\right] \qquad \ldots(\text{K.1})$$

where:

LIFEKMPCT is the lifetime kilometreage as a percentage

a0 and a1 are regression coefficients

Figure K.5 shows the input screen for the program. The program uses the following data from the VEHICLES data file:

- **Replacement value of the vehicle**
- **Cost of maintenance labour**, and
- **Lifetime utilisation** (km)

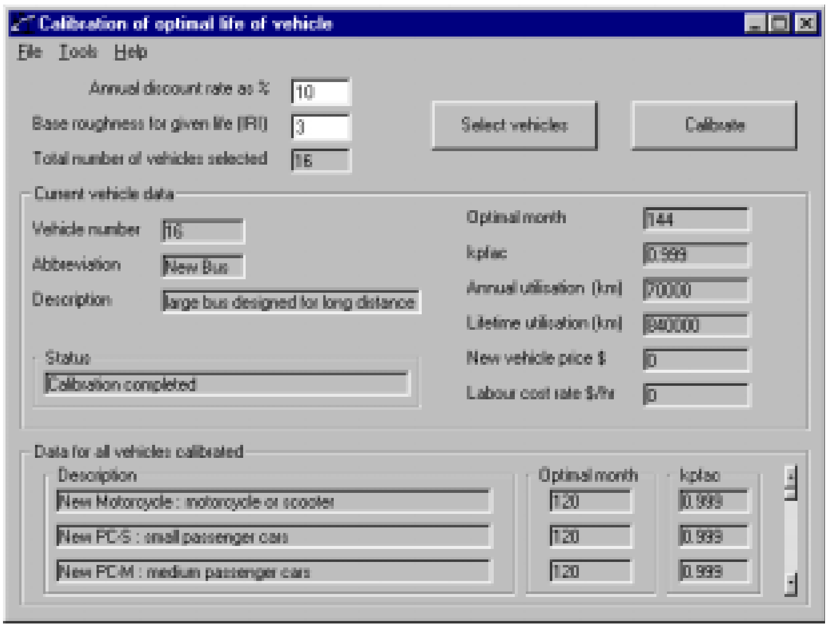


Figure K.5 Optimal life simulation

To run the program it is necessary to supply a base roughness level. This is the roughness at which the lifetime utilisation applies. For example, if the average roughness on the network is 6 IRI m/km and the average lifetime utilisation is 100,000 km you would supply the value of 6. However, if you estimated that on smooth roads the lifetime utilisation would be 200,000 km and this value is entered in the data file, you would enter 3 IRI m/km for the roughness.

The user selects the vehicles to analyse and the optimal life at different roughnesses is generated. Varying the parts consumption coefficient kp based on the user-supplied values and this roughness level does this. Once it has the value of kp that matches the optimal life information, it then calculates the life at a roughness of the 3 IRI m/km (which if the base roughness is 3 equals the lifetime utilisation). It then goes on to calculate the percentage of life at various roughness levels relative to the life at a roughness of 3 IRI m/km (that is, not the supplied lifetime utilisation).

Table K.3 is an example of the program output.

Table K.3
Optimal life analysis output

Representative vehicle	Roughness (IRI m/km)	Optimal life as a percentage of baseline life
1	3.0	97.1
1	3.5	89.6
1	4.0	83.3
1	4.5	77.9
1	5.0	73.3
1	5.5	69.6
1	6.0	65.8
1	6.5	62.9
1	7.0	60.0
1	7.5	57.5
1	8.0	55.4
1	8.5	53.3
1	9.0	51.3
1	9.5	49.6
1	10.0	47.9

The data from OPTIMAL need to be analysed to quantify the coefficients a0 and a1. This is done with the non-linear regression program supplied with HDM Tools through the script file illustrated in Figure K.6.

```
Title "Optimal Life Analysis";
Variable RI; // Roughness in IRI m/km
Variable Lifekm; // Optimal Life in km
Parameter Lifekm_a0; // Coefficient a0
Parameter Lifekm_a1; // Coefficient a1
Function Lifekm = min(100,100/(1 + EXP(lifekm_a0 * RI ^ lifekm_a1)));
Plot xlabel="Roughness in IRI m/km",ylabel="Optimal Life as Percentage";
Data;
3.0     97.1
3.5     89.6
4.0     83.3
4.5     77.9
5.0     73.3
5.5     69.6
6.0     65.8
6.5     62.9
7.0     60.0
7.5     57.5
8.0     55.4
8.5     53.3
9.0     51.3
9.5     49.6
10.0    47.9
```

Figure K.6 Script file for fitting HDM-4 OL equation to raw data

K.3.4 Engine speed simulation

The engine speed simulation program is used to generate a matrix of engine speed versus road speed. These data are then analysed to establish the following polynomial equation. The coefficients are supplied to HDM-4[1].

$$RPM = a0 + a1SP + a2\,SP^2 + a3\,SP^3 \qquad \ldots(K.2)$$

$$SP = MAX\,(20, SPEED) \qquad \ldots(K.3)$$

where:

RPM is the engine speed (revolutions per minute)

SPEED is the road speed (km/h)

Figure K.7 shows the input screen for the program. The user defines the gear ratio data, which are readily available from manufacturer's specifications, along with certain other key attributes. *Bennett and Greenwood (1999)* discuss these values in detail. Selecting **calibrate** the program then applies the simulation logic described in *Bennett and Greenwood (1999)* to generate a table of the mean engine speed and mean effective mass ratio as a function of road speed.

[1] *NDLI (1995a)* proposed a different formulation based on three zones but as shown in *Bennett and Greenwood (1999)*, the above polynomial formulation gives more than adequate results.

Table K.4 is an example of this output.

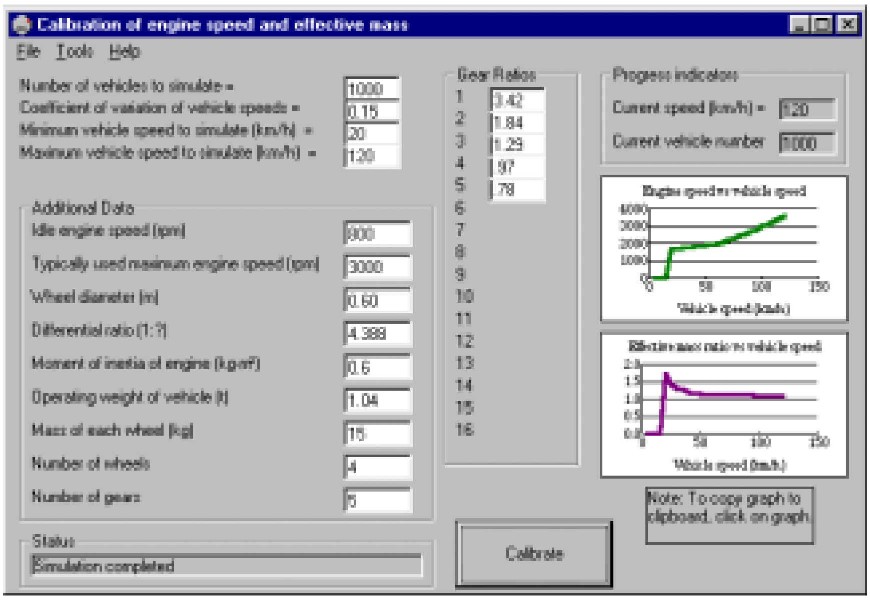


Figure K.7 Engine speed simulation

K.3.5 Wind adjusted aerodynamic drag coefficient

The basis for the wind averaged drag coefficient (CDmult) is described in *Bennett and Greenwood (1999).*

CDmult is calculated for angles of χ between 0 and 359 degrees, in intervals of 1 degree. From the 360 values of yaw angle, the value for CDmult and relative velocity are calculated for each angle. The average[1] of these values was then divided by the square of vehicle speed to obtain the value for CDmult. The HDM Tools database is then updated with this value, which in turn can be imported to HDM-4 to update its vehicles database. Figure K.8 shows the input data screen for this application.

[1] The weight of the product at a wind angle of 0 and 180 degrees was halved to account for the full 360 degrees of possible wind angles.

Table K.4
Engine speed simulation results

Road speed (km/h)	Engine speed (RPM)	Effective mass ratio	Road speed (km/h)	Engine speed (RPM)	Effective mass ratio
20	1693	1.719	75	2347	1.114
25	1677	1.433	80	2462	1.111
30	1739	1.331	85	2592	1.109
35	1783	1.271	90	2722	1.108
40	1809	1.219	95	2850	1.108
45	1843	1.183	100	3029	1.108
50	1913	1.160	105	3171	1.108
55	1954	1.143	110	3324	1.107
60	2017	1.130	115	3466	1.107
65	2134	1.124	120	3618	1.107
70	2230	1.117			

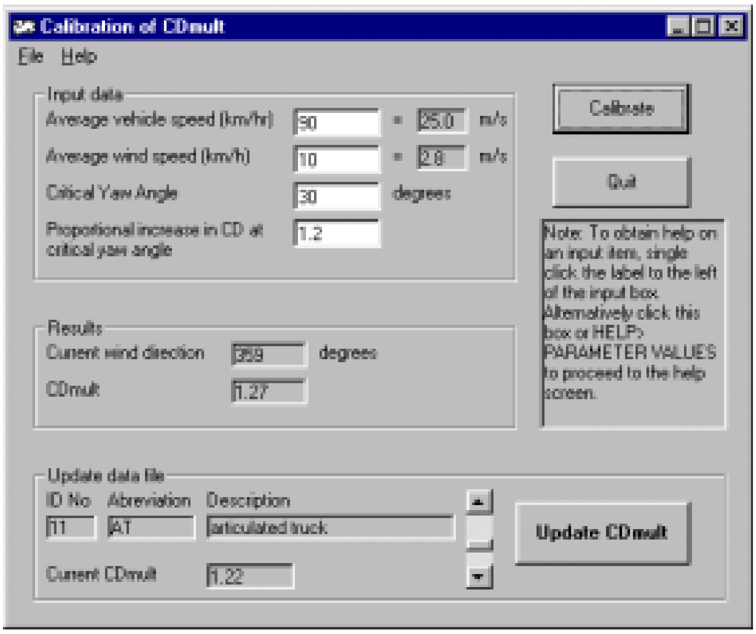


Figure K.8 HDM Tools CDmult calibration program screen

There are four data items required:

- **Average vehicle speed**;
- **Average wind speed**;
- **Critical yaw angle** (*ψc*); and,
- **Proportional increase in CD at the critical yaw angle** (*h*).

It is recommended that ψc be assumed to be a constant value of 30° for all vehicle classes. The vehicle and wind speeds will vary depending on applications. As shown in Table K.5, the value for *h* varies between vehicle classes.

Table K.5
Aerodynamic resistance default parameters

Vehicle number	Type	h	CDmult[1]	CD	AF (m^2)
1	Motorcycle	0.4	1.12	0.70	0.8
2	Small Car	0.4	1.12	0.40	1.8
3	Medium Car	0.4	1.12	0.42	1.9
4	Large Car	0.4	1.12	0.45	2.0
5	Light Delivery Vehicle	0.5	1.16	0.50	2.9
6	Light Goods Vehicle	0.5	1.16	0.50	2.8
7	Four Wheel Drive	0.5	1.16	0.50	2.8
8	Light Truck	0.6	1.19	0.55	4.0
9	Medium Truck	0.6	1.19	0.60	5.0
10	Heavy Truck	0.7	1.22	0.70	8.5
11	Articulated Truck	1.2	1.38	0.80	9.0
12	Mini-bus	0.5	1.16	0.50	2.9
13	Light Bus	0.6	1.19	0.50	4.0
14	Medium Bus	0.7	1.22	0.55	5.0
15	Heavy Bus	0.7	1.22	0.65	6.5
16	Coach	0.7	1.22	0.65	6.5

Source: *Bennett and Greenwood (1999)*

Notes:

1 Calculated assuming wind speed of 14.4 km/h and vehicle speed of 75 km/h

K.3.6 Work zone simulation model

Work zones are modelled using the program ROADWORK (*Bennett and Greenwood, 1999*). This program calculates the additional delay and fuel due to traffic being interrupted for a work zone.

The program works as follows. The user supplies:

- A range of values for the work zone capacity, work zone length, work zone speed and AADT.
- An hourly flow distribution over the 24-hour period.

For each AADT level and the hourly flow distribution, the program generates the arrivals at the work zone. These arrivals will apply irrespective of the work zone capacity or length.

For each combination of work zone capacity, work zone length and work zone speed the program will calculate the average delay and queue size, in two directions.

The WORKZONE simulation model is capable of analysing the impact of road closures on road users. The user inputs the configuration of the lane closure from any of the following:

- **Lane closure -** typically motorway situations
- **Directional closure -** two lane road situation
- **Complete closure -** complete blockage of road

The start-up screen for the software is illustrated in Figure K.9. The first tab is used to execute the analysis while the other tabs are used for data.

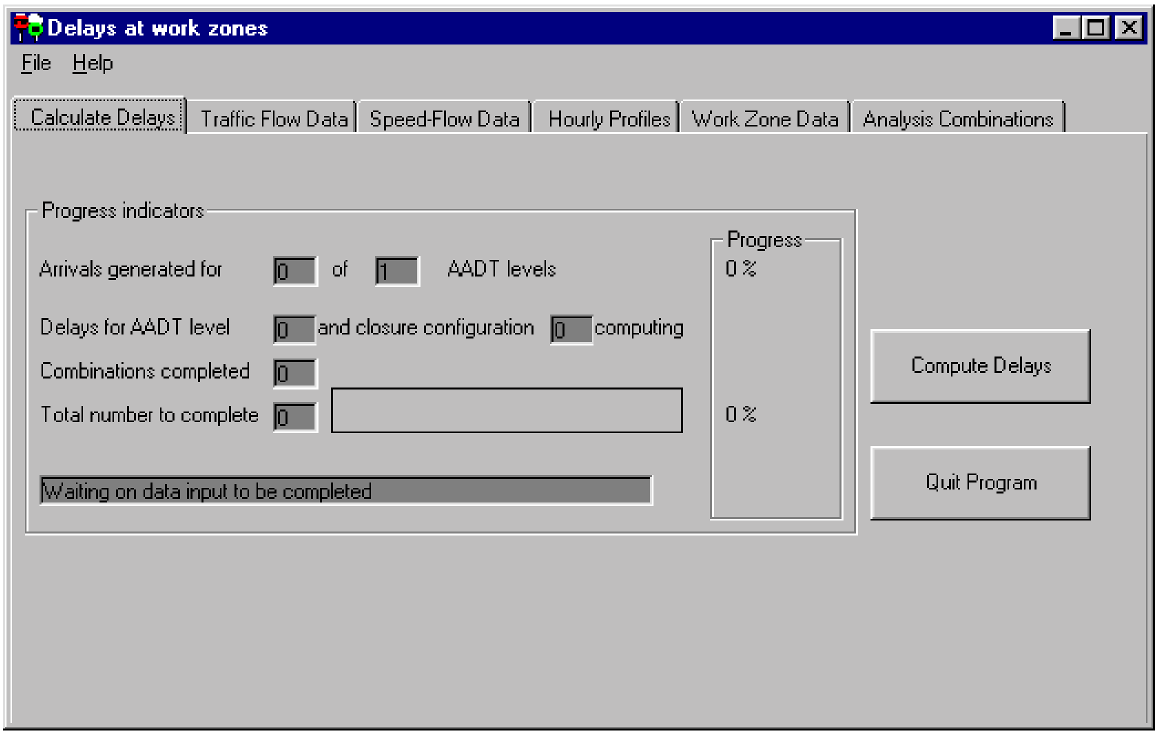


Figure K.9 Screenshot of Start-up screen to the Work Zone simulation model

As described in *Bennett and Greenwood (1999)*, WORKZONE uses simulation to generate arrival times of vehicles based on a headway distribution model, and then using the available capacity of the work zone, determines when the vehicle will be able to depart. Based on this information, and the configuration of the work zone, the model then produces estimates of travel times and queue lengths.

To use the program, information needs to be supplied on the following items:

- **Daily traffic flow**, directional split, 24-hour profile

- **Normal lane configuration**
- **Speed-flow model data**
- **Type and duration of closure**

Figure K.10 shows the traffic flow data. Instead of defining the number of individual vehicles of each class to simulate, the program works in terms of an aggregate number of vehicles. Accordingly, it is necessary to provide the average PCSE for each vehicle. This should be based on the PCSE values for individual vehicles, weighted based on their frequency in the traffic stream.

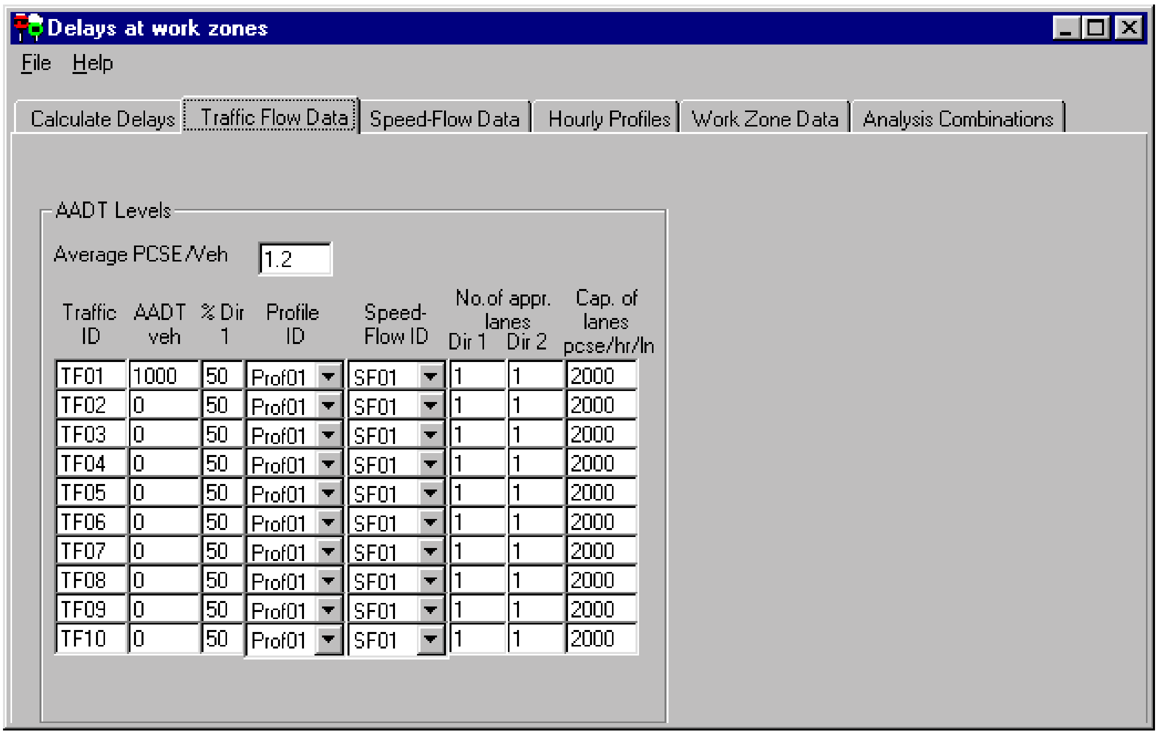


Figure K.10 Screenshot of Traffic Flow Information

The user must define speed-flow data, which is based on the HDM speed-flow model presented in Chapter 6. The hourly flow profiles give the variation in flow over the course of the day. Both of these data are used to create a unique traffic ID that consists of a volume, speed-flow profile, hourly flow profile, and a capacity for the site.

As shown in Figure K.11, the user must define the work zone closure type, the length of the work zone, the maximum speed through the work zone, the work zone capacity, and the number of lanes in each direction. If the capacity will vary over a day (for example if the road will be closed for periods of time) this can also be specified.

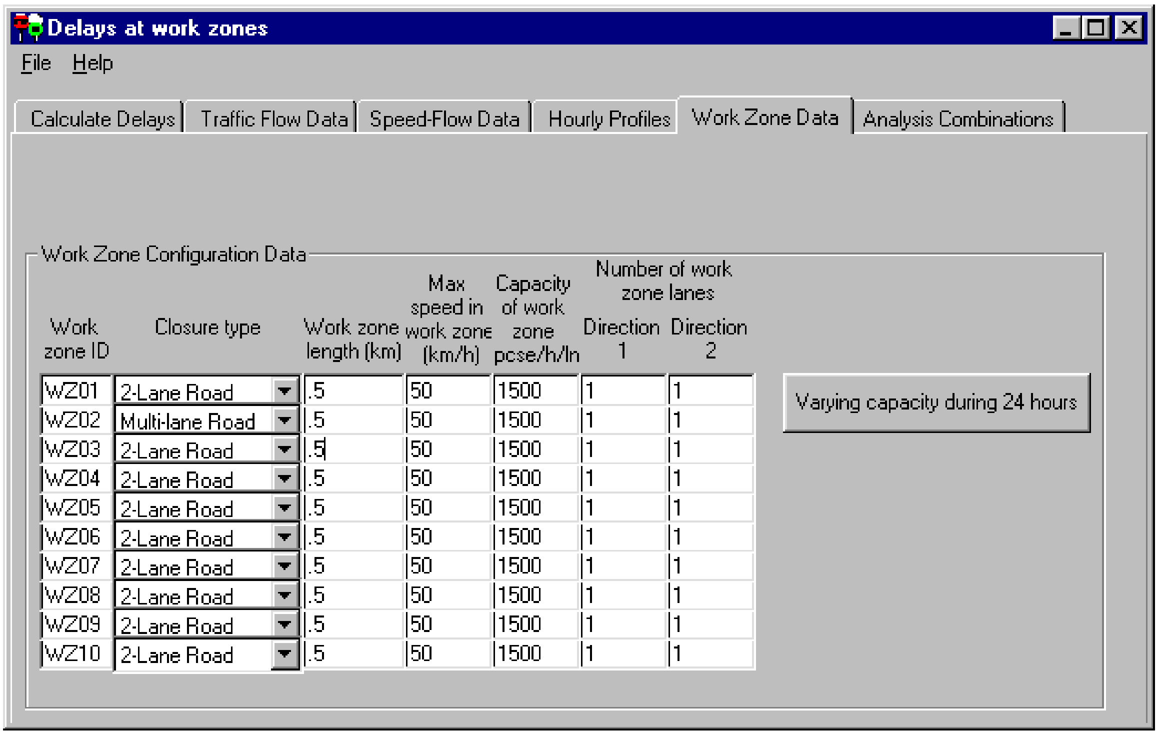


Figure K.11 Screenshot of Closure Configuration Screen

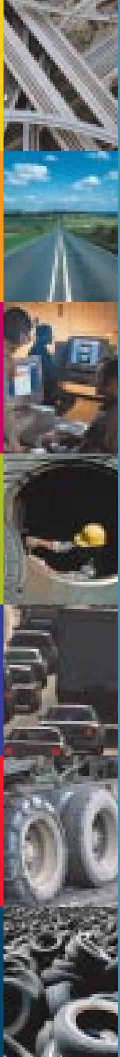
Once all the input data have been defined the program is used to calculate the delays. The output from the program is in a format suitable for performing a regression analysis to determine various model coefficients for input to the HDM-4 road user effects model or to apply as an exogenous cost.

K.4 Pavement deterioration and maintenance effects

To be included in a subsequent edition of this document

K.5 Analysis

The non-linear regression program is NLREG. This is a shareware program and you are expected to register it with the author should you decide to use the program. Registration details are provided with the software.



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These HDM-4 products have been produced by the International Study of Highway Development and Management Tools (ISOHDM), sponsored by The World Bank, the Asian Development Bank, the Department for International Development (UK), the Swedish National Road Administration, and other sponsors. HDM-4 is jointly published by The World Road Association (PIARC), Paris, and The World Bank, Washington, D.C.

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