Road Deterioration and Maintenance Effects
ROAD DETERIORATION AND MAINTENANCE EFFECTS
MODELS FOR PLANNING AND MANAGEMENT

ERRATA

1. Page 39 Chart (d): the equation should read MO = IRI/1.5.
2. Page 103 In caption (b): "traffic" should read "terrain".
3. Page 128 Under Brazil: modified structural number range should read 1.5 - 8.7.
4. Page 203 In equation 5.41: a "}" should be added at the end of the equation.
5. Page 207 Equation for parameter "a" in 1 should read 375 SNC$^{2,17}$.
6. Page 304 In equation 8.20: the coefficient 725 should read 72.5.
7. Page 307 Captions should read "(a) Pavements with Modified Structural Number 5", and "(b) Pavements with Modified Structural Number 7".
10. Page 388 Table 10.6: in item 4, potholing, line 4, the factor (1 - CQ) should read (1 + CQ).
11. Page 391 In equation 8.13: the coefficient, 0.000758 should read 0.0000758.

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Road Deterioration and Maintenance Effects
Models for Planning and Management

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Foreword

An effective road transportation network is an important factor in economic and social development. It is also costly. Road construction and maintenance consume a large proportion of the national budget, while the costs borne by the road-using public for vehicle operation and depreciation are even greater. It is therefore vitally important that policies be pursued which, within financial and other constraints, minimize total transport costs for the individual road links and for the road network as a whole. To do this meaningfully, particularly when dealing with large and diverse road networks, alternatives must be compared and the trade-offs between them carefully assessed. This in turn requires the ability to quantify and predict performance and cost functions for the desired period of analysis.

Because of the need for such quantitative functions, the World Bank initiated a study in 1969 that later became a large-scale program of collaborative research with leading research institutions and road agencies in several countries. The Highway Design and Maintenance Standards Study (HDM) has focused both on the rigorous empirical quantification of the trade-offs between the costs of road construction, road maintenance, and vehicle operation, and on the development of planning models incorporating total life-cycle cost simulation as a basis for highway decision making.

This volume is one in a series that documents the results of the HDM study. The other volumes are:

Vehicle Operating Costs
Evidence from Developing Countries

Vehicle Speeds and Operating Costs
Models for Road Planning and Management

The Highway Design and Maintenance Standards Model
Volume 1. Description of the HDM-III Model

The Highway Design and Maintenance Standards Model
Volume 2. User's Manual for the HDM-III Model

The performance of road pavement and the effects of maintenance are important parameters of the cost of road transport; both determine the direct outlays of the highway authority and affect the operating cost of vehicles plying the roads. Although pavement engineering and design is by now an old art, the specific pavement performance and response to maintenance are less well known, at least in quantitative terms. This is particularly true for roads in developing countries, with their diverse physical and climatic environments, construction methods, and traffic characteristics.

This volume presents the results of a methodical series of analyses on an important data set from Brazil. The data were scant in many respects, and, because of the nature of the phenomena investigated, often exhibited large scatter. Through a judicious combination of theory, empiricism, statistical finesse, and engineering judgment, the author has been able to establish important relationships. Of particular significance is the establishment of causality of events: a pavement starts to crack and to ravel (in a random fashion, after a few years of service); the cracking then increases in extent and intensity; this leads to potholing and other surface disfigurement, which together with rutting, leads to increased roughness—the principal parameter affecting vehicle operating costs.

Although the relationships described in this volume form part of the HDM-III model, they can also be used on their own. They have a great didactic value and help to explain pavement performance phenomena. As engineering tools, they take on importance in the technical, financial, and economic fields, such as in the life-cycle costing of roads, road pricing and regulation policies, pavement management, and verification of design methodologies.

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Principal Transport Economist

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Highways Adviser
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The Government of Brazil, the United Nations Development Programme, and the World Bank sponsored the major research project entitled "Research on the Interrelationships between Costs of Highway Construction, Maintenance and Utilization" (PICR) which provided the primary database for the study. The experimental design, large data collection effort, and initial analyses were undertaken by the Brazilian Transport Planning Agency (GEIPOP) with an international research team, in collaboration with the Texas Research and Development Foundation (TRDF) and the World Bank. The extent and depth of the analyses reported here were made possible by the high calibre of this effort, particularly in respect to the experimental design and the calibration of road condition monitoring which made it possible to achieve a high degree of transferability of the results to other countries and conditions. Of the many individuals from the PICR project who gave invaluable assistance and warm hospitality at GEIPOP in Brasilia, I especially thank Jose Teixeira, Teodoro Lustosa, Maria Isabel P. Machado, Ildeu L. Martins, Libero G. Martins, Marcio L. Paiva, Stanley Buller, and Paulo Roberto Rezende Lima for willing and excellent support. The information and comments from project staff, W. Ronald Hudson, Bertell Butler, Robert Harrison, Cesar Queiroz, and Alex Visser were greatly appreciated. The cooperation of many of these was also fundamental in realizing the International Road Roughness Experiment, hosted by GEIPOP in June 1982, which was greatly facilitated by the roughness measurement practices established in the PICR project. Thomas Gillespie and Michael Sayers (University of Michigan) illuminated many roughness issues and were instrumental in our achievement of the International Roughness Index (IRI) standard.

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The dedication is to my wife, Rosalind, and children, Matthew, Megan, Simon, and Emily, who have been faithfully encouraging and understanding throughout.
CHAPTER 1

The Need to Predict Deterioration

A recent study estimates that a third of the unpaved roads and a quarter of the paved roads outside urban areas in developing countries are in need of reconstruction, and that a further forty percent of paved roads require strengthening now or in the next few years (World Bank 1987a). The study also estimates that much of the $90 billion cost of this work could have been saved by spending $12 billion on earlier preventive work. The crisis has reached such dimensions, it is asserted, in part because the rate of deterioration of roads is not immediately evident. New paved roads deteriorate very slowly and almost imperceptibly in the first ten to fifteen years of their life, and then deteriorate much more rapidly unless timely maintenance is undertaken. The study goes on to evaluate a wide range of technical options, maintenance strategies and standards with their associated economic consequences.

Similar situations have been arising in industrialized countries to varying degrees. For example, the accelerating deterioration of federally-aided highways in the United States required a forty-four percent increase in funding in 1982 to $13 billion annually to meet the repair and rehabilitation costs of the system through to 1990. Extensive rehabilitation programs have been planned also in most European countries.

Such projections on international and national scales exemplify the problems facing highway planners, financiers, managers and engineers everywhere at national or local levels, and to varying degrees. The problem concerns deterioration of an aging road infrastructure and how best to control it, taking into account the best interests and constraints of the economy and resources.

Largely because of the worldwide need for extensive rehabilitation programs in the 1980s and 1990s and in order to avoid such sharp bunching in the demands for highway expenditures, increasing efforts are being made to develop and implement improved road management and planning tools. These tools are required for evaluating the allocation of financial resources in the highway sector, for estimating the timing and financial needs of the road maintenance and rehabilitation programs, for evaluating the design and maintenance standards appropriate for the funding available to the highway sector, and for planning and prioritizing works in a program. Tools are also needed for evaluating the costs of road use as a basis for pricing and taxation in the transport sector.

All such projections and evaluations depend upon predictions of the rate at which roads in the network will deteriorate and of the effectiveness of different maintenance options, dependent on the current state and projected trends of traffic, economic growth and available resources. At the heart is a model of road deterioration, which may be as simple as a fixed estimate of life (for example, "paved roads need rehabilitation every twenty years") or may be more complex, for example, taking into account the traffic projections, existing road structure, and specific standards of service and design.
The increasing demands for improved management and planning techniques, and for economic justification of expenditures and standards in the highway sector, are placing much more exacting requirements on models of road deterioration. This book addresses those demands, examining the specific needs and the adequacy of current models, and focusing on the development of a major new set of predictive relationships for both unpaved and paved roads.

1.1 MAJOR DEMANDS

The demands involve not only the decisions to be made and questions to be evaluated, but also the data on which predictions are to be based. Reliable statistics of the road network, traffic, and condition of the road infrastructure are costly to collect and maintain, and vary widely in scope and quality from country to country. The need for reliable data and systematic monitoring of the network was identified as urgent in the study of developing countries cited earlier. Without adequate data, the road needs cannot be quantified or evaluated accurately, and planning decisions tend to become short-term and crisis-oriented. It is important, therefore, to identify which parameters are essential and relevant for predictive models, and this issue clearly involves tradeoffs between the accuracy of predictions and the cost and amount of data required.

These modelling and data needs are best identified by examining four major applications of predictive models.

1.1.1 Planning, Policy and Standards

Forecasting the needs of the road network over a planning horizon requires the ability to assess both the current and future condition and the demands on the network. It also requires a coherent set of maintenance standards and a maintenance/construction policy, which together determine the level of service to be provided and the funding required to support that level.

Predominantly in the past, such maintenance standards and policies have been established on primarily technical, political and fiscal grounds. Pavement design standards, surface quality and maintenance strategies have been geared to perceived standards of service such as riding comfort, skid resistance, and appearance, and technical considerations such as durability and service-life, and have been selected on the grounds of works costs. Economic efficiency, however, suggests that tradeoffs should be made between the costs of alternative strategies and the economic return that comes from reduced user costs and lower total transportation costs. In other words, maintenance standards and policies, and the associated budget requirements, should be economically justifiable.

Such questions as "At what stage should the pavement be resurfaced (or strengthened)?," "What should be the design life (hence thickness)?," "Which project has priority?," "When should unpaved roads be upgraded?," etc. are economic questions, as well as technical, because they have impact on the economic returns and the financial requirements. Budget planning is thus as much a question of seeking appropriate, affordable standards and policies as it is of forecasting condition.

To evaluate such questions in economic terms, it is necessary to estimate the condition, cost, and benefit streams of different technical options over the life-cycle of the road. The approach is to minimize the total transportation
costs, including the road agency costs of construction and maintenance works, and the user costs of vehicle operation, congestion and accidents. For a given design and maintenance strategy, the estimation of road costs requires knowledge of the present condition and prediction of the deterioration and maintenance effects on the future condition over the life-cycle. The estimation of user costs requires not only the average operating and accident costs, but also how those costs vary with road characteristics and with road condition over time in response to the maintenance strategy. The net present value of benefits is the saving in discounted user costs due to implementing the given strategy when compared to the minimum maintenance, or "null", strategy, which may be "do nothing" or may involve routine surface maintenance.

In analyses of total transportation costs, therefore, accuracy is required of condition forecasts on two grounds. First, it influences the timing and present value of future maintenance, and the ranking of alternatives. Second, it influences the economic benefits deriving from reductions of vehicle operating costs.

1.1.2 Pavement Management

Formal pavement management systems, of varying degrees of complexity, are being applied at regional or national levels to improve the planning and effectiveness of maintenance works and expenditures. In the widest sense they include the planning systems just described, but most examples in practice are directed simply at programming works in accordance with preset standards or budget. The two basic elements are:

1. An information system, comprising a database of network inventory, current and historical pavement condition, traffic volume and loadings, maintenance works, and regular monitoring of the network to update the data; and

2. A decision-support system, which analyses the data, and identifies current and future needs either according to prescribed criteria (screening process) or by the ranking of alternatives (prioritization).

Decision systems which rely on prescribed standards (intervention criteria and treatments), and which essentially only screen the network to identify current needs, only require deterioration models for prioritizing works when the funds are inadequate.

The majority of systems, however, use predictive models to forecast future road conditions, the timing and type of maintenance needs, and the consequences of deferring maintenance. Some use simple extrapolation models based on the historical trend of condition established in past regular condition surveys. Some use basic correlative models from whatever local performance data are available. In either case, the reliability of such models is low until a considerable history of data has been established. Furthermore, the scope of the data base which is available on a local level is usually insufficient to establish all the predictive capability that is possible from major factorial studies. Thus, predictive models which have been derived from a broad empirical base, and which use the current condition and physical parameters to estimate deterioration, are extremely valuable because they are versatile, and relatively little effort is required to adapt them to local conditions.
1.1.3 Pricing and Taxation

Given the large sums of money spent on constructing and maintaining roads, many governments are seeking an appropriate basis for recovering the costs of supporting a road network from road users. Users' groups are equally concerned that road use charges be reasonable and allocated equitably among the various classes of vehicles. Recent major studies include the Cost Allocation Study of the Federal Highway Administration in the United States (Rauhut, Lytton and Darter 1984) and a World Bank Study on the pricing and taxing of transport fuels (Newbery and others 1988). The allocation of costs amongst various classes of road users involves two primary issues. First, what are the effects of vehicles and environment on road damage and repair costs? Second, on what basis should costs be determined and allocated?

The first issue involves the difficult task of developing causal relationships between the many vehicle, environmental and pavement factors which influence the amount of damage caused. The relative damage caused by different axle loadings, axle configurations, tire types, sizes and pressures, and suspension systems are significant vehicle-related issues. The amount of damage depends also on many pavement factors such as the pavement type, strength and condition. Roads deteriorate slowly in the early stage of their life and thereafter at an increasing rate so that the timeliness and level of maintenance may influence the amount of damage caused by a vehicle. Environmental factors such as climate, foundation movements, and oxidation of bituminous materials, all affect the damage caused in a way which is partly independent of vehicular use. Thus, the underlying form of predictive models of road deterioration must be rational, reflecting the physical parameters and processes that are relevant. Simple correlative models predicting the average trends of deterioration are inadequate for this purpose.

The second issue concerns the total cost to be recovered from users and whether user charges are equitable and efficient (see Newbery and others 1988). An equitable charge based on average cost allocates the total cost amongst users in proportion to the damage they cause, the "total cost" being interpreted variously as the current highway agency budget, or the long-term road costs of both construction and maintenance. An efficient charge is the marginal social cost incurred by the vehicle when it uses the road, and includes not only the extra damage done to the road, which advances the time when maintenance expenditure will be required, but also the externality cost of increased operating costs occasioned to other road users by the extra damage. The efficient charge should also be such that it encourages users to select the most efficient vehicle design so as to minimize road damage and the consequent costs. By its nature therefore the determination of an efficient charge requires reliable models to predict the damage caused by vehicles and non-vehicle factors, and to predict user costs as a function of road conditions. Often, the road use cost is less than the average cost and then efficient user charges do not result in full cost recovery. Any part of the difference which is applied is considered a tax instead of a charge, which may or may not be allocated in the same way as road use costs. Under certain conditions the average and marginal costs are equal (Newbery 1988).

1.1.4 Verification of Design Methodologies

Engineering methods for designing road pavements and analyzing the effects of vehicle loading and climate on pavement condition have developed consi-
derably due to the results of major road tests and the development of satisfactory theoretical, mechanistic techniques. However, feedback from the performance of roads under longterm service conditions is essential for verifying and improving the design methodologies, particularly when they have been based on either experimental studies (usually involving acceleration of the trafficking effects) or a combination of theoretical analysis and experimental study. The effects of mixed traffic and longterm environmental, non-traffic-related factors are especially important factors that are derived only from studies of in-service pavements.

The theoretical or mechanistic design methodologies provide a greatly enhanced capability for evaluating the effects of individual parameters on different modes of deterioration. Thus, they provide an important source for determining the rational form of empirical models. On the other hand, they generally do not predict roughness, or take account of aging effects and variability, and so need to be supplemented by empirical models. Thus, empirical studies of road performance, which form the basis of empirically-derived predictive models, serve a vital role in the validation of design methodologies.

1.2 HIGHWAY DESIGN AND MAINTENANCE STANDARDS STUDY

The total transport cost approach to life-cycle cost analysis has been developed and applied since 1969 through the World Bank's Highway Design and Maintenance Standards Study, of which this book is a part. The Bank's primary concerns have been to assist developing countries in determining how to allocate scarce financial resources in the highway sector to the best economic advantage, what budgets should be allocated, what standards were appropriate and affordable for the highway network, and how to prioritize and select highway projects.

The computer tool developed by the study is the "Highway Design and Maintenance Standards Model", now in its third version, HDM-III. The model simulates the trends of condition and costs over time under maintenance and design strategies specified by the user. The capabilities are outlined in World Bank (1987) and the model is fully described by Watanatada and others (1987a). Essentially, the model is for network-level analysis of policies and standards for both paved and unpaved roads, but it can as easily be applied to a single project, determining the economic internal rate of return, for example. For a number of road segments of different road and traffic characteristics and for several user-specified strategies, the model computes for each year of the analysis period:

1. The traffic by vehicle class and loadings (as specified);
2. The change in road condition (by prediction);
3. The maintenance or construction quantities, as required by the strategy being analysed;
4. The vehicle operating costs (by prediction);
5. The exogenous benefits and costs (delays, accidents, etc.) (as specified, not estimated); and
6. The total costs and quantities.
Finally, for the full analysis period, the model computes a number of economic criteria and provides various summary reports for individual strategies, links and the network.

Early in the Study, the knowledge available for estimating the effects of road condition and geometry on vehicle operating costs, and for predicting longterm pavement performance, was found to be lacking an adequate empirical foundation. Hence, several major field studies were undertaken, in cooperation with various national organizations, to gain a sound empirical base for the development of predictive functions. On vehicle operating costs, studies in Kenya, Brazil, the Caribbean, and India found strong effects of road roughness on operating costs, and resulted in sound relationships predicting the effects of roughness and geometric characteristics on vehicle speed and individual components of operating costs, such as fuel, tires, maintenance parts, and utilization (Chesher and Harrison, 1987; Watanatada, Dhareshwar and Rezende Lima 1987).

For road deterioration, the original model utilized the AASHO (American Association of State Highway Officials) model of pavement performance (Highway Research Board 1962), but the validity of this for the range of conditions in developing countries was considered limited by the freezing climate, high standard of pavements and accelerated nature of the experiment on which it was based. The field studies in Kenya (Hodges, Rolt and Jones 1975) and Brazil (GETPOT 1982) provided valuable data on the longterm performance of roads in service under mixed traffic, for both paved and unpaved roads and a more appropriate range of conditions, including thin pavements and a warm climate. The Kenya study was the basis for deterioration predictions in the British Road Transport Investment Model (RTIM2) (Parsley and Robinson 1982) and the second version of HDM. The Brazil study provided a still broader range of in-service conditions, forming the main basis for the deterioration predictions in the present version, HDM-III, which are the subject of this book.

1.3 THE ROLES OF EMPIRICAL AND MECHANISTIC METHODS

While much of the knowledge of pavement behavior historically has been based on theoretical considerations, ranging from the application of soil mechanics theory on the shear strength and behavior of pavement materials under load at the turn of the century to the detail of multilayer structural analysis techniques applied during the past two decades, empirical observations have always provided the basis for formulating the criteria to be applied in practice. The reasons for this are clear: the longterm behavior of natural and treated materials in a road under traffic and climate is influenced by numerous and complex factors and is highly variable, and the criteria for acceptable performance involve subjectively-determined limits of riding quality and other modes of distress.

Various road tests of full-scale pavements, including accelerated experiments such as the AASHO Road Test in Illinois (Highway Research Board 1962), and longterm monitoring of in-service highways in Britain (Lister and Powell 1987), have been the basis for developing relationships between pavement strength, axle loading and number of axle transits that have formed the foundation of pavement design methods throughout the world.

The large number of variables involved however strains the empirical method, and the capability to improve the structural efficiency of pavements and to extrapolate design to magnitudes of loading and types of material that are
THE NEED TO PREDICT DETERIORATION

beyond the scope of available field data, has been the major contribution of mech-
anistic analysis techniques in recent years (as exemplified by papers to the
international conferences on structural pavement design). Mechanistic design
methods are based on a theoretical analysis of the stresses induced in a pavement
under load, mechanical properties of materials, and experimental models of the
behavior of materials under repeated loadings and different environmental condi-
tions. However, the methods need validation and calibration to the full range of
real conditions, and are particularly lacking in the prediction of roughness and
surface disintegration which are important determinants of the need for mainte-
nance.

Empirical study is the only method by which the long-term parallel
effects of traffic of mixed loading and of environmental factors on pavement per-
formance can be quantified and distinguished. It is the only method by which the
real rates of distress development, the interactions between distress types, and
the relative effectiveness of different intervention timings and types of mainte-
nance activity, can be quantified. It is also the only method by which the
performance of unpaved roads can be realistically quantified. On the other hand,
mechanistic analyses and accelerated loading studies have been invaluable in
identifying the fundamental variables and the appropriate functional forms for the
development of each type of distress.

The technology is thus at a confluence where both approaches are
required. The mechanistic method makes the statistical design and analysis more
efficient by structuring the variables and functional forms. The empirical
method, aided by new statistical techniques, has the objectives of quantifying
roughness trends, joint aging and trafficking effects, the variability of pavement
behavior and the confidence of predictions, and of distinguishing the various
sources of damage for allocating the costs of road use.

The approach adopted in the current study is an example of the empirical
method enhanced by mechanistic principles. It uses comprehensive field data from
in-service roads and advanced statistical techniques to estimate models that have
been structured on mechanistic principles. The approach thus differs from that of
Rauhut, Lytton and Darter (1984) who used a mechanistic model, calibrated empiri-
cally by field data, to evaluate traffic damaging effects for the major United
States study on highway cost allocation. Each approach is appropriate to its
purpose, but the two are evidence of the convergence which is occurring.

1.4 SCOPE AND OUTLINE

The book focuses on the development of a set of road deterioration pre-
dictive models for a variety of applications, including life-cycle cost analysis,
road pricing studies, pavement management systems, and the evaluation of pavement
design methods. The initial objective of the study was to provide predictive
relationships for the World Bank's HDM-III Model (Watanatada and others 1987a),
and during the development of the models a further objective was added, namely, to
quantify the marginal deterioration effects attributable to traffic, materials and
climate for application in a study of taxation and pricing in transport (Newbery
and others 1988).

The primary data base for this study comes from the major Road Costs
Study undertaken in Brazil with assistance from the United Nations Development
Program and the World Bank (hereinafter referred to as the Brazil-UNDP study).
The study was undertaken between 1976 and 1981 by the Brazilian Transport Planning Agency (GEIPOT) and an international team from the Texas Research and Development Foundation (TRDF), and is fully reported in GEIPOT (1982). The data base is comprehensive, comprising detailed time-series observations of road condition over a period of three to five years on 164 selected sections of the existing network, and detailed measurements of pavement structural and traffic characteristics. The sections, on paved and unpaved roads, were selected by factorial design, and cover generally wider ranges of pavement type, age, traffic volume and loading, and pavement strength than previous empirical studies. Most importantly, the condition measurements were systematic, frequent and reproducible (a point particularly important with regard to roughness). The measures of pavement strength characteristics were also sufficiently diverse to permit comparison of the different measures and some mechanistic analysis. The range of materials and climate in the study, however, were limited to those of the central region of Brazil, where lateritic and quartzitic aggregates predominate and the climate is subhumid to humid with moderate, and highly seasonal, rainfall of 1,100 to 1,800 mm per year and moderate, nonfreezing temperatures.

In order to evaluate the validity of the models and to extend their applicability to other regions and climates, independent data sources from several other deterioration studies are drawn upon. This exercise was complicated by the need to establish conversions between sets of differing physical measures. For paved roads, data is drawn from the factorial study in Kenya, network samples from Arizona, Kenya, Texas and Tunisia, and special studies from Colorado, Illinois (the AASHO road test), Ghana, Canada and the Caribbean. For unpaved roads, comparisons are made with data drawn from the factorial study in Kenya, and special studies in Ghana, Ethiopia and Bolivia. These comparative studies make it possible to evaluate the transferability and predictive accuracy of the models, and the effect of factors not included within a particular study, such as climate, material types and construction practices.

Predictive relationships are developed for the individual modes of distress and types of maintenance that are most relevant to predicting performance. Although this approach means that many separate models must be developed it is preferred because of its versatility for different applications and because it is rigorous, allowing the mechanisms and interactions of various factors to be estimated directly. It is also practical, because the decision for maintenance takes into account different types of distress for different types of maintenance.

1.4.1 Outline

As road roughness influences the economic benefits accruing from maintenance and the acceptability of service to the user, it is a crucial measure of deterioration, and so the concepts and measurement of roughness are addressed first in Chapter 2. The deterioration and maintenance of unpaved roads are considered in Chapter 3, including the mechanisms of deterioration, models for roughness progression, blading maintenance and surface material loss, and criteria for the selection of surfacing material and thickness.

1/ Processed and compiled data files (World Bank 1985) are available through Infrastructure and Urban Development, World Bank, Washington, D.C. Raw data files and documentation are available through GEIPOT, Brazil (see GEIPOT 1982).
Paved roads are addressed in Chapters 4 to 8, beginning with a discussion of concepts and definitions in Chapter 4. Chapter 5 deals with cracking distress in bituminous surface roads, developing models for the initiation and progression of cracking for various pavement types. Chapter 6 deals with the disintegration mode of distress including ravelling, potholing and loss of surface friction. Chapters 7 and 8 deal with permanent deformation in pavements, the first discussing theory and developing models for rutting, and the second addressing the prediction of roughness progression. The latter chapter is a focal point of the book because of the importance of roughness and the advances made in predicting its progression, going well beyond the capability of existing models.

The basis for evaluating vehicle loading effects on deterioration, and the often controversial issues of the relative damaging power of different axle loadings and of non-traffic-associated factors, such as the environment, are addressed in Chapter 9. This introduces a special empirical analysis of these effects under long-term service conditions with findings that are of considerable relevance to pricing studies and design methodologies.

Finally, a concise overview of the findings and models is given in Chapter 10, along with conclusions on their transferability and the state of knowledge.

1.4.2 Structure

The general structure of most chapters is:

- a preface on the relevance of the distress mode or topic;
- definition and description of the distress mode, and objective measures which are appropriate to planning and management;
- a summary of current knowledge on the physical causes and mechanisms of distress, evaluating both empirical and mechanistic knowledge;
- choice and description of modelling approach and model form taking account of the various applications for such models;
- a general assessment of the available field data, detecting primary trends and features;
- the statistical estimation of the predictive model;
- evaluation of how well the model fits the original data, and the size and source of errors;
- illustration of predictions;
- discussion of the engineering implications of the model, its primary effects and limitations, highlighting those of importance to planning, management or engineering design;
- validation of the model by applying it to independent data sets from other regions, and evaluating its transferability and limitations; and
- concluding comment.

1.4.3 Reading Guide

The following guide is intended to facilitate quick or selective readings where desired. The chapter prefaces could usefully supplement selected readings.
For a quick reading, go to Chapter 10, since there are summarized the philosophy, the technical models and the chief technical, economic and planning implications.

Those planners and economists who wish to focus on models suitable for forecasting and the reliability of the predictions may concentrate on the sections concerning

i) modelling approach;
ii) illustration of the predictions; and
iii) validation.

Sections 2.1 and 2.2 (significance of roughness), and 9.4 and 9.5 (attribution of damage) are relevant to performance standards and taxation.

Those engineers and planners interested in the physical issues, may wish to skin past the sections on statistical and modelling issues and concentrate on:

i) the introductory sections on objective measures and current knowledge of causes and mechanisms;
ii) the general assessment of field data and trends;
iii) illustration of predictions and the engineering implications; and
iv) validation and transferability.

Chapters 2 (Measures of Roughness), 3 (Unpaved Roads), 8 (Paved Road Roughness) and 9 (Relative Damaging Effects) mark important directions in thinking aimed at improving our ability to evaluate and forecast deterioration.

Researchers and modellers already familiar with the field, may note initiatives in Sections 2.3, 3.5, 4.3, 5.3 - 5.7, 6.2 - 6.3, 7.2, 8 (all), 9.3 - 9.6, and Appendix B.

For the dedicated reader, it is hoped that the development of thought and improved tools, throughout the text, will stimulate progress in both implementation and knowledge.
Roughness is the irregularity of the road surface familiar to all road users, and perceptions of the riding quality have long been considered important criteria for the acceptability of the service provided by the road. Roughness affects the dynamics of moving vehicles, increasing the wear on vehicle parts and the handling of a vehicle, and so having an appreciable impact on vehicle operating costs and the safety, comfort and speed of travel. It also increases the dynamic loadings imposed by vehicles on the surface, accelerating the deterioration of the pavement structure. Roughness can have adverse effects on surface drainage, causing water to pond on the surface, with consequently adverse impacts on both the performance of the pavement and on vehicle safety.

We begin the chapter by considering the reasons why roughness has become such a dominant criterion in the evaluation of road policies and standards, adding persuasive economic reasons to the familiar, longstanding subjective reason of riding quality. While roughness measurements have been made for many years, the recent advances have been in understanding and defining roughness and in standardizing the measurements. After summarizing the concepts and principles involved in analysing the road profile and vehicle dynamics, the selection of a standard is discussed, focussing on the International Roughness Index (IRI), which was established during this study through international cooperation and is transferable worldwide and applicable to all measuring instruments.

Finally, the chapter addresses practical problems associated with measurements of roughness, the effectiveness of calibration, the accuracies associated with different types of equipment and the effects of measurement speed. These issues are relevant to the processing and interpretation of roughness data in any network monitoring system, and particularly in the development of the predictive models made later in Chapters 3 and 8.

2.1 IMPORTANCE OF ROUGHNESS

2.1.1 Acceptability to Users

Early concerns related to the provision of acceptable riding quality, and the formalization of subjective assessment by panel rating in the late 1950s provided a basis for performance standards. The Present Serviceability Rating (PSR) (Carey and Irick 1960) for example, was a five-point scale (0 poor, 5 excellent) quantifying the subjective rating of ride and pavement condition by a panel of experienced highway users. Assessment of the "acceptability" of a sample of seventy four flexible pavements in Illinois, Minnesota and Indiana (Appendix F in Highway Research Board 1962) was related to the PSR by the distribution shown in Figure 2.1. The 50th percentile of "acceptability" was 2.9 PSR and for "unacceptability" (not shown) was 2.5 PSR. The "acceptability" is relative to the functional class of the road, in this case state highways, and corresponding values have been determined for other classes, for example 2.0 or less for secondary roads.
The PSR correlated very highly with roughness (see Chapter 4) and much of the thrust of pavement design and management to date has been directed towards meeting such standards of acceptable serviceability and riding quality. The design life of a pavement, for example, would be expressed in terms of the traffic and time taken for the pavement to deteriorate to the minimum acceptable serviceability.

However, panel rating is not a sufficiently objective, consistent basis for the planning of maintenance, so a physical measurement of roughness is the preferred basis for standards. Changes in vehicle technology over time and the changing expectations of users (which are conditioned by either selective or general exposure to different standards), mean that the relationship of subjective rating to roughness and the levels of acceptability vary over time and with region. Hence recent studies in the United States (Janoff and others 1985) have been directed at relating the subjective ratings to objective measures.

### 2.1.2 Economic Impact

The economic impact of roughness is considerable, usually outweighing the considerations of riding comfort, and thus providing the strongest objective basis for evaluating road policies. The cost of operating vehicles and transporting goods rises as road roughness increases, and as the total operating costs of all vehicles on a road outweigh the agency costs of maintaining the road by typically ten- to twenty-fold, small improvements in roughness can yield high economic returns. These returns are not immediately apparent to a highway agency, however, because most of the benefits accrue to road users, but then so also are the costs of neglected maintenance incurred by the users. Thus the benefits are realized in the economy and, ultimately, in lower transport costs.
The crucial relationship between vehicle operating costs and road roughness has now been well established through four major empirical studies conducted in Kenya, Brazil, the Caribbean and India, beginning in the early 1970s (Chesher and Harrison 1987, Watanatada and others 1987). The studies quantified the impacts of road geometry and roughness on the fuel, tires, parts and labor, depreciation and interest, and time (speed) components of operating costs of several different classes of vehicle, based on data collected for individual vehicles over two- to four-year periods. Parametric models, developed from these data, permit the operating costs to be computed for vehicles in other countries also, with a generally high degree of confidence, because they are based on quantifiable vehicular characteristics, and physical units of consumption, to which are applied local unit costs.  

A general example of this relationship for one application is shown in Figure 2.2 based on the model forms developed from the Brazil study (Chesher and Harrison 1987 and Volume 5 in GEIPOT 1982). The relationships show that operating costs rise at a rate of about two to four percent per IRI unit of roughness as roughness increases, with slightly different rates applying for different vehicle classes. Typically, over the range of condition of paved roads, operating costs rise by the order of 15 percent between the extremes of excellent condition (2 m/km IRI) and poor condition (8 m/km IRI). On unpaved roads, the rise in costs is much greater and is typically of the order of 40 to 60 percent between the extremes of good condition (4 m/km IRI) and very poor condition (16 m/km IRI).

**Economic standards**

The initial roughness of new road construction, which depends on the construction method and quality, ranges from 1 m/km IRI for high quality paver-laid asphalt to 4 m/km IRI for poor quality paved construction, and from the order of 4 to 8 m/km IRI for medium standard gravel or earth roads. Applications of the HDM model (for example, Bhandari, Fossberg and Harral 1984) have indicated that economic intervention levels for pavement rehabilitation range from about 2.5 m/km IRI (3.2 PSI) for traffic volumes greater than about 3,000 veh/day to 5 m/km IRI (2.0 PSI) for traffic volumes of less than about 500 veh/day, depending on price relativities and budget constraints. From this it can be deduced that intervention levels based on economic criteria tend towards slightly higher standards than those indicated by the subjective acceptability criteria of riding quality. Thus criteria for acceptability to users are usually more than satisfied by economic criteria.

**Vehicle speed**

Free-flow vehicle speeds are also influenced by roughness but only at relatively high levels, as shown in Figure 2.3 (see Watanatada and others 1987, Paterson and Watanatada 1985). As a rule of thumb, the product of speed in km/h

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1/ Computer versions of these models are available, incorporated in the HDM-III model (Watanatada and others 1987a), and for microcomputers in Dhareshwar and Archondo-Callao (1987).

2/ The International Roughness Index (IRI) is the primary measure of roughness used throughout this book. Its definition and relation to other roughness scales are addressed in detail in section 2.3.
Figure 2.2: Example of the influence of roughness on vehicle operating costs excluding taxes estimated for Tunisia 1983

![Diagram showing vehicle operating costs in relation to roughness (m/km IRI)].

Note: Costs in January 1983 Tunisian Dinars (TD); US$1 ~ TD 0.616. GVM = (maximum) gross vehicle mass.

Source: Newbery, Hughes, Paterson and Bennathan (1988); Application of relationships from Chesher and Harrison (1987).

and roughness in m/km IRI rarely exceeds about 700 for cars or 550 for heavy trucks. For example, car speeds usually do not exceed 85 km/h on roughness above 8 m/km IRI. A study in Canada (Karan, Haas and Kher 1976) found that free-flow vehicle speeds were even influenced at slightly lower levels of roughness in the order of 5 to 6 m/km IRI³/.

³/ Note, however, that a firm basis for converting the RCI roughness scale used in that study to IRI is not yet available; the level of 5 to 4 on the Canadian subjective RCI scale, was converted using Figure 2.15 and RCI = 2 SI.
2.1.3 The Need for a Relevant Measure

Road roughness therefore emerges as a key property of road condition to be considered in any economic evaluation of design and maintenance standards for pavements, and also in any functional evaluation of the standards desired by road users. For this purpose a relevant measure of roughness is needed.

Over time many measures have been utilized, varying greatly with the instrument technology. Standardization of a relevant measure has been slow, however, due to the complexities of defining roughness and its impact on vehicles and occupants. In order to understand the standardization that has now been achieved in the international roughness index (IRI), we first use the benefit of hindsight to review the principles and elements involved, and then consider measurement issues.

2.2 DEFINING ROAD ROUGHNESS

Roughness is a characteristic of the longitudinal profile in the wheelpaths of the travelled surface, and is best defined with respect to its impact on both the functional and structural performance of the road. Thus road roughness is defined as:

"the deviations of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads and drainage." (American Society for Testing and Materials (ASTM) specification E867-82A).

This definition implies firstly that "roughness" is a defect, comprising those surface deviations which influence, and are relevant to, the motion and operation of a moving vehicle; that is, through the user's perception of riding quality, the wear and operating costs of vehicles, road safety, and the impact of the vehicle on the road through excitation of the vehicle mass. Second, it implies that the natural origin for a roughness scale is a true planar surface, for which the roughness is nil by definition.

In Europe and some francophone countries, the term "evenness" is used, which is conceptually the reverse of roughness. In Britain, the term "unevenness" has come to be adopted for the major routes, which at least has the same sense as roughness.

Defining a relevant measure of roughness must therefore consider three elements, namely the road surface profile, the vehicles, and the users or vehicle occupants.

### 2.2.1 The Road Surface Profile

The surface deviations in the longitudinal profile tend to be random in nature, but they can be characterized conveniently by a combination of waveforms of various amplitudes and wavelengths. Typically, the spectrum of wavelengths is complex and covers a broad range. A practical range which eliminates surface texture (very short waves) and longitudinal gradient (very long waves) is given by:

- **Surface wavelengths**: 0.1 to 100 m
- **Surface amplitudes**: 1 to 100 mm.

Roughness increases as the amplitudes of the waveforms in the profile elevations increase. The full spectrum can be represented by the power spectral density (PSD) as shown in Figure 2.4(a), which depicts the mean squared elevations (or amplitudes) as a function of the wavenumber (the inverse of wavelength) for several different roads. The figure shows that the greatest amplitudes generally occur in long wavelengths (low wavenumbers), that is, the long humps in a road; and that the amplitudes are generally smaller for the shorter wavelengths (or higher wavenumbers). All roads tend to have a similar shape of PSD, but for rough roads the PSD would appear higher in the figure than for smooth roads.

Roughness can also be expressed in terms of the road slopes, instead of elevations, and Figure 2.4(b) shows the PSD of the slopes. This represents vertical velocities in space and time, and has the convenient property that the PSD is approximately uniform over all wavelengths (spanning only two decades instead of eight as for the elevation PSD). Thus the relative impacts of different wavebands within the spectrum are rather insensitive to the bandwidth chosen for defining roughness when compared with the elevation or acceleration PSDs. Mathematically, the slopes PSD is the derivative of the elevations PSD.
The second derivative of the elevations gives the PSD of accelerations, as shown in Figure 2.4(c), which relates to the vibrations induced in vehicles and vehicle occupants. The highest accelerations are seen to come from short wavelengths and very low levels to come from long wavelengths (low wavenumbers). However, this PSD has a wide span of seven decades so that measures of the acceleration level tend to be dominated by amplitudes in the short wavelength band (high wavenumber).

It is the tradeoffs between the relative impacts of elevations, velocities and accelerations on users and vehicles which complicate the definition of a relevant statistic for roughness.

**Surface types**

The spectrum tends to vary with the type of road surface, and it is possible to identify characteristics for each type. In Figure 2.5, the slope PSDs of two paved and two unpaved types of road surface have been normalized to remove the effects of magnitude of roughness, so what remains indicates the roughness...
Figure 2.5: Typical spectral compositions of road profiles for flexible pavements and unpaved roads

(a) Asphalitic concrete

(b) Surface treatment

(c) Gravel

(d) Earth

Source: Sayers, Gillespie and Queiroz (1986).
content being contributed by each wavelength in the spectrum. The data were obtained during the International Road Roughness Experiment (Sayers, Gillespie and Queiroz 1986), which covered a balanced range of surfaces from very smooth to very rough within each road type.

Paved roads with asphalt surfaces, in chart (a) of the figure, typically have only a very small portion of the roughness in short wavelengths (high wavenumbers), because these are effectively eliminated by the levelling action of the mechanical paver-finisher used in construction. Roads with surface treatment (or chip seal) surfaces, in contrast, have a much more variable spectrum with more roughness found at the short wavelengths below 2 m. Corrugations or ripples with a more or less uniform wavelength appear as a spike in the PSD, as seen in chart (b), for example, which includes some surfaces that had a corrugation spacing of about 2 m.

Unpaved road surfaces, shown in (c) and (d) of the figure, have a broad spectrum of roughness with appreciable content in all wavebands, and noticeably more roughness in the short wavelengths (high wavenumbers) than paved roads. In the case of gravel roads, the spectrum is rather similar in composition to that of surface-treated paved roads, due probably to the similarities in the methods of construction (by mechanical grader) and in the performance of coarse granular materials under traffic. Earth road surfaces show the greatest content in the short wavelength range below 1 m, which reflects the tendency of fine-grained materials to develop small depressions, humps and potholes.

Concrete pavements, shown for example in Figure 2.4, tend to have much smaller amplitudes in the long wavelengths due to level controls during construction, though marked periodicities can sometimes occur. Strong amplitudes in short wavelengths characterize some texturing methods and can develop due to slab faulting or joint deterioration. Thus the PSDs may be skewed compared with those for flexible pavements.

2.2.2 Effects on the Vehicle

The way a moving vehicle responds to roughness depends on vehicle properties, the speed and the roughness content of the road; this is well-summarized in the study by Gillespie, Sayers and Segel (1980).

In a moving vehicle, the effect of roughness in the road profile is translated into a vertical velocity input in the wheels, and the spatial frequency of the road profile (wavenumber) is translated into a temporal frequency (cycles per sec, Hz). The vehicle response to roughness is illustrated in Figure 2.6, which shows the interaction between the road input in (a) with the dynamic characteristics of the vehicle in (b) to produce the response shown in (c). The dynamic characteristics are expressed in terms of gain, with a gain of one meaning that the axle-body displacement has the same magnitude as the deviation in the road surface. The response is the spectrum of vibrations occurring in the vehicle, as represented by the relative displacement between an axle and the vehicle body.

An important feature to notice in (b) is that the vehicle is not equally responsive to all frequencies. Most frequencies of the roughness input tend to be attenuated by the vehicle suspension (gains of less than one), so that most of the surface deviations are suppressed in the vertical movements of the vehicle body. However two major resonant frequencies exist that tend to amplify the roughness
At frequencies of 1 to 2 Hz, resonance of the vehicle body on the suspension occurs, so that the roughness in this range is amplified in the bouncing of the vehicle body. The amplification depends on the damping characteristics of the suspension and typically ranges from 1.5 to 3 - stiff shock absorbers give the lower value of 1.5, whereas as soft shock absorbers cause gains of up to 3 (Gillespie and others 1980). The second resonance exists at higher frequencies in the range of 8 to 12 Hz and corresponds to the axle resonance between the "springs" of the tire and the suspension system. The amplification of roughness is in the order of 1.5 again for stiff shock absorbers and 2.5 for soft shock absorbers.

At frequencies above this range, much of the roughness input is absorbed by deflections within the tires. A third resonant frequency exists at about 30 Hz, corresponding to resonance in the tire walls, but has often been ignored in roughness and riding quality studies because the amplitudes are very small. However there is some evidence that this frequency, which depends on tire inflation pressures, may relate to the development of road surface corrugations. Spatial frequencies of corrugations observed on unpaved roads in the order of 1 to 1.5 cycles per meter, under vehicle speeds ranging from 100 down to 60 km/h, correspond to a temporal frequency of about 30 Hz.

Still higher temporal frequencies, above 60 Hz, approach the audible range and contribute to road noise - this range corresponds to spatial frequencies of more than four cycles/m or wavelengths of less than 0.25 m.

2.2.3 Effects on Occupants: Riding Quality

Studies on the effects of roughness on riding quality have made extensive use of subjective assessments (Carey and Irick 1960, Walker and Hudson 1973) and the importance of correct methods for collecting and analysing psycho-physical data has been well established (Weaver 1979, Holbrook and Darlington 1973).
From an analysis of individual wavebands in the roughness spectrum, Holbrook and Darlington (1973) found that the 1.8 to 2.4 m wavelength band caused maximum reactive forces in vehicles. Williamson and others (1975) found that roughness in the 1.2 to 3.0 m band predicted the subjective rating as well as all other bands combined, but noted that the correlation with ratings remained appreciable for longer wavelengths of up to 30 m.

The correlation with roadmeters, which measure the relative axle-body displacements on a vehicle, is generally high, being in the order of 0.75 to 0.95 (e.g., Yoder and Milhous 1964, Highway Research Board 1973, and see Section 4.1). Contrary results come from Nakamura and Michael (1962) with high correlation (0.9) for rigid pavements but low correlation (0.5) for flexible pavements, and opposite findings from Janoff and others (1985) with correlations of 0.47 and 0.86 for rigid and flexible pavements, respectively. These are explicable by specific characteristics of the data; for example, the rigid pavements in the Janoff study covered only a narrow range of roughness (1 to 3.7 m/km IRI) which was only one-third that of the flexible pavements. Sayers, Gillespie and Queiroz (1986) in the International Road Roughness Experiment found high correlations (0.78 to 0.96) between roadmeter-type measures and subjective rating on both flexible pavements and unpaved roads over much wider ranges of roughness (from 1 to 17 m/km IRI), as shown in Figure 2.7.

In the detailed analysis of their data, Janoff and others (1985) examined the correlations with subjective ratings for individual narrow wavebands of one-third of an octave over the range from 0.008 to 2.0 cycles/m (0.5 to 125 m wavelengths). They found that the 0.5 to 2.4 m wavelength band gave the most consistent correlations across all surface types, while longer wavelengths also

**Figure 2.7: Correlation between subjective ratings and roadmeter measurements of roughness**

<table>
<thead>
<tr>
<th>Key and Statistics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road Surface</td>
</tr>
<tr>
<td>--------------------</td>
</tr>
<tr>
<td>Asphalt</td>
</tr>
<tr>
<td>Surface Treatment</td>
</tr>
<tr>
<td>Gravel</td>
</tr>
<tr>
<td>Earth</td>
</tr>
<tr>
<td>All</td>
</tr>
</tbody>
</table>

**Note:** The individual ratings have been normalized by the mean ratings.

**Source:** Adapted from Sayers, Gillespie and Queiroz (1986), Figure D.2(d).
correlated highly for flexible pavements but degraded the correlation for rigid and composite pavements. They concluded that the short wavelength band was the most indicative of riding quality, and defined a profile index (PI) from the root mean square (rms) of elevations in the 0.5 to 2.4 m wavelength band. This conclusion, however, is questionable, firstly because the longer wavelengths clearly influenced the ratings on flexible pavements, and secondly because the low amplitudes of the longer wavelengths which are typical of rigid pavements (see Figure 2.4) would inherently have little influence on subjective ratings.

Research on human response to vibration under controlled laboratory conditions (Oborne 1976) has shown that humans are sensitive to whole-body vibrations at low frequencies in the range of 3 to 8 Hz. Cooper and Young (1980), using large samples of the public in Britain, found that an rms acceleration level of 40 milli-g in the frequency range 0.2 to 20 Hz would be rated as "acceptable" by 90 percent of car occupants and 100 percent of occupants in heavy vehicles, Figure 2.8. Translating this into terms of roughness, Jordan (1984) derived the criteria for riding quality shown in Figure 2.9; the acceptable profile variance (squared deviations of the profile elevations with respect to a moving average over a baselength %) is much smaller for the short wavelength bands than for long wavelength bands. For discrete pavement defects such as corrugations, subsidence and ramps, accelerations of up to 0.6 m/s\(^2\) have been found to be acceptable, but over 1.5 m/s\(^2\) to be uncomfortable (Jordan, 1984); specific criteria for each case are summarized in Table 2.1. Ride comfort boundaries adopted by the International Standards Organization (ISO) (1974) indicate that the most critical range of frequencies is in the 3 to 20 Hz band, and a review by Gillespie and others (1980) suggests that this may be as wide as 1.5 to 37 Hz.

Figure 2.8: Riding comfort characteristics for buses (coaches), cars and heavy goods vehicles on major roads

![Figure 2.8: Riding comfort characteristics for buses (coaches), cars and heavy goods vehicles on major roads](image)

Figure 2.9: British criteria for surface unevenness (roughness) and ride on major roads

![Diagram showing surface quality and ride categories for major roads.]


Table 2.1: Criteria for acceptable ride over specific pavement defects

<table>
<thead>
<tr>
<th>Defect</th>
<th>Criteria for acceptable ride</th>
</tr>
</thead>
<tbody>
<tr>
<td>Systematic unevenness (corrugations, ripples)</td>
<td>Peak variance of profile elevation at ripple frequency ≤ 8 times the profile variance in adjacent frequency bands.</td>
</tr>
<tr>
<td>Subsidence</td>
<td>Depth &lt; 0.04 ( \frac{L^2}{V^2} ); Depth (m), where ( L ) = length of subsidence (m) (&lt; 30 m), ( V ) = traffic speed (m/s).</td>
</tr>
<tr>
<td>Overlay ramps</td>
<td>Length of ramp ≥ 4.06 ( V \sqrt{H} ); where length (m), ( V ) = traffic speed (m/s), ( H ) = overlay thickness (m).</td>
</tr>
</tbody>
</table>

French human-engineering studies (Abrache 1974) found the relationship between occupant discomfort and the induced vertical accelerations shown in Figure 2.10. The peak discomfort at low accelerations occurred at a frequency of about 7 Hz, and Jordan's "acceptability" criteria of 40 milli-g (0.39 m/s²) falls at the boundary between "bothersome" and "unpleasant." Extrapolation of the results to frequencies higher than 20 Hz suggests that only accelerations of more than 40 milli-g are likely to be considered unpleasant.

There is thus reasonable consensus that riding quality is predominantly sensitive to frequencies in the band of 3 to 20 Hz, with frequencies below and above that range being significant only when the accelerations induced are high. The range reported by Janoff (1985), which translates as 10 to 50 Hz, is clearly skewed to higher frequencies than reported elsewhere, apparently due to a skewed distribution of the observed vertical accelerations in their data. It appears to suggest that users are reacting to the higher frequencies more than indicated by previous studies, possibly because vehicle technology has compensated for roughness in the medium frequency waveband, and possibly perhaps because these are sensations other than whole-body vibrations. Within these frequency bands, acceptable ride is achieved when the vertical accelerations experienced by the occupants are less than 40 milli-g.

**Figure 2.10:** Relationship of ride discomfort to frequency and peak vertical acceleration of induced vibrations

2.2.4 Methods of Measurement

Just as the measures of roughness evolved with the different methods, so also must a roughness standard be measurable by available types of equipment, and the equipment must provide a relevant measure of roughness. The numerous instruments and methods available are summarized in four categories in Table 2.2 and as follows.

Absolute profile instruments

These measure the elevations of the profile relative to a horizontal level datum. The precision of the measurements, and the interval between measurements determine the accuracy with which the profile is reproduced. The data need to be processed by a mathematical procedure to give a summary statistic of roughness. The methods give the highest accuracy but most tend to be laborious and slow.

Table 2.2: Methods of measuring road roughness

<table>
<thead>
<tr>
<th>Category</th>
<th>Method and examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absolute profile</td>
<td>Measure profile elevation relative to a true horizontal datum, e.g., rod-and-level survey, &quot;Dipstick&quot; profiler, British profile beam (measures in 3m segments).</td>
</tr>
<tr>
<td>Moving-datum profile</td>
<td>Measure deviations of profile relative to a datum moved along the road, e.g., sliding straightedge, rolling straightedge, profilographs.</td>
</tr>
<tr>
<td>Vehicle-motion instruments</td>
<td>Measure: 1. relative displacement between axle and body of car, summing upward, or upward and downward, movements with readout at regular distances giving a slope statistic (displacement/length, e.g., m/km, mm/km, inch/mile) e.g., Maysmeter, Cox meter, NAASRA meter, Bump Integrator (trailer or car-mounted) 2. Accelerations of axle or body by accelerometer, and integrate signal, e.g., ARAN (Automatic Road Analyser).</td>
</tr>
<tr>
<td>Dynamic profile instruments</td>
<td>Measure profile elevations electronically relative to an artificial &quot;horizontal&quot; datum providing elevation-distance data, at intervals depending on electronic sampling rate, and filtered to span a practical range of frequencies; e.g., General Motors research (GMR) and K. J. Law profilometers (accelerometer as inertial reference); French APL (longitudinal profile analyser) (mechanical inertial reference); British High Speed Road Monitor (HSM) (laser-sensed profile relative to leading sensor).</td>
</tr>
</tbody>
</table>

Source: Author.
Moving datum instruments

The familiar straightedge, and rolling versions on wheels, measure deviations relative to a moving datum. The wavelengths measurable are limited by the baselength of the datum, and the signal gain is highly tuned and variable, as shown in Figure 2.11(a) (the ideal is a uniform gain of one). For example, a rolling straightedge measures each bump three times and is completely insensitive to wavelengths exactly equal to its baselength, and a profilograph overcomes this to a degree by averaging the end reference points.

Vehicle-motion instruments

Roadmeters are the most common instruments, able to measure long lengths of road quickly at highway speeds. The axle-body movements are usually summed so as to give the cumulative "bumps" per unit distance, which is a "slope" measure of the vehicle response to roughness. Some integrate an accelerometer signal. The

Figure 2.11: Response characteristics and typical output of profile devices

(a) Straightedge and profilograph devices

![Graph showing response characteristics of straightedge and profilograph devices](image)

(b) Profile output from different dynamic profile instruments

![Graph showing profile output from different dynamic profile instruments](image)

Source: Gillespie (1986).
results typically depend on the dynamic characteristics of the vehicle and speed, as shown by the response in Figure 2.6. Detailed information on the location, amplitude and frequency of movements usually is not obtained, so the result represents the average rectified slope of the axle-body motion (rectified because both upward and downward movements are counted) over the segment between successive outputs. The length of the tire contact naturally filters out very short wavelength effects.

Dynamic profile instruments

Dynamic profilers measure the profile from a moving vehicle or trailer. They differ in the reference used to represent the horizontal datum (Table 2.2) and in the method of sensing the profile. The early GMR profilometer and French APL profiler use direct contact through a following wheel on the pavement, while recent versions use indirect or noncontact methods such as visible light lasers, infrared light sensors and ultrasonic sensors. As these sensors can measure very short wavelengths, including surface texture and down into cracks (which are bridged by a vehicle tire and do not affect vehicle motion), high frequency filters or averaging needs to be applied to the data to suppress these effects. Figure 2.11(b) shows that the profilometers do not return the absolute profile exactly because of a lack of the lowest frequencies and slight distortions in the instruments, but the recorded profiles have been shown to contain all the information needed to calculate most roughness indices with adequate accuracy.

2.3 STANDARD AND COMMON MEASURES

The wide differences between the outputs of different devices used throughout the world, and the often poor reproducibility of results by the same type of equipment, have severely hindered the use of roughness data in decision-making, and particularly in research attempting to compare results from different studies. Awareness had grown that equipment hardware was generally unsuitable as a roughness "standard" because the characteristics change over time. Hudson (1979), with the proposal that roadmeters be calibrated over a series of road sections for which a standard roughness had been measured (successfully demonstrated in the Brazil-UNDP study (GEIPOT 1982)), and Gillespie and others (1981), with extensive study of the vehicle and road characteristics, laid the basic groundwork for standard calibration procedures.

2.3.1 International Road Roughness Experiment

In order to establish correlation between the different roughness measures and to select a standard for calibration, the World Bank convened the International Road Roughness Experiment (IRRE) in 1982 in Brazil, with sponsorship and participation by several international organizations (Sayers, Gillespie and Queiroz 1986). The experiment (see Table 2.3) was conducted on forty nine test sites of flexible pavements and unpaved roads covering a very wide range of roughness. Four profile measures, five types of roadmeter, and two types of subjective panel rating were run on all sections. The major conclusions were:

1. The average rectified slope (ARS) outputs of all roadmeters differ numerically but correlate highly when run at similar speeds (the correlations degraded when the speeds differed);
Table 2.3: Scope of the 1982 International Road Roughness Experiment

1. Road Test Sites

<table>
<thead>
<tr>
<th>Road surface</th>
<th>Number of Sections</th>
<th>Range of roughness (IRI)</th>
<th>Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>Minimum</td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>13</td>
<td>4.22</td>
<td>1.9</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>12</td>
<td>4.05</td>
<td>2.5</td>
</tr>
<tr>
<td>Gravel</td>
<td>12</td>
<td>7.63</td>
<td>3.7</td>
</tr>
<tr>
<td>Earth</td>
<td>12</td>
<td>8.35</td>
<td>4.1</td>
</tr>
<tr>
<td>All</td>
<td>49</td>
<td>6.03</td>
<td>1.9</td>
</tr>
</tbody>
</table>

2. Measurements and Methods

<table>
<thead>
<tr>
<th>Method</th>
<th>Number</th>
<th>Description and test speeds</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roadmeters, car-mounted</td>
<td>5</td>
<td>Maysmeter (3), NAASRA (1), Bump Integrator (1): 5 runs at each speed 20, 32, 50, 80 km/h.</td>
</tr>
<tr>
<td>Roadmeters, trailer</td>
<td>2</td>
<td>Bump Integrator, BPR roughmeter: 6 runs at each speed 20, 32, 50 km/h.</td>
</tr>
<tr>
<td>Static profilers</td>
<td>2</td>
<td>Rod and level survey, TRRL beam.</td>
</tr>
<tr>
<td>Dynamic profilers</td>
<td>2</td>
<td>APL trailer (speeds 21.6, 72 km/h); GMR profilameter.</td>
</tr>
<tr>
<td>Panel rating</td>
<td>2</td>
<td>18-person PSR-panel, 4-person IRI-panel.</td>
</tr>
</tbody>
</table>

3. Participants

Transport Planning Agency (GEIPOT), Road Research Institute (IPR), Brazil; Central Bridges and Pavements laboratory (LCPC), France; Road Research Center (CRR), Belgium; Transport and Road Research Laboratory, Overseas Unit (TRRL), United Kingdom; University of Michigan Transportation Research Institute (UMTRI), USA; The World Bank; and contributions from Australian Road Research Board (ARRB); and Federal University of Rio de Janeiro, Brazil.

Source: Based on Sayers, Gillespie and Queiroz (1986).
2. Roadmeters generally performed satisfactorily over the full range of roughness (except the BPR roughometer was not robust enough for the rough roads, and the Maysmeter lost counts on very smooth sections), but dynamic profilometers were limited to paved roads (GMR) and all but the roughest unpaved roads (APL);

3. After calibration to a road profile statistic, there is a high equivalence and correlation amongst roadmeters and profilometers;

4. Of six profile statistics evaluated, most were satisfactory for calibration purposes, and the best correlations were given by the ARS of a quarter-car simulation (the reference simulation, RQCS, derived by Gillespie and others 1981);

5. The international roughness index (IRI) was selected to be the slope output (ARS) of the RQCS, with a simulation speed of 80 km/h, derived from the absolute profile of the road surface.

Further discussion on the experiment can be found in Sayers, Gillespie and Queiroz 1986b, 1987).

2.3.2 International Roughness Index

The international roughness index (IRI) is a mathematically-defined summary statistic of the longitudinal profile in the wheelpath of a travelled road surface. The index is an average rectified slope statistic computed from the absolute profile elevations. It is representative of the vertical motions induced in moving vehicles for the frequency bandwidth which affects both the response of the vehicle and the comfort perceived by occupants.

The IRI is defined by a mathematical simulation of a quarter-car (that is, one wheel with the associated dynamic characteristics of the suspension and sprung mass of a typical passenger car), as shown in Figure 2.12 and defined in Sayers, Gillespie and Paterson (1986). The simulated travelling speed is 80 km/h, which determines the bandwidth of the responses shown in (b) and (c) of the figure. These can be seen to cover the range of frequencies most affecting the users' perception of comfort and the impact on moving vehicles.

The IRI describes a scale of roughness which is zero for a true planar surface, increasing to about 6 for moderately rough paved roads, 12 for extremely rough paved roads with potholing and patching, and up to about 20 for extremely rough unpaved roads, as shown in Figure 2.13. The units of IRI are actually dimensionless, because it is a slope statistic, but it has been scaled by a factor of 1,000 so that it represents m/km, mm/m or inches/1,000 inches. The standard presentation is thus 2.1 m/km IRI, generally reported to one decimal place.

Details of the computation of the IRI, and guidelines for applying it to the calibration of equipment and the conduct of roughness measurements are given by Sayers, Gillespie and Paterson (1986). The calibration method refines the ones adopted in the Brazil-UNDP study (GEIPOT 1982) and proposed by Hudson (1979).
Figure 2.12: Dynamic model simulating a quarter car is the reference of the International Roughness Index

(a) Reference Quarter-Car Simulation (RQCS)

(b) Frequency Response of RQCS to Elevation Input

(c) Frequency Response of RQCS to Slope Input

Source: Sayers, Gillespie and Queiroz (1986).
Figure 2.13: The International Roughness Index (IRI) scale of road roughness

![IRI scale diagram]

Source: Sayers, Gillespie and Paterson (1986).

Figure 2.14: Comparison of various profile statistics by their correlation to roadmeters for all road surfaces and various roadmeter speeds

![Profile statistics comparison diagram]

Note: Refer to Figure 2.15 and Table 2.4 for definitions of the profile statistic acronyms.
Source: Data from IRRE (Sayers, Gillespie and Queiroz 1986).
Discussion

The IRI was adopted as a standard measure of roughness because it met essential criteria. First, it is time-stable and reproducible anywhere from elevation data because it is a mathematical summary statistic of the absolute road profile. Second, the origin for a planar surface is zero and the scale is open-ended at high roughness levels (equipment-based statistics generally had a nonzero origin due to mechanical imperfections). Third, it was the statistic which gave the most consistently high correlations with the output of all roadmeters on all surfaces at all speeds, as shown in Figure 2.14, and also correlated highly with subjective ratings, Figure 2.7. Fourth, it is relevant to the impact of roughness on vehicles and users because the waveband covered, and its sensitivity to amplitude variations within that waveband, are representative of vehicular response and the comfort perceived by users - the bandwidth is predominantly in the frequency range of 1 to 20 Hz or 0.04 to 1 cycles/m (wavelengths of 1 to 25 m). Fifth, it is applicable to all profilometers and roadmeters, because it can be calculated directly from profile data (and is not limited to specific intervals as are some statistics) and correlates well with all roadmeters tested. Sixth, it is relevant to a wide range of traffic conditions from slow speeds to fast speeds and for a variety of vehicle types: even though the specific bandwidth included in the statistic applies primarily to uncongested, interurban highway travel with speeds in the order of 80 km/h (which represent the most prevalent conditions for highway networks in both industrialized and developing countries), the statistic also correlates extremely highly with the roughness perceived by vehicles at both slower speeds (under congestion) and faster speeds (motorways or freeways).

Some shortcomings of the IRI have been noted. First, it does not give a direct measure of the accelerations affecting the riding comfort perceived by users, although it correlates very highly with them. Comfort is affected by different levels of acceleration in different frequency bands and, while Hudson and others (1985) claim that the IRI-type simulation emphasizes 10 Hz frequencies, and Janoff and others (1985) claim that frequencies up to 50 Hz affect comfort, the IRI is basically consistent with the ranges shown in the ISO Standard (1974). Detailed studies of comfort however, need to identify the amplitudes or accelerations in different bandwidths, for which spectral or bandwidth profile analysis is preferable (Section 2.2.3). Second, there is concern that the vehicle characteristics embodied in the IRI mathematical model may not be specifically representative of all vehicle classes or of future vehicle technology. While this is valid in respect of specific vehicle characteristics, Sayers and Gillespie (1981) demonstrated for heavy trucks that chassis and floor vibrations correlated very highly, and seat vibrations correlated well, with the simulation on which IRI is based. In this respect, a direct statistic of the slopes or "accelerations" in the profile may have been preferable for the IRI but this would have foregone the advantages of including the filtering effects of a moving vehicle.

As the advantages of other profile statistics were not comparable to those of IRI in all respects, the IRI was found to be the most valid summary statistic of roughness for general applications. It is appropriate to the primary applications in road deterioration and user cost studies, and to the calibration of nearly all roughness instruments. It is complementary to, but not directly equivalent to, separate waveband statistics.
Alternative statistics

Profile-related statistics fall into three categories, as shown in Table 2.4, which lists the statistics considered during the IRRE and others. In the first category, which includes the IRI, the full profile is processed mathematically to simulate vehicle response.

In the second category, the summary statistic is an estimate of the response of a particular piece of equipment by correlation to a waveform statistic taken from one or more selected wavelengths within the full spectrum. The two examples are RMSVA and RMSD, defined in the table. In this approach different wavelengths are selected, depending on the reference equipment and travel speed. These were 1.2 and 2.5 m for MO, 1.0 and 4.9 m for QIr, and 1.8 m for BIRr (the Bump Integrator trailer), where MO and QIr simulate speeds of 80 km/h and BIRr simulates 32 km/h. Also, the origin for a planar surface in each case is non-zero. Thus this kind of statistic "freezes" in time the mechanical and operative imperfections of a particular item of hardware (or panel of raters in some cases). This is an undesirable feature because it is preferable for the standard itself to be pure, so that specific equipment imperfections are reflected instead in the calibration equation.

The third category offers more flexibility for special studies of the effects across the full spectrum by defining roughness with respect to different wavebands. The higher correlations with roadmeters and ratings in the IRRE were given by CPV.5 and WSW, that is the shorter wavelengths of 1 to 3.3 m. The British unevenness statistic (PU) represents 3.0 m for major roads (100 km/h), whereas the PI (profile index) represents only shorter wavelengths of 0.5 to 2.4 m. Taken alone, therefore these statistics ignore certain parts of the spectrum affecting vehicle and users, whereas the IRI is more encompassing, covering 1 to 25 m wavelengths appropriately weighted. Individual waveband statistics have special uses, however, because they isolate specific effects which can aid the special interpretation of deterioration. For example:

1. Short wavelength roughness represents defects in the upper pavement layers;

2. Medium wavelength roughness represents defects deriving from the pavement subgrade; and

3. Long wavelength roughness represents subsidence or heave deriving from the formation or roadbed.

In future research, such subdivisions of the roughness spectrum may help to identify the different impacts of each bandwidth on comfort, operating costs and pavement deterioration. But they also expand the number of variables involved, and the first priority of research has been to establish good relationships for the primary effects of roughness, as represented in one summary statistic by the IRI.

2.3.3 Conversion between Roughness Scales

In order to facilitate technology transfer and the comparison of results between studies, conversion relationships were established between the IRI and scales such as the QI and Bump Integrator trailer (BI) roughness measures used in
Table 2.4: Description of various road profile statistics by category

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Source</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. Mathematical simulation of vehicle response</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RQCS</td>
<td>NCHRP Report 228</td>
<td>Reference Quarter Car Simulation with parameters representing passenger car (Gillespie and others 1981); ARS output in &quot;inches/mile.&quot;</td>
</tr>
<tr>
<td>QCS</td>
<td>GMR Profilometers</td>
<td>Quarter Car Simulation with vehicle constants derived by K.J. Law (Inc.); ARS output in &quot;inches/mile.&quot;</td>
</tr>
<tr>
<td>IRI</td>
<td>World Bank</td>
<td>RQCS as above with scaled dimensionless ARS output (nominally in &quot;m/km,&quot; where 1 m/km = 63.36 inches/mile). (Sayers, Gillespie and Queiroz 1986).</td>
</tr>
<tr>
<td><strong>2. Estimation of vehicle response by correlation to wavelength statistics</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MO</td>
<td>Texas</td>
<td>Estimate of &quot;Maysmeter Output&quot; ARS (inches/mile) from root mean squared vertical acceleration (RMSVA) of profile in 1.2 m and 4.9 m baselengths (McKenzie and Hudson 1982): MO = 20 + 23 RMSVA$<em>{1.2}$ + 58 RMSVA$</em>{4.9}$.</td>
</tr>
<tr>
<td>QIr</td>
<td>Brazil-UNDP study</td>
<td>Estimate of QCS output (ARS) of GMR profilometer from RMSVA statistics of profile on 1.0 and 2.5 m baselengths (Queiroz 1979): QIr = -8.54 + 6.17 RMSVA$<em>{1}$ + 19.38 RMSVA$</em>{2.5}$.</td>
</tr>
<tr>
<td>BIr</td>
<td>TRRL (Overseas Unit)</td>
<td>Estimate of Bump Integrator trailer by root mean square deviations (RMSD) from best-fit line through elevations at 300 mm intervals on 1.8 m baseline: BIr = 472 + 1,437 RMSD + 225 RMSD$^2$ (Abaynayaka in Sayers, Gillespie and Queiroz 1986).</td>
</tr>
<tr>
<td><strong>3. Statistics of Discrete Wavebands</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PI</td>
<td>NCHRP Report 275</td>
<td>Root mean square elevation statistic from the 0.5 to 2.4 m wavelength band (Janoff and others 1985).</td>
</tr>
<tr>
<td>PU$_{3.0}$</td>
<td>TRRL</td>
<td>Variance of elevation from 3 m moving average (Jordan 1985).</td>
</tr>
<tr>
<td>CP$_{2.5}$</td>
<td>CRR Belgium</td>
<td>Average rectified elevation on 2.5 m moving average baseline (Appendix G in Sayers, Gillespie and Queiroz 1986). Also for 10 and 40 m baselengths.</td>
</tr>
<tr>
<td>$W_{sw}$</td>
<td>LCPC France</td>
<td>Mean square energy of profile signal in wavebands $sw$ (1 - 3.3 m), $mw$ (3.3 - 13 m) and $lw$ (13 - 40 m) (Appendix G in Sayers, Gillespie and Queiroz 1986).</td>
</tr>
</tbody>
</table>

**Source:** Author.
the major costs studies, and others. These are presented as a chart in Figure 2.15. The International Roughness Index (IRI) is the primary reference scale, and the variations to be expected when applying the conversion estimates to individual sections are represented by the 15th and 85th percentile confidence limits shown by the diagonal lines on each scale.

The relationships were derived from data of the IRRE (Sayers, Gillespie and Queiroz, 1986) and the Ann Arbor profilometer meeting (Sayers and Gillespie 1986), and the estimates and prediction statistics are listed in Table 2.5 (see also Paterson 1986 for details). These are reversible relationships which yield the same result when converting from either scale to the other. The estimates were developed from linear regressions by averaging the coefficients predicting scale A from scale B, and scale B from scale A in each case, and the statistics were determined by applying the conversion relationship directly to the data. The predictions and relationships between the various scales against the data are presented in Figure 2.16, relating the roadmeter-based QIm and BI scales (as used in the major road costs studies) with the profile-based IRI and French/ Belgian C2B scales; and in Figure 2.17, relating the profile-based scales of the French APL statistics and the Texan MO statistic to the IRI.

Approximate conversions for two other scales have been based on indirect analyses and await refinement by rigorous correlation studies. The British unevenness statistic (FU) is a squared-deviation statistic which is approximately related to IRI by:

\[ IRI = 2.0 \times FU^0.9 \]  

(2.1)

where FU is the profile variance from a 3 m moving average (Jordan 1984), based on a correlation to CPB (Cooper, Young and Gorski 1986). The Australian NAASRA roadmeter (National Association of Australian State Road Authorities), which is currently related to a standard vehicle, has about 37 counts per m/km IRI.

Straightedge equivalent

Surface tolerances are frequently expressed in terms of the deviation under a simple straightedge. The relationship between this deviation and IRI has been determined mathematically for various baselengths from the profile data of seven road sections covering the full range of roughness from 2 to 12 m/km IRI. The correlations are very high, as shown for the most common straightedge lengths of 2 m and 3 m in Figure 2.18. Typical relationships are presented below for the mean maximum deviation under a straightedge, the usual way a tolerance is expressed:

<table>
<thead>
<tr>
<th>Mean Maximum Deviation</th>
<th>Standard error</th>
<th>( r^2 )</th>
<th>Average deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>SED_{1.0} = 1.40 RI</td>
<td>0.60 mm</td>
<td>0.985</td>
<td>0.72 RI</td>
</tr>
<tr>
<td>SED_{2.0} = 2.29 RI</td>
<td>0.42 mm</td>
<td>0.997</td>
<td>1.23 RI</td>
</tr>
<tr>
<td>SED_{3.0} = 2.85 RI</td>
<td>0.76 mm</td>
<td>0.994</td>
<td>1.58 RI</td>
</tr>
</tbody>
</table>

where SED_{\lambda} = mean maximum (95th percentile) deviation under a straightedge of length \( \lambda \) meters, in mm; and RI = roughness, in m/km IRI. These relationships are particularly convenient for making simple inexpensive estimates of roughness in the field without the use of mobile equipment, and for relating construction standards to roughness.
Figure 2.15: Chart for approximate conversions between major roughness scales and the International Roughness Index (IRI)

<table>
<thead>
<tr>
<th>IRI (m/km IRI)</th>
<th>Q&lt;sub&gt;1m&lt;/sub&gt; (count/km)</th>
<th>Bl&lt;sub&gt;t&lt;/sub&gt; (mm/km)</th>
<th>CP&lt;sub&gt;2.5&lt;/sub&gt; (0.01 mm)</th>
<th>W&lt;sub&gt;SW&lt;/sub&gt;</th>
<th>CAPI&lt;sub&gt;25&lt;/sub&gt;</th>
<th>SI (PSI)</th>
<th>IM&lt;sub&gt;T&lt;/sub&gt; (in/mile)</th>
<th>IRI (m/km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>4</td>
<td>40</td>
<td>2.000</td>
<td>2.000</td>
<td>2.000</td>
<td>2.000</td>
<td>2.000</td>
<td>2.000</td>
<td>2.000</td>
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<tr>
<td>6</td>
<td>80</td>
<td>4.000</td>
<td>4.000</td>
<td>4.000</td>
<td>4.000</td>
<td>4.000</td>
<td>4.000</td>
<td>4.000</td>
</tr>
<tr>
<td>8</td>
<td>120</td>
<td>6.000</td>
<td>6.000</td>
<td>6.000</td>
<td>6.000</td>
<td>6.000</td>
<td>6.000</td>
<td>6.000</td>
</tr>
<tr>
<td>10</td>
<td>160</td>
<td>8.000</td>
<td>8.000</td>
<td>8.000</td>
<td>8.000</td>
<td>8.000</td>
<td>8.000</td>
<td>8.000</td>
</tr>
<tr>
<td>12</td>
<td>200</td>
<td>10.000</td>
<td>10.000</td>
<td>10.000</td>
<td>10.000</td>
<td>10.000</td>
<td>10.000</td>
<td>10.000</td>
</tr>
<tr>
<td>14</td>
<td>240</td>
<td>12.000</td>
<td>12.000</td>
<td>12.000</td>
<td>12.000</td>
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<tr>
<td>16</td>
<td>280</td>
<td>14.000</td>
<td>14.000</td>
<td>14.000</td>
<td>14.000</td>
<td>14.000</td>
<td>14.000</td>
<td>14.000</td>
</tr>
<tr>
<td>18</td>
<td>320</td>
<td>16.000</td>
<td>16.000</td>
<td>16.000</td>
<td>16.000</td>
<td>16.000</td>
<td>16.000</td>
<td>16.000</td>
</tr>
<tr>
<td>20</td>
<td>360</td>
<td>18.000</td>
<td>18.000</td>
<td>18.000</td>
<td>18.000</td>
<td>18.000</td>
<td>18.000</td>
<td>18.000</td>
</tr>
</tbody>
</table>

Notes:
On the 3-line scales, the center line represents the estimated value, and the left and right margins represent the low (15th percentile) and high (85th percentile) limits of individual values about the estimated value.

NOTES:
Conversions estimated on data from the International Road Roughness Experiment, (Sayers, Gillespie and Queiroz, 1986) as follows:

2. Q<sub>1m</sub> — Quarter-car Index of calibrated Maysmeter, Brazil-UNDP Road Costs Study
   \[ IRI = Q_{1m}/13 \pm 0.3 IRI \] (IRI<17)
3. Bl<sub>t</sub> — Bump Integrator trailer at 32 km/h, Transport and Road Research Laboratory, UK
   \[ B_t = 0.0032 B_{10.8} \pm 0.0014 B_{10.8} \] (IRI<17)
4. CP<sub>2.5</sub> — Coefficient of planarity over 2.5m baseline for APL72 Profilometer, Centre de Recherches Routiers, Belgium
   \[ IRI = CP_{2.5}/16 \pm 0.27 IRI \] (IRI<11)
5. W<sub>SW</sub> — Short Wavelength Energy for APL72 Profilometer, Laboratoire Central des Ponts et Chaussées, France
   \[ W_{SW} = 0.78 W_{SW} \pm 0.09 W_{SW} \] (IRI<9)
6. CAPI<sub>25</sub> — Coefficient of APL25 Profilometer, Laboratoire Central des Ponts et Chaussées, France
   \[ IRI = 0.45 \pm 0.16 \] (IRI<11)
7. SI — Serviceability Index, American Association of State Highway and Transportation Officials
   \[ SI = 5.5 \ln (S/I) \pm 25\% \] (IRI<12)
8. IM<sub>T</sub> — Inches/mile equivalent of IRI from Reference Quarter-Car Simulation at 50 mile/hr (see 'HR-reference' in Gillespie, Sayers and Segel NCHRP report 228, 1985; and 'RARS' in Sayers, Gillespie and Queiroz, World Bank Technical Paper 45, 1986)
   \[ IM_T = IM_T/63.36 \]
### Table 2.5: Relationships and statistics for conversions between roughness scales

<table>
<thead>
<tr>
<th>Conversion relationship</th>
<th>Standard error</th>
<th>C. V.</th>
<th>Bias slope</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRI = QI&lt;sub&gt;m&lt;/sub&gt;/13</td>
<td>0.919</td>
<td>15.4</td>
<td>0.989</td>
<td>m/km</td>
</tr>
<tr>
<td>= (QI&lt;sub&gt;r&lt;/sub&gt; + 10)/14</td>
<td>0.442</td>
<td>7.3</td>
<td>0.975</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 0.0032 BI&lt;sup&gt;0.88&lt;/sup&gt;</td>
<td>0.764</td>
<td>12.7</td>
<td>1.008</td>
<td>&quot;</td>
</tr>
<tr>
<td>= CP&lt;sub&gt;z,s&lt;/sub&gt;/16</td>
<td>0.654</td>
<td>12.4</td>
<td>0.993</td>
<td>&quot;</td>
</tr>
<tr>
<td>≥ 5.5 log&lt;sub&gt;e&lt;/sub&gt; (5.0/PSI)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 0.80 RARS&lt;sub&gt;5.0&lt;/sub&gt;</td>
<td>0.478</td>
<td>7.9</td>
<td>1.002</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 0.78 W&lt;sub&gt;SW&lt;/sub&gt;&lt;sup&gt;0.63&lt;/sup&gt;</td>
<td>0.693</td>
<td>11.5</td>
<td>0.994</td>
<td>&quot;</td>
</tr>
<tr>
<td>= CAPE&lt;sub&gt;2.5&lt;/sub&gt;/(2.2 + 0.8A)</td>
<td>1.050</td>
<td>17.4</td>
<td>1.030</td>
<td>&quot;</td>
</tr>
<tr>
<td>QI&lt;sub&gt;m&lt;/sub&gt; = 13 IRI</td>
<td>12.0</td>
<td>15.3</td>
<td>0.993</td>
<td>counts/km</td>
</tr>
<tr>
<td>= 9.5 + 0.90 QI&lt;sub&gt;r&lt;/sub&gt;</td>
<td>14.5</td>
<td>18.7</td>
<td>0.985</td>
<td>&quot;</td>
</tr>
<tr>
<td>= BI/(55 + 18 E)</td>
<td>11.7</td>
<td>15.0</td>
<td>1.002</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 0.81 CP&lt;sub&gt;z,s&lt;/sub&gt;</td>
<td>11.7</td>
<td>17.2</td>
<td>0.986</td>
<td>&quot;</td>
</tr>
<tr>
<td>≥ 72 log&lt;sub&gt;e&lt;/sub&gt; (5.0/PSI)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 7.9 W&lt;sub&gt;SW&lt;/sub&gt;&lt;sup&gt;0.70&lt;/sup&gt;</td>
<td>8.78</td>
<td>11.2</td>
<td>0.996</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 6.2 CAPE&lt;sub&gt;2.5&lt;/sub&gt;</td>
<td>18.29</td>
<td>23.3</td>
<td>1.13</td>
<td>&quot;</td>
</tr>
<tr>
<td>QI&lt;sub&gt;r&lt;/sub&gt; = −10 + 14 . IRI</td>
<td>6.32</td>
<td>8.3</td>
<td>1.024</td>
<td>&quot;</td>
</tr>
<tr>
<td>BI = 630 IRI&lt;sup&gt;1.12&lt;/sup&gt;</td>
<td>694</td>
<td>14.7</td>
<td>0.998</td>
<td>mm/km</td>
</tr>
<tr>
<td>= 36 QI&lt;sub&gt;m&lt;/sub&gt;&lt;sup&gt;1.12&lt;/sup&gt;</td>
<td>1100</td>
<td>22.8</td>
<td>0.985</td>
<td>&quot;</td>
</tr>
<tr>
<td>= (55 + 18 E) QI&lt;sub&gt;m&lt;/sub&gt;</td>
<td>673</td>
<td>14.2</td>
<td>0.976</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 62 QI&lt;sub&gt;r&lt;/sub&gt;</td>
<td>850</td>
<td>18.1</td>
<td>0.971</td>
<td>&quot;</td>
</tr>
<tr>
<td>CP&lt;sub&gt;z,s&lt;/sub&gt; = 16 IRI</td>
<td>10.5</td>
<td>12.4</td>
<td>0.994</td>
<td>0.01 mm</td>
</tr>
<tr>
<td>= 11 + 1.12 QI&lt;sub&gt;r&lt;/sub&gt;</td>
<td>14.8</td>
<td>17.6</td>
<td>0.995</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 1.23 QI&lt;sub&gt;m&lt;/sub&gt;</td>
<td>14.4</td>
<td>17.2</td>
<td>0.986</td>
<td>&quot;</td>
</tr>
<tr>
<td>= 11.7 W&lt;sub&gt;SW&lt;/sub&gt;&lt;sup&gt;0.68&lt;/sup&gt;</td>
<td>8.87</td>
<td>10.5</td>
<td>1.018</td>
<td>&quot;</td>
</tr>
<tr>
<td>MO&lt;sub&gt;m&lt;/sub&gt; = IRI/1.5</td>
<td>0.25</td>
<td>0.9</td>
<td>1.04</td>
<td>m/km</td>
</tr>
<tr>
<td>MO&lt;sub&gt;i&lt;/sub&gt; = 42 IRI</td>
<td>16.0</td>
<td>0.9</td>
<td>1.04</td>
<td>in/mile</td>
</tr>
</tbody>
</table>

**Note:**
- E = 1 if earth surface, = 0 otherwise.
- A = 1 if asphalt surface, = 0 otherwise.
- BI = TRRL Bump Integrator trailer at 32 km/h (mm/km).
- CP<sub>z,s</sub> = APL Profilometer coefficient of evenness (.01 mm).
- IRI = International Roughness Index (m/km).
- QI<sub>m</sub> = Roadmeter-estimate of QI roughness (counts/km).
- QI<sub>r</sub> = Profile RMSVA-function of QI roughness (counts/km).
- RARS<sub>5.0</sub> = ARS response of reference roughness simulation at 50 km/h (Sayers, Gillespie and Queiroz, 1986).
- W<sub>SW</sub> = Short wavelength (1-3.3 m) energy index of APL72.
- MO<sub>i</sub> = Maysmeter Output function of RMSVA (Table 2.4) (m/km; inch/mile).

**Source:**
Computer analysis of data from Sayers, Gillespie and Queiroz (1986) and Sayers and Gillespie (1986).
Figure 2.16: Relationships for conversion between Qlm (Brazil road costs study), BI (TRRL Bump Integrator trailer) and CP2.5 (French/Belgian APL profilometer scales of road roughness)

(a) Brazil Calibrated Maymeter, Qlm and Profile Roughness, IRI

(b) Brazil Calibrated Maymeter, Qlm and APL72 Profilometer Coefficient, CP2.5

(c) TRRL Bump Integrator Trailer at 32km/h and Profile Roughness, IRI

(d) TRRL Bump Integrator Trailer at 32km/h and Brazil Calibrated Maymeter, Qlm

Source: Paterson (1986).
Figure 2.17: Relationships of various coefficients of the French APL profilometer systems APL72 and APL25, and the Texan MD profile statistic, to the International Roughness Index (IRI)

(a) APL25 Coefficient, CAP25, and IRI

(b) APL72 Short Wavelength Energy, W_s and IRI

(c) APL72 Coefficient CP2.5 and IRI

(d) Texas MD (RMSA)-Statistic and IRI

Source: Paterson (1986) and data from Sayers and Gillespie (1986).
Figure 2.18: Relationship of deviations under a simple straightedge to IRI roughness statistic

(a) Under a 2-Meter Straightedge

(b) Under a 3-Meter Straightedge

Note: Maximum = 95th percentile deviation.
Source: Mathematical analysis of real road profiles.
Discussion

While such conversion relationships meet the practical need for comparing road performance and vehicle cost results across studies, there are important caveats to their application. First, they are representative primarily of the types of surface and ranges of roughness included in the data bases, which covered a very wide range but excluded surfaces placed by manual labor, and cobble-like surfaces (either boulder gravel surfaces or set-stone surfaces) which tend to have high amplitudes in the short wavelengths. The profile statistic conversions apply to both flexible and concrete pavements, but the roadmeter conversions have not yet been validated on concrete pavements. Corrugated surfaces ("washboarding") are particularly likely to cause deviation from the standard relationships because some roadmeters tune in (or resonate) on certain wavelength and vehicle speed combinations. Second, for scales based on response-type roadmeter systems, such as the BI and QIm scales, the relationships are valid for the "standard" equipment and operating procedures as they were applied during the IRRE. The relationships between profile-based scales (such as IRI, CP, MO and QIr) are generally more widely transferable because of their mathematical basis, although some constraints still apply on account of the mathematical filtering applied to the profile data. The MO - IRI relationship was derived from only paved roads but encompassing both flexible and rigid pavements and several abnormal defects (the units of MO need to be noted).

The relationships given here for QIm represent the standard calibration method of control section profiles used in the Brazil-UNDP road costs study, and are thus appropriate for all interpretation of the road deterioration, road user survey and speed and fuel experiment data from that study. The relationship given for BI represents the TRRL standard operating procedure for the Bump Integrator trailer at a speed of 32 km/h, as applied at the IRRE in 1982. As no profile calibration system was used to control the BI scale during the Kenya, Caribbean and India road costs studies, the same relationships must be assumed to apply, and to the best of our knowledge the application is valid. Instances of a bias of up to +20 or 30 percent in Bump Integrator measures have been observed in a few other cases however, and this serves to emphasize the importance of profile calibration even for "standardized" hardware such as the BI trailer.

The relationship between the QIm and BI scales is particularly important for the comparisons between the major road costs studies. Over the full range, the relationship is slightly nonlinear as given by:

\[ \text{BI}_r = 36 \ Q_{Im}^{1.12} \quad (2.2) \]

The nonlinearity is due mainly to the soft suspension characteristics and the slower speed of travel (32 versus 80 km/h) of the Bump Integrator trailer, which cause it to respond more strongly to the short wavelength roughness on earth roads than does the Maysmeter vehicle (the BI trailer has resonant frequencies corresponding to 0.75, 2.2 and 4.4 m wavelengths (Jordan and Young 1981)). For paved roads and gravel roads, on roughness up to about 100 QIm (8 m/km IRI), a linear relationship is reasonable, as follows:

\[ \text{BI}_r = 55 \ Q_{Im} = 720 \ IRI \quad (2.3) \]

and, for earth roads, the coefficient on QIm increases from 55 to 73.
On comparing the separate plots of $B_{Ir}$ and $Ql_m$ against the IRI profile reference in (a) and (c) of Figure 2.16, it is apparent that the errors of the two roadmeter systems are compounding when they are compared together, because they are responding differently to the same sections. Also the Maysmeter vehicles experienced resonance on two corrugated surface treatment sections that appear as outliers. These differences would have been reduced if both systems were to have had stiffer shock absorbers, since that suppresses the response at resonance frequencies.

The two APL statistics that correlate best with vehicle response, and in particular the IRI roughness scale, are the $CP_{2,s}$ and short wavelength energy ($W_{sw}$) indices as shown in Figure 2.17 and Table 2.4. The APL25 coefficient (CAPL25) has a generally poor correlation with IRI and other response-type measures because it is sensitive mostly to long rather than short wavelengths, and the correlation is thus best on asphalt concrete surfaces. All the APL statistics, except $CP_{2,s}$, tend to reach signal saturation and are not applicable to roughness levels above 9 m/km IRI in the case of the $W_{sw}$ (APL72) index, and above 11 m/km IRI in the case of the CAPL25 index, as can be seen from diagrams (a) and (b) in Figure 2.17.

In conclusion, it is apparent that highly acceptable transferability is now possible between past major studies involving road roughness where one of the above calibration references exist. However, occasional distortions can occur when stiff shock absorbers are not used, particularly on corrugations and the high frequency roughness associated with earth roads, surfaces placed by manual labor (macadams, cobbles or set-stones), boulder-size gravel roads, and so forth.

2.3.4 Relation between Roughness and Riding Quality

No definitive relationship between the Serviceability Index as defined in the AASHO Road Test (Carey and Irick 1960) and IRI has yet been established. Panel ratings, which were the initial basis for defining the Present Serviceability Index (SI) function, tend to vary considerably with the expectation of the users and their previous exposure to very high roughness levels, and thus the ratings can vary from country to country. SI was not defined for unpaved roads. Relationships derived from four panel rating sources, namely, Brazil and Texas (Working Document 10, in GEIPOT 1982), South Africa (Visser 1982) and Pennsylvania (Nick and Janoff, 1983), between PSR and the $Ql_m$ and IRI roughness scales are given in Figure 2.19. In the first three, ratings were related to direct physical measures of QI (in the Texan case, the panel rating was an estimate derived from a Texas waveband correlation with profile data as applied to Brazil section profile data), and for Pennsylvania, an approximate conversion of 1 count/km $Ql_m = 6.6$ inch/mile was used.

Considerable variations exist in the serviceability rating scales derived from the different sources: the Texan, Pennsylvanian and South African ratings represent users who were used to high-standard paved roads, but the means nevertheless varied by up to one rating interval for a given roughness, whilst the Brazilian raters attached much higher ratings for rough roads than did the other groups. A linear relationship between rating and roughness seems adequate over the range of 1 to 4 rating units on paved roads (Janoff and others 1985). By extrapolation, the scales indicate that a roughness of 130 to 175 $Ql_m$ is equivalent to 0 SI, except for the Brazilian case which included unpaved roads and
Figure 2.19: Approximate relationships between AASHO serviceability index, PSI and the QIm and IRI roughness scales, based on panel ratings from four sources.

Source: Paterson (1986).

A roughness of 175 was rated as better than 1 SI. The best continuous function meeting the scale's perfect score of 5 at a roughness of zero is as follows:

\[ Q_{Im} = 72 \log_e \left( \frac{5.0}{SI} \right) \]

\[ IRI = 5.5 \log_e \left( \frac{5.0}{SI} \right) \]  

However, a linear function may be more convenient over normal ranges of paved road roughness, and the following approximation is valid for a range of 1.8 to 4.2 PSI or 0.8 to 6 m/km IRI:

\[ Q_{Im} = 120 - 30 \ SI \]

\[ IRI = 9.2 - 2.2 \ SI. \]
The slope of the IRI-SI relationship averages -2.2 in this range and varies from -1.5 at low roughness to -3 at roughness worse than 5 m/km IRI. The common terminal levels of serviceability are therefore approximately:

- 4.2 SI \approx 13 \text{ counts/km QI}_m \approx 1.0 \text{ m/km IRI}
- 2.5 SI \approx 50 \text{ counts/km QI}_m \approx 3.8 \text{ m/km IRI}
- 2.0 SI \approx 65 \text{ counts/km QI}_m \approx 5.0 \text{ m/km IRI}
- 1.5 SI \approx 86 \text{ counts/km QI}_m \approx 6.6 \text{ m/km IRI}

Subsequent use of relationship (2.4) has shown poor results for SI less than 1.5 (IRI more than 6.6), and IRI = 12 - 3.6 SI is suggested for that range.

2.4 MEASUREMENT METHOD AND ACCURACY: BRAZIL STUDY

The accuracy with which roughness can be measured has an important bearing on the success of developing empirical models for predicting and explaining roughness trends, and also on the confidence with which roughness can be used as a planning criterion for maintenance intervention and economic evaluation. Thus it is important to quantify and evaluate the accuracy of the various measurement methods because that influences the choice of measurement method and ultimately the analytical technique applied to the data. Potentially there are sources of error between repeat measurements (deriving from variations in both the instrument and method), and between instruments (whether like or not). The effectiveness of calibration and control of the methods is relevant. In this section, therefore, the control methods and measurement errors are evaluated. This is done with particular respect to the Brazil-UNDP road costs study and the International Road Roughness Experiment both because of the immediate relevance to the empirical analyses and because the studies provided a large comprehensive source of data.

In the Brazil-UNDP road deterioration study, the roughness of the study sections was measured at 4 to 6-month intervals using calibrated roadmeters, comprising a Maysmeter (a conventional Mays Ride Meter sensor adapted to a digital readout), fitted in Chevrolet sedans (Brazilian Opala model) and station wagons (Brazilian Caravan model). Examples of the data collected, shown in Figure 2.20, indicate the measurement variability that was experienced even under controlled calibration. Deviations that appear similar for both lanes are systematic errors, presumably due to calibration drift in the Maysmeter vehicles, whereas other deviations are random errors, presumably due to operational variations.

2.4.1 Calibration

The nine Maysmeter vehicles were calibrated on a series of twenty control road sections for which a reference roughness had been measured. Elevation data from longitudinal profiles measured by a General Motors Research (GMR) profilometer (Volume 3 in GEIPOT 1982) were processed through a quarter-car simulation of the Bureau of Public Roads (BPR) roughometer. The resulting slope (ARS) output was defined as the quarter-car index (QI), expressed in counts per km. The QI value was the reference roughness on each control section against which the Maysmeters were calibrated. Once calibrated, the Maysmeters were checked daily on two road sections, if testing was being done in the vicinity, or else before and after a mission. These routine tests used control charts which required the mean and range of five run readings to fall within the following limits:
Figure 2.20: Examples of time-series roughness measurements using calibrated Maysmeter roadmeters in Brazil-UNDP road costs study

Source: Paterson (1985)

Control limits for the mean = \( \pm 0.58 \, R \), and

Upper limit for the range = \( 2.11 \, R \),

where \( R \) is the mean of the ranges of readings from five runs on each applicable control section. Operational standards were set for the ballast, tire pressure, and so forth, and the measured error for controlled operating conditions was of the order of 3 to 5%, which is comparable to that cited in the NCHRP study by Gillespie, Sayers and Segel (1980). Whenever a Maysmeter failed the control test, or when tires on shock absorbers on the vehicle were changed, the vehicle was recalibrated.

An indication of the medium term variations in the dynamic characteristics of the cars, which were compensated for by the calibration, can be assessed from the calibration factors. The calibration equation took the following form:

\[
Q_{ij}^* = a_i + b_i \, M_{ij}
\]  

(2.6)
where M_{ij} = response in number of 5.08 mm counts per 320 m length for an Opala Maysmeter i at 80 km/h on section j;

QI_j = estimate of reference roughness (QI) on section j; and

a_i, b_i = parameters to be estimated (intercept a_i was usually insignificant) for Maysmeter vehicle i.

Analysis of the calibration equations for nine survey vehicles over a three-year period (Paterson 1985), showed that the calibration factor for each vehicle changed over successive calibration periods, typically by less than 10 percent but sometimes by as much as 35 percent between successive periods, (see Figure 2.21). The larger changes were either associated with identifiable events such as changes in the tires or shockabsorbers, or have subsequently been attributed to calibration error. The coefficients of variation of the calibration factor over the two to four year periods of vehicle use ranged from 8.9 to 16.8% for the nine vehicles. The range of the factor about the mean for a given vehicle was between 30 and 60 percent of the mean value.

Figure 2.21: Variation of roughness-response calibration factor over medium-term for Opala Maysmeters at 80 km/h speed

2.4.2 Measurement Errors

Roughness measurements vary across repeat runs, across time, and across vehicles, due to both random and systematic errors.

Random errors

Repeatability, which includes the errors of transverse and longitudinal location, speed, and the sensor performance, is related to the variance of replicate runs making up an individual measurement. The average standard error and coefficient of variation of the mean of the three replicate runs of each measurement, computed from a total of 25,000 runs on unpaved road sections, were only 0.1 m/km IRI and 4.4 percent respectively in the Brazil study and essentially independent of speed and vehicle (Paterson 1985). The plotted values in Figure 2.22 show that the error increased approximately in proportion to the square root of the

Figure 2.22: Roughness measurement error across-runs as a function of speed and mean response for Opala Maysmeters in Brazil road costs study

Source: Paterson (1985)
mean, though with considerable scatter. This is slightly at variance with the IRRE finding that the repeatability error of calibrated vehicles was essentially independent of the roughness level, and may reflect the reality of practical operating conditions as distinct from the well-controlled conditions of the IRRE. Data for paved roads from the IRRE show a smaller repeatability error averaging 2 percent of the mean (3.7 m/km IRI, 48.4 QI).

The reproducibility of measurements across different calibrated vehicles and across survey dates tends to be worse than the repeatability. Over the duration of the Brazil-UNDP study, the log mean squared error was 0.018 on paved roads, which is equivalent to a coefficient of variation of 14 percent (Visser and Queiroz 1979), and 0.028 on unpaved roads, which is equivalent to a coefficient of variation of 18 percent (Paterson 1985). Hence, the 95th percentile confidence intervals for a calibrated roughness measurement were -24 to +31 percent for paved roads and -31 to +40 percent on unpaved roads. Using data from the IRRE, it was shown that this error reduced to a coefficient of variation of 11.2 percent when five replicate runs per measurement were used instead of three, and that the variances on paved and unpaved roads were approximately equal (Paterson 1985).

The higher error on unpaved roads was due to the high variability of roughness across each section, variations in the alignment of the vehicle in the "wheelpaths," variations in speed, and (possibly) variation in the dynamic characteristics of the vehicle. A study of the shock absorber response showed that damping characteristics were diminished at operating temperatures above 50°C, and that appreciable changes in vehicle response could occur if the shock absorber temperatures rose significantly during successive runs. For the Brazilian Opala vehicles, the equilibrium shock absorber temperature was found to rise rapidly with the roughness level, as follows (Paterson 1985):

\[ \Delta T_{\text{um}} = 3.88 + 6.24 \text{RI} \]  \hspace{1cm} (2.7)

where \( \Delta T_{\text{um}} \) = difference between the ambient and equilibrium shock absorber temperatures for the Opala vehicles, °C; and RI = road roughness, m/km IRI.

The change in vehicle response, with respect to operations at ambient conditions, was found to be a linear function of the temperature difference, given by (Paterson 1985):

\[ \Delta \text{ARS} = 0.1 (\Delta T_{\text{um}} - 23) \]  \hspace{1cm} (2.8)

where \( \Delta \text{ARS} \) is the difference between the steady state response of the warmed-up vehicle at the equilibrium shock absorber temperature and the initial response of the vehicle at the cool, ambient temperature condition (in m/km). Thus significant errors can occur on roughness exceeding 4 m/km IRI, unless the equilibrium temperature has been achieved through twenty to thirty minutes of continuous running. With no warming up, the errors would be a maximum of 0 percent at 4 m/km IRI and -40 percent at 15 m/km IRI.

In summary, the random errors tend to increase with roughness, as follows:

\[ \text{Standard error of roughness} = 0.16 \sqrt{\text{RI}} \]

where RI = roughness in m/km IRI, for response-type measurements.
Systematic errors

Systematic errors are potentially the most damaging to the validity of empirical models. They may be either short-term, deriving from errors in the profile reference or calibration method and described simply as "calibration error", or long-term, deriving from a long-term bias or drift in the profile reference.

Short-term systematic errors in the profile reference were evaluated from the dynamic profilometer control section measurements over the 3.5 year main study period (Paterson 1985). Measurement variations, normalized by the long-term section mean, had a coefficient of variation of 7 percent (log variance of 0.0055) overall. However, a date-specific bias was evident which caused all profile readings on a given date to be either high or low, as shown in Figure 2.23, and this appeared to be due to the method of electronic calibration of the sensors before measurement. The average error was 6 percent up to 1979 dropping to 3.2 percent thereafter with maximum deviations of 13 percent. In the study, these errors were partially compensated for by adopting a rolling average as the reference roughness value of a given control section. Other short-term systematic errors apparently arose in the vehicle-specific calibration equations, evidenced by a consistent bias which occasionally occurred in measurements on adjacent subsections and sections within a short period (see Figure 2.20 for example). Some of the variations in vehicle dynamic sensitivity, shown in Figure 2.21 were therefore partly due to calibration errors.

Figure 2.23: Short-term systematic bias of the GMR profilometer used to measure reference roughness (QI) on calibration sections in Brazil-UNDP road costs study.

Long-term systematic bias in the dynamic profilometer between 1977 and 1979 was evaluated from the trends of roughness values on the ostensibly stable control sections, because no independent profile measure existed for that period. It was found that the profilometer QI of the control sections decreased slightly over that period by an average of 4 percent, with changes ranging from -10 percent to +1 percent for the eighteen sections remaining in service (Table 3 in Paterson 1981). This slight negative trend was effectively suppressed for the Maysmeter calibration process through the rolling average technique that was applied to the profilometer QI trend data and thus the resulting systematic error was small and less than -1 percent per year. During the period between 1979 and 1980, profiles were measured by both the dynamic profilometer and by rod and level survey. Comparison of the QI statistic, which was developed on 1979 data and is time-stable by virtue of being a mathematical function of profile elevations, and the profilometer QI, showed a standard error difference of only 0.9 percent in 1979 and 1.1 percent in 1980. It was concluded therefore that the QI and QIr roughness reference statistics were equivalent in mean values, and free from significant long-term bias.

However, other data show that QIm, the Maysmeter estimate of QI and QIr from calibration on only asphalt surfaces, is not exactly equivalent to QI across all surface types. Data from the IRRE, which included three of the Brazil road costs study Maysmeter vehicles, are shown in Figure 2.24, comparing QIm and QIr on four surfaces, namely asphalt concrete, surface treatment, gravel and earth. The regression line shown (which is approximately orthogonal and estimated on the mean of the three vehicles) has the form:

\[ Q_{Im} = 9.50 + 0.902 Q_{Ir} \] (2.9)

with \( r^2 = 0.892 \); standard error = 15.3 counts/km QIm; sample = 49 sections.

The fact that the positive intercept has nearly the same magnitude as the negative intercept in the RMSVA-definition of QIr (-8.54, Table 2.3) indicates that the QIr profile statistic ought not to have included an intercept if it were to correctly represent the roadmeter vehicles. Thus, the transfer of predictive models derived from the Brazilian data must be made through the QIm scale (which was used for all empirical estimation analyses) instead of the QIr scale. The relevant conversion relationships are therefore (Table 2.4):

\[ IRI = Q_{Im} / 13 \]

\[ \hat{B}_{Ir} = [36 Q_{Im}^{1.12}, \text{ or } 55 Q_{Im} \text{ for non-earth roads.} \] (2.10)

2.4.3 Source of Variability in Trends

Finally we return to consider the source of the trend variations shown in Figure 2.20 and, in particular, explanations for the apparently negative change in roughness on some sections observed over the three- to five-year duration of the study, in the absence of maintenance. A total of 27 of the 380 study subsections showed a negative change of roughness in the absence of maintenance, with an average value of -2.0 QIm (-0.16 m/km IRI), and a worst value of -6.7 QIm (-0.5 m/km IRI) or 30 %, as compared with the overall mean of 8.2 QIm (0.63 m/km
IRI). Based on the error review above, it is apparent that most of the variability was derived from the reproducibility error, and that much of this was probably calibration error, as shown by the systematic effects which were evident in measurements by a specific vehicle in a specific time period. As it was not possible to correct these errors retrospectively, and as the short-term systematic errors were irregularly distributed across sections and time, it is necessary in the analyses of the data to regard them as essentially random errors and to select the analytical method so as to minimize the error effect.

The resulting standard error of individual roughness measurements, after some averaging through merger with other pavement condition variables on staggered dates, was 4.1 Q\text{fm} (0.4 m/km IRI). As this error is approximately four times the average change in roughness between successive observations (see Table 8.1), its effect on the analyses can be substantial. With such a result coming from controlled calibrated measurements it is clear that response-type systems are inadequate for road deterioration research studies, and thus it is considered essential that profilometry systems be used for this purpose in future research (Paterson 1985; Sayers, Gillespie and Paterson 1986).

2.4.4 Speed Effects and Correction

Speed affects a vehicle's response to roughness because as the speed increases, so also do the wavelengths corresponding to the frequency bandwidth of the vehicle. For example, a vehicle is typically most sensitive (unit gain) to the 0.5 to 6 m wavelength band at 20 km/h, and to the 1.8 to 22 m wavelength band at 80 km/h. Consequently, the response of the vehicle at different speeds depends...
upon the spectral contents of the roughness in the different wavebands perceived by the vehicle. Thus the response of a vehicle on earth roads may be relatively higher at low speeds than high speeds because of the high short wavelength content of earth road profiles, while on asphalt concrete roads the reverse could be true because of the low content of short wavelength roughness.

Extensive data on speed-roughness effects collected in the Brazil-UNDP study verified these effects (Paterson and Watanatada 1985), and a general relationship between vehicle speed, roughness and their effect on vehicle response is shown in Figure 2.25. The response (average rectified slope) is normalized relative to the response at 80 km/h, termed the response ratio (KM). It can be seen that there is very little change (less than ten percent) in the vehicle response over speeds ranging from 50 to above 80 km/h. At lower speeds, however, the impact of speed becomes appreciable. Increases in the level of road roughness have a strong impact, raising the relative response from negative to positive, due to the higher amplitudes found in the short wavelength band at high roughness levels.

Frequently in road surveys, roughness measurements must be made at speeds lower than the reference speed, on account of traffic congestion or severe geometry, and thus correction to the reference roughness is required. In the current guidelines (Sayers, Gillespie and Paterson 1986), the recommended procedure is to establish separate calibration equations at each survey speed for the vehicle against IRI and to apply the appropriate calibration to the survey data.

**Figure 2.25: Effects of vehicle speed and road roughness on response of roadmeters and typical road vehicles**

![Figure 2.25](image)

*Source: Paterson and Watanatada (1985).*
During the Brazil-UNDP study, however, a different approach was adopted in which the adjustment for survey speed was made by a general equation relating the Maysmeter responses at the survey and reference speeds. This was necessary only on unpaved roads, where three survey speeds of 20, 50 and 80 km/h were generally used (though the highest speed achieved depended on the level of roughness).

Based on an initial sample of data, the original study used linear conversion functions (GEIPOT 1982), but later analysis on a much larger sample of 8,915 observations found that nonlinear functions were necessary (Paterson and Watanatada 1985). The relationships are shown with the data in Figure 2.26, and are as follows:

1. Gravel surfaces:
   \[ M_{80} = 1.39 M_{80} - 0.831 = 1.64 M_{50} - 0.712 \]  
   (2.11)

2. Earth surfaces:
   \[ M_{80} = 1.36 M_{80} - 0.809 = 1.92 M_{20} - 0.577 \]  
   (2.12)

where \( M_{80}, M_{50}, \) and \( M_{20} \) are the roadmeter counts in m/km for speeds of 80, 50 and 20 km/h respectively (where one 5.08 mm-count per 80 m subsection = 0.0635 m/km).

Corrections were therefore applied to the unpaved road roughness data in those cases where the maximum survey speed had been less than 80 km/h. The corrections amounted to increases of 20 to 40 percent at low roughness of 25 QI_m (2 m/km IRI) and decreases of 8 to 30 percent at high roughness of 200 to 250 QI_m (17 to 19 m/km IRI). The resulting Maysmeter estimate of QI was designated QI_m to distinguish it from the original QI*. For all paved roads, and for unpaved roads where the roughness was measured at 80 km/h, QI_m equals QI*.

2.4.5 Concluding Comment

A danger inherent in any scientific discussion of errors is that it may convey the impression of a lack of confidence in the data. The discussion in this case shows the opposite. The road deterioration data in the Brazil-UNDP study is particularly valuable because the roughness measurements were calibrated using a methodology that was the forerunner of the recently established guidelines, and so confidence in the long-term reference datum of the roughness data is high. The discussion has revealed however that short-term measurement errors from various sources were appreciable in the data, and so these need special attention during the analytical process. It is clear that control procedures for calibration and measurement operations need to be carefully and tightly enforced if good quality data are to be gathered by roadmeters.

A second important value of the roughness data from Brazil is that the Brazilian reference statistic (QI) correlates very highly with the international roughness index (IRI), because of their similar definitions. Thus most of the strengths and applicability of the IRI to road deterioration, vehicle response and subjective ride rating, apply also to the QI_m data. Hence, throughout this volume, IRI is used as the roughness measure, the data having been converted through the relevant equations in Table 2.5.
Figure 2.26: Effects of survey speed on roadmeter response on gravel and earth roads in Brazil

Note: ARSM is the averaged rectified slope output of the roadmeter; the subscript is the roadmeter's speed in km/h.

Source: Paterson and Watanatada (1985).
CHAPTER 3
Unpaved Roads: Concepts and Models

Unpaved roads comprise the major part of most road networks, amounting
for example to eighty one percent of all roads in a major survey of developing
countries (World Bank 1987a) but varying with the per capita income and population
density from about ninety percent in Africa and Latin America down to about
seventy percent in Asia, Europe and the Middle East. The proportion of total
traffic carried on unpaved roads varies widely from the order of only two percent
to over fifty percent of the total vehicle-kilometers travelled (VKT), but the
economic importance of these is often appreciable since many provide the farm to
market access in rural areas. Thus unpaved roads are an important part of road
network management, and the maintenance standards and policies applied to control
their deterioration have important economic consequences.

Unpaved roads are broadly classified into engineered roads, and tracks,
and into gravel and earth surfacings, since these factors influence both the level
of service and the deterioration of the road. This chapter describes the life-
cycle of engineered unpaved roads, the modes of distress that characterize deteri-
oration, the effects of maintenance, and the road, traffic and environmental
factors which influence them. The requirements for predictive models are consid-
ered in the context of previous studies, and then models for predicting roughness
and material loss, and the effects of blading and spot-regravelling maintenance,
are developed from data collected in Brazil. Finally, the predictive models are
validated against data from independent studies.

3.1 LIFE-CYCLE OF UNPAVED ROADS

3.1.1 Classification

Unpaved roads comprise the lower classes of the road network hierarchy,
and generally carry low volumes of traffic ranging from a few vehicles to up to
several hundred vehicles per day. The geometric standards vary considerably, and
it is necessary to make a primary classification of unpaved roads into engineered
roads, which have controlled alignment, formation width, cross-section profile and
drainage; and tracks, which are essentially ways formed by trafficking along
natural contours with or without the removal of topsoil. The unpaved roads that
are classified as part of a country's network are usually engineered or partly-
engineered roads, and tracks are usually not classified.

This study of deterioration and maintenance effects is applicable
primarily to engineered unpaved roads because the available data bases dealt only
with such roads and not with tracks. Some of the findings are likely to be appli-
cable to tracks as well, certainly as a first estimate. However, it is likely
that the environmental effects of drainage and rainfall may be poorly represented
for tracks in regions where these factors are important.
A variety of definitions have been used to classify unpaved roads into gravel and earth roads. The term "earth road" is sometimes used to denote a track as opposed to an engineered road. In the Kenya study of road deterioration, "earth road" described all unpaved engineered roads for which the surfacing material was outside the material gradation specification for gravels of the Kenya Ministry of Works (Hodges, Rolt and Jones 1975). In the Brazil study "earth road" denoted those unpaved roads having a surface of predominantly fine soil materials with more than 35 percent finer than 0.075 mm particle size (GEIPOT 1982). In the present study, this last definition has been adopted because of its simple physical definition and transferability, and because the Brazilian data were used as the primary data base.

3.1.2 Deterioration

The deterioration of unpaved roads is governed by the behaviors of the surfacing material and the roadbed under the combined actions of traffic and the environment. The surfacing is typically 100 to 300 mm thick and serves as both the wearing course and the basecourse of the pavement, providing sufficient structural strength and cover thickness to distribute the applied traffic loads to the roadbed material. As the surfacing comprises a natural material, it is usually permeable, although in some cases the permeability may be very low, such as in densely-graded plastic gravel or cemented material (which includes self-cementing materials like laterites, ferricretes and calcretes. Thus material properties, rainfall, and surface drainage influence the behavior of the surfacing under traffic; likewise, surface water runoff and side drainage usually affect the moisture penetration to the roadbed and thus its bearing capacity.

There are three fundamental mechanisms of deterioration, namely wear and abrasion of the surface material under traffic, deformation of the surface material under stresses induced by traffic loading and moisture condition, and, finally, erosion of the surface by traffic, water and wind. Consequently, the modes of deterioration differ in dry weather and wet weather, on the one hand, and depend on the strength of the surfacing and roadbed material (which are most critical in wet weather), on the other hand. The modes and the approaches for modelling then can be placed in four categories, noted by Visser (1981), as follows.

Dry weather deterioration

Under dry weather conditions, the most prominent deterioration mechanisms are:

1. Wear and abrasion of the surface, which generates loose material and develops ruts;
2. Loss of the surfacing material by whip-off and dust;
3. The movement of loose material into corrugations under traffic action; and
4. Ravelling of the surface, in cases where there is insufficient cohesion in the material to keep the surface intact. This could be caused either by the abrasive action of vehicle tires, or by
injudicious blading of the surface. At points where ravelling occurs, tire action continues the abrasion process, and loose material is removed from the abraded areas. This results in depressions and increased roughness.

These mechanisms result in roughness and material loss, both of which are best studied and modelled by empirical methods, analysing the field performance of a sufficient variety of roads to identify and quantify the factors involved.

While rigorous guidelines to control such deterioration are not readily discernible from the literature, there is general consensus that these would focus on properties of the surface material. A sufficient proportion of fines (which provide cohesion) appears a primary factor to prevent ravelling and looseness and to suppress any tendency for corrugating (Robinson 1980, Heath and Robinson 1980, Visser 1981). A minimum level of

\[ P_{075} \geq 14 \]  

(3.1)

where \( P_{075} = \) percentage of material finer than 0.075 mm, was recommended by Visser based on a review of empirical studies and is consistent with the other reviews cited.

The proneness of a material to corrugate is still not clearly understood but, in addition to a minimum fines content, the angularity and gradation of particles in the material appear to be important factors in suppressing the proneness, probably through enhancing the shear strength of the material under wheel slip (Heath and Robinson 1980). Corrugations are formed and perpetuated by forced oscillations at the resonant frequencies of vehicles' suspension and tire systems, according to most researchers (see Heath and Robinson 1980), but the author's own observations indicate that corrugation frequencies relate most closely to the tires' resonance, being equivalent to frequencies in the order of 30 Hz (Section 2.2.2). Extensive research by the U.S. Forest Service on central tire inflation vehicles has shown that low inflation pressures reduced or eliminated the occurrence of corrugations (Stuart and others 1987), a finding which supports the importance of tire resonance and material shear strength factors.

Wet weather deterioration of adequate pavements

Under wet weather conditions the shear strengths of the materials determine the pattern of deterioration. When the shear strengths of the surfacing and roadbed materials are adequate for the stresses induced by traffic, deterioration occurs only at the surface. This is prevalent in regions where either road drainage is good, or good quality materials are found. The major modes of deterioration under these conditions are:

---

1/ Note that upper limits on the fine fraction are dictated by considerations of shear strength for wet weather performance, as detailed later.

2/ The term pavement is defined as the combination of compacted roadbed or formation, and layers of materials placed on the roadbed, configured so as to carry traffic; it is not restricted to pavements with a sealed surface of bituminous or concrete materials.
1. environmental and traffic influences on surface erosion;

2. wear and abrasion of the surface by traffic causing rutting and loss of the surfacing material; and

3. the formation of potholes under traffic action. Free water on the surface accumulates in the depressions, and the passage of a vehicle tire stirs up the water causing fine material to pass into suspension. Water, with the suspended fine material, is also forced out of the depression. Under the action of many wheel passages and sufficient water, this is a rapidly accelerating phenomenon.

**Wet weather deterioration with weak surfacing layer**

When the surfacing layer has inadequate shear strength under the operative drainage conditions to sustain the stresses applied by traffic loadings, shear failure and deformation occur. The road surface will be soft and slushy under wet conditions so that, while it may be possible for a few light vehicles to pass, the road will become impassable after a relatively small number of vehicle passages. Traditionally, a simple shear strength test such as the California Bearing Ratio (CBR) has been used to identify materials that resist shear failures, but other material properties such as plasticity and fineness also influence the behavior under these conditions.

Empirical studies by Visser (1981) showed that the soaked CBR of the surfacing material was the most reliable indicator of passability, and preferable to the plasticity index or percentage of fines. Based on the data shown in Figure 3.1, criteria for ensuring that a road remains passable during a wet season (given there is no flooding) were developed as follows:

\[
SFCBR > 8.25 + 3.75 \log_{10}(ADT)
\]

(3.2)

where

- \( SFCBR \) = the soaked California Bearing Ratio at standard Proctor laboratory compaction (600 kJ/m²), in percent, which is the minimum for ensuring passability; and
- \( ADT \) = the average daily vehicular traffic in both directions, in vehicles per day.

**Wet weather deterioration with weak roadbed material**

Where the in situ roadbed soil is weak, a pavement needs to be placed to protect the roadbed and limit the deformation developing under traffic to acceptable levels. When the pavement is inadequate and the subgrade or roadbed is overstressed, deterioration takes the form of rutting, or permanent deformation in the wheelpaths. This type of deterioration is prevalent in areas of poor surface and subsurface drainage, or during spring thaw conditions in freezing climates when the roadbed reaches relatively high moisture contents, or in areas of weak soils when design standards are inadequate.

The thickness and stiffness of the pavement layer(s) (typically, only one layer, the surfacing, is required for unpaved roads) need to be sufficient to distribute the applied loads so that the stresses and strains induced within the roadbed have been reduced to levels at which the permanent deformation of the
Figure 3.1: Suggested material selection criteria to ensure the passability of unpaved roads in wet weather

Legend:
- Road Passable
- Road Impassable


roadbed material is acceptable. These stress levels depend to a large extent on the volume and loading of traffic and the shear strength of the roadbed material in situ which, in turn, depends on the compacted density and the moisture content associated with the climate and drainage conditions.

Traditionally, the thickness and material strength required have been determined by empirical methods, and Figure 3.2 shows the criteria developed by the United States Corps of Engineers for the thickness of cover required depending on the strengths of the roadbed and surfacing materials. The criteria take the form of the following model (based on Hammitt (1970) and Barber, Odom and Patrick (1978), metricated and simplified for equivalent single wheel loadings of 40 kN):

$$\log_{10} \text{HG} = 1.40 + 12.3 \text{Cl}^{-0.466} \text{C}_2^{-0.142} \text{NE}^{0.124} \text{RD}^{-0.5}$$  \hspace{1cm} (3.3)

where

- \text{HG} = \text{the thickness of gravel surfacing, in mm;}
- \text{Cl} = \text{soaked CBR of surfacing material, in percent;}
- \text{C}_2 = \text{soaked CBR of roadbed soil, in percent;}
- \text{NE} = \text{design number of cumulative equivalent 40 kN single wheel loads at 550 kPa tire pressure;}
- \text{RD} = \text{maximum allowable mean rut depth, in mm.}
Figure 3.2: U.S. Corps of Engineers' design criteria for thickness of surface material of gravel roads based on soaked California Bearing Ratio

Note: Criteria: 75mm rut depth; 10,000 coverages equivalent to 40 kN single wheel loads.

In the more general case, the coefficient 12.3 was replaced by

\[ 0.856 P^{0.235} Q^{0.285} \]

where \( P \) = equivalent single wheel load, kN;
\( Q \) = tire inflation pressure, kPa; and NE would be replaced by \( N \),
the number of coverages of load \([P,Q]\).

Other design criteria have been developed by Greenstein and Livneh (1981) for data in Thailand and Ecuador, including the following estimate of material strength required for earth roads under a criterion of 75 mm maximum rut depth:

\[ C_2 = 0.0138 N^{0.172} P^{0.580} Q^{0.490} \]

(3.4)

where the variables are as defined above. This is shown in Figure 3.3.

We note, in passing, that these equations indicate a very strong influence of tire pressure on pavement deformation. For example, Equation 3.4 estimates a threefold improvement in life from a 32 percent reduction in pressure,
and Equation 3.3 estimates slightly less. In general, they show

\[ N = Q^k \]

where \( k = 2.30 \) in Equation 3.4 and \( k = 2.86 \) in Equation 3.4.
A theoretical mechanistic approach, in which the strains induced in the subgrade are computed through analysis of an elastic layered model of the pavement and evaluated with respect to the resilient modulus and deformation properties of the subgrade, has been presented by Visser (1981). While the theoretical approach is powerful for special studies, such as the design of unpaved roads for very heavy loadings, the empirical approach is preferred for most general applications.

### 3.1.3 Modes of Distress and Maintenance

Based on this general classification of deterioration mechanisms and design criteria, specific modes of distress can be identified which should form the basic scope of the predictive models required for an economic evaluation of deterioration, maintenance and associated user costs. For unpaved roads with generally adequate material specifications and pavement thickness, the principal modes of distress are:

1. **Roughness**, which increases over time under the actions of traffic and environment, and is defined in units of a standard roughness scale such as IRI, and

2. **Material loss** from the surfacing, which occurs under the actions of traffic (through whip-off of stones and dust loss) and of erosion by water and wind, and is defined by the change in average thickness of the surfacing material over time.
These two modes of distress are the ones which are corrected by regular maintenance activities, comprising blading by motorized or towed grader, spot regravelling, dust palliatives, and full-width regravelling (although this last is sometimes also classified as a rehabilitation activity).

The other modes of distress are ones which need to be addressed at the "design" or material selection stage of the construction or rehabilitation of unpaved roads, namely:

3. Rutting, which develops under traffic when the surface or roadbed materials have inadequate shear strength under the traffic loading and moisture conditions prevailing, and which is measured, for example, as the average rut depth in the wheelpaths under a 1.2 m straight edge, in mm;

4. Surface looseness, which affects the tracking, skidding and safety of vehicles and is measured by loose depth, in mm (see Hodges, Jones and Rolt 1975); and

5. Impassability, which occurs when the surfacing material has inadequate strength (usually through saturation or inundation) to allow a vehicle to pass over the surface.

These modes of distress are controlled through the material strength and thickness design criteria discussed in the previous section.

3.1.4 Maintenance

The maintenance activities on unpaved roads are classified and summarized in Table 3.1. The categories of activity vary greatly in usage, but these definitions of routine maintenance, resurfacing, rehabilitation and betterment are part of a coherent terminology for road expenditures (see also Table 4.2).

In essence, spot regravelling, drainage and verge maintenance, dragging, dust control, and "shallow" blading, are all regular or routine maintenance activities normally carried out under annual financing and requiring only operational programming at the local level. In some instances, however, where equipment resources are scarce and require special financing, dragging and shallow blading are only undertaken when specifically programmed and funded in the same way as periodic maintenance. Resurfacing, comprising regravelling, or deep blading with reprofiling and (preferably) recompaction, is a less frequent, periodic maintenance activity which restores and maintains the existing road standards. Rehabilitation is typically a major resurfacing exercise, combined with reformation of the existing pavement and overhaul or renewal of the drainage facilities, designed to fully restore the road standards and enhance them to meet current structural needs. Betterment works include rehabilitation with the enhancement of geometric standards, and the upgrading of earth roads by the provision of all-weather gravel surfacing.

3.1.5 Life Cycle of Deterioration and Maintenance

The life cycle of deterioration and maintenance of unpaved roads is thus depicted primarily by the trends of roughness and surfacing material loss, and the effects of maintenance on them, as illustrated in Figure 3.4.
Table 3.1: Maintenance categories and activities for unpaved roads

<table>
<thead>
<tr>
<th>Mode</th>
<th>Activity</th>
<th>Effect</th>
</tr>
</thead>
<tbody>
<tr>
<td>Routine maintenance</td>
<td>Spot regravelling</td>
<td>Fill potholes and small depressions; reduce roughness, exclude surface water.</td>
</tr>
<tr>
<td></td>
<td>Drainage and verge maintenance</td>
<td>Control runoff of surface water, reduce erosion and material loss, improve surfacing and subgrade strengths by lowering moisture contents.</td>
</tr>
<tr>
<td>Dragging</td>
<td></td>
<td>Redistribute surface gravel, fill minor depressions, improve safety.</td>
</tr>
<tr>
<td>Shallow blading</td>
<td></td>
<td>Redistribute surface material, fill minor depressions, reduce roughness.</td>
</tr>
<tr>
<td>Dust control</td>
<td></td>
<td>Controls depth of loose fine material and dust loss.</td>
</tr>
<tr>
<td>Resurfacing</td>
<td>Full regravelling</td>
<td>Restore required thickness of surfacing.</td>
</tr>
<tr>
<td></td>
<td>Deep blading with repaving and/or</td>
<td>Reshape road profile, reduce roughness, and rate of deterioration, improve crown and drainage.</td>
</tr>
<tr>
<td></td>
<td>recompaclon</td>
<td></td>
</tr>
<tr>
<td>Rehabilitation</td>
<td>Major regravelling after ripping,</td>
<td>Improve strength shape, drainage and performance.</td>
</tr>
<tr>
<td></td>
<td>recompaclion and drainage</td>
<td></td>
</tr>
<tr>
<td></td>
<td>rehabilitation</td>
<td></td>
</tr>
<tr>
<td>Betterment</td>
<td>Rehabilitation and geometric</td>
<td>Improve the geometric and structural standards.</td>
</tr>
<tr>
<td></td>
<td>improvement, drainage rehabilitation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Upgrading earth road to gravel</td>
<td>Improve structural standards, performance and all-weather passibility.</td>
</tr>
<tr>
<td></td>
<td>road</td>
<td></td>
</tr>
</tbody>
</table>

Source: This study.

The trend for roughness is one of generally frequent phases of increasing roughness followed by a reduction due to blading maintenance. Roughness tends to increase substantially and often rapidly under traffic, and blading maintenance may be applied at intervals ranging from one week to one year, depending on the traffic and other conditions. When the roughness reaches a high level, blading maintenance using a towed or motor grader is usually undertaken to reduce the roughness, though with variable effectiveness. Usually the operation comprises minor reshaping and a redistribution of the surface gravel, filling the wheelpath ruts and any potholes without major reshaping or repaving. The frequency of
blading operations in practice is related either to keeping the roughness down at an acceptable level ("condition-responsive"), or to the season ("scheduled"), e.g., at the beginning and end of the rainy season. On gravel roads, over a number of such blading cycles, there is a net loss of surfacing gravel, which is usually at least 100 mm thick initially. Regravelling, with the import of additional material, is undertaken at infrequent intervals to restore the protection of the subgrade.

When deep blading, or ripping and blading, or resurfacing, are supplemented with controlled or heavy compaction, there appears to be a substantial effect of reducing the rate of roughness in the early stages of the cycle, according to Butler, Harrison and Flanagan (1985). Resurfacing and rehabilitation effectively mark the commencement of a new life cycle.

The trend of condition thus shows a strong cyclic character under a regular maintenance policy, whether as "scheduled maintenance" undertaken at regular time-intervals, or as "condition-responsive maintenance" undertaken whenever the condition reaches a specified threshold. Maintenance policies thus tend to be cited in terms of a fixed frequency; for example, the number of bladings per
year (or the average interval in days between bladings, or the number of vehicles between successive bladings), and the years between resurfacings of a specified thickness.

### 3.2 EMPIRICAL DETERIORATION STUDIES

The two major empirical studies in developing countries which afford adequate data bases for the development of predictive models were undertaken in Kenya by the Overseas Unit of the British Transport and Road Research Laboratory (TRRL) (Hodges, Jones and Rolt 1975), and in Brazil under the Brazil-UNDP road costs study (GEIPOT 1982), both with World Bank support. Other studies in Ethiopia (Robinson 1980a, Newill and others 1982), Ghana (Roberts 1983), Bolivia (Carmichael and others 1979, Butler and others 1985), and by the United States Forest Service (Lund 1973, Lund 1977), provide more limited data which are nevertheless useful for evaluating the transferability of empirical models.

#### 3.2.1 Kenya

In the Kenya study (Hodges and others 1975), forty-six test sections were selected for monitoring and analysis to enable the deterioration to be related to maintenance, traffic, environment and material type. The ranges of geometric and climatic factors included an annual rainfall from 400 mm to 2,000 mm, a maximum gradient over any test section of 5.5 percent, and a maximum curvature of 200 degrees per kilometer. Four types of surfacing material, found in different areas of the country, were included, namely lateritic nodular gravels, quartzitic rounded gravels, volcanic angular gravels, and coral angular gravels.

The test sections, each 1 km long, were selected from the road network according to an experimental factorial matrix which ensured that the interactions and ranges of the most important parameters were well represented in the data as shown by Tables 3.2(a) and Table 3.3. While the distributions of gravel sections over the ranges of rainfall and gradient were generally uniform, it is notable that there was a clear association between material type and climate, with laterites predominant in the medium rainfall (western) region, and the others in the low rainfall (central and coastal) regions. The three levels of maintenance studied included blading by motorgrader at regular intervals of about 6,000 vehicles (normal), 12,000 vehicles (intermediate), and no blading (nil), with only one of these levels being applied on a given section.

The results expressed the progression of roughness, rut depth, gravel loss and looseness (depth of loose surface material) as a function of traffic volume only, but grouped by material type.

#### 3.2.2 Brazil-UNDP Study

In the Brazil-UNDP road costs study (GEIPOT 1982) also, data for the development of empirical models were collected by monitoring the condition of existing roads in service. The factorial basis for selecting the forty-eight sections, shown in Table 3.2(b), was similar to that of the Kenyan study, but with several important differences aimed at improving the interpretation of the results. This was a "partial factorial" in which the low and high extremes of the primary factors (traffic, gradient, and curvature) were fully represented for each surface type (lateritic and quartzitic gravels, earth), but supplemented selectively by intermediate levels (or "star points"), shown encircled in the table, and by replicates of some cells.
Table 3.2: Factorial sampling matrices for empirical studies of unpaved road deterioration in Kenya and Brazil

(a) Kenya road transport cost study, 1971-74

<table>
<thead>
<tr>
<th>Gravels</th>
<th>400 - 1,000</th>
<th>1,000 - 2,000</th>
<th>Totals</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;30</td>
<td>3Q,V</td>
<td>3Q,V</td>
<td>&lt;1.5</td>
</tr>
<tr>
<td>30-90</td>
<td>Q,C</td>
<td>2Q</td>
<td>1.5-3.5</td>
</tr>
<tr>
<td>&gt;90</td>
<td>C</td>
<td>2V</td>
<td>3.5-6</td>
</tr>
<tr>
<td></td>
<td>L</td>
<td>V</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Earth</th>
<th>&lt;30</th>
<th>3Q,V</th>
<th>&lt;1.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>30-90</td>
<td>Q</td>
<td>-</td>
<td>&gt;6</td>
</tr>
<tr>
<td>&gt;90</td>
<td>Q,C</td>
<td>L</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Maintenance</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal</td>
<td>6 4 5</td>
</tr>
<tr>
<td>Intermediate</td>
<td>2 3 2</td>
</tr>
<tr>
<td>Nil</td>
<td>2 1 1</td>
</tr>
</tbody>
</table>

Notes: Material code: L=lateritic, Q=quartzitic, V=volcanic, C=coral, E=earth. "Earth" comprised gravels outside gradation specification with maximum particle sizes of 0.6 to 37 mm. - = Nil. Maintenance blading frequencies: Normal = per 6,000 vehicles; Intermediate = per 12,000 vehicles.

Source: Adapted from Hodges and others (1975).

(b) Brazil-UNDP road costs study, 1976-81

<table>
<thead>
<tr>
<th>Surfacing type</th>
<th>&gt;100</th>
<th>100-350</th>
<th>&gt;350</th>
<th>Totals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateritic gravel</td>
<td>0 *</td>
<td>1 2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Quartzitic gravel</td>
<td>0 *</td>
<td>1 2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Earth</td>
<td>0 *</td>
<td>1 2</td>
<td>1 0</td>
<td></td>
</tr>
</tbody>
</table>

Notes: * = intermediate levels of parameters ("star points"). ** = selected levels. Totals () = number of sections included in high-maintenance study. Curvature (degrees/km) = 180,000/RC, where RC = radius of curvature (m).

Source: Adapted from Volume 2 in GEIPOT (1982).
Table 3.3: Scope and range of parameter values for the primary empirical deterioration studies on unpaved roads in Kenya and Brazil

<table>
<thead>
<tr>
<th></th>
<th>Kenya</th>
<th>Brazil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total no. of sections</td>
<td>46</td>
<td>48</td>
</tr>
<tr>
<td>No. of gravel roads</td>
<td>37</td>
<td>37</td>
</tr>
<tr>
<td>No. of earth roads</td>
<td>9</td>
<td>11</td>
</tr>
<tr>
<td>Period of observation (years)</td>
<td>2</td>
<td>2.5</td>
</tr>
<tr>
<td>Length of sections (m)</td>
<td>1,000</td>
<td>320-720</td>
</tr>
<tr>
<td>Road width (m)</td>
<td></td>
<td>7 to 11</td>
</tr>
<tr>
<td>Gradient (%)</td>
<td>0 to 5.5</td>
<td>0 to 8.2</td>
</tr>
<tr>
<td>Horizontal curvature (degrees/km)</td>
<td>0 to 200</td>
<td>0 to 318</td>
</tr>
<tr>
<td>Traffic volume (veh/day)</td>
<td>42-403</td>
<td>18-608</td>
</tr>
<tr>
<td>Truck volume (veh/day)</td>
<td>12-136</td>
<td>5-477</td>
</tr>
<tr>
<td>Road roughness (m/km IRI)</td>
<td>4-17</td>
<td>1.5-29</td>
</tr>
<tr>
<td>Surface material</td>
<td>Lateritic nodular;</td>
<td>Lateritic nodular;</td>
</tr>
<tr>
<td></td>
<td>quartzitic rounded;</td>
<td>quartzitic subangular;</td>
</tr>
<tr>
<td></td>
<td>volcanic angular;</td>
<td>clayey silts</td>
</tr>
<tr>
<td></td>
<td>coral angular</td>
<td></td>
</tr>
<tr>
<td>Annual rainfall (mm/year)</td>
<td>400-2,000</td>
<td>1,200-2,000</td>
</tr>
<tr>
<td>Thornthwaite's moisture index</td>
<td>-30 to 0</td>
<td>35 to 100</td>
</tr>
<tr>
<td>Climate</td>
<td>semiarid to dry subhumid</td>
<td>moist subhumid to humid</td>
</tr>
</tbody>
</table>

Note: Roughness conversions are given by:

\[
RQI_m = \frac{13}{RI} \quad (\text{m/km IRI})
\]

\[
BIR = 630 RI^{1.12} \quad (\text{m/km IRI})
\]

where \( RBI \) denotes roughness in TRRL Bump Integrator trailer units of \( \text{mm/km} \)

\( \text{BIR} \); \( RI \) is in International Roughness Index units of \( \text{m/km IRI} \); \( RQI_m \) is in Brazil Quartercar Index units of \( \text{counts/km QIm} \).

Sources: Brazil: GEIPOT (1982); World Bank (1985); Organization of American States (1968).

The ranges studied were wider than in the Kenyan study, as seen from Table 3.3, with the maximum levels of traffic, gradient and curvature being 50 percent higher, lower minimum levels, and a strong sample of earth roads of fine-grained silty-clayey soils (which were barely represented in the earlier study). Traffic volume replaced climate as a primary factor because the study was restricted to the central plateau region of Brazil, as seen in Figure 3.5. The climate is almost entirely classed as humid (\( B_1 \) to \( B_4 \)), with small areas classed as moist subhumid (\( C_2 \)) or perhumid (\( A \)), according to the classification of Thornthwaite (1955). The rainfall pattern varied, as shown in Figure 3.6, from precipitations of less than 20 mm per month and low air humidity during a continuous six to eight months of a year, to precipitations of 200 to 600 mm per month and high air humidity over the remainder.
Figure 3.5: Location and climate of road deterioration sections in Brazil

Note: Contours of Thornthwaite Moisture Index (intervals of 20):
A: Perhumid (>100); B: Humid (20-100); C: Subhumid (<20).
Source: After GEIPOT (1982) and Organization of American States (1968)

Figure 3.6: Monthly precipitation during road deterioration study in Brazil

Source: Data from seven stations in the Brazil-UNDP study.
Maintenance effectiveness was initially studied through applying different levels of maintenance (normal and nil) on two subsections of each section. In practice, the subsections were often bladed simultaneously in error and so a "satellite" experiment was later conducted on a sample of nine study sections with blading maintenance being performed at about 2- and 6-week intervals. As a result, very wide ranges of maintenance frequency and condition were achieved, as shown by the detailed summary statistics in Table 3.4. The number of days between successive bladings ranged from 2 to 659, the number of vehicles between bladings ranged from 60 to 136,000, the roughness from 0.8 to 32 m/km IRI and the rut depth up to 75 mm. Details of the section characteristics, given in Table 3.5, show the wide range of material properties also encompassed in the study, and together these tables indicate the broad range over which the derived empirical models are applicable.

3.2.3 Assessment of Findings

The models developed from the Kenyan data (Hodges and others 1975) were polynomial functions of traffic (either linear or cubic), grouped by material type (lateritic, quartzitic, volcanic, coral, earth) but without explicit inclusion of any material properties or climate. In the GEIPOT (1982) analysis of the Brazilian data, some inclusion of physical properties was achieved, supplemented by class variables of material type, and also season or rainfall, as presented in the next section. The transferability of empirical relationships to differing regions and materials can be greatly enhanced if they are explicit functions of physical properties, and so this was an aim of the latest analyses which considered the maximum particle size, particle size distribution, plasticity and shear strength parameters in preference to material class parameters.

As looseness was found to have no measurable influence on vehicles' speed in the range of the Kenyan study, it was considered of little direct value in deterioration modelling for economic analysis and was thus not modelled in the Brazilian study. Rutting, which was modelled in both the Kenyan and GEIPOT (1982) Brazilian studies, is not considered further here because of a preference for the general model given earlier in Equation 3.3; the earlier analyses did not evaluate strength or thickness parameters (which are considered essential) and achieved poor model fits, and further analysis along these lines has not been done. Subsequent discussion therefore focuses on the primary distress modes of roughness and gravel loss and the effects of blading maintenance.

Environmental factors appear to have strong influences on deterioration and maintenance effectiveness but to date have not been well accounted for in empirical models. In the new analyses, macroenvironment is represented by parameters of season, monthly rainfall, and Thornthwaite's moisture index (1955). The index measures the water balance and evapotranspiration potential on a scale of -100 (arid) through 0 (subhumid) to 100 (perhumid) and is known to be a good indicator of soil moisture conditions (see Section 4.3.6). The microenvironment is difficult to define because it depends on drainage in addition to climate and material properties. Characteristics such as adjacent gradients (run-on), cross-fall, runoff points, side drain capacity, etc. affect the local condition and may vary over time but were generally not available in the analyses. In Brazil, the roughness on level tangent sections that were poorly drained was very high during wet periods due largely to the rapid development of potholes under such conditions. On vertical grades, roughness was frequently low despite extensive erosion by surface runoff because the longitudinal profile was affected less than the
Table 3.4: Summary range of values of data in the unpaved road deterioration study in Brazil

<table>
<thead>
<tr>
<th>Variable</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>Number of sections = 48</td>
<td>3.8</td>
</tr>
<tr>
<td>Grade, %</td>
<td>3.9</td>
</tr>
<tr>
<td>Curvature on curved sections, km⁻¹</td>
<td>9.8</td>
</tr>
<tr>
<td>Material properties</td>
<td></td>
</tr>
<tr>
<td>Maximum particle size, mm</td>
<td>18.3</td>
</tr>
<tr>
<td>Percentage passing the 0.075 mm sieve</td>
<td>36</td>
</tr>
<tr>
<td>Plasticity index, %</td>
<td>11</td>
</tr>
<tr>
<td>Liquid limit, %</td>
<td>32</td>
</tr>
<tr>
<td>Average daily traffic (both directions)</td>
<td></td>
</tr>
<tr>
<td>Passenger cars</td>
<td>88</td>
</tr>
<tr>
<td>Buses</td>
<td>7</td>
</tr>
<tr>
<td>Pickups</td>
<td>37</td>
</tr>
<tr>
<td>Two axle trucks</td>
<td>56</td>
</tr>
<tr>
<td>Trucks and trailer combinations with more than 2 axles</td>
<td>15</td>
</tr>
<tr>
<td>Time-related information for gravel loss</td>
<td></td>
</tr>
<tr>
<td>Number of observations = 604.</td>
<td></td>
</tr>
<tr>
<td>Time of observation relative to start of observation or regravelling (days)</td>
<td>238</td>
</tr>
<tr>
<td>Number of bladings relative to start of observation or regravelling</td>
<td>2.3</td>
</tr>
<tr>
<td>Information related to roughness measurement</td>
<td></td>
</tr>
<tr>
<td>Number of observations = 8,095</td>
<td></td>
</tr>
<tr>
<td>Roughness (counts/km Q₂₅₅)</td>
<td>113</td>
</tr>
<tr>
<td>Roughness (m/km IRI)</td>
<td>8.7</td>
</tr>
<tr>
<td>Number of days between bladings</td>
<td>110</td>
</tr>
<tr>
<td>Number of vehicle passes since blading for the last observation in each blading period</td>
<td>16,080</td>
</tr>
<tr>
<td>Information related to rut depth measurements</td>
<td></td>
</tr>
<tr>
<td>Rut depth, mm</td>
<td>11.1</td>
</tr>
<tr>
<td>Number of days since blading for the last observation in each blading period</td>
<td>61</td>
</tr>
<tr>
<td>Number of vehicle passes since blading for the last observation in each blading period</td>
<td>12,490</td>
</tr>
<tr>
<td>Climate</td>
<td></td>
</tr>
<tr>
<td>Mean annual precipitation, mm</td>
<td>1571</td>
</tr>
<tr>
<td>Monthly precipitation, mm</td>
<td>131</td>
</tr>
<tr>
<td>Moisture index (Thornthwaite, 1955)</td>
<td>59</td>
</tr>
</tbody>
</table>

Source: Compiled from GEIPOT (1982), study data files.
Table 3.5: Geometric, traffic, environmental and material characteristics of unpaved road sections in Brazil-UNDP road costs study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Surface type</th>
<th>Lateritic gravels</th>
<th>Quartzitic gravels</th>
<th>Earth roads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Item</td>
<td>Sym- Units</td>
<td>Min- Dev. imm</td>
<td>Max- imm</td>
</tr>
<tr>
<td>No. of sections</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Gradient</td>
<td>G %</td>
<td>3.48</td>
<td>2.70</td>
<td>0.8</td>
</tr>
<tr>
<td>Curvature of curved sections</td>
<td>KCV km⁻¹</td>
<td>3.97</td>
<td>2.57</td>
<td>5.32</td>
</tr>
<tr>
<td>No. curved sections</td>
<td>-</td>
<td>12</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Road width</td>
<td>W m</td>
<td>9.89</td>
<td>0.92</td>
<td>9</td>
</tr>
<tr>
<td>Average daily traffic</td>
<td>ADT veh/ day</td>
<td>225</td>
<td>177</td>
<td>21</td>
</tr>
<tr>
<td>Average daily heavy traffic</td>
<td>ADH veh/ day</td>
<td>104</td>
<td>139</td>
<td>5</td>
</tr>
<tr>
<td>Annual precipitation</td>
<td>MAP m/ yr</td>
<td>1.57</td>
<td>0.05</td>
<td>1.51</td>
</tr>
<tr>
<td>Maximum stone size</td>
<td>D95 mm</td>
<td>21.9</td>
<td>6.7</td>
<td>8.1</td>
</tr>
<tr>
<td>Percentage finer than 2 mm</td>
<td>FO2 %</td>
<td>51.1</td>
<td>8.3</td>
<td>35.7</td>
</tr>
<tr>
<td>Percentage finer than 0.425 mm</td>
<td>P425 %</td>
<td>41.6</td>
<td>7.5</td>
<td>28.7</td>
</tr>
<tr>
<td>Percentage finer than 0.075 mm</td>
<td>P075 %</td>
<td>25.5</td>
<td>5.9</td>
<td>17.0</td>
</tr>
<tr>
<td>Material gradation coefficient</td>
<td>MG</td>
<td>0.25</td>
<td>0.04</td>
<td>0.20</td>
</tr>
<tr>
<td>Dust ratio</td>
<td>MSD</td>
<td>0.62</td>
<td>0.13</td>
<td>0.39</td>
</tr>
<tr>
<td>California Bearing Ratio</td>
<td>CBR MD %</td>
<td>57.3</td>
<td>25.6</td>
<td>31.7</td>
</tr>
<tr>
<td>California Bearing Ratio soaked</td>
<td>CBROMD %</td>
<td>45.0</td>
<td>15.6</td>
<td>22.7</td>
</tr>
<tr>
<td>Plasticity index</td>
<td>PI %</td>
<td>10.1</td>
<td>4.9</td>
<td>0.0</td>
</tr>
<tr>
<td>Liquid limit</td>
<td>LL %</td>
<td>29.8</td>
<td>6.6</td>
<td>20.0</td>
</tr>
</tbody>
</table>

Note: For definitions of parameters, see Table 3.9. - Not applicable.
Source: Analysis of data from computer files and working documents Nos. 9, 14, 13, in GEIPOT (1982). See also World Bank (1985) for listing of data.
transverse profile. The study sections generally had moderate standards of drainage and drainage maintenance, and positive crowns. The prediction relationships therefore apply to unpaved roads with moderate to good cross-sectional geometry and for dry to wet conditions, but may not apply to roads that have a "bathtub" type of cross-section, with negative crown or inadequate surface runoff capacity under high intensity rainfall.

The analytical development in the following sections (3.3 to 3.6) was done on the Brazilian data because of the comprehensive scope and range of the data. Comparisons with the findings of the other empirical studies (in the succeeding section, 3.7) provide an evaluation from an even broader, transregional base.

### 3.3 ROUGHNESS PROGRESSION

#### 3.3.1 Data Characteristics

The roughness of unpaved roads increases through the shear, mechanical disintegration, and erosion of the surfacing material caused by traffic and surface water runoff. Roughness levels are usually 4 to 15 m/km IRI (50 to 200 QI) although lower levels sometimes occur with fine materials. Roughness in excess of 13 m/km IRI (180 QI) is usually related to depressions, potholes or transverse erosion gullies, and levels above 22 m/km IRI (300 QI), which correspond to wheel-sized potholes, are rare and usually apply only on short sections or unclassified tracks. The roughness modelled for economic evaluation is the profile in the wheelpaths of the traffic, since this determines the vehicle operating costs. The location of the wheelpaths tends to vary when roughness reaches high levels as vehicles seek to minimize the dynamic impact, hence the prediction of roughness progression must take this self-regulating tendency into account.

On account of the high variability of material properties, drainage, surface erosion, and the location of the wheelpaths, the roughness of unpaved roads over time tends to be variable and the progression may not be regular. When to this trend is added a measurement error in the order of 18 percent for a response-type roughness instrument (which is currently the only type sufficiently robust to monitor all unpaved roads), and a speed correction error in the order of 2 to 8 percent (Section 2.4), the variations observed tend to be very large.

Typical observations from the Brazil-UNDP study are shown in Figure 3.7 for two different sections, and superimposed on the figure are the times that blading maintenance was applied. It is apparent that the trends are not always clear from the few observations taken during a blading cycle, that the trend varies from cycle to cycle, and that blading does not always reduce the roughness substantially. In the modelling process, therefore, considerable reliance is placed on the error-minimizing effects of taking large numbers of observations from separate blading cycles and an experimentally-designed matrix of parameter values.

A total of 8,095 roughness measurements were made from 26,000 run-readings in the study. Statistics summarizing the range of roughness behavior observed in the study for each material group are given in Table 3.6 (Note: All values include the adjusted speed correction described in Section 2.4.4). The means and ranges of roughness change per blading cycle were similar for all
Figure 3.7: Examples of roughness progression and effects of blading observed in Brazil-UNDP study

(a) Section 263 COM CS

(b) Section 259 COM CS

Note: (a) Traffic, 284 veh/day; grade, 0.6%; tangent; plastic quartzitic gravel. (b) Traffic, 156 veh/day; grade 0.8%; curved; plastic quartzitic gravel.

Source: Brazil-UNDP study data.
Table 3.6: Summary of rates of roughness progression observed on unpaved roads in Brazil-UNDP road costs study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Lateritic gravels</th>
<th>Quartzitic gravels</th>
<th>Earth roads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rates of roughness change 1/</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Increment per cycle (m/km IRI)</td>
<td>1.25</td>
<td>2.05</td>
<td>-2.8</td>
</tr>
<tr>
<td>Increment per cycle (ct/km QI₉₀)</td>
<td>18.2</td>
<td>26.6</td>
<td>-37</td>
</tr>
<tr>
<td>Per day (QI₉₀/day)</td>
<td>0.73</td>
<td>1.56</td>
<td>-5.84</td>
</tr>
<tr>
<td>Per 1,000 vehicles (QI₉₀/1000 veh.)</td>
<td>4.79</td>
<td>13.65</td>
<td>-31.1</td>
</tr>
<tr>
<td>Fractional rates of roughness progression</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percentage change (%)</td>
<td>14.4</td>
<td>20.9</td>
<td>-38</td>
</tr>
<tr>
<td>Percent per day</td>
<td>0.7</td>
<td>15</td>
<td>-4.1</td>
</tr>
<tr>
<td>Percent per 1,000 veh.</td>
<td>5.9</td>
<td>18.8</td>
<td>-51.6</td>
</tr>
<tr>
<td>Observed conditions 1/</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Progression period (days)</td>
<td>36.3</td>
<td>40.5</td>
<td>2</td>
</tr>
<tr>
<td>Initial roughness (ct/km QI₉₀)</td>
<td>92.1</td>
<td>39.5</td>
<td>16.8</td>
</tr>
<tr>
<td>Final roughness (ct/km QI₉₀)</td>
<td>108.3</td>
<td>49.5</td>
<td>19.4</td>
</tr>
<tr>
<td>Average daily rainfall (mm/day)</td>
<td>4.64</td>
<td>3.94</td>
<td>0</td>
</tr>
<tr>
<td>Average daily traffic (veh/day)</td>
<td>267</td>
<td>210</td>
<td>21</td>
</tr>
<tr>
<td>Average heavy traffic (veh/day)</td>
<td>151</td>
<td>180</td>
<td>5</td>
</tr>
<tr>
<td>No. subsections x cycles</td>
<td>568</td>
<td>312</td>
<td></td>
</tr>
</tbody>
</table>

1/ Incremental, initial and final conditions are here defined by the average of the first-three and last-three observations of roughness, time, traffic and rainfall, respectively, in order to reduce the effects of measurement errors. **Source:** Analysis of data from Brazil-UNDP road costs study (GEIPOT 1982).

groups, but the progression rates tended to be slower for the fine-grained soil (earth) surfaces than for gravel (particularly lateritic) surfaces, on either a per day or per vehicle basis.

**Seasonal effects**

An interesting feature of the data is the negative or neutral trend of roughness that was often observed, as shown by the second and fourth cycles in Figure 3.7(a) for example. Some incremental reductions were as much as 30 to 50 percent. While some of the reductions may be attributable to measurement error and some, at least in part, to the changing location of the wheelpaths as the
traffic sought to avoid the roughest areas, much appears to be due to rainfall, presumably through promoting compaction of the moistened surface under traffic. Lower roughness levels in the wet season than in the dry season have also been observed in Bolivia by Carmichael and others (1979). The apparent paradox here is that the common user's experience of unpaved roads being in their worst condition in the wet season does not seem to be supported by the field data, at least in respect of roughness.

Figure 3.8 shows the cumulative frequency distributions of roughness in the Brazilian study, summarized by surface type, season and measurement speed. These all show that the mean roughness during the wet season was 6 to 20 percent lower than during the dry season, and the same was generally true for the median value of roughness. However, we also note two other important features. First, the average maximum roughness was highest in the wet season, as evidenced by the top portion of the cumulative frequency distribution curves, even though the mean value was lower. Second, the number of measurements missing at speeds of 50 and 80 km/h indicate that more difficulty was encountered in reaching the maximum measuring speed in the wet season (all measurements were done at 20, 50 and 80 km/h where possible). These both imply that the road conditions can reach a worse state in the wet season than in the dry season, notably through the development of potholes (most probably on level sections), and this is consistent with experience. The lower speeds probably indicate for example that the driver needed to make more deviations in order to follow the smoothest wheelpaths.

The wet season therefore appears to be influencing also the utilization and the attainable free-flow speed of vehicles. This is illustrated in Table 3.7, which presents the observed chances of attaining a given test speed in either the wet or the dry season, given the constant geometric characteristics of each section. The data indicate a reduction of about 14 percent in the average attained velocity from 71.9 km/h in the dry season to 62 km/h in the wet season. The finding is important because it suggests that there is an additional constraint on vehicle speed in the wet season which is not being explained by the roughness alone, but which is due to some other element of surface condition, presumably standing water, slipperiness or softness of the surface. This additional constraint has not yet been quantified for either the road deterioration or the vehicle-speed prediction models.

Table 3.7: Wet season effects on unpaved road service as shown by observed success of attaining various test speeds for roughness measurements

<table>
<thead>
<tr>
<th>Desired velocity (km/h)</th>
<th>Chance of attaining desired test speed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wet season</td>
</tr>
<tr>
<td>80</td>
<td>0.60</td>
</tr>
<tr>
<td>50</td>
<td>0.80</td>
</tr>
<tr>
<td>20</td>
<td>1.00</td>
</tr>
</tbody>
</table>

*Source: Analysis of data from Brazil-UNDP study.*
Figure 3.8: Seasonal distributions of aggregated roughness data from unpaved roads in Brazil-UNDP study

(a) Gravel Roads: Survey Speed 20 km/h

(b) Earth Roads: Survey Speed 20 km/h

(c) Gravel Roads: Survey Speed 80 km/h

(d) Earth Roads: Survey Speed 80 km/h

Note: Obs = subsection measurement, corrected for survey speed by Equations 2.11 and 2.12.

Source: Data from the Brazil-UNDP study.
3.3.2 Analytical Approach

- The period between bladings was considered a cycle since every time an unpaved road was bladed, the roughness was generally reduced and the new deterioration during the cycle depended on the length of time between bladings. Consequently, deterioration was considered a function of the number of days since the last blading. In the Kenya study (Hodges and others 1975) the same approach was used, but in that study they assumed that blading a road returned its roughness to some standard value. Inspection of the data collected in Brazil showed that the roughness measured after blading varied, so that the assumption of a standard value was not appropriate. Because roughness after blading was variable, and the number of days between blading varied widely, the analysis was executed in two parts: the first part consisted of predicting the change in roughness with time, while the second part involved predicting the roughness after blading.

3.3.3 Predictive Models

Polynomial form

The TRRL model for lateritic, quartzitic and volcanic gravels from the Kenyan data (Hodges and others 1975) was:

\[ RBI = 3250 + 84 T - 1.6 T^2 + 0.016 T^3 \]  

where \( RBI \) = roughness, in mm/km Bump Integrator trailer; and \( T \) = in cumulative traffic volume in both directions since blading, thousands of vehicles.

Other coefficients applied to coral gravel and earth roads (the trends are compared later in Figure 3.22). The model shows low initial progression rates and very steep final rates without any upper limit; these steep rates (from the higher order terms) were dictated largely by the performance under nil maintenance. Such a correlative model represents only the average behavior, without distinction of individual material properties, season, or rate of trafficking, and thus may have poor transferability. Indeed, later studies in Kenya found generally steeper rates even for the average, as discussed in Section 3.7.

Logit and exponential forms

In the GEIPOT (1982) and Visser (1981) analyses of the Brazilian data, the choice of model form was based on a hypothesis of traffic action. As irregularities develop under traffic and weathering, the dynamic loadings imposed on the road increase, causing an accelerating rate of deterioration and particularly of roughness. At very high levels of roughness, vehicles tend to slow down or avoid the worst areas and thus the roughness would tend to remain essentially constant at a high level. The concept is illustrated in Figure 3.9.

From their interpretation of the data, the rate of increase in roughness with respect to time or traffic \( (RG') \) was considered a function of the current roughness level \( (RG) \), not of the initial roughness level after blading \( (RG_0) \), i.e.:

\[ RG' = f (RG) \]
The logit and exponential forms, which possess this property, were the two models estimated. The logit model has the general form of:

$$RG = RG_{\min} + \frac{(RG_{\max} - RG_{\min})}{[1 + \exp(-tf)]}$$

(3.6)

and the exponential model, the form:

$$RG = \exp(fl + tf)$$

(3.7)

where

- $RG = \text{the road roughness; }$
- $RG_{\min}, RG_{\max} = \text{asymptotic minimum and maximum expected values of roughness; }$
- $t = \text{time; }$
- $f = \text{regression function which is a linear combination of independent variables; and }$
- $fl = \text{regression function, a linear combination of independent variables. }$

One advantage of both the logit and the exponential models is that since the roughness errors are related to the magnitude of the roughness (see Figure 2.15), a logarithmic transformation results in homogeneous variances and also allows linear regression techniques to be used (which was important because the data sets were very large).

In the estimation of the GEIPOT logit model, roughness limits of 15 and 450 counts/km QI* were assumed based on the extreme observed values (there are problems with this assumption discussed later in Section 3.5). For the exponential model, the scarcity of data in the upper range of 300 to 450 counts/km QI* prevented the estimation of two curves piecewise, so a single curve was estimated.
representing the lower concave portion of the model only. For application of the model, an imposed upper limit of 450 counts/km QI* was proposed. In the data processing this was seen to be the maximum value on any section. In the final evaluation, the two models were found to comprise similar parameters and coefficients and to give similar predictions over the common range of prediction below 300 QI* (23 m/km IRI) (see Volume 7 in GEIPOT 1982). Consequently the exponential model was preferred for its greater ease of computation.

Two methods of incorporating climatic effects were presented in the GEIPOT (1982) study, one incorporating a season variable for wet and dry seasons, the other incorporating the rainfall directly from monthly rainfall data. The latter form, being the most suitable for transferability to other climates and regions, is the one reviewed here. The GEIPOT (1982) model, re-estimated using the corrected roughness data file, was

\[
LDQ = D (0.376 - 0.191 TE + 0.000320 ADL + 0.001014 ADH)
+ CP (-0.16 - 0.0354 G + 0.00883 P075 - 0.0218 PI) \tag{3.8}
\]

with \( r^2 = 0.31 \); standard error = 0.211; sample = 8,276 observations; and

where

- \( LDQ \) = change in natural logarithmic value of roughness, i.e. \( \ln (RG_2/RG_1) \), dimensionless, where \( RG_1 \) and \( RG_2 \) are roughness values at times \( t_1 \) and \( t_2 \), respectively;
- \( D \) = number of days between observations at times \( t_1 \) and \( t_2 \), in hundreds, i.e., \( (t_2 - t_1)/100 \);
- \( TE \) = surfacing type dummy variable, where \( TE = 1 \) if surfacing is earth, and \( TE = 0 \) otherwise;
- \( ADH \) = average daily bus and truck traffic in both directions, in veh/day;
- \( ADL \) = average daily car and utility traffic in both directions, in veh/day;
- \( G \) = absolute value of grade, in percent;
- \( P075 \) = surfacing material passing the 0.075 mm sieve, in percent;
- \( PI \) = plasticity index of surfacing material, in percent; and
- \( CP \) = cumulative precipitation since the previous blading, in m.

The model is almost identical to the original Visser-GEIPOT model. Evaluation of the model's predictive capability, illustrated in Figure 3.10 for one section under two maintenance strategies, shows that the model tended to overestimate the rate of progression of roughness at high levels under infrequent blading, and to slightly underestimate at low levels of roughness under frequent blading, for the maximum volume of traffic of 608 veh/day. The tendency to overestimate the roughness under infrequent blading policies is particularly evident for section 205, shown in Figure 3.11 which had a low traffic volume of 92 veh/day and was reportedly never bladed during the study period.

A subsequent analysis (Paterson 1983) attempted to suppress the explosive tendency of the models by estimating the average roughness over the duration of a blading cycle as follows:

\[
\ln RGM = 1.607 + 0.605 \ln RG_0 + 0.174 TQ + DB (0.000393 ADL
+ 0.00119 ADH) + MP (0.370 - 0.069 G - 0.0567 PI
+ 0.00855 P075) \tag{3.9}
\]
Figure 3.10: Examples of fit of exponential model to roughness progression observed on unpaved road with high volume of traffic

(a) Infrequent Blading (approximately 90-day cycle)

(b) Frequent Blading (approximately 14-day cycle)

Note: Section No. 251 of Brazil-UNDP study. For presentational purposes, the initial roughness used in each cycle prediction is the observed value, not a predicted value. SV = % passing 0.075 mm (P075).

Source: Volume 7 in GEIPOT (1982).
Figure 3.11: Comparison of predictions of exponential and average roughness models on unpaved road without blading for 20 months

Notes: Section No. 205 of Brazil-UNDP study. SV = PO75.
Source: Adapted from Volume 7 in GEIPOT (1982).

with \( r^2 = 0.76 \); standard error = 0.239; sample = 1,089 observations, and t-statistics of 24, 43, 9.9, 4.6, 13, 3.0, 4.6, 5.1 and 2.7, respectively;

where

- \( RGM \) = mean roughness during blading cycle, counts/km QIm;
- \( RG_0 \) = roughness after blading, at beginning of blading cycle, counts/km QIm;
- \( DB \) = interval between bladings, hundreds of days;
- \( MP \) = average rainfall intensity during blading cycle, (m/month);
- \( TQ \) = 1 if surface is quartzitic gravel, otherwise \( TQ = 0 \).

and other variables were as previously defined. Several of the coefficients in the model are similar to those of Equation 3.8, though the time constant has disappeared. The improved fit of the model \( (r^2 = 0.76) \) is largely due to elimination of the high variability and measurement errors of the individual observations through the averaging process.

The relationships provided by this model are illustrated in Figure 3.12 for two extremes, namely, a nonplastic gravel on zero grade and a plastic gravel on 8 percent grade, on tangent sections. The effects of traffic volume are very
Figure 3.12: Predictions of average roughness since blading for unpaved roads: exponential average roughness model

(a) Non-plastic Gravel, Zero Grade

(b) Plastic Gravel, 8% Grade

Source: Equation 3.9.
strong so that at 50 cars/day there is negligible change in roughness, and with 400 trucks/day plus 150 cars/day the roughness increases 100 percent in 100 days. The effect of high rainfall intensity is to increase the roughness slightly on the zero grade, and to decrease the roughness when the grade is steep and/or the material is slightly plastic.

The model was successful in suppressing the explosive tendencies, and for example fits the extreme conditions typified by the low traffic volume section much better than the exponential model (see Figure 3.11). The model form was designed primarily for application in economic evaluations where the average roughness during a cycle is used to estimate the vehicle operating costs. Linear progression is assumed from the initial roughness to the final roughness of the cycle. However, the discrete, step-like nature of its predictions creates problems when applying it to a marginal cost evaluation and when combining it with varying rainfall and a blading effect model. In its place a combined steady-state model, described in Section 3.5, was estimated.

3.4 EFFECTS OF BLADING MAINTENANCE ON ROUGHNESS

Blading maintenance reduced road roughness by an average of about 20 percent in the Brazil-UNDP study, but the comparison of roughness observed before and after blading in Figure 3.13 shows that the effectiveness was extremely variable. Two characteristics are clear; first, that blading did not return the roughness to some approximately constant value regardless of the roughness before blading, and second, that blading did not always reduce the roughness, even at apparently high roughness levels.

In fact, the roughness after blading ranged from 1.7 to 19 m/km IRI and the effect from a reduction of 78 percent to small apparent increases of up to 3 m/km IRI, as detailed in Table 3.8. The average reductions were 26 percent for laterites, 19 percent for quartzites and 15 percent for fine-grained soils. This may be compared with the findings in the Kenyan study of a constant average roughness after blading of 4.3 m/km IRI, and reductions of 7 to 25 percent in Ghana and 13 to 52 percent in Bolivia (as detailed further in Section 3.7). The effectiveness of blading in reducing roughness is highly dependent on human factors, and particularly on the expertise of the grader operator for regular motor graders without sophisticated automation. Much of the variability that can be seen in Figure 3.13 derives from variable performance by the operators; for example, it was shown that the variation in roughness after blading for a constant level of roughness before blading on the same section was in the order of 34 percent (Volume 7 in GEIPOT 1982).

The data represented the roughness measured within a few days either before or after blading. The exact timing of the measurements was variable because of the logistics involved and the lag times are shown in the table. In the analyses, the non-synchronization was taken into account in two ways, either by using the roughness progression prediction model to extrapolate to the blading date, or by partitioning the data by the lag time. Both the GEIPOT (1982) and Carmichael and others (1979) studies have shown that the roughness usually decreases further during the first one to three days immediately after blading through compaction under traffic, and the data here generally represent that lowest roughness instead of the roughness on the day of blading.
Two exponential models have been estimated, in each case using logarithmic transformation to homogenize the variances, as for roughness progression. These take the form:

$$ R_{G_a} = a R_{G_b}^b $$

where $R_{G_a}$ and $R_{G_b}$ are the roughness after and before blading, respectively. The GEIPOT (1982) estimates of $a$ and $b$ were:

$$ a = \exp(1.404 - 0.0239 W - 0.0048 P075 + 0.0169 PI + 0.15 TQ + 0.31 TE + 0.0002 NHV + 0.206 BS - 0.0118 PI BS) $$

$$ b = 0.631 $$

with $r^2 = 0.61$; standard error = 0.340 (on the log transform); sample = 1,308 observations; and where roughness ($R_G$) is in counts/km $Q_{IM}$; $W$ = road width, m; $NHV$ = number of heavy vehicles passed since blading; $BS = 1$ if wet season, $BS = 0$ otherwise; and other variables are as previously defined.

The second model incorporates rainfall and the coarseness of the surfacing material:

$$ a = \exp (0.649 + 0.0135 R25 + 0.723 MP - 0.0387 PI MP) $$

$$ b = 0.771 $$
Table 3.8: Characteristics of data on the effect of blading on roughness of unpaved roads in Brazil

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Units</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateritic gravels</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of observations = 568 subsections x bladings</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number bladings/subsection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roughness before blading,</td>
<td>$R_{b}$</td>
<td>m/km IRI</td>
<td>9.74</td>
<td>3.23</td>
<td>1.98</td>
<td>16.34</td>
</tr>
<tr>
<td>Roughness after blading,</td>
<td>$R_{a}$</td>
<td>m/km IRI</td>
<td>7.12</td>
<td>3.40</td>
<td>1.69</td>
<td>17.99</td>
</tr>
<tr>
<td>Roughness change,</td>
<td>$D_{RG}$</td>
<td>m/km IRI</td>
<td>-2.62</td>
<td>2.69</td>
<td>-3.98</td>
<td>3.07</td>
</tr>
<tr>
<td>Fractional change in roughness,</td>
<td>$F_{RG}$</td>
<td>-</td>
<td>-0.259</td>
<td>0.238</td>
<td>-0.665</td>
<td>0.391</td>
</tr>
<tr>
<td>Observation lag before blading,</td>
<td>$DAYS_{b}$</td>
<td>days</td>
<td>7.29</td>
<td>8.72</td>
<td>1</td>
<td>28</td>
</tr>
<tr>
<td>Observation lag after blading,</td>
<td>$DAYS_{a}$</td>
<td>days</td>
<td>8.76</td>
<td>10.21</td>
<td>0</td>
<td>33</td>
</tr>
<tr>
<td>Monthly precipitation</td>
<td>$MP$</td>
<td>mm/month</td>
<td>217</td>
<td>119</td>
<td>0</td>
<td>336</td>
</tr>
<tr>
<td>Quartzitic gravels</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of observations = 312 subsections x bladings</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number bladings/subsection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roughness before blading,</td>
<td>$R_{b}$</td>
<td>m/km IRI</td>
<td>12.12</td>
<td>4.14</td>
<td>5.25</td>
<td>31.76</td>
</tr>
<tr>
<td>Roughness after blading,</td>
<td>$R_{a}$</td>
<td>m/km IRI</td>
<td>9.59</td>
<td>3.42</td>
<td>3.09</td>
<td>18.90</td>
</tr>
<tr>
<td>Roughness change,</td>
<td>$D_{RG}$</td>
<td>m/km IRI</td>
<td>-2.53</td>
<td>2.85</td>
<td>-13.08</td>
<td>2.40</td>
</tr>
<tr>
<td>Fractional change in roughness,</td>
<td>$F_{RG}$</td>
<td>-</td>
<td>-0.189</td>
<td>0.210</td>
<td>-0.779</td>
<td>0.449</td>
</tr>
<tr>
<td>Observation lag before blading,</td>
<td>$DAYS_{b}$</td>
<td>days</td>
<td>12.76</td>
<td>11.37</td>
<td>1</td>
<td>36</td>
</tr>
<tr>
<td>Observation lag after blading,</td>
<td>$DAYS_{a}$</td>
<td>days</td>
<td>8.13</td>
<td>6.20</td>
<td>0</td>
<td>21</td>
</tr>
<tr>
<td>Monthly precipitation</td>
<td>$MP$</td>
<td>mm/month</td>
<td>144</td>
<td>85</td>
<td>7</td>
<td>277</td>
</tr>
<tr>
<td>Earth roads</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of observations = 160 subsections x bladings</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number bladings/subsection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roughness before blading,</td>
<td>$R_{b}$</td>
<td>m/km IRI</td>
<td>8.96</td>
<td>3.20</td>
<td>2.84</td>
<td>17.45</td>
</tr>
<tr>
<td>Roughness after blading,</td>
<td>$R_{a}$</td>
<td>m/km IRI</td>
<td>7.42</td>
<td>3.27</td>
<td>2.57</td>
<td>16.48</td>
</tr>
<tr>
<td>Roughness change,</td>
<td>$D_{RG}$</td>
<td>m/km IRI</td>
<td>-1.54</td>
<td>2.23</td>
<td>-5.94</td>
<td>2.16</td>
</tr>
<tr>
<td>Fractional change in roughness,</td>
<td>$F_{RG}$</td>
<td>-</td>
<td>-0.147</td>
<td>0.261</td>
<td>-0.593</td>
<td>0.544</td>
</tr>
<tr>
<td>Observation lag before blading,</td>
<td>$DAYS_{b}$</td>
<td>days</td>
<td>10.80</td>
<td>6.07</td>
<td>1</td>
<td>21</td>
</tr>
<tr>
<td>Observation lag after blading,</td>
<td>$DAYS_{a}$</td>
<td>days</td>
<td>9.88</td>
<td>11.37</td>
<td>0</td>
<td>50</td>
</tr>
<tr>
<td>Monthly precipitation</td>
<td>$MP$</td>
<td>mm/month</td>
<td>155</td>
<td>112</td>
<td>15</td>
<td>319</td>
</tr>
</tbody>
</table>

1/ Positive changes of roughness after blading usually were associated with long lag times between the date of blading and the dates of roughness observations. Otherwise, they were associated with low roughness before blading or measurement error.

Note: Roughness values converted from QI m scale of Brazil study by: 1 m/km IRI = 13 counts/km QI m (see Chapter 2).

Source: Individual observation data from files of Brazil road costs study (GEIPOT 1982) as processed in World Bank (1985).
with $r^2 = 0.65$ (log model); standard error $= 0.327$; sample $= 666$ observations; and
$r^2 = 0.56$ for the natural data (excluding lag times $> 20$ days); and where
$R_{25} =$ percentage of material particles coarser than 25 mm.

The second model, which was estimated after the GEIPOT study, indicated that the
coarseness and stone size influenced the blading effectiveness, but the other
terms in the GEIPOT model then became insignificant. The fact that few physical
parameters were found to explain the blading effects confirmed that most of the
variance was indeed attributable to the operators' performance.

3.5 STEADY-STATE SOLUTION FOR ROUGHNESS UNDER REGULAR BLADING MAINTENANCE

In policy analysis, the models predicting roughness progression and the
effects of blading maintenance on roughness are used together to predict the
cyclic trend of roughness over the analysis period. From the cyclic nature of
these activities, it is evident that, for a given section for which the material
properties, traffic volume and climate remain essentially constant over long
periods of time, a characteristic trend of roughness emerges. In the long term
this trend can be regarded as a steady state of cyclic increases and decreases of
roughness, with a characteristic long-term average roughness that represents the
influence on vehicle operating costs.

The exponential models given in the previous sections can be applied
iteratively, determining the mean roughness for each cycle from the average of the
roughness values after and before blading. Over the long term, the cycle-means
converge to an asymptotic value that represents the "steady state" of the road,
under the given policy of regular blading. However, it is computationally conve-
nient if the process can be solved by simultaneous solution of the two underlying
models, resulting in a closed-form model with a unique solution. This aim,
together with the need to restrain the explosive tendency of the exponential
progression model at low blading frequencies, led to the development of a steady-
state model as follows.

3.5.1 Closed-Form Steady-State Model for Average Roughness

As a convenient mathematical solution could not be found using the
exponential model forms, a number of simplifying assumptions were made regarding
the form and constraints of the model, as follows.

Roughness progression

First, it was assumed that the roughness experienced by vehicles on
unpaved roads was constrained by a maximum value, and that the maximum roughness
was not a constant high value (such as $450 \, Q_{1m}$ selected in the GEIPOT analysis),
but was itself a variable whose value depended upon material properties, road
geometry, etc. It was observed in the field, for example, that the roughness on
positive grades was frequently less than that on level sections (particularly at
the top and bottom of grades), where potholes and depressions develop if surface
drainage is poor.

Second, it was assumed that the rate of roughness progression ($R_{G'}$)
decreased as the roughness approached the maximum value, as shown in Figure 3.14

---

3/ In collaboration with Thawat Watanatada.
Figure 3.14: Conceptual solution for closed-form steady-state model of roughness progression and blading maintenance

(a) Roughness Progression

(b) Rate of Progression

(c) Roughness Progression Function

(d) Roughness after Blading Function, Solution with Progression Function

Note: $RG_a$ = roughness at the beginning of a cycle; $RG_b$ = roughness at the end of a cycle, before blading maintenance.

Source: Watanatada with author.
(a) and, for simplicity, that \( RG' \) was a linear function of roughness reducing to zero at the maximum value \( RG_{\text{max}} \) (diagram(b)). This may be expressed as

\[
RG' = y \{ RG_{\text{max}} - RG \} \quad \text{for} \; 0 < RG < RG_{\text{max}} \; \text{and} \; y \geq 0
\]

where \( y \) = a vector of explanatory variables to be estimated, e.g., road, traffic and climate characteristics. Integration with respect to time \( t \) gives:

\[
[RG_{\text{max}} - RG] = c \exp[-y t]
\] (3.13)

between limits on \( RG \) and \( t \), where \( c \) is a constant to be estimated. Under steady-state conditions, the roughness at the end of the blading cycle (\( RG_b \)) then becomes a linear function of the roughness at the beginning of the cycle (\( RG_a \)) as follows (Figure 3.14(c)):

\[
RG_b = RG_{\text{max}} - p \{ RG_{\text{max}} - RG_a \}
\] (3.14)

where \( p = \exp(-y \Delta t) \) such that \( 0 < p < 1 \);
\( \Delta t = t_b - t_a \), the time between blading activities; and
\( RG_a, RG_b \) = roughness at the beginning and end of the period between consecutive bladings, respectively.

Effect of blading

The reduction of roughness due to blading, already shown to be related to the roughness before blading, was assumed to be linearly related to the difference above an estimated minimum roughness level. Again, instead of assuming the same minimum for all sections, it was considered that the minimum roughness might depend on material properties and their influence on the effectiveness of blading, particularly in respect of particle size. The difference between linear and exponential models of the blading effect were minor in the preliminary analysis so the assumption of linearity was considered to be satisfactory. Thus the roughness after blading was represented by the following linear function of the difference between the roughness before blading (\( RG_b \)) and the minimum (\( RG_{\text{min}} \)), as illustrated in (d) of Figure 3.14:

\[
RG_a = RG_{\text{max}} + q \{ RG_b - RG_{\text{min}} \}
\] (3.15)

where \( q \) = vector of explanatory variables to be estimated; for \( 0 < q < 1 \).

Solution

The simultaneous solution of the equations above is the intersection of the two functions as shown in (d), i.e.:

\[
RG_b = \frac{RG_{\text{max}} (1 - p) + p (1 - q) RG_{\text{min}}}{1 - p q}
\] (3.16)

and

\[
RG_a = \frac{q (1 - p) RG_{\text{max}} + (1 - q) RG_{\text{min}}}{1 - p q}
\] (3.17)
The average roughness over a cycle is found by integrating the roughness function (Equation 3.13) over the period $\Delta t$ between consecutive bladings at times $t_a$ and $t_b$, i.e.

$$R_{G_{avg}} = \frac{1}{\Delta t} \int_{t_a}^{t_b} R_G \, dt$$

to derive the general expression:

$$R_{G_{avg}} = R_{G_{max}} - \frac{(R_{G_{max}} - R_{G_{min}})}{\gamma \Delta t}.$$  

The longterm average roughness is then found by substituting the steady state values of $R_{G_{a}}$ and $R_{G_{b}}$, the roughnesses at the beginning and end of the cycle from Equations 3.16 and 3.17, as follows:

$$R_{G_{avg}} = R_{G_{max}} + \frac{(R_{G_{max}} - R_{G_{min}})}{(1 - p) \ln p}$$  

where $\Delta t$ = interval between bladings, in days, and $\Delta t > 0$; and $p$ and $q$ are defined in Equations 3.14 and 3.15 respectively. In the extreme case of no blading maintenance, we note that $R_{G_{avg}}$ tends towards the maximum roughness, $R_{G_{max}}$ and, in the case of extremely frequent blading, that $R_{G_{avg}}$ tends towards the minimum roughness, $R_{G_{min}}$.

### 3.5.2 Estimation and Characteristics of Model

The models for roughness progression, Equation 3.14, and for roughness after blading, Equation 3.15, were estimated separately by nonlinear estimation techniques. In order to determine the factors influencing the maximum roughness, $R_{G_{max}}$ was estimated as a function of parameters, simultaneously with the estimation of roughness progression by Equation 3.14. Likewise, the minimum roughness, $R_{G_{min}}$, was estimated as a function simultaneously with the estimation of blading effects by Equation 3.15. In this way, any limitations in the data of the highest or lowest values of roughness actually observed did not limit the model estimations, and the estimates were based on the observed behavior between lower and upper bounds of roughness that were statistically estimated rather than arbitrarily imposed.

The estimation statistics of the models are summarized in Table 3.9, and the parameters are defined in Table 3.10. So that the models would be transferable to other regions, the physical properties of the materials were used in preference to the broad classifications of material type, including several parameters describing the particle size distribution (gradation) of the surfacing material. The maximum particle size was defined by the 95th percentile size, $D_{95}$, determined from sieve analysis data. The gradation was defined by the dust ratio ($N_{GD}$) for the fine fractions, and by the mean gradation shape ($M_{G}$), as indicated in Table 3.10. The gradation shape ($M_{G}$) relates the distribution of sizes to the particle size, $D$, in the general form:

$$P = p^{M_{G}}$$

where $P$ is the percentage of material finer than size $D$, $M_{G}$ has a value between 0 and 1, and $M_{G}$ normalizes $M_{G}$ about a value of 0.5 which represents gradations approaching maximum density.
Table 3.9: Statistics of model estimations for roughness progression and blading effects under steady-state concept

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimate</th>
<th>t-Stat.</th>
<th>Std. Mean</th>
<th>Std. Dev.</th>
<th>Min.</th>
<th>Max.</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>At</td>
<td>0.000461</td>
<td>2.9</td>
<td>44.1</td>
<td>51</td>
<td>2.3</td>
<td>480</td>
<td>days</td>
</tr>
<tr>
<td>(ADL At/1000)</td>
<td>0.0174</td>
<td>5.9</td>
<td>13.0</td>
<td>19</td>
<td>0</td>
<td>77</td>
<td>vehicles</td>
</tr>
<tr>
<td>(ADH At/1000)</td>
<td>0.0114</td>
<td>5.7</td>
<td>6.5</td>
<td>12</td>
<td>0</td>
<td>46</td>
<td>vehicles</td>
</tr>
<tr>
<td>(ADT MMP At/1000)</td>
<td>-0.0287</td>
<td>4.2</td>
<td>3.6</td>
<td>4.2</td>
<td>0</td>
<td>23</td>
<td>veh m/mo</td>
</tr>
</tbody>
</table>

1. ROUGHNESS PROGRESSION, \( RG(t) \)

Dependent variable \( p \)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimate</th>
<th>t-Stat.</th>
<th>Std. Mean</th>
<th>Std. Dev.</th>
<th>Min.</th>
<th>Max.</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>21.4</td>
<td>15.5</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>(0.5-MGD) (^2)</td>
<td>32.4</td>
<td>5.0</td>
<td>0.046</td>
<td>0.06</td>
<td>0.0</td>
<td>0.24</td>
<td>–</td>
</tr>
<tr>
<td>KCV</td>
<td>0.97</td>
<td>5.2</td>
<td>2.1</td>
<td>2.0</td>
<td>0</td>
<td>5.6</td>
<td>km(^{-1})</td>
</tr>
</tbody>
</table>

Model statistics

Full sample: \( r^2 = 0.856 \); standard error = 1.52 m/km IRI; 1,044 observations.
Subsection means: \( r^2 = 0.916 \); standard error = 0.92 m/km IRI; 192 observations.

2. ROUGHNESS AFTER BLADING, \( RG_a \)

Dependent variable \( q \)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimate</th>
<th>t-Stat.</th>
<th>Std. Mean</th>
<th>Std. Dev.</th>
<th>Min.</th>
<th>Max.</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>0.553</td>
<td>13.0</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>MGD</td>
<td>0.230</td>
<td>4.0</td>
<td>0.64</td>
<td>0.16</td>
<td>0.15</td>
<td>0.99</td>
<td>–</td>
</tr>
</tbody>
</table>

Dependent variable \( RG_{\min} \)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimate</th>
<th>t-Stat.</th>
<th>Std. Mean</th>
<th>Std. Dev.</th>
<th>Min.</th>
<th>Max.</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>D95</td>
<td>0.361</td>
<td>7.1</td>
<td>20</td>
<td>9.2</td>
<td>0.07</td>
<td>39</td>
<td>mm</td>
</tr>
<tr>
<td>(D95 MG)</td>
<td>1.00</td>
<td>4.6</td>
<td>5.5</td>
<td>2.7</td>
<td>0.02</td>
<td>12</td>
<td>mm</td>
</tr>
</tbody>
</table>

Model statistics

Full sample: \( r^2 = 0.550 \); standard error = 2.39 m/km IRI; 1,044 observations.
Subsection means: \( r^2 = 0.793 \); standard error = 1.09 m/km IRI; 188 observations.

- Not applicable

Note: Estimated by nonlinear least squares regression (SAS 1979), one observation per cycle. Parameters defined in Table 3.10.

Source: Analysis of data from Brazil-UNDP study.
Table 3.10: Definition of variables used in prediction models of unpaved road roughness and material loss

| Traffic, geometry and rainfall                                                                                                               |
| ADH   = the average daily heavy vehicle traffic (gross weight ≥ 3,500 kg) in both directions, in vehicles/day.                               |
| ADL   = the average daily light vehicle traffic (gross weight < 3,500 kg) in both directions, in vehicles/day.                              |
| ADT   = the average daily vehicular traffic in both directions, in vehicles/day.                                                             |
| G     = the average longitudinal vertical gradient of the road, in percent.                                                                     |
| KCV   = the average horizontal curvature of the road, in km⁻¹                                                                                   |
| W     = average road width, in meters.                                                                                                          |
| MMP   = mean monthly precipitation rate over observation period, m/month;                                                                       |
| IM    = moisture index, after Thornthwaite (1955).                                                                                             |
| RG₀, RG₁ = roughness at beginning and end of period between bladings, respectively, m/km IRI.                                                |
| RGₐ, RGₐ = roughness after blading, and before blading, respectively, in m/km IRI.                                                            |

| Material properties                                                                                                                           |
| D₉₅   = maximum particle size of material, defined by size with 95% finer, in mm.                                                              |
| P₄₂₅   = amount of material finer than 0.425 mm, in percent.                                                                                   |
| P₀₇₅   = amount of material finer than 0.075 mm, in percent.                                                                                   |
| MGD   = material gradation dust ratio, defined as                                                                                             |
| MGD = (P₀₇₅ / P₄₂₅) if P₄₂₅ > 0, or 1 otherwise.                                                                                              |
| MG   = slope of mean material gradation, such that                                                                                             |
| MG = min (MGM, 1 - MGM) where MGM = (MGO₇₅ + MGO₂₅ + MGO₂) / 3.                                                                                |
| MGO₇₅  = log (P₀₇₅ / D₉₅) / log (0.075/D₉₅) if D₉₅ > 0.4, 0.3 otherwise.                                                                          |
| MGO₂₅  = log (P₄₂₅ / D₉₅) / log (0.425/D₉₅) if D₉₅ > 1.0, 0.3 otherwise.                                                                          |
| MGO₂  = log (P₀₂ / D₉₅) / log (2.0/D₉₅) if D₉₅ > 4.0, MGO₂₅ otherwise.                                                                             |
| PI    = plasticity index of fine material, in percent.                                                                                          |

| Quantities                                                                                                                                     |
| VGS   = volume of spot regravelling material in place, in m³/km.                                                                               |

Source: This study.
Roughness progression

The rate of roughness progression, represented by the parameter $p$, was found to be a function of traffic and rainfall, with no significant effects of material properties being found. The maximum roughness, $R_{G\text{max}}$, was found to be a function of material properties, road geometry and rainfall, which are all characteristics of the road that are essentially independent of age, traffic and maintenance, and represents the potential roughness that the road could reach under no maintenance. Although material properties were not explicit in $p$, they are implicit in the rate of progression through the boundary $R_{G\text{max}}$. The model is a bounded-exponential function, as follows:

$$R_{G}(t_2) = R_{G\text{max}} - p \left[ R_{G\text{max}} - R_{G}(t_1) \right]$$  \hspace{1cm} (3.19)

where

- $R_{G}(t_1) =$ roughness at time $t_1$ in m/km IRI;
- $R_{G}(t_2) =$ roughness at time $t_2$ in m/km IRI;
- $t_1$, $t_2 =$ times elapsed since latest blading, in days;
- $p =$ exp \[- 0.001 \left( 0.461 + 0.0174 \text{ ADL} + 0.0114 \text{ ADH} \\
- 0.0287 \text{ ADT MMP} \right) (t_2 - t_1) \]; and
- $R_{G\text{max}} =$ max $\left[ 21.4 - 32.4 \left( 0.5 - \text{MDG}_j \right)^2 + 0.97 \text{ KCV} \\
- 7.64 \text{ G MMP; 12} \right]$

and other parameters are as defined in Table 3.10.

The statistics show that the model is well-determined and, from the scattergram in Figure 3.15(a), it can be seen that the predictions fit the observed data very closely with a high $r^2$ of 0.86 and standard error of 1.5 m/km IRI (the scattergram requires careful interpretation due to the high density of multiple observations on the line of equality). Much of the remaining variance derives in differences from cycle to cycle, for which there are very few explanatory parameters. For planning and management purposes the aim is to select the best policies for different roads, and the scattergram in chart (b) (which removes the between-cycle variance) shows that the model does this extremely well, with a significantly better fit, a standard error down to only 0.9 m/km IRI, and no bias apparent between the three material classes. The model thus seems to take good account of the major factors influencing policy decisions.

Predictions for the model are shown in Figure 3.16. The maximum roughness, in chart (a), is highest in rolling or hilly terrain, particularly in dry climates, but lowers as the rainfall or gradient increase, presumably because the gradient facilitates runoff of the surface water and because curvature accentuates transverse erosion (which influences roughness more than does longitudinal erosion of the roadway). Materials having a high dust ratio (i.e., clayey, or poorly-graded fines) appear to yield lower maximum roughness levels than better graded materials, probably because earth roads tended to have lower roughness than gravel roads. The rate of progression, in chart (b), is seen to be very dependent on traffic volume, and essentially linear in time or traffic over half to two-thirds of the roughness range. Light and heavy vehicles appear to have rather similar damaging effects, the slight difference in the model being not significant, which is a curious difference from the earlier models (Equations 3.8 and 3.9).
Figure 3.15: Goodness of fit of bounded models for roughness progression and for the effect of blading maintenance on roughness

(a) Progression: Full Sample

\[ r^2 = 0.86 \]
\[ S.E. = 1.5 \text{ IRI} \]
\[ 1,044 \text{ obs.} \]

(b) Progression: Subsection Means

\[ r^2 = 0.92 \]
\[ S.E. = 0.9 \text{ IRI} \]
\[ 192 \text{ obs.} \]

(c) Blading Effect: Full Sample

\[ r^2 = 0.55 \]
\[ S.E. = 2.4 \text{ IRI} \]
\[ 1,044 \text{ obs.} \]

(d) Blading Effect: Subsection Means

\[ r^2 = 0.79 \]
\[ S.E. = 1.1 \text{ IRI} \]
\[ 188 \text{ obs.} \]

Notes:
- Multiple observations: A = 1, B = 2, etc.
- Mean across all cycles of subsection.

Source: Brazil-UNDP study data; Equations 3.19 for (a), (b), and 3.20 for (c), (d).
Figure 3.16: Prediction of maximum roughness and roughness progression for unpaved roads: bounded-exponential progression model

(a) Maximum Roughness $R_{G_{\text{max}}}$

- Hilly Terrain: $MGD = 0.5$, $0.75$, $1.0$ (4% grade, 300 deg/km)
- Flat Tangent: $MGD = 0.5$, $0.75$, $1.0$ (1% grade, 0 deg/km)

Source: Equation 3.19.

(b) Roughness Progression, $R_{G}(t)$

- Maximum
- Initial

Note: MMP = 0.04 m/month; $R_{G_{\text{max}}}$ = 16 IRI
Source: Equation 3.19.
Effect of blading

The effect of blading maintenance on roughness was found to depend on the roughness before blading, the material properties and the minimum roughness, \( R_{G\min} \). This minimum roughness, which is the minimum achievable by blading, was itself found to be a function of material properties. The model estimating roughness after blading is a bounded linear function, based on Equation 3.15, as follows:

\[
R_{G_a} = R_{G\min} + q [R_{G_b} - R_{G\min}]
\]  

(3.20)

where

\( R_{G_a} = \) roughness after blading, in m/km IRI;
\( R_{G_b} = \) roughness before blading, in m/km IRI;
\( q = 0.553 + 0.230 \times MGD; \) and
\( R_{G\min} = \max \{0.8; \min \{8; 0.361 \times D95 (1 - 2.78 \times MG')\}\}; and
\( MG' = \min \{MG, 0.36\} \).

The model statistics, given in Table 3.9, and the goodness of fit shown in Figure 3.15(c), indicate that the fit is as good as the earlier models (Equation 3.11 and 3.12) (in natural, not logarithmic, form), so that the variance observed again derives largely from the operators' performance. This is made clear by the still much better fit shown in chart (d) of Figure 3.15, in which the between-cycles variance has been removed. The error reduces by 60 percent to 1.1 m/km IRI, or about the same level as for roughness progression, and lateritic and quartzitic gravel, and earth surfaces are all well-represented by the one model without undue bias.

Typical predictions are illustrated in Figure 3.17. The minimum roughness, in chart (a), is seen to be quite sensitive to both the maximum particle size and the gradation of the material. Very low roughness levels can be achieved in fine or well-graded materials, e.g., less than 2 m/km IRI in all materials finer than 6 mm maximum size, or in well-graded materials (\( MG = 0.25 \) to 0.30) with up to 20 - 30 mm maximum size. With poorly-graded (\( MG \) less than 0.15) or very coarse materials, blading maintenance cannot reduce the roughness below much higher levels in the order of 5 to 8 m/km IRI. The limits placed on \( R_{G\min} \) were imposed to keep the predictions within the reasonable bounds of the inference space. The reduction of roughness achieved by blading, shown in chart (b), averages about 34 percent of the difference between the before-blading and minimum roughness levels, with only moderate sensitivity to material properties (ranging from 17 to 45 percent reductions as the dust ratio drops from 100 (clayey materials) to 0 (sandy materials)).

3.5.3 Predictions of Average Roughness under Various Policies

When blading maintenance is performed regularly at constant time intervals, or a fixed roughness level, or fixed traffic intervals, the trends of roughness described by these relationships eventually lead to a steady state, characterized by a saw-toothed pattern of the roughness-time profile. The highs and lows represent the roughness immediately before and after grading, respectively defined by \( R_{G_b} \) in Equation 3.16 and \( R_{G_a} \) in Equation 3.17, and the long-term average roughness, \( R_{G\text{avg}} \), is given by Equation 3.18. The parameters \( p, q, R_{G\max} \) and \( R_{G\min} \) are given by the estimates in Equations 3.19 and 3.20, with the time interval \((\Delta t = t_2 - t_1)\) being defined according to the type of maintenance policy.
Figure 3.17: Predictions of minimum roughness and blading maintenance effect: bounded-linear model

(a) Minimum Roughness, $R_{G_{\text{min}}}$

(b) Blading Effect on Roughness: Example for $R_{G_{\text{min}}} = 3$ IRI

Note: $R_{G_{\text{min}}} = 3$ IRI for example.

Source: Equation 3.20.
The predictions for time-scheduled maintenance (At = time interval between bladings) are illustrated in Figure 3.18 for (a) a road under regular 90-day blading maintenance and different levels of traffic, and (b), a road under different (30, 90, 360-day) blading policies and one level of traffic. The surfacing material is a medium-size (D95 = 20 mm) slightly plastic gravel with high dust ratio (MGD = 0.80) and moderate gradation (MG = 0.20).

Increasing the traffic volume under a constant blading frequency has the dual effect of raising the average roughness and of advancing the time at which the long-term average is reached from the minimum possible roughness. Increasing the blading frequency lowers the average roughness and advances the time at which the long-term average is reached. It is apparent from the figure that the roughness progression is essentially linear in most cases, and only becomes noticeably concave at blading frequencies as low as once per year. While the model is thus a reasonable approximation to reality for the long-term effects, it is also apparent that the initial period of a slow rate of progression immediately following construction, recompacon or deep blading (Figure 3.4), is not present, so that short-term effects may be poorly represented for such an initial cycle.

For traffic-scheduled maintenance expressed in terms of the number of vehicle passes between bladings, RG avg is computed from the time interval (At = t₂ - t₁) given by the required number of vehicle passes divided by the ADT.

Under condition-responsive maintenance, blading is done whenever a maximum allowable roughness, RG ha, is reached. The time interval, At (t₂ - t₁), in Equation 3.19, is then as follows:

\[ \Delta t = k \left[ \frac{(Rg_{max} - Rg_{ha})}{(Rg_{max} - (1-q) Rg_{min} - q Rg_{ha})} \right] c \quad (3.21) \]

where \( c = -0.001 \times (0.461 + 0.0174 ADL + 0.0114 ADH - 0.0287 ADT MMP) \).

The impacts of maintenance, traffic and road characteristics on the average roughness are illustrated in Figure 3.19. In charts (a) and (b), the performance is illustrated for an average case with moderate quality of materials, moderate climate and moderate geometry, defined in Table 3.11. The maintenance policy is scheduled by regular time intervals in chart (a), and in terms of the number of vehicle passes in chart (b). For traffic volumes of up to 200 to 400 veh/day, the average roughness is relatively insensitive to traffic volume or to blading frequency, for bladings more frequent than every 120 days; but, at higher traffic volumes or under less frequent blading, the average roughness levels increase more sharply. When the maintenance policy is defined in terms of vehicle passes, as in chart (b), a clearer picture emerges showing the average roughness to be virtually independent of traffic volume except for low volumes. Economic analyses using the model (Bhandari and others 1987) have shown that a policy of blading at intervals of about 4,000 vehicles is close to optimal.

The influences of material properties, and road or climatic characteristics are illustrated in the lower part of the figure. With materials of good gradation, even when these are fairly coarse as for the example with 50 mm maximum size, and with moderate conditions of climate and geometry, the levels of roughness predicted by the model are relatively low, as shown by chart(c). Conversely, with poorly graded materials, and adverse conditions such as an arid climate, strong curvature, etc., the range of roughness levels tends to be much wider and relatively high, as shown in chart(d). By way of comparison, the maintenance
Figure 3.18: Predictions of roughness progression under various traffic volumes and blading policies for unpaved roads

(a) Effects of Traffic Volume under Regular 90-Day Blading Policy

(b) Effects of Blading Frequency for Traffic Volume of 300 Veh/Day

Note: Heavy traffic = 30% ADT; rainfall = 0.04 m/month; $R_{max} = 19.2$ IRI; MGD = 0.8.
Source: Equations 3.18, 3.19, and 3.20.
Figure 3.19: Predictions of average roughness as a function of traffic volume, under various conditions and maintenance policies.

(a) Moderate Conditions; Time-Scheduled Blading

(b) Moderate Conditions; Traffic-Scheduled Blading

(c) Well-Graded Materials; Traffic-Scheduled Blading

(d) Adverse Conditions; Traffic-Scheduled Blading

Note: The characteristics of the materials, geometry, and climate are defined in Table 3.11.
Source: Equations 3.18, 3.19, and 3.20.
Table 3.11: Parameter values used in examples of predicted average roughness from the steady-state model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Moderate</th>
<th>Well-graded material</th>
<th>Poor material semiarid, hilly</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>%</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>KCV</td>
<td>km(^{-1})</td>
<td>3</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>(curvature)</td>
<td>(deg/km)</td>
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<td>(170)</td>
<td>(290)</td>
</tr>
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<td>-</td>
<td>0.2</td>
<td>0.3</td>
<td>0.1</td>
</tr>
<tr>
<td>MGd</td>
<td>-</td>
<td>0.6</td>
<td>0.3</td>
<td>0.6</td>
</tr>
<tr>
<td>D95</td>
<td>mm</td>
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<td>13</td>
</tr>
<tr>
<td>ADH/ADT</td>
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<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>MMP</td>
<td>m/month</td>
<td>0.15</td>
<td>0.2</td>
<td>0.05</td>
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<tr>
<td>(rainfall)</td>
<td>(mm/yr)</td>
<td>(1,800)</td>
<td>(2,400)</td>
<td>(600)</td>
</tr>
</tbody>
</table>

Note: Parameter names defined in Table 3.10.

policies needed to meet a standard of 12 m/km IRI average roughness under a traffic volume of 200 veh/day require regular blading at intervals of 54,000 vehicles for the well-graded materials in (c), of 22,000 vehicles for the moderate conditions in (b), and of 16,000 vehicles for the poor material and adverse conditions in (d). These correspond to blading frequencies of 270 days, 110 days and 80 days, respectively, or a range of maintenance costs varying by a factor of about three.

3.5.4 Spot Regravelling and Roughness

Maintenance by spot regravelling may also be expected to reduce the average roughness on the assumption that the gravel is applied in the major depressions and potholes that have appeared in the surface in the upper ranges of roughness. Roughness levels above 15 m/km IRI are invariably associated with the presence of visible birdbath type depressions or potholes, which become larger or more frequent as the roughness level increases, and these can be effectively patched, with high benefits, by spot regravelling. Over the roughness range of 11 to 15 m/km IRI, such repairable birdbath depressions are frequently, but not always, present so that spot regravelling may not always be effective in this range. For example, spot regravelling is not considered effective on shallow corrugations or on runoff-induced surface erosion, two conditions that commonly cause roughness within this range.

An algorithm developed to represent spot regravelling effects, based on the above rationale, is shown in Figure 3.20, and expressed by:

\[
RG_a = \max \left\{ \left[ RG_b - 0.1(VGS/W) \min(3; RG_b - 12) \right]; 12 \right\} \tag{3.22}
\]
3.6 MATERIAL LOSS

3.6.1 Scope of Study

Regravelling is the major maintenance operation on unpaved roads, analogous in importance to the overlaying of a paved road, so the frequency required is an important planning decision. The gravel loss studies were aimed at predicting the loss of surfacing material on sections with lateritic and quartzitic gravels, and earth roads. The surfacing material of the earth road sections contained more than 35 percent material finer than 0.075 mm. The scope of the study data was summarized in Tables 3.4 and 3.5. Original data are contained in Working Documents 9, 13, 14 and 15 of the project (GEIPOT 1982).

3.6.2 Approach for Gravel Loss Analysis

Gravel loss is defined as the change in gravel thickness over a period of time. On a well compacted subgrade the change in gravel level or gravel height is the change in gravel thickness. Although gravel thickness is not necessarily equivalent to gravel level or gravel height under all conditions, gravel thickness is used here as a synonym for gravel level or height. Since gravel loss is a change of gravel thickness over time, it was not necessary to determine an absolute value at some initial point in time. Gravel loss was evaluated for the interval between regravellings, which initiated a new analysis cycle, or from the time of the first observation until a regravelling occurred.

Three major factors identified as affecting gravel loss were weathering, traffic, and the influence of blading maintenance. Material properties, and road alignment and width influence the gravel loss generated by each of these factors.
The average elevation of a subsection relative to the bench mark was used to evaluate gravel loss. These elevations were obtained at about three monthly intervals, and it was not possible to separate seasonal influences. In the Kenyan study (Hodges, Rolt and Jones 1975), it was shown that no seasonal pattern existed in the data, and this also appeared to be the case for the Brazilian data. Furthermore, seasonal influences do not have any practical implications since the agency responsible for regravelling wishes to know its frequency in terms of years, and has little interest in the influences of each particular season.

3.6.3 Estimation of Model

The following relationship was estimated for predicting the annual quantity of material loss as a function of monthly rainfall, traffic volume, road geometry and characteristics of the surfacing material:

\[
MLA = 3.65 \left[ 3.46 + 2.46 \text{ MMP G} + \text{ KT ADT} \right] \tag{3.23}
\]

where MLA = the predicted annual material loss, in mm/year;

KT = the traffic-induced material whip-off coefficient, expressed as a function of rainfall, road geometry and material characteristics

where

\[
KT = \text{max} \left[ 0; \left( 0.022 + 0.969 \text{ KCV} + 0.00342 \text{ MMP P075} - 0.0092 \text{ MMP PI} - 0.101 \text{ MMP} \right) \right]
\]

with \( r^2 = 0.313; \) standard error = 49 mm/year; sample = 456 observations; and the t-statistics for MLA were 3.1 and 2.6, and for KT were 3.7, 1.1, 3.9, 3.0 and 2.8, for the respective coefficients.

3.6.4 Predictions of Gravel Loss

The predictions are illustrated in Figure 3.21 for a surfacing material of slightly plastic, fine silty gravel, showing the effects of first, traffic and rainfall for flat terrain, and second, rainfall and geometry for a traffic volume of 200 veh/day. The effect of traffic volume dominates the rate of gravel loss, with the annual rate increasing by about 10 mm for every 100 vehicles/day increase in ADT, and the average rate being about 30 to 40 mm per 100,000 vehicles, depending on other factors. Increasing horizontal curvature increases the loss rate through whipoff under traffic, but the effect is not large and amounts to only a 20 percent increase over the full range of curvature from 0 to 5.5 km\(^{-1}\) (0 to 300 degrees/km). Rainfall also affects the loss rate, but by amounts that vary with gradient and with the fines' plasticity of the material: for materials with more than 50 percent fines the loss rate is likely to increase, and for others it may decrease; typically it may increase the loss rate by about 10 percent per m/month of rainfall.

3.7 VALIDATION

The validity and transferability of the models have been evaluated by comparing the predictions with data from the Kenyan study (Hodges and others 1975), and also smaller amounts of data from Ghana (Roberts 1983), Ethiopia (Robinson 1982), Bolivia (Butler and others 1985) and a recent study in Kenya (Jones 1984).
Figure 3.21: Predictions of surfacing material loss related to traffic, rainfall and geometry for unpaved roads

(a) Effects of Traffic and Rainfall on Flat Terrain

(b) Effects of Traffic and Rainfall under 200 Veh/Day

Note: PI = 10%; P075 = 66%; C = 50 deg/km; G = 0%.
3.7.1 Effect of Blading on Roughness

Comparison of the predicted effect of blading on roughness with data from Ghana is presented in Table 3.12. The exponential model (Equations 3.10 and 3.12), predicts the effect of blading well, with an average bias of only -1 percent and a prediction error in the order of 14 percent. The bounded-linear model of the steady-state solution (Equation 3.20) also predicts well, with an average bias of 5 percent and a prediction error of 15 percent. In both cases the prediction errors are less than half of the error in the original estimations (33 and 31 percent, respectively). In the bounded-linear model, the large errors for sections 1G1, 3G4 and 4G4 seem to be related to the estimate of the minimum roughness boundary \( \text{RG}_{\text{min}} \), since the observed roughness after blading is less than the estimated \( \text{RG}_{\text{min}} \). With the exponential model, the overestimates derive from the coarser materials (1G1 and 3G4) and the underestimates from the finer materials (2G2 and 4G4), which implies that the effect of maximum particle size (R25) may be exaggerated in the model.

With negligible bias and prediction errors well within the statistical limits, both models are thus well validated on these data from Ghana. Moreover, the data encompass a wide range of conditions, for example, with MG of 0.13 to

<table>
<thead>
<tr>
<th>Section</th>
<th>Observed roughness</th>
<th>Predictions by exponential model</th>
<th>Predictions by bounded-linear model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \text{RG}_b )</td>
<td>( \text{RG}_a )</td>
<td>( \hat{a} )</td>
</tr>
<tr>
<td>1G1</td>
<td>8.8 - 9.5</td>
<td>6.7 - 7.2</td>
<td>1.50</td>
</tr>
<tr>
<td>2G2</td>
<td>9.0 - 9.6</td>
<td>8.6 - 8.7</td>
<td>1.21</td>
</tr>
<tr>
<td>2G3</td>
<td>9.0 - 12.2</td>
<td>7.5 - 8.5</td>
<td>1.32</td>
</tr>
<tr>
<td>3G4</td>
<td>6.9 - 7.9</td>
<td>6.6 - 7.0</td>
<td>1.54</td>
</tr>
<tr>
<td>4G4</td>
<td>9.6 - 10.2</td>
<td>8.2 - 8.5</td>
<td>1.21</td>
</tr>
<tr>
<td>5G4</td>
<td>10.8 - 11.2</td>
<td>9.6 - 10.5</td>
<td>1.55</td>
</tr>
<tr>
<td>Average bias:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prediction error:</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: Exponential Model: \( \text{RG}_a = a \text{RG}_b^{0.771} \) where \( a \) is defined in Equation 3.12.

Bounded-Linear Model: \( \text{RG}_a = \text{RG}_{\text{min}} + q (\text{RG}_b - \text{RG}_{\text{min}}) \) where \( q \) and \( \text{RG}_{\text{min}} \) are defined in Equation 3.20.

Roughness converted to IRI units according to Figure 2.9.

Source: Analysis using data from Roberts (1983).
0.43, D95 in a range of about 9 to 80 mm, plasticity index (PI) of 1 to 19 percent, rainfall from 1,020 to 1,910 mm per year, and roughness before blading of 6.9 to 12.2 m/km IRI.

In the Kenyan study (Hodges and others 1975), the roughness after blading was modelled as a constant, with values of 4.3 IRI for lateritic, quartzitic and volcanic gravels, and 7.9 IRI for coral gravels. The first category is most applicable to the Ghanaian materials, but the Kenyan roughness level of 4.3 IRI is much lower than the average 8.1 IRI observed in Ghana. Equations 3.10 and 3.20 are therefore preferable to the TRRL-Kenyan Model.

### 3.7.2 Roughness Progression

The method of modelling roughness progression differs among the various studies, some using a time base and some a traffic base, with most separating the observations by material type. As a basis for comparison, therefore, the average rates of progression observed in the Brazil study are compared with the long-term (over 400 days) mean rates reported in five other studies in Table 3.13, using the International Roughness Index as the common scale of roughness.

In the Brazilian data, the rates of progression per vehicle were higher on lateritic gravels than on either quartzitic gravels or earth roads; and on a time basis, the rates for both gravels tended to be higher than for earth roads because the traffic volumes were higher. This result is related partly to blading frequency, because stratification of the data revealed greater similarity and lower values in the vehicle-based rates across the different materials for blading intervals of longer than 30 days. High progression rates were therefore often associated with high blading frequencies in the study, which implies that frequent disturbance of the gravel surface caused high progression rates and that compacted surfaces showed comparatively lower rates.

Comparing the mean rates of roughness progression in Table 3.13, it can be seen that they range widely, from 0.03 to 0.37 m/km IRI per thousand vehicles or a factor of about 10, and from 0.003 to 0.20 m/km IRI per day. Taking account of the fact that the Brazilian results were influenced by blading frequency, the observed long-term rates in Brazil are lower than shown, ranging from 0.10 (quartzitic gravels) to 0.24 (lateritic gravels) m/km IRI per thousand vehicles, which puts them more in line with the other studies. The Brazilian rates for lateritic gravels were still three times higher than those observed in both Kenyan studies. The results for quartzitic gravels are more similar, being 0.10, 0.08 and 0.13 IRI per thousand vehicles in the Brazil, 1975 Kenya and 1984 Kenya studies, respectively.

There is insufficient information available at this stage to apply the model and evaluate the wide range of roughness progression rates being observed. Part of the range is attributable to physical material properties and rainfall, but in this case one would expect that the models estimated from the Brazilian study, which had the scope to permit such determination, would have shown even stronger effects than they did. The differences observed across material types in the two Kenyan studies were also large, as illustrated in Figure 3.22 on a basis of cumulative vehicles.

The most plausible explanation for the remaining differences concerns the looseness or compaction of the surface materials which affect their effective
### Table 3.13: Comparison of observed rates of roughness progression on unpaved roads from various studies in Africa and Latin America

<table>
<thead>
<tr>
<th>Study and material type</th>
<th>Mean rates of roughness progression</th>
<th>Material properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IRI per 1000 vehicles</td>
<td>IRI per period</td>
</tr>
</tbody>
</table>

**Brazil 1/**
- Lateritic gravel 0.37 0.056 109 142 21-609 9-12 139 (608) 8-34 17-35 0-21
- Quartzitic gravel 0.25 0.065 108 78 18-390 7-11 128 (608) 15-34 10-40 0-15
- Earth roads 0.24 0.018 219 41 21-512 8-11 130 (608) 0.1-16 28-97 0-33

**Kenya, 1975 2/**
- Lateritic, quartzitic, volcanic gravels 0.078 0.016 350 5 42-403 - 33-170 12-25 13-45 0-26
- Coral 0.034 0.006 450 1 99 - 33 1-25 20-24 12-14

**Kenya, 1984 3/**
- Lateritic gravel 0.063 0.011 530 4 110-224 - 40-170 30 25-30 12-14
- Volcanic gravel 0.24 0.024 550 2 70-140 - 40-170 30 12-16 16-17
- Quartzitic gravel 0.13 0.006 600 2 25-95 - 40-170 25 14-19 14-18
- Sandstone gravel 0.18 0.009 600 2 42-96 - 40-170 25-30 16-25 6-8

**Ethiopia 4/**
- Basalt gravel 0.24 0.043 270 5 175 7 120 (250) 75 7 23
- Basalt gravel 0.21 0.042 270 5 175 7 120 (250) 30 11 29
- Cinder gravel 0.20 0.040 500 5 180 10 73 (170) 20 5-15 np-sp

**Bolivia 5/**
- Natural gravel 0.20 0.20 30 2 \{ 575- 10 - \} 40 4 0
- Reconstructed 0.07 0.065 120 2 \{ 1,690 10 - \} 10 40 16

**Ghana 6/**
- Climatic zone 3 0.19 0.009 500 3 30-330 7-9 75 \{ 20-22 5-8 \}
- Climatic zone 1 0.17 0.008 600 3 78-120 8-9 85 \{ 18-29 1-8 \}
- Climatic zone 2 0.05 0.003 550 3 70-120 8-11 138 \{ 22-33 16-20 \}
- Climatic zone 4 0.14 0.008 350 3 50-240 5-8 159 \{ 12-22 1-6 \}

1/ Source: Brazil–UNDP road costs study (GEIPOT 1982) as analyzed in this volume. In-service sections. Observed progression rates include sections under both high- and low-frequency maintenance.

2/ Source: Kenya road costs study (Hodges, Rolt and Jones 1975). In-service sections. The values reported represent low-frequency maintenance sections, but the study indicated that rates under high-frequency maintenance were similar.


5/ Source: Butler, Harrison and Flanagan (1985). In-service sections. Reconstruction was experimentally-controlled with gravel surfacing ripped, modified by addition of cohesive silty material with cited properties, and compacted.

6/ Source: Roberts (1983). In-service roads. Roughness progression rates are maximum values of one-year averages.

Maximum size not available; 40 to 60% of material was coarser than 2.80 mm. np-sp: not available. np. nonplastic. sp. slightly plastic.
Figure 3.22: Roughness progression relationships for gravel roads developed in 1975 and 1984 TRRL studies in Kenya

Note: Roughness conversion: \( R_I = 0.0032 \times \text{RBI} \), where \( \text{RBI} = \text{mm/km TRRL Bump Integrator Trailer} \), and \( R_I = \text{m/km RI} \). Source: Hodges, Rolt, and Jones (1975) and Jones (1984).

Shear strength. The study in Bolivia (Butler and others 1985), though limited to only two road sites, demonstrated that the process of compaction and modifying the gravel with fines (termed "rehabilitation" in that study) effected a considerable reduction in the initial rate of roughness progression, as illustrated in Figure 3.23 (a) and (b). The predictions of the model (Equation 3.19) are seen to be reasonable for the roughness under regular blading and high traffic volumes of 1,692 vehicles per day, but to be underestimating the progression rate under the lower volume of 575 vehicles per day (there was surprisingly little difference in the observed behaviors of the two sections even though the traffic flows differed by a factor of three). In the first cycle after compaction with silt-modification of the surface, in (b), the initial rate is poorly predicted, but the predicted rate matches the data almost exactly from a period of forty days onwards (when the curves are translated). In the other studies, the road sections monitored were variously of new construction or old construction as noted in Table 3.13, but no clear relationship existed between this and the progression rates.

A small validation study in Niger on three roads totalling 186 km in length found that the predictions of average roughness were within about 7 percent of the observed average roughnesses, as shown in Table 3.14. These were generally straight, flat roads carrying 38 to 117 veh/day in a semiarid climate of 700 mm annual rainfall and with a variety of fine to coarse materials.

In summary, there is confidence in the applicability of the steady-state model to those soil and gravel materials which have subangular particle shapes, but the applicability to strongly rounded, angular or vesicular materials has yet to be determined. At a detailed level of predicting roughness progression during an individual cycle, the applicability of the model is limited to phases subsequent to the initial phase after major works.
Figure 3.23: Comparison of model predictions of roughness progression with observed data from Bolivia

(a) Regular Blading Cycle

(b) First Cycle after Rehabilitation (compaction or regravelling)

Note: Roughness conversion: RI = RQI/13, where RQI = count/km Ql, and RI = m/km RI.
Source: Bolivian data from Butter and others (1985); model predictions given by Equations 3.18, 3.19, and 3.20.
Table 3.14: Validation of average roughness predictions in Niger

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Road A</th>
<th>Road B</th>
<th>Road C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic volume</td>
<td>veh/day</td>
<td>38</td>
<td>117</td>
<td>65</td>
</tr>
<tr>
<td>Gravel type</td>
<td></td>
<td>Laterite</td>
<td>Laterite</td>
<td>Laterite</td>
</tr>
<tr>
<td>Maximum stone size</td>
<td>mm</td>
<td>5-20</td>
<td>10-80</td>
<td>10-40</td>
</tr>
<tr>
<td>Length surveyed</td>
<td>km</td>
<td>36</td>
<td>80</td>
<td>70</td>
</tr>
<tr>
<td>Roughness:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>observed range</td>
<td>IRI</td>
<td>5-14</td>
<td>7-14</td>
<td>7-11</td>
</tr>
<tr>
<td>observed average</td>
<td>IRI</td>
<td>8</td>
<td>10-13</td>
<td>9</td>
</tr>
<tr>
<td>predicted average</td>
<td>IRI</td>
<td>7.4-7.7</td>
<td>11.5-13.7</td>
<td>9.1-9.9</td>
</tr>
<tr>
<td>error</td>
<td>%</td>
<td>-6</td>
<td>+7</td>
<td>+5</td>
</tr>
</tbody>
</table>

Notes: Assigned values: rainfall 0.06 m/mo; curvature 0.2 km⁻¹; grade 0%; material gradation MG=0.3, MGD=0.5; two bladings/yr.


3.7.3 Material Loss

A comparison of four separate studies on gravel loss shows conflicting evidence on the effects of road gradient and rainfall, but broadly similar effects of traffic, as summarized in Table 3.15. In the relationships derived from the Kenya costs study (Hodges, Rolt and Jones 1975), strong rainfall effects were found raising the loss rate threefold on flat gradients to fivefold on moderate gradients (3 percent) for an increase from 1,000 mm to 2,000 mm per year of rainfall. In the Brazilian study (Equation 3.22) the effects were very minor with comparable effects of only seven and ten percent, respectively. In the Ghanaian study (Roberts 1983), lower loss rates were observed in the higher rainfall area though positive effects of gradient were observed. In all these three studies, the effects of increasing gradient were positive, with a change from 0 to 3 percent under moderate rainfalls. Studies in Kenya by Jones (1984) and in Oregon by Lund (1973) did not detail any rainfall or gradient effects.

The best indications of gravel loss rates are given on a per vehicle basis, and in Table 3.15 the rates have been normalized to mm loss per 100,000 vehicle passages. When the results are adjusted to a common basis of mixed vehicles with about 50 percent heavy vehicles (which applied in the Brazil study), the results converge to values in the order of:

0 percent gradient: 20 to 30 mm/100,000 vehicles.
3 percent gradient: 40 to 50 mm/100,000 vehicles.

for an average rainfall of 1,500 mm per year. This is also in general agreement with Jones (1984), who shows a range of 30 to 70 mm per 100,000 vehicles in a comparison of seven Africa studies, including the Ghanaian and two Kenyan studies mentioned here, Ethiopia, Cameroon, Niger and Ivory Coast. The British studies in Kenya suggest, however, that the rate is not linearly proportional to traffic, as the Brazilian and Oregonian studies indicate, but instead reaches a peak in the order of 15 to 35 mm per year for traffic volumes of over 200 vehicles per day.
Table 3.15: Comparison of gravel loss rates from various studies

<table>
<thead>
<tr>
<th>Study location</th>
<th>Units</th>
<th>0%</th>
<th>3%</th>
<th>Percentage heavy vehicles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1,000</td>
<td>2,000</td>
<td>1,000</td>
<td>2,000</td>
</tr>
<tr>
<td>Brazil</td>
<td>V</td>
<td>30</td>
<td>32</td>
<td>39</td>
</tr>
<tr>
<td>Ghana</td>
<td>HV</td>
<td>(20-100)</td>
<td>(10-60)</td>
<td>40-160</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>30</td>
<td>13</td>
<td>41</td>
</tr>
<tr>
<td>Oregon</td>
<td>AV</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Kenya A</td>
<td>V</td>
<td>(7)</td>
<td>(21)</td>
<td>(12)</td>
</tr>
<tr>
<td>Kenya B</td>
<td>V</td>
<td>10</td>
<td>30</td>
<td>17</td>
</tr>
</tbody>
</table>

¹/ Assumed value.

Notes: V = per 100,000 vehicles; HV = per 100,000 heavy vehicles when light vehicles are present but not counted; AV = articulated logging vehicles, rated as AV = 3 HV = 6 V. --. Not available. ( ). Original data not adjusted for vehicle mix.

Sources: Brazil: Figure 3.21; Ghana: Roberts (1983); Oregon: Lund (1973); Kenya A: Hodges, Jones and Rolt (1975); Kenya B: Jones (1984).

Material properties are likely to be important factors in the loss rate but only the Brazilian study has estimates of these, and these are not sufficient as they stand to explain the range of rates evident in the data. The rainfall effects in the 1975 Kenyan study appear to be overstated, probably due to the analytical approach adopted in the modelling which accentuated water erosion (Jones 1984).

Based on this discussion and Table 3.15, a general model which includes the major traffic, gradient and rainfall effects but excludes material properties, is given by the following:

\[
GL = (30 + 180 MMP + 72 MMP G) h ADT At. 10^{-5}
\]

(3.24)

where
- \(GL\) = surface material loss, in mm;
- \(MMP\) = mean monthly precipitation, in m;
- \(G\) = average longitudinal gradient, in percent;
- \(h\) = proportion of heavy vehicles in traffic, fraction;
- \(ADT\) = average daily traffic, in veh/day; and
- \(At\) = time period, in days.

The general effects of material properties, as understood to date, are:

1. Increasing the plasticity index reduces the loss rate (Brazil, Ghana);
2. Increasing the relative compaction of the surfacing reduces the loss rate significantly (Lund 1977); and

3. Coral gravels have 40 percent higher loss rates than lateritic, quartzitic and sandstone gravels, which in turn have 40 percent higher loss rates than volcanic (versicular) gravels (Kenya).

3.8 CONCLUSION

The study of unpaved road deterioration is clearly complicated by the high variances deriving both from material behavior (and the many material, geometric and traffic factors influencing that) and from experimental measurement errors. Nevertheless, advances have been made in developing parameterized models of roughness progression, the effect of blading maintenance, and material loss, and the validation study has shown that these models are sufficiently reliable to apply to a reasonable range of conditions with only few caveats.

For roughness progression, the model based on Brazilian data (Equation 3.19) utilizes physical parameters of geometry, climate and material properties and thus provides a sound basis for transferability to other regions and materials. The model indicates that well-graded materials (MG of 0.3 or greater) perform best especially when the fine fraction (less than 2 mm) is well graded. However, a failing of the model which may need correction in specific cases is that the rates of roughness progression predicted correspond to surfaces that are bladed regularly and have not been recently recompacted or reconstructed. The Bolivian studies showed (and the Kenyan studies give implicit support) that the rate of roughness progression, after compaction or special binding treatment with silt, may be in the order of one quarter or less of the rate predicted by the model for the first forty days or more. Thus an important omission from the model at present appears to be the factor of surface compaction, but compensation for this can be readily made.

For the effect of blading maintenance, two models (Equations 3.11 and 3.20) perform well, with prediction errors in the order of 14 percent and negligible bias, although the first, an exponential model, is probably the preferred one for individual use. The second, a linear bounded model, has special application in the steady-state solution for roughness under regular blading maintenance, and performs well within the range of its bounds; however, the relationship for minimum roughness (Equation 3.20) may be limited in extrapolation beyond those bounds.

On material loss, the various studies differ on the impacts of rainfall and geometry, but a reasonable consensus on the impact of traffic could be demonstrated. The impact of material properties remains poorly quantified at this stage though some general trends have been identified (in the previous section). While the Brazilian model, Equation 3.23, utilizes the most explanatory parameters, the general model derived from all the studies reported, and given in Equation 3.24, gives predictions which are perhaps the most broadly applicable, especially when adjusted for material type.

The effect of material properties on passability and the design of adequate thickness and material properties to control rutting and ravelling have been well studied and the criteria given in Section 3.1 are recommended.
Issues remaining to be properly quantified include the effects of compaction (on roughness and material loss), of shallow versus deep blading, of transverse profile (crown and side drainage), and so forth. There is scope also for extending the implementation of the material gradation parameters, defined here, to future studies.
CHAPTER 4
Paved Roads: Concepts and Empirical Methodology

As paved roads carry typically more than ninety percent of the total vehicle-kilometers travelled in a network, and as the vehicle operating costs are strongly influenced by road condition, the deterioration and maintenance of paved roads have a major impact on the economy in the road transport sector. The pattern of deterioration of paved roads is distinctly different from that of unpaved roads and is dominated by the behavior of a largely impervious surfacing that is considerably more resistant to wear by traffic and water than either gravel or earth materials. The rate of deterioration is normally slow, the changes are small, and variability is high, so that special study techniques are required to measure these changes without resorting to lifetime study periods.

This chapter introduces the concepts of paved road deterioration and the study methods and modelling issues involved in developing predictive relationships. It describes the modes and mechanisms of distress, the modes of maintenance intervention and a summary of previous studies. It then describes the primary data base and methodology used for developing the present set of predictive models, discussing particular issues which influence the functional form and parameterization of the models. Ensuing chapters consider the major modes of distress individually.

4.1 LIFE CYCLE OF PAVED ROADS

4.1.1 Performance and Deterioration

The primary objective of both pavement design and pavement maintenance is to ensure that the pavement gives adequate service to the road users. The performance of the pavement is measured in relation to the quality of service provided and the achievement of acceptable levels of service. Measures of service and performance are difficult to define, however, because roads deteriorate through a variety of different mechanisms.

Initially the primary concern of structural design was to control rutting, the development of excessive channelized deformation in the wheelpaths. Acceptable rut depths in the order of 20 to 25 mm or greater for secondary roads and 10 to 12 mm for primary roads evolved, based in part on the hazard of ponding water and its relation to the crossfall of the pavement surface. The early structural design methods concentrated on estimating the cover thickness of high quality materials that would be sufficient to protect the natural subgrade from excessive deformation within these limits. Relationships were developed between measures of the material shear strength, the layer thicknesses and various measures of traffic and wheel loading. While rut depth was the primary criterion of structural performance, a major criterion of service was riding quality, assessed qualitatively at first but with physical measures of roughness evolving by the 1920s. Riding quality, however, was not included as a criterion for design
until the 1960s, by when experimental research had developed empirical relationships between it, pavement properties and traffic.

The concept of serviceability of a pavement was defined by Carey and Irick (1960) in order to quantify the subjective measure of the level of service to users and to provide a basis for relating this to various physical measures of pavement condition, not only rut depth. The five-point scale defined the perfect level of service by a score of 5, and an "impassable" condition by the origin, zero, and the Present Serviceability Rating (PSR) was assessed by a panel of people according to a defined methodology (an approach that paralleled the Riding Comfort Index (RCI) scale developed about the same time in Canada, though with a ten-point scale). The performance of a pavement was then defined as the trend of serviceability with time, which was commonly depicted as a concave curve with an accelerating negative slope, with the level of service declining more and more rapidly as the pavement aged, as shown in Figure 4.1. The minimum level of service expected by users of a facility is one criterion for maintenance, and serviceabilities of about 1.8 to 2 for minor and secondary roads, about 2.5 for primary roads, and 3.0 or greater for major routes, motorways and freeways, have been established (in many cases arbitrarily, and in others, quantified from panel surveys, see Section 2.1).

The concept of performance has gained widespread recognition around the world as a useful way of depicting the general deterioration of a pavement from good condition to poor, and the effects of maintenance in improving the serviceability, producing a saw-toothed trend. However, few agencies outside North

Figure 4.1: Performance defined by the trend of serviceability with time
America use quantitative measures of it, and still fewer (even within North America) quantify serviceability by panel rating. Panel ratings are difficult to control so as to ensure repeatability, and their reproducibility, especially across regional and national boundaries, is particularly poor, as was discussed in Section 2.3. The recent NCHRP study by Janoff and others (1985) is an effort to revive and rationalize this approach. Most agencies make physical or visual measures of distress, and serviceability, where it is used as a criterion, is estimated indirectly by correlation with distress (usually with roughness).

A physical interpretation of serviceability was established at the AASHO Road Test by statistical correlation between the FSR and various measures of pavement distress. For flexible pavements, the strongest correlation was given by roughness, there quantified by a slope variance statistic, with minor improvements given by adding the mean rut depth and the area of cracking and patching. The resulting correlation equation defined the Present Serviceability Index (PSI), being the statistical estimate of FSR, as follows (Highway Research Board 1962):

$$\text{PSI} = 5.03 - 1.91 \log_{10}(1 + \text{SV}) - 1.38 \text{RD}^2 - 0.01 (\text{C + P})^{0.8}$$

(4.1) (13.7) (4.1) (1.3)

where

- $\text{SV}$ = slope variance, a measure of longitudinal roughness;
- $\text{RD}$ = average rut depth, in inches; and
- $\text{C + P}$ = area of class 2 and class 3 cracking\(^1\) plus patching, in ft/1,000 ft\(^2\) (or tenths of a percent);

with $r^2 = 0.844$; standard error = 0.38; sample = 74 flexible pavements; and t-statistics as given in parentheses. In this correlation, the roughness (SV) term explained about 81 percent of the variance, and the remaining terms only a further 3 percent. Individually, the cracking and patching term correlated much more strongly with FSR than this would suggest, and the rut depth less strongly, with correlation coefficients of 0.62 and 0.16, respectively, compared with 0.90 for the slope variance term (see also Finn 1973). This is due to the strong correlation ($r = 0.65$) existing between the slope variance and cracking terms. Subsequent studies (Yoder and Milhous 1964) developed fairly similar relationships for PSI on pavements elsewhere, although the coefficients varied depending upon the measuring equipment used.

There are three major problems with using a summary statistic such as PSI as a performance parameter, namely:

1. Different types of maintenance are appropriate for different levels, and relative levels, of each type of distress. For example, resurfacing is considered timely when cracking reaches about 30 percent of the area, but even at 100 percent cracking the loss of serviceability by the PSI formula is only 0.33. The maintenance, and the consequences of deferring maintenance, will differ for an uncracked rough pavement, and for a severely cracked and rutted pavement with low to moderate roughness, even though both may have the same serviceability.

----

\(^1\) Cracks of 1 to 3 mm width, and more than 3 mm width, respectively (see Chapter 5 for definition).
2. The relative seriousness of different defects varies with the pavement type, the environment, the rate of deterioration, and the maintenance standard. Thus, the relative weightings developed for asphalt pavements in the wet-freezing climate of the Illinois region is not necessarily applicable for pavements with thin surfacings or semirigid pavements, nor for pavements in a dry nonfreezing climate, for example.

3. Each type of distress evolves at different rates in different types of pavements and under different conditions of traffic and environment. Thus modelling the performance by the PSI (or PSR) alone must average over many different combinations of these types, with a consequently high variance that may mask or suppress the effects which are of interest (see Nunez and Shahin 1986, for example).

The more versatile approach to predicting deterioration and maintenance effects is to model the major modes of distress individually. Extensive research in the mechanistic analysis of pavement behavior over the past twenty five years has demonstrated the fundamental parameters and functional forms that determine behavior in the major distress modes. The individual types of distress are physically measurable and definable without supposition as to relative weightings. Further, the approach permits the analysis of a variety of design and maintenance strategies under different scenarios, without suppositions as to environment, pavement type, design standards or maintenance standards.

Thus the deterioration of paved roads is defined here in terms of individual modes of distress, as described next. This study thus departs from the approach of a single summary performance statistic adopted, for example, in the AASHTO performance model with PSI (AASHTO 1981) or in the "PAVER" model with Pavement Condition Index (PCI) (Nunez and Shahin 1986). Instead, it follows a line which examines the underlying causalities of distress and their interactions. The resulting predictions can be combined subsequently in any manner so as to produce a desired summary statistic or, indeed, to estimate appropriate relative weightings of the individual distress types.

4.1.2 Modes and Types of Distress

The deterioration of paved roads is defined by the trend of its surface condition over time. The defects in a pavement surface, usually quantified through a pavement condition survey, are classified under three major modes of distress, namely:

1. Cracking (or fracture);
2. Disintegration; and
3. Permanent deformation,

and further by distress type, as listed in Table 4.1. The most important types to be predicted for planning purposes are those which trigger decisions to undertake maintenance, namely the following:

1. Cracking (primarily crocodile cracking);
2. Ravelling;
3. Potholing;
4. Skid resistance;
Table 4.1: Classification of pavement distress by mode and type

<table>
<thead>
<tr>
<th>Mode</th>
<th>Type</th>
<th>Brief description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking</td>
<td>Crocodile(^1)/</td>
<td>Interconnected polygons of less than 300 mm diameter.</td>
</tr>
<tr>
<td></td>
<td>Longitudinal</td>
<td>Line cracks longitudinal along pavement</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>Line cracks transverse across pavement</td>
</tr>
<tr>
<td></td>
<td>Irregular</td>
<td>Unconnected cracks without distinct pattern</td>
</tr>
<tr>
<td></td>
<td>Map</td>
<td>Interconnected polygons more than 300 mm diameter</td>
</tr>
<tr>
<td></td>
<td>Block</td>
<td>Intersecting line cracks in rectangular pattern at spacing greater than 1 m.</td>
</tr>
<tr>
<td>Disinte-</td>
<td>Ravelling</td>
<td>Loss of stone particles from surfacing</td>
</tr>
<tr>
<td>gregation</td>
<td>Potholes</td>
<td>Open cavity in surfacing (&gt; 150 mm dia.; &gt; 50 mm depth).</td>
</tr>
<tr>
<td></td>
<td>Edgebreak</td>
<td>Loss of fragments at edge of surfacing</td>
</tr>
<tr>
<td>Deforma-</td>
<td>Rut</td>
<td>Longitudinal depression in wheelpaths</td>
</tr>
<tr>
<td>tion</td>
<td>Depression</td>
<td>Bowl-shaped depression in surfacing</td>
</tr>
<tr>
<td></td>
<td>Mound</td>
<td>Localized rise in surfacing</td>
</tr>
<tr>
<td></td>
<td>Ridge</td>
<td>Longitudinal rise in surfacing</td>
</tr>
<tr>
<td></td>
<td>Corrugation</td>
<td>Transverse depressions at close spacing</td>
</tr>
<tr>
<td></td>
<td>Undulation</td>
<td>Transverse depressions at long spacing (&gt; 5 m)</td>
</tr>
<tr>
<td></td>
<td>Roughness</td>
<td>Irregularity of pavement surface in wheelpaths</td>
</tr>
</tbody>
</table>

\(^1\) Alligator cracking, in North American usage.

Source: Author.

5. Rutting; and
6. Roughness.

Cracking, ravelling and potholing are often collectively termed surface distress because they are defects which usually originate and develop within or near, the surfacing layer (though this is not to imply that other components do not influence their development). They are characterized by two phases of development, an initiation phase after construction before the defects first appear on the surface, and a progression phase during which the defects progressively develop in extent or amount of the surface area and in severity, as illustrated in Figure 4.2.

For the purposes of predictive modelling, the initiation of distress is defined by the time when the defect is first visible because this is the only feasible choice compatible with the data on road condition that is practicably measurable by network monitoring. The amount of distress is expressed by the surface area that it covers, usually most conveniently as a percentage of the area of the travelled way. For cracking, severity is measured in terms of the width of opening and the intensity, or length of crack per unit area, which are simplified here to two classes, namely "all cracking" (which includes cracking of all severities) and "wide cracking"; further definition and the modelling of cracking are described in Chapter 5.
Figure 4.2: Primary modes of pavement distress and their trends as defined for the modelling of road deterioration and maintenance effects

(a) Cracking and Potholing

Limit 100%

All Cracking

Wide Cracking

Potholing

Limit 30%

Area (%)

Initiation

Progression

Rehabilitation

(b) Ravelling

Limit 100%

Area (%)

Initiation

Progression

Rehabilitation

(c) Rutting

Limit 50 mm

Mean

Standard Deviation

Depth (mm)

Rehabilitation

(d) Roughness

Limit 12 m/km IRI

IRI

Rehabilitation

Source: Author.
Ravelling and potholing develop through disintegration of the surfacing material, and the severity is a function of the depth of penetration. These are modelled in Chapter 6, together with skid resistance, which is a function of the texture of the pavement surface and deteriorates through abrasive wear.

Rutting and roughness develop through permanent deformation of materials throughout the depth of the pavement. Both develop progressively from the time of construction but, in both cases, the rate of progression may be accelerated by weakening of the pavement due to cracking, as illustrated in (c) and (d) of the figure. The severity of rutting is characterized by the rut depth, and its extent by the variation of rut depth over the surface; these are discussed in Chapter 7. Roughness, defined by a numeric relevant to its impact on vehicle ride and costs (Chapter 2), is the primary criterion of performance and economic benefits, and is modelled in Chapter 8.

4.1.3 Mechanisms of Distress

These types of distress develop through a number of different mechanisms, as illustrated in Figure 4.3. Traffic axle loadings induce levels of stress and strain within the pavement layers which are functions of the stiffness and layer thickness of the materials and, which under repeated loading, cause the initiation of cracking through fatigue in bound materials and the deformation of all materials to various degrees, dependent upon the material properties. Weathering causes bituminous surfacing materials to become brittle, and thus more susceptible to cracking and to disintegration (which includes ravelling, spalling, and edge-breaking). Once initiated, cracking progresses in area and severity to the point where spalling and, ultimately, potholes develop. Open cracks on the surface and poorly maintained drainage systems permit excess water to enter the pavement, hastening the process of disintegration, reducing the shear strength of unbound materials, and thus increasing the rate of deformation under the stresses induced by traffic loading. The cumulative deformation throughout the pavement depth is manifested in the wheelpaths as ruts and more generally in the surface as an unevenness or distortion of profile defined by roughness. Environmental effects of drainage, weather and seasons influence the strength and behavior of the pavement materials under traffic, and can cause distortions and volume changes which contribute to the roughness.

Pavement roughness is therefore the result of a chain of distress mechanisms and combines the effects of various modes of distress. This process of interactive causes and effects, resulting ultimately in roughness, is a key concept in the approach to modelling which is adopted here. Roughness cannot be considered in isolation from these other causes, as is evidenced by the correlation between different distress types (noted with regard to serviceability earlier, for example).

Maintenance has two effects, an immediate impact on the condition of the pavement, and an impact on the future rate of deterioration of the pavement. Usually maintenance is intended to improve the condition and performance, but certain forms, such as patching, may initially increase the roughness.

4.1.4 Maintenance

Maintenance activities for paved roads are classified according to their frequency and their impact on the standards of the road, as defined in Table 4.2.
Figure 4.3: The mechanisms and interactions of distress in paved roads

- Cracking
- Water ingress
- Lower shear strength and stiffness
- Spalling
- Ravelling
- Uneven deformation
- Variability of properties and behavior
- Potholes
- (Patching)
- Roughness
- (Deep Patching)
- Rut depth
- Age
- Faster deformation
- Variability of shear and volume changes
Table 4.2: Classification of road maintenance and improvement works for paved roads

1. **Routine Maintenance:** Localized repairs (typically less than 150 m in continuous length) of pavement and shoulder defects, and regular maintenance of road drainage, side slopes, verges and furniture. (Examples: pothole patching, reshaping side drains, repairing and cleaning culverts and drains, vegetation control, dust control, erosion control, snow and sand removal from travelled way, repainting pavement strips and markings, repairing or replacing traffic signs, guardrails, signals, lighting standards, etc., roadside cleaning and maintenance of rest areas).

2. **Resurfacing:** Full-width resurfacing or treatment of the existing pavement or roadway (inclusive of minor shape correction, surface patching or restoration of skid resistance) to maintain surface characteristics and structural integrity for continued serviceability. (Examples: slurry seals, fog seals, or enrichment treatments; surface treatments (chip seal); friction courses; thin asphalt surfacings typically 25 mm or less in thickness. The terms "preventive maintenance" and "periodic maintenance" had approximately synonymous meaning in previous usage.

3. **Rehabilitation:** Full-width, full-length surfacing with selective strengthening and shape correction of existing pavement or roadway (inclusive of repair of minor drainage structures) to restore the structural strength and integrity required for continued serviceability. (Examples: asphalt concrete overlays, selective deep patching and overlays, granular overlay and surfacing, surface treatment with major shape correction, recycling of one or more pavement layers. The term "strengthening" is sometimes used for a particular category of rehabilitation works.

4. **Betterment (or Improvement):** Geometric improvements related to width, curvature or gradient of roadway, pavement, shoulders, or structures, to enhance traffic capacity, speed or safety; and inclusive of associated "rehabilitation" or "resurfacing" of the pavement.

5. **Reconstruction:** Full-width, full-length reconstruction of roadway pavement and shoulders mostly on existing alignment, including rehabilitation of all drainage structures generally to improved roadway, pavement and geometric standards.

6. **New Construction:** Full-width, full-length construction of a road on a new alignment, upgrading of a gravel or earth road to paved standard, and provision of additional lanes or carriageways to existing roads.

**Source:** Infrastructure Department, World Bank.
A wide variety of different classifications have evolved in various countries and
highway agencies, some based on fiscal criteria (recurrent and capital expendi-
ture), some on phases of the pavement's life cycle (routine, preventive, correc-
tive, rehabilitation), and some on both frequency and fiscal criteria (periodic
maintenance and strengthening), etc. The classification in Table 4.2 represents
an important initiative by the World Bank to bring clarity and consistency into
the terminology, particularly into the various categories of maintenance and
rehabilitative activities which are frequently combined in current projects. It
has been developed to be compatible with various distinctions between capital and
non-capital expenditures, and to encourage a more flexible interpretation of
financing criteria.

The routine maintenance and resurfacing categories represent maintenance
functions, the one financed through annual expenditures and the latter through a
planned program. Resurfacing and rehabilitation works are undertaken through
planned programs, and are an integral part of maintaining the physical service-
ability of a road network to meet current needs, and of preserving the infrastruc-
tural investment. Betterment, and reconstruction, like new construction, enhance
the functional and physical standards of the existing network and thus clearly add
to the capital stock; these works are typically part of long-range development
programs.

4.1.5 Life Cycle of Deterioration and Maintenance

A typical life cycle of a pavement is shown in Figure 4.4 depicting
pavement condition in (a) and (b) and the costs in (c) and (d). For the purposes
of the example and clarity in the condition diagrams, all aspects of cracking,
ravelling and potholes have been aggregated under surfacing distress, and rut
depth has been omitted.

During the early phase of the pavement's life, prior to the occurrence
of surface distress, the only changes in condition are slight increases in rut
depth and roughness, there are no road costs except the small annual routine main-
tenance cost, and there are only slight changes in the average vehicle operating
cost due to the small increase in roughness.

Following the initiation of surfacing distress, the roughness and the
associated vehicle operating costs increase more rapidly. Patching maintenance
reduces the roughness and costs slightly, but not back to the levels that would
have applied in the absence of surfacing distress because the patching itself is a
defect, deviating in profile from the perfect planar surface.

A thin resurfacing or reseal has the immediate effects of reducing the
area of surfacing distress to nil, and reducing the roughness by an amount depen-
dent upon the thickness and technique applied. A surface treatment reseal causes
only a minor or negligible reduction in roughness (except when preliminary shape
correction by spot patching has been done). A thin overlay causes an appreciably
greater reduction in roughness, primarily in the short wavelength band. Neither
of these thin resurfacing options add appreciably to the stiffness of the pave-
ment, and thus their impacts on future performance are the benefits derived pri-
marily from the repair and control of surfacing distress, and the initial rate of
deterioration is likely to be similar to, or slightly less than, that for the
equivalent maintenance-free pavement.
Figure 4.4: Time-profiles of pavement condition, maintenance and costs over life-cycle of a paved road

(a) Surfacing Distress

(b) Road Roughness

(c) Road Costs

(d) Vehicle Operating Costs

Source: Author.
Rehabilitation works such as a thick overlay (whether asphaltic or granular with bituminous surfacing) have the immediate effects of reducing the roughness to approximately the level of new pavements and the surfacing distress to nil, and the future effect of reducing the rate of deterioration through strengthening the pavement.

The associated cost-streams, shown in (c) and (d) of the figure, when combined with the volume and growth of traffic and discounted to present worth, provide the basis for determining the economic benefits and returns of a given maintenance strategy. Typically, the total vehicle operating costs outweigh the road costs by large factors in the order of ten to twenty so that small savings yield high returns.

4.2 EMPIRICAL STUDIES OF PERFORMANCE

The central importance of empirical studies of pavement deterioration for quantifying the long-term environmental, mixed traffic, and stochastic variation effects, and also for calibrating theoretical analyses to field conditions, was a theme of the opening chapter. Tremendous advances have been made in the past three decades on the mechanistic approach, including the theoretical analysis of the reaction of layered pavement structures under load, the identification of the fundamental properties of materials, and the experimental derivation of relationships for the behavior of the materials and pavement under load (documented, for example, in the proceedings of the international conferences on asphalt pavements). However, validation of the mechanistic models must be made against empirical studies of long-term pavement performance. The major empirical studies, which cover wide ranges of operating conditions, are therefore valuable not only for the empirical relationships derived directly from them but also as independent sources for testing the validity of predictive relationships.

4.2.1 Study Methods

The long duration of the life cycle of paved roads, with surface distress occurring after eight to twelve years and roughness reaching limiting levels after fifteen to forty years, necessitates special attention to study techniques. The variety of techniques that have been employed, summarized in Table 4.3, fall into two categories, namely those that accelerate the rate of deterioration so as to observe a full deterioration cycle, and those that observe portions of the deterioration cycle under in-service conditions.

Accelerated deterioration

In the first category, the acceleration of deterioration is achieved either by underdesigning the pavements with respect to the traffic to be carried (e.g., the AASHO Road Test, in which a variety of design lives provided a variety of rates of deterioration, but nearly all of them accelerated), or by applying supra-normal loadings of 40 to 100 kN per wheel unit (as in the case of the Heavy Vehicle Simulator (HVS), Accelerated Loading Facility (ALF) and circular test tracks). The two problems with this approach which make it unsuitable as a basis for predictive models are, first, that long-term effects are virtually eliminated (these are primarily environmental but also include effects of rest periods or vehicle headway), and second, the unrepresentative loading regimes can distort the behavior of the pavement materials, which are often stress-dependent.
Table 4.3: Techniques for empirical study of pavement deterioration

<table>
<thead>
<tr>
<th>Techniques</th>
<th>Traffic</th>
<th>Environment</th>
<th>Pavements</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Accelerated Deterioration</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Accelerate deterioration</td>
<td>Regular transits of selected vehicles, normal load</td>
<td>Single locale, short term</td>
<td>Specially constructed, underdesigned</td>
<td>AASHD1/</td>
</tr>
<tr>
<td>in 0.5 to 2 year period</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Accelerate deterioration</td>
<td>Simulation by regular transits of one wheel with supra-normal load</td>
<td>Local and simulated in-service, conditions</td>
<td>Selected</td>
<td>HVS2/, ALK3/, TRRL track4/, LCPC track5/</td>
</tr>
<tr>
<td>in 0.2 to 0.5 year period</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Deterioration In-service</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Monitor lifetime (20 years)</td>
<td>Mixed, growing, uncontrolled, monitored</td>
<td>Matrix of locales, long-term</td>
<td>Specially-construted</td>
<td>United Kingdom7/, Ordway, Colorado8/, Brampton Road9/</td>
</tr>
<tr>
<td>of full-strength pavements under service conditions</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Monitor 3 to 5-year</td>
<td>Mixed, growing, uncontrolled, measured</td>
<td>Matrix of locales, long-term</td>
<td>In-service standards; factorial of types, strengths, ages.</td>
<td>Kenya10/</td>
</tr>
<tr>
<td>&quot;window&quot; of lifetime of experimentally-selected in-service pavements</td>
<td></td>
<td></td>
<td></td>
<td>Brazil-UNDE11/</td>
</tr>
<tr>
<td>Monitor 3 to 10-year</td>
<td>Mixed, growing, uncontrolled, monitored</td>
<td>Matrix of locales, long-term</td>
<td>Representative sample of network</td>
<td>Arizona12/</td>
</tr>
<tr>
<td>&quot;window&quot; of life cycle of in-service pavements representative of network</td>
<td></td>
<td></td>
<td></td>
<td>Texas13/</td>
</tr>
<tr>
<td>Spot observation (one-year)</td>
<td>Mixed, growing, uncontrolled, historical records</td>
<td>Matrix of locales, long-term</td>
<td>Representative sample of network</td>
<td>Tunisia14/</td>
</tr>
<tr>
<td>of selected sections of in-service pavements with back-analysis of life cycle.</td>
<td></td>
<td></td>
<td></td>
<td>India15/</td>
</tr>
<tr>
<td>Source: Author.</td>
<td></td>
<td></td>
<td></td>
<td>United States16/</td>
</tr>
</tbody>
</table>

2/ Walker and others (1977).  
5/ Autret and others (1987).  
7/ Powell and others (1984).  
9/ Haas and others (1972).  
10/ Hodges and others (1975).  
12/ Way and Eisenberg (1980).  
13/ Lytton and others (1982).  
14/ SETEC in Paterson (1985 a).  
15/ CRRI (1986).  
16/ Nunez and Shahin (1986).
Deterioration in-service

The monitoring of deterioration of pavements in-service is a lengthy process, limited by the problems and errors of measuring small changes of condition and by the range of pavement designs and standards available in the network. Full lifetime monitoring has served as a useful basis for pavement design methods in the United Kingdom (Lister and Powell 1987) but only when supported by mechanistic analysis to interpret and extrapolate the results to other materials and loading conditions. A spot observation of condition and back-analysis of performance for a sample of pavements of different ages can be useful for a coarse validation or calibration of predictive models, provided that reasonable deductions can be made on the original condition and the traffic and maintenance history; it is generally not adequate for developing predictive models of distress, but has been applied to predict a summary performance index (e.g., CRRI 1986, O'Brien and others 1983, Nunez and Shahin 1986).

For in-service deterioration, the greatest utility is realized by monitoring a sample of pavements during a medium period in the order of five years, which provides a "window" or "snapshot" of part of the lifecycle of those pavements. By including a range of pavement ages, types and strengths, of traffic, and of climate, selected according to an experimentally-designed factorial which will permit a sound statistical analysis of the primary factors, it is possible to achieve reliable models of the whole life-cycle from a manageable sample of pavements in a comparatively short period of time. The approach thereby includes the fullscale, longterm effects of environment, age and mixed traffic in realistic loading regimes, and is constrained only by the logistics involved for the number of pavements in the sample, which grows rapidly as the number of factors in the factorial is increased.

The "window monitoring" technique was first applied in Kenya (Hodges and others 1975) to develop the empirical predictive models that formed the basis of the British Road Transport Investment Model (RTIM) (Robinson and others 1975), and subsequently of Release II of the Highway Design and Maintenance Model (HDM) (World Bank 1981). The study comprised 49 sections studied over a period of four years with a partial factorial of 2 (rainfall) x 3 (pavement type) x 3 (gradient) x 3 (curvature). This was followed by a similar but larger study in Brazil (GEIPOT 1982) which forms the primary basis of version III of the HDM model (Watanatada and others 1987b). That study comprised 116 sections studied over a period of three to five years in a partial factorial of 1 (climate) x 5 (pavement types) x 2 (rehabilitation states) x 2 (traffic levels) x 2 (gradients) x 2 (ages). Other similar studies include Arizona (Way and Eisenberg 1980) in which 51 pavements were monitored regularly over a period of ten years, and Texas (Lytton and others 1982) where 337 pavements were monitored intermittently over a period of seven years; each of these provided the basis for local, statewide predictive models for use in pavement management. Finally, it is noteworthy that this technique has been selected for the major five- to ten-year Long Term pavement Performance study of the American Strategic Highway Research Program (SHRP); this study, commencing in 1987, will be considerably larger than the previous ones with a total of 1,560 sections for general pavement studies related to performance modelling, and 1,630 sections of special pavement studies of individual factors, all selected according to a tiered-factorial design (Transportation Research Board 1986).
The success of using the "window monitoring" technique for developing predictive models is critically dependent on:

1. The quality and accuracy of the data;
2. The number and relevance of parameters measured;
3. The distribution and range of values of the major factors across the factorial matrix; and
4. sufficient replicates within the factorial to provide a basis for quantifying the stochastic variability which is inherent in the properties and behavior of materials.

Based on these and other considerations, the data from the Brazil-UNDP-World Bank study (GEIPOT 1982) were used as the primary empirical base for the development of predictive models for paved road distress. The Kenyan, Arizonan, Texan, and AASHO data sets were used in two ways, namely:

1. To quantify environmental and other effects across climatic regions, and
2. To serve as independent data sources for assessing the validity of the predictive models developed from the Brazilian data.

4.2.2 Selected Empirical Studies

The scopes of the Brazilian study and the four other empirical studies just mentioned are summarized in Table 4.4. Although the Texas and Illinois (AASHO road test) studies are largest by the number of sections studied, the Brazilian study is clearly the most comprehensive in terms of scope, numbers of sections, and frequency of observation, for nonfreezing climates.

AASHO road test

The AASHO road test was an accelerated, controlled-trafficking experiment using typical road vehicles on specially-constructed pavements over a period of two years (1958 to 1960). The primary objective was to determine the relationships between the numbers of axle transits of different loadings and the performance of flexible and rigid pavements. A total of 368 flexible pavements were included, located on six independent test loops of which 5 were trafficked and one was not trafficked, as a measure of climatic effects. Different axle loadings and configurations were applied on separate lanes on the loops, ranging from 9 kN on single-tire single axles to 213 kN on dual-tire tandem axles, reaching a total of 1,114,000 axle applications on each section over two years at a rate of approximately one vehicle per minute.

Pavement condition was quantified in terms of slope variance, rut depth, cracking area and patching area, and summarized in the "present serviceability index" (PSI), as defined in Equation 4.1. The PSI was an estimate of the serviceability rating (PSR), being the subjective evaluation by a panel of the ride, condition and need for maintenance of the pavement. An equivalent thickness index of pavement strength, the structural number (SN), was developed, combining the products of material strength coefficient and layer thickness for each layer in a
Table 4.4: Scope of some empirical studies on the deterioration and maintenance of flexible pavements

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Kenya</th>
<th>Brazil</th>
<th>Arizona</th>
<th>Texas</th>
<th>Illinois</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of sections</td>
<td>49</td>
<td>116</td>
<td>51</td>
<td>337</td>
<td>384</td>
</tr>
<tr>
<td>granular base</td>
<td>10</td>
<td>74</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>asphalt base</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>cemented base</td>
<td>39</td>
<td>11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>overlaid</td>
<td>0</td>
<td>33</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length of section (m)</td>
<td>1,000</td>
<td>720</td>
<td>1,600</td>
<td>3,200</td>
<td>70</td>
</tr>
<tr>
<td>Modified structural number</td>
<td>2.5-5.1</td>
<td>1.5-7.0</td>
<td>0.8-8.7</td>
<td>0.9-10</td>
<td>2.9-6.9</td>
</tr>
<tr>
<td>Period of study (yrs)</td>
<td>4</td>
<td>3-5</td>
<td>5-9</td>
<td>7</td>
<td>2</td>
</tr>
<tr>
<td>Weeks between observations</td>
<td>25-60</td>
<td>12</td>
<td>50</td>
<td>100</td>
<td>2</td>
</tr>
<tr>
<td>Traffic(^1)/veh/day</td>
<td>mixed</td>
<td>mixed</td>
<td>mixed</td>
<td>mixed</td>
<td>selective</td>
</tr>
<tr>
<td>MESA/lane/year</td>
<td>323-1,618</td>
<td>73-5,600</td>
<td></td>
<td></td>
<td>1,400</td>
</tr>
<tr>
<td>ESA/heavy vehicle</td>
<td>0.012-3.6</td>
<td>0.003-1.20</td>
<td>0.0002-0.27</td>
<td>0.00-1.2</td>
<td>0.00-4.8</td>
</tr>
<tr>
<td>MESA total</td>
<td>0.004-3.3</td>
<td>0.003-18</td>
<td>0.04-4.5</td>
<td>0.003-9.4</td>
<td>0.00-9.9</td>
</tr>
<tr>
<td>Climate(^2)/moisture index(^3)/precipitation (mm/yr)</td>
<td>dry NF</td>
<td>wet NF</td>
<td>dry F, NF</td>
<td>all</td>
<td>wet F</td>
</tr>
<tr>
<td></td>
<td>-80, 0</td>
<td>12, 100</td>
<td>-80, -20</td>
<td>-70, 30</td>
<td>40</td>
</tr>
<tr>
<td>Age (year)</td>
<td>0-14</td>
<td>0-24</td>
<td>1-34</td>
<td>2.2-49</td>
<td>0-2.1</td>
</tr>
<tr>
<td>Roughness (^4)/absolute (IRI)</td>
<td>2.9-6.0</td>
<td>1.8-10.2</td>
<td>2.3-4.0</td>
<td>0.1-12.0</td>
<td>0.3-16</td>
</tr>
<tr>
<td>increment (IRI)</td>
<td>0.3-1.7</td>
<td>0-4.9</td>
<td>-0.3-2.2</td>
<td>-1.0-1.6</td>
<td>-4.5-10.4</td>
</tr>
</tbody>
</table>

\(^1\)/ESA - equivalent 80 kN single axle load; MESA = million ESA.
\(^2\)/NF = nonfreezing, F = freezing; all: combinations of wet or dry, F or NF.
\(^3\)/After Thornthwaite (1955), ranging from -100 (arid) to 100 (perhumid).
\(^4\)/Roughness conversions based on Figure 2.15, and 1 m/km IRI = 86 inches/mile ride for Arizona.

Sources: Kenya (Hodges and others 1975); Brazil (GEIPOT 1982 and World Bank); Arizona (Way and Eisenberg 1980); Texas (Texas Transportation Institute 1986); Illinois (AASHTO road test data, by courtesy of Asphalt Institute).
linear function such that pavements of similar SN would give similar performance in terms of PSI. Later development added a component for subgrade support to SN. The predictive models developed for the trend of serviceability as a function of equivalent axle loads and pavement structural number are listed in Appendix A.

The applicability of the AASHO road test results to roads in developing countries is severely limited by several factors. First, the freezing environment of the test, which had a major influence on deterioration, is distinctly different from the tropical and subtropical climates of most developing countries. Second, the range of pavement types, which was limited primarily to thick asphalt concrete and rigid surfacings on one weak subgrade, was not representative of the thin surfacings (predominantly surface treatments) and range of material and subgrade types (particularly tropical soils) which are common in developing countries. Third, it is uncertain how applicable the relationships -- based on accelerated, experimentally controlled loading -- are to roads with mixed light and heavy traffic, or to roads with low traffic volumes. Fourth, in order that different maintenance actions, intervention criteria and standards could be evaluated, it is desirable to predict the trends of roughness, rut depth and cracking separately rather than in a composite index such as the serviceability index, as argued in Section 4.1. Fifth, the effects of alternative maintenance policies on deterioration were not considered in the AASHO test.

Kenya and Brazil

The road deterioration studies that were conducted in Kenya, 1971 to 1974, and in Brazil, 1976 to 1982, were designed therefore to collect data on the changes of roughness, cracking and rut depth of flexible and semirigid pavements in nonfreezing climates, over a wide range of pavement strengths and mixed traffic loadings, and under different maintenance standards. The three- to five-year study periods were the minimum periods necessary to achieve adequate resolution of the trends of condition and the development of empirical distress models, when using a combination of cross-section and time-series methods as discussed below.

The pavement sections included in the Kenya and Brazil studies were selected by experimental design, making up a partial factorial of major variables. The design of the Kenya study, given in Table 4.5, used pavement type, vertical and horizontal geometry, and rainfall as factorial parameters. The central design of the Brazil study, given in Table 4.6(a), excluded horizontal geometry (all were tangent sections) and climate as factorial parameters, but added traffic flow and had more levels of pavement age and pavement type (with a breakdown by surfacing, base and rehabilitation status). The central sampling matrix was supplemented by a "star point" matrix, Table 4.6(b), which added sections having intermediate values of the main parameters, making a total of 74 sections. A further 42 special test sections were defined for studies of maintenance effects.

In the Kenya study, most of the pavements studied were of cement-stabilized base construction and covered a rather narrow range of pavement strengths of 2.7 to 3.7 modified structural number. The volumes and loading of traffic, however, were sufficiently high on eight of the sections that almost complete deterioration histories were obtained. The study on crushed-stone base pavements was hampered by the elimination of seven sections due to drainage difficulties.

2/ The definition of modified structural number is given in Section 4.3.
Table 4.5: Sampling factorial for paved roads in Kenya study

<table>
<thead>
<tr>
<th>Geometric Classification</th>
<th>Low rainfall &lt; 1000 mm/year</th>
<th>High rainfall &gt; 1000 mm/year</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Flat (&lt;1.5%)</td>
<td>Flat (&lt;1.5%)</td>
</tr>
<tr>
<td></td>
<td>Intermediate (1.5% - 3.5%)</td>
<td>Steep (≥3.5%)</td>
</tr>
<tr>
<td></td>
<td>Steep (≥3.5%)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Flat (&lt;1.5%)</td>
<td>Flat (&lt;1.5%)</td>
</tr>
<tr>
<td></td>
<td>Intermediate (1.5% - 3.5%)</td>
<td>Steep (≥3.5%)</td>
</tr>
<tr>
<td></td>
<td>Steep (≥3.5%)</td>
<td></td>
</tr>
<tr>
<td>Test surface - dressed roads (P)</td>
<td>Low (&lt;30°/km)</td>
<td>P8</td>
</tr>
<tr>
<td>Test surface - dressed roads (P)</td>
<td>Medium (≥30°/km) (≤90°/km)</td>
<td>P10</td>
</tr>
<tr>
<td>Test surface - dressed roads (P)</td>
<td>High (≥90°/km)</td>
<td>–</td>
</tr>
<tr>
<td>New surface - dressed roads (NB)</td>
<td>Low</td>
<td>NB10</td>
</tr>
<tr>
<td>New surface - dressed roads (NB)</td>
<td>Medium</td>
<td>–</td>
</tr>
<tr>
<td>Old surface - dressed roads (OB)</td>
<td>Low</td>
<td>OB17*</td>
</tr>
<tr>
<td>Old surface - dressed roads (OB)</td>
<td>Medium</td>
<td>OB23*</td>
</tr>
<tr>
<td>Old surface - dressed roads (OB)</td>
<td>High</td>
<td>–</td>
</tr>
</tbody>
</table>

* Nil maintenance sections  
** Surface-dressed in error

Source: Hodges, Rolt and Jones (1975).

While the deterioration rates that were observed agreed reasonably well with current pavement design criteria, the data base was very narrow and the resulting relationships did not extrapolate well beyond that range, particularly for thin pavements.

In the Brazilian study of paved roads, the number of sections and pavement types was more than double that of Kenya, and the range of pavement strength covered virtually the whole range currently used in most developing countries. The pavement construction generally comprised thin surfacings (less than 100 mm in thickness), and a basecourse of natural gravel, crushed stone or cemented gravel materials; not included in the study were thick bituminous pavements, inverted-design pavements with granular base and cemented subbase (which are often used for very heavily trafficked pavements), or layers of water bound macadam or bituminous penetration macadam. The ranges of pavement age, roughness and observed roughness change were also double those of the Kenya study, but the maximum traffic loading (1.2 million ESA/lane/year at the beginning of the study) and axle loadings per vehicle were slightly lower.

The climates of the two study regions are more different than indicated by the annual rainfall. The climate of the central plateau of Brazil where the study was conducted is classed as moist subhumid to perhumid (12 to 100 Thornthwaite moisture index) (See Figure 3.5) and Kenya's is classed as arid to semiarid (-80 to 0 moisture index). Neither the Kenya nor Brazil studies encompassed either very low (less than 400 mm) or very high (more than 2,000 mm) annual rainfall. Horizontal curvature was varied in Kenya but not in Brazil. Vertical gradient ranged from 0 to 8 percent in both studies. Pavement width was not
Table 4.6: Sampling factorials for paved roads in Brazil-UNDP road costs study

(a) Primary sampling matrix

<table>
<thead>
<tr>
<th>Surface Type</th>
<th>Base Type</th>
<th>Asphaltic Concrete</th>
<th>Double Surface Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic (ADT)</td>
<td>Vertical Geometry (%)</td>
<td>Gravel</td>
<td>Crushed Stone</td>
</tr>
<tr>
<td>Age (Years)</td>
<td>State Remar</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overlaid</td>
<td>&gt; 6</td>
<td>126</td>
<td>129</td>
</tr>
<tr>
<td></td>
<td>0-1.5%</td>
<td>008</td>
<td>035</td>
</tr>
<tr>
<td></td>
<td>0-2</td>
<td>118</td>
<td>123</td>
</tr>
<tr>
<td></td>
<td>&gt;12</td>
<td>003</td>
<td>005</td>
</tr>
<tr>
<td></td>
<td>0-1.5%</td>
<td>022</td>
<td>028</td>
</tr>
<tr>
<td></td>
<td>0-4</td>
<td>001</td>
<td>034</td>
</tr>
</tbody>
</table>

Note: The numbers in each cell are the section numbers.

(b) Star point sampling matrix

<table>
<thead>
<tr>
<th>Levels of traffic (TR), vertical geometry (VG), and age (AGE)</th>
<th>Asphalt concrete</th>
<th>Surface treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Gravel base</td>
<td>Crushed stone base</td>
</tr>
<tr>
<td>TR V G AGE</td>
<td>Co</td>
<td>Ov</td>
</tr>
<tr>
<td>* * 1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>* 1 *</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>1 * *</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>* * *</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>* * 2</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>* 2 *</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>2 * *</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

Notes: Parameter Level: 1 | * | 2 |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>TR: ADT (veh/day)</td>
<td>&lt;500</td>
<td>600-900</td>
</tr>
<tr>
<td>VG: Grade (%)</td>
<td>0-1.5</td>
<td>3-5</td>
</tr>
<tr>
<td>AGE: Age (yr)</td>
<td>Co: &lt;4</td>
<td>8-10</td>
</tr>
<tr>
<td></td>
<td>Ov: &lt;2</td>
<td>3-5</td>
</tr>
</tbody>
</table>

Co = As constructed; Ov = overlaid. Cell numbers indicate the number of road sections.

Source: After Volume 2 in GEIPOT (1982).
varied in Brazil (constant 7.0 m), and varied only slightly in Kenya (6.0 to 7.5 m). All the sections were located on existing roads in-service with generally adequate engineering design standards and adequate drainage (with the exception of seven sections eliminated from the Kenya study).

The empirical distress relationships derived from the Kenyan data for the RTIM2 model (Parsley and Robinson 1982), and from the Brazilian data in the course of the Brazil-UNDP study (Queiroz 1981, and Volume 7 in GEIPOT 1982), are summarized in Appendix A.

Arizona

The Arizonan data base represents a time-series of data on fifty one sections, each 1.6 km long, gathered over a period of up to 9 years, and comprising roughness, cracking and rut depth condition data, pavement structural number and traffic loading. These data served as a validation base for predictive models developed for a pavement management system from an earlier cross-sectional short-term study of one year's duration on 430 km road length (Finn, Kulkarni and McMorran 1976). The roughness data were controlled, but not rigorously calibrated against a profile as in the Brazil study; the conversion used (86 inches/mile = 1 m/km IRI) was based on a subsequent profile calibration in 1984. Recursive distress models, developed by linear regression, are presented in Appendix A.

Texas

The data were collected intermittently over a period of seven years from 1972 to 1980, with condition data collected in accordance with a rating method and quantified by a score (Texas Transportation Institute 1974). Conversions were based on Lytton and others (1982) and discussions with the Institute. Maintenance activities were included, but the roughness data are difficult to interpret because large fluctuations are often not explained by the maintenance records. Sigmoidal damage models developed by Lytton and others (1982) are presented in Appendix A.

4.3: DETERIORATION MODELLING CONCEPTS

4.3.1 Research Methodology

The weakness in the empirical approach of using statistical correlations to establish the relationship between distress, as a dependent variable, and various pavement, traffic and environmental factors as explanatory variables, is that the result may represent only a "fingerprint" of the local situation and not necessarily identify the true underlying relationship between the variables. For example, the average rate of rut depth progression in mm per year may be determined from a set of data, but without pavement, traffic and climatic variables to explain variations in the rate of rutting, the relationship is valid only for the local sample observed and is not transferable to different situations. For example, Potter (1982) and Way and Eisenberg (1980) developed models for roughness progression that were functions of age, and age and climate, respectively, being unable to identify pavement strength and traffic effects within the data available, even though the roughness-strength-traffic relationship is the primary basis of the AASHTO (1981) and other major pavement design methods.
These problems can be avoided, or at least minimized, if the primary variables and the functional form are identified either independently from an external source or by the use of statistical techniques such as cross-sectional analysis.

In this study, therefore, the mechanistic concepts of material properties and behavior under traffic loading and climatic factors, and the results of experimental research, are used in formulating the model forms and in selecting the analytical methods. In this way, the strengths of both the mechanistic (theoretical and experimental) and empirical approaches can be combined. The approach has parallels in the recent British pavement design method (Lister and Powell 1987) in which mechanistic theory has been used to interpret thirty years' worth of empirical data, and in the calibration of a theoretical method through the empirical data of the AASHO road test and other sources by the Asphalt Institute (Shook and others 1982), to take just two examples.

4.3.2 Model Form

In life-cycle predictions, pavement management systems, or in the pricing of road use, the models need to predict the expected change of condition in the future over a given period of time or under the transit of one extra axle load, when the current pavement condition is known. Thus the models should be essentially incremental and recursive in form, for example:

\[
\text{Change in condition} = f \left( \text{current condition, pavement strength and age characteristics, environment, incremental time, and incremental traffic} \right). \quad (4.2)
\]

Models which predict absolute levels of distress are of limited use because they are typical only of the average construction technique and quality particular to the study area. When suitably structured, such models can be converted to a derivative form by differentiation, but the usefulness of the results is extremely dependent on the suitability of the original functional form.

The concept of incremental distress is depicted in Figure 4.5. For a level of distress of a given type, D, at time \( t_1 \), we wish to know what the level of distress \( D_2 \) will be at time \( t_2 \). The change in distress \( \Delta D \) is usually conveniently expressed on a time base, \( \Delta t \), since this is the most convenient dimension for road management and planning models. In some cases, however, such as in the pricing of road use and damage, or in the modelling of fatigue cracking, it is preferable to express the "time" dimension in terms of traffic, for example by the number of cumulative axle transits (N).

4.3.3 Dimensioning Pavement Condition

Two distinct approaches have been taken for dimensioning pavement condition for predictive models.

Damage functions for performance prediction

The first approach is to normalize the pavement condition to a dimensionless state so that, for example, a pavement in its "new" or initial, state has a damage value of zero, and in its "terminal" state has a damage value of one. Such predictive models are termed damage functions.
Source: Author.

The attraction of a normalized damage function is that it expresses the fraction of terminal damage, and so relates readily to an interpretation of the "consumed life" or "remaining life" of a pavement. In the example of the AASHTO functions, the damage $g$ defined the fractional change of serviceability index as follows:

$$ g = \frac{p_1 - p}{p_1 - p_U} \tag{4.3} $$

where $p$ = the present serviceability index; and $p_1$ = the initial serviceability of the original pavement; and $p_U$ = the terminal serviceability of the pavement.

The trend of the damage function therefore expresses the performance of the pavement in relation to two standards, the quality of original construction or initial condition ($p_1$) and the "terminal" level of distress at which maintenance or rehabilitation is deemed necessary ($p_U$). The base of the function therefore changes as one or other of these two standards alters.

The predictive models of AASHTO, Lytton and others (1982) and Rauhut and others (1984) all take this form of dimensionless damage functions.

**Distress functions and deterioration prediction models**

The dependence of damage functions on prescribed standards is a drawback for some objectives of economic evaluation. The advantages of incremental or
derivative type models that do not require knowledge of the original condition were noted earlier. Further, when we wish to compare maintenance policies involving different intervention criteria, and to quantify the economic benefits through the causal relationship of absolute roughness and vehicle operating costs, dimensionless damage functions are no longer useful.

The alternative is to estimate distress functions which have dimensions that are characteristic of the distress type, and to express maintenance intervention criteria in similar dimensions. Thus distress models estimate roughness in slope dimensions, rut depth in depth dimensions and cracking in length per unit area and percentage area dimensions. The predictive models of the studies in Kenya, Brazil and Arizona are all of this type.

As the versatility and flexibility provided by utilizing physical measures of distress were considered essential for a model evaluating the economic standards of maintenance and design, the distress-function approach was adopted for this study.

4.3.4 Pavement Strength Parameters

One major difficulty in modelling pavement behavior is the representation of pavement strength. Because the pavement is a semi-infinite continuum comprising layers of materials with often greatly differing properties and behavior under load, and because light loads have a shallower depth of influence than heavy loads, some uniform basis is required for representing pavement strength in predictive models.

There are three categories of structural parameters for pavements, namely:

1. Equivalent thickness parameters: these sum the thicknesses of the pavement layers weighted by material-layer strength coefficients. They are predicated on the principle of load-spreading or stress-distribution, and are related to performance through either rut depth or roughness trends under traffic loading. Most prominent is the AASHO structural number, modified to account for subgrade strength and here defined as follows:

\[
SNC = 0.04 \sum a_i h_i + SN_{sg} \tag{4.4}
\]

where

- \( SNC \) = modified structural number;
- \( a_i \) = material and layer strength coefficients;
- \( h_i \) = layer thickness, mm (where \( \sum h_i \leq 700 \) mm);
- \( SN_{sg} \) = subgrade contribution after Hodges and others (1975):
  \( = 3.51 \log_{10} CBR - 0.85 (\log_{10} CBR)^2 - 1.43 \);
- \( CBR \) = in situ California Bearing Ratio of subgrade, %;

and values for the coefficients \( a_i \) are defined in Table 4.7 and illustrated in Figure 4.6. Note that special provision is made here for the contribution of the basecourse, ranging from negligible \( a_2 = 0 \) when the material is very soft or saturated, to high \( a_2 > 0.14 \) when the material is supported by a rigid subbase and thus acts under compression, based on the empirical data.
Table 4.7: Pavement layer strength coefficients for structural number

<table>
<thead>
<tr>
<th>Pavement layer</th>
<th>Strength coefficient $a_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Surface course</strong></td>
<td></td>
</tr>
<tr>
<td>Surface treatments</td>
<td>0.20 to 0.40</td>
</tr>
<tr>
<td>Asphalt mixtures (cold or hot premix of low stability)</td>
<td>0.20</td>
</tr>
<tr>
<td>Asphalt concrete (hot premix of high stability) $^1$</td>
<td></td>
</tr>
<tr>
<td>$MR_{so} = 1,500$ MPa</td>
<td>0.30</td>
</tr>
<tr>
<td>$MR_{so} = 2,500$ MPa</td>
<td>0.40</td>
</tr>
<tr>
<td>$MR_{so} = 4,000$ MPa or greater</td>
<td>0.45</td>
</tr>
<tr>
<td><strong>Base course</strong></td>
<td></td>
</tr>
<tr>
<td>Granular materials $^2$</td>
<td>For maximum axle loading:</td>
</tr>
<tr>
<td></td>
<td>$&lt; 80$ kN $&gt; 80$ kN</td>
</tr>
<tr>
<td>CBR = 30% $^3$</td>
<td>0.07</td>
</tr>
<tr>
<td>CBR = 50%</td>
<td>0.10</td>
</tr>
<tr>
<td>CBR = 70%</td>
<td>0.12</td>
</tr>
<tr>
<td>CBR = 90%</td>
<td>0.13</td>
</tr>
<tr>
<td>CBR = 110%</td>
<td>0.14</td>
</tr>
<tr>
<td>Cemented materials $^4$</td>
<td></td>
</tr>
<tr>
<td>UCS = 0.7 MPa</td>
<td>0.10</td>
</tr>
<tr>
<td>UCS = 2.0 MPa</td>
<td>0.15</td>
</tr>
<tr>
<td>UCS = 3.5 MPa</td>
<td>0.20</td>
</tr>
<tr>
<td>UCS = 5.0 MPa</td>
<td>0.24</td>
</tr>
<tr>
<td>Bituminous materials $^5$</td>
<td></td>
</tr>
<tr>
<td>Subbase and selected subgrade layers</td>
<td></td>
</tr>
<tr>
<td>(to total pavement depth of 700 mm)</td>
<td></td>
</tr>
<tr>
<td>Granular materials $^6$</td>
<td></td>
</tr>
<tr>
<td>CBR = 5%</td>
<td>0.06</td>
</tr>
<tr>
<td>CBR = 15%</td>
<td>0.09</td>
</tr>
<tr>
<td>CBR = 25%</td>
<td>0.10</td>
</tr>
<tr>
<td>CBR = 50%</td>
<td>0.12</td>
</tr>
<tr>
<td>CBR = 100%</td>
<td>0.14</td>
</tr>
<tr>
<td>Cemented materials</td>
<td></td>
</tr>
<tr>
<td>UCS &gt; 0.7 MPa</td>
<td>0.14</td>
</tr>
</tbody>
</table>

1/ Applicable only when thickness $> 30$ mm. $MR_{so}$ = resilient modulus by indirect tensile test at 30°C.
2/ $a_i = (29.14 \, CBR - 0.1977 \, CBR^2 + 0.00045 \, CBR^3) \times 10^{-4}$; the coefficient $a_i$ may be increased by 60 percent if CBR $> 70$ and the subbase is cement- or lime-treated. Note: $a_i = 0$ for CBR $< 60$ when maximum axle loading exceeds 80 kN.
3/ CBR = California Bearing Ratio (in percent) determined at the equilibrium in situ conditions of moisture content and density.
4/ $a_i = 0.075 + 0.039 \, UCS - 0.00088 \, UCS^2$; where UCS = unconfined compressive strength in MPa at 14 days. "Cemented" implies development of tensile strength through portland cement- or lime-treatment, or the use of certain flyash, slag, lateritic or ferricrete materials that are self-cementing over time.
5/ Dense-graded bitumen-treated base of high stiffness, e.g., $MR_{so} = 4000$ MPa, resilient modulus by indirect tensile test at 20°C.
6/ $a_i = 0.01 + 0.065 \, \log_{10} \, CBR$.
Figure 4.6: Layer strength coefficients for computation of the pavement structural number

(a) Bituminous and Cemented Materials

(b) Unbound Materials and Subgrade Support

Source: Table 4.7.
2. Surface deflections: The deflection and deflected shape, which vary with applied load and loading rate, are measures of pavement stiffness and the sum of vertical strains over the pavement depth. Examples include the maximum deflection of the Benkelman beam under an 80 kN dual wheel single axle creep load; the Dynaflect deflection basin at 300 mm spacings from 455 kg mass 8 Hz cyclic loading, the deflection basin under a Falling Weight Deflectometer, etc.

3. Mechanistic parameters: the strain and stress levels induced in pavement materials are deduced from the deflection basin through theoretical or semi-empirical analysis. Performance predictions are derived from laboratory-derived material behavior models that relate fatigue-cracking to tensile strain in the surfacing, and rut depth to vertical compressive strain or shear stresses in the subgrade and other layers.

Of these parameters, the surface deflection is the most readily measured, and probably the most widely applied. Computation of the structural number is either estimated from pavement design or construction records (which may be unreliable), or computed from measured data on layer thickness and strength. Mechanistic parameters cannot be measured directly in existing pavements, but are estimated through simulation of the reaction of the pavement to load, using a multilayer elastic model calibrated to match measurements of the surface deflection basin.

**Structural number and deflection**

It is a matter of some dispute as to whether the structural number or surface deflection is the better indicator of expected pavement performance, and this is one of the issues addressed in this study. In the literature there is support for both. Although the correlation between the two is good, it is not high, because the two parameters measure different attributes of the pavement. The structural number measures the strength of the pavement, ranking performance by permanent deformation under repeated loading to material characteristics that are related to shear strength. The peak surface deflection, which depends on the applied load and loading period, measures the stiffness of the pavement and that depends on the resilient stiffness and thickness of the material in each layers. The two are related only insofar as the resilient characteristics correlate with the permanent deformation behavior of the component materials; the correlation is high for a given material but may be poor when comparing across different materials.

The relationship between modified structural number and Benkelman beam deflection observed in the Brazil-UNDP study is shown in Figure 4.7. Considerable scatter is evident, and separate relationships apply to cemented and granular base pavements, but there is a clearly inverse, nonlinear relationship between the two. Conversion relationships developed from these data (by minimizing residuals on both SNC and DEF) are as follows:

1. For granular base pavements:

   \[
   \text{DEF} = 6.5 \text{ SNC}^{1.6} \\
   \text{SNC} = 3.2 \text{ DEF}^{0.63}
   \]
Figure 4.7: Relationship between the Benkelman beam surface rebound deflection and modified structural number in the Brazil-UNDP study

\[
\begin{align*}
\text{Granular Base} & \quad \text{DEF} = 6.5 \text{ SNC}^{-1.6} \\
\text{Cemented Base} & \quad \text{DEF} = 3.5 \text{ SNC}^{-1.4} \\
\end{align*}
\]

\[
\begin{align*}
\text{SNC} & = 2.2 \text{ DEF}^{0.63} \\
\end{align*}
\]

where \( r^2 = 0.36 \); standard error = 0.34 mm on DEF, and 1.24 on SNC; and t-statistics of the coefficient and power are about 8 and 12 respectively, in each case.

From the size of the standard errors of the prediction and from the scatter in the data, it is evident that the modified structural number and deflection are not directly interchangeable parameters. Better predictions can be obtained with the addition of data on subgrade strength, depth of pavement layers and particularly the thickness of the surfacing. Examples of models estimated from Brazilian study data are tabulated in Table 4.8, with linear correlations and ranges of values shown in Table 4.9.

Benkelman beam and Dynaflect deflections

The relationship between Benkelman beam and Dynaflect deflection measures depends on the reaction of the pavement to the very different loading characteristics of each test. Figure 4.8 (a) shows the relationship observed in the Brazil study on granular-base pavements, each observation representing the mean peak deflection over a 320 m long subsection averaged over the four-year study period. The general relationship is clear, but the scatter (linear correlation coefficient of 0.67) is significant and indicates that the two measures give different rankings of pavement stiffness in some cases. Queiroz (1981) has shown that the stress-dependency of the subgrade material properties affects the relationship and explains some of the variation. The symbols in the figure, which represent three ranges of subgrade CBR, indicate that subgrade strength alone
Table 4.8: Relationships between parameters of pavement strength and stiffness: modified structural number, Benkelman beam deflection, and Dynaflect deflections

<table>
<thead>
<tr>
<th>Estimated relationship 1/</th>
<th>Standard error</th>
<th>r²</th>
<th>No. obs.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Granular base: Benkelman beam deflection</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 3.48 DEF -0.311 ²/</td>
<td>0.22²/</td>
<td>0.37</td>
<td>392</td>
</tr>
<tr>
<td>(80) (14)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 0.48 DEF 0.216 SGDEF 0.248 SGCBR 0.165</td>
<td>0.20³/</td>
<td>0.39</td>
<td>348</td>
</tr>
<tr>
<td>(3) (9) (6) (7)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 1.87 DEF 0.242 HS 0.172</td>
<td>0.20³/</td>
<td>0.43</td>
<td>348</td>
</tr>
<tr>
<td>(10) (11) (10)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 0.48 DEF 0.247 SGDEF 0.219 HS 0.172</td>
<td>0.19³/</td>
<td>0.48</td>
<td>348</td>
</tr>
<tr>
<td>(3) (12) (6) (11)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 0.17 DEF 0.158 SGDEF 0.259 SGCBR 0.227 HS 0.209</td>
<td>0.16³/</td>
<td>0.63</td>
<td>348</td>
</tr>
<tr>
<td>(8) (8) (8) (12) (15)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Cemented base: Benkelman beam deflection</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 2.40 DEF -0.311 ²/</td>
<td>0.22²/</td>
<td>0.37</td>
<td>58</td>
</tr>
<tr>
<td>(11) (14)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 0.017 DEF -0.264 SGDEF 0.840</td>
<td>0.12³/</td>
<td>0.81</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 0.024 DEF 0.264 SGDEF 0.726 SGCBR 0.113</td>
<td>0.10³/</td>
<td>0.88</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 0.025 DEF 0.154 SGDEF 0.652 SGCBR 0.158 HS 0.112</td>
<td>0.09³/</td>
<td>0.91</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Granular base: Dynaflect deflection indices</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 4.7 DMD -0.17 HS 132</td>
<td>0.72</td>
<td>0.35</td>
<td>171</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNC = 12.2 DMD -0.285 DBCI -1.65</td>
<td>0.77</td>
<td>0.25</td>
<td>171</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DEF = 26.5 DMD ⁴/</td>
<td>0.18</td>
<td>0.45</td>
<td>171</td>
</tr>
<tr>
<td>(46)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(95% confidence limits 16 DMD to 41 DMD)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1/ Models were estimated by linear regression on natural logarithmic transformations of the variables, except where indicated by ²/. Correction for the logarithmic mean has not been made. t-Statistics are given in parentheses.

2/ The conversion relationships recommended in Equation 4.5 and 4.6 were derived by averaging this relationship and the inverse relationship having DEF as the dependent variable and SNC as the independent variable.

3/ Root mean squared error of logarithmic transform, see 1/.

4/ Linear regression without transformation of variables.

Note: Definitions of the variables are given in footnotes to Table 4.9.

Source: Analysis of Brazil-UNDP study data.
explains none of the variation, and thus several factors, particularly surfacing and pavement thicknesses, influence the relationship. The average relationship observed in Brazil was given by:

\[ \text{DEF} = 26.5 \ DMD \]

(4.7)

with 5th and 95th percentile confidence limits of 16.5 to 41, respectively, on the coefficient. Within these confidence limits, the result is highly consistent with other published data from North America (RTAC 1977, Asphalt Institute 1983, Lytton and Smith 1985), as shown in Figure 4.8(b), although the range of deflections over which these are applicable is not always clear.

Table 4.9: Linear correlations between major parameters of pavement strength and stiffness

<table>
<thead>
<tr>
<th>Pavement structural parameters</th>
<th>SNC</th>
<th>SNCK</th>
<th>DEF</th>
<th>DMD</th>
<th>BCI</th>
<th>SCI</th>
<th>HS</th>
<th>SGCBR</th>
<th>SGDEP</th>
<th>MCRX</th>
</tr>
</thead>
<tbody>
<tr>
<td>SNC</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SNCK</td>
<td>.99</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DEF</td>
<td>-.33</td>
<td>-.38</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DMD</td>
<td>-.41</td>
<td>-.44</td>
<td>.67</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BCI</td>
<td>-.34</td>
<td>-.36</td>
<td>.48</td>
<td>.72</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCI</td>
<td>-.41</td>
<td>-.42</td>
<td>.61</td>
<td>.87</td>
<td>.42</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HS</td>
<td>.47</td>
<td>.43</td>
<td>-.06</td>
<td>.05</td>
<td>.04</td>
<td>-.10</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SGCBR</td>
<td>.17</td>
<td>.20</td>
<td>-.39</td>
<td>-.26</td>
<td>-.16</td>
<td>-.26</td>
<td>-.00</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SGDEP</td>
<td>.25</td>
<td>.26</td>
<td>.02</td>
<td>-.22</td>
<td>-.28</td>
<td>-.05</td>
<td>-.22</td>
<td>-.22</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>MCRX</td>
<td>-.02</td>
<td>-.14</td>
<td>.49</td>
<td>.23</td>
<td>.21</td>
<td>.16</td>
<td>.09</td>
<td>-.25</td>
<td>.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Notes:  
SNC = Modified structural number.  
SNCK = Modified structural number reduced for cracking (see Equation 8.11).  
DEF = Benkelman beam deflection under 80 kN dual-wheel axle load, mm.  
DMD = Dynaflect maximum deflection, mm.  
BCI = Dynaflect base curvature index, mm.  
SCI = Dynaflect surface curvature index, mm.  
HS = Surfacing thickness, mm.  
SGCBR = Subgrade CBR in situ, percent.  
SGDEP = Depth from surface to top of subgrade, mm.  
MCRX = Mean extent of indexed cracking, percent area (see Section 5.1).  
Source: Data from Brazil-UNDP study.
Figure 4.8: Relationships between Benkelman beam and Dynaflect peak pavement surface deflections in Brazil and North America

(a) Observed in Brazil-UNDP Road Costs Study

Source: Data from GEIPOT (1982).

(b) Comparison of Various Sources

Notes:
- A Saskatchewan '75 DEF = 18.7 DMD
- B Asphalt Institute DEF = 22.3 DMD - 0.07
- C Brazil '77-'80 DEF = 26.5 DMD
- D Alberta '73-'74 DEF = 30.9 DMD
- E Ontario DEF = 0.06 + 19 DMD + 374 DMD^2
- F Louisiana '67 DEF = 20.6 DMD
- G Arizona DEF = 22.5 DMD
- H Virginia '66 DEF = 30.5 DMD - 0.31

All deflections in mm

Source: A,D,E from CGRA (1977); B from Asphalt Institute (1983); F,G,H from Lyon and Smith (1985); C from analysis of GEIPOT (1982).
Deflection measurements by Benkelman Beam and Dynaflect are therefore not directly interchangeable. The surface profile of deflection given by the Dynaflect is frequently used to analyze layer stiffnesses in pavement evaluation, but the very low stress level and shallow depth of influence of the applied load, combined with the significantly stress-dependent properties of most unbound pavement materials, mean that the results are not directly representative of pavement reaction under heavy loads. The results of the present analysis of the Brazilian data showed that the Dynaflect deflections were very poor explanatory variables for predicting pavement deterioration and were rarely significant in the model estimations. In contrast, the Benkelman Beam deflections had much stronger explanatory power (although not as strong as the modified structural number in many cases), so that a high magnitude of loading in the deflection test is clearly important. This conclusion thus supports other findings which show that heavy loading methods (such as Benkelman beam, Lacroix Deflectograph, Falling Weight Deflectometer, etc.) are more appropriate to the evaluation of pavement strength for the purposes of performance prediction than light load devices (such as Dynaflect, Road Rater, etc.). The measurements from light loading devices need compensation for any stress-dependent or frequency-dependent properties of the pavement materials in order to yield a relevant measure. The correlation between the deflections from light-loading and heavy loading devices thus depend on the pavement structure, and may be either high or low depending on the range of structures tested.

Falling Weight Deflectometer and Benkelman beam deflections

The loading applied by Falling Weight Deflectometer (FWD) is currently considered to be more similar to traffic loading in both the load and time domains than either the Benkelman beam test (which applies similar loads at creep speed) or the light-loading, high frequency devices. Correlations between FWD and other deflections are often high (e.g., $r = 0.9$ to $0.96$), however the ratio of the two deflections may often vary widely depending on the pavement structure (Tholen, Sharma and Terrel 1985). Under similar applied loads, the ratio of FWD to Benkelman beam deflections ranges from 0.8 to 1.35 for asphalt-surfaced pavements. Thus, a reasonable first approximation, in the absence of specific local correlations, is to equate the FWD deflection (after correction for the applied load) to the Benkelman beam deflection, for the purposes of applying Equations 4.5 and 4.6 and interpreting the models in this book.

4.3.5 Traffic

Mixed traffic of a variety of vehicle classes, axle loadings and wheel configurations is commonly converted to equivalent standard axle loads (ESA), which is the number of 80 kN (18,000 lbf) single axle loads causing the same amount of total damage to the pavement as the mixed traffic, up to the time of rehabilitation. The concept of the relative damaging power of different axle loads and configurations is considered in detail in Chapter 9, but some preliminary remarks are needed at this point.

In simple terms, the number $N_p$ of axle loads $P$ can be converted to the number of ESA (NE) by a function of the form:

$$NE_n = N_p \left(\frac{P}{80}\right)^n$$

(4.7)
where $P$ is the axle load in kilonewtons. Based upon an analysis of AASHO road test data by Liddle (1962), utilizing a more complex function, the value of the power $n$ has been taken typically as about 4.2 or 4.0. However, the validity of this value is open to some question because theoretical and experimental studies have indicated that $n$ may have either much smaller or much larger values under certain conditions. In empirical data, $n$ can not be determined directly because the traffic is mixed and the effects are not separated as they are in controlled or theoretical studies. Thus to evaluate the value of $n$, the practice adopted in this study was to index $NE$ by $n$, compute a set of values $NE_n$ for various values of $n$ from the loading spectrum of traffic on the section and then test to determine which value best explains the traffic effects on pavement deterioration.

4.3.6 Environment and Moisture

The environment of a pavement comprises climatic and terrain factors, as follows:

<table>
<thead>
<tr>
<th>Climatic factors</th>
<th>Terrain factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>precipitation rates</td>
<td>roadbed soil</td>
</tr>
<tr>
<td>ambient temperatures and ranges</td>
<td>formation (fill, cut, slope)</td>
</tr>
<tr>
<td>incidence of freezing</td>
<td>moisture regimes</td>
</tr>
<tr>
<td>solar radiation</td>
<td>surface drainage</td>
</tr>
<tr>
<td>humidity, evaporation and water surplus</td>
<td>subsurface drainage</td>
</tr>
</tbody>
</table>

The general approach adopted in the study was to define the climate by as few relevant statistics as possible, and to incorporate the terrain factors into the pavement strength characteristics.

Climate

Monthly, instead of annual, rainfall was used to define precipitation, in order to permit the analysis of seasonal effects. Ambient temperatures are best quantified by the weighted mean annual average temperature ($w$-MAAT) (Claessen and others 1977) which weights the seasonal temperatures in relation to their impact on pavement temperatures. The incidence of freezing can be quantified by the number of freeze-thaw cycles, the period of ground freezing per year, and the depth of frost penetration.

More relevant to the moisture regimes in the pavement and roadbed is the water balance during the year. Whether there is a water surplus or deficiency depends on the evapotranspiration rate, and that is a function of humidity, rainfall, and soil type. Thornthwaite's moisture index (Thornthwaite 1955) takes into account the periods of water surplus and deficiency, and soil moisture storage and utilization in a scale ranging from -100 (very arid) to 100 (perhumid), with the division between dry and wet occurring at zero, namely:
This classification of climate shows much stronger discrimination than annual rainfall alone because it takes into account temperature, humidity and seasonal effects. For example, the climate in Brazil, shown in Figure 3.5, ranging from moist subhumid to perhumid (moisture index 12 to 100), differs considerably from the climate of Kenya (Table 4.4) which is arid to subhumid (-80 to 0), whereas the annual precipitations appear more similar (1,040 to 1,800 mm in Brazil and 400 to 2,000 mm in Kenya). Maps showing the index are available (e.g., Organization of America States 1968, Thornthwaite Associates, 1958).

As the variables involved in defining climate are numerous, a practical approach being adopted in recent American studies (e.g., Rauhut and others 1984, and Transportation Research Board 1986) is to define just four major climatic classes, namely:

- dry, nonfreezing (DNF)
- dry, freezing (DF)
- wet, nonfreezing (WNF)
- wet, freezing (WF),

where the dry/wet division is defined by the moisture index of zero, and "freezing" is defined by temperatures below freezing for more than 60 days per year (typically, a freezing index of greater than 100).

The climatic parameters adopted for this study were the monthly precipitation (and its annual mean, MMP), Thornthwaite's moisture index (IM), and the wet/dry, freezing/nonfreezing classification above.

Pavement moisture

As the stiffness and strength of pavement materials depend significantly on the moisture content, with dramatically early failures being possible when the materials become saturated, the moisture regime of the pavement is of vital relevance to the modelling of performance. Poorly maintained or blocked drains are often cited as the cause of local failures, and the trapping of water in a base-course with no vertical drainage through the subbase, or horizontal drainage through the shoulder, invariably results in early fatigue cracking of the surfacing and shallow failures.

The approach adopted in this study was to define the pavement materials by their properties in situ, since these are the properties influencing performance. Thus material strength was determined under the equilibrium moisture
conditions *in situ*, and not at either the optimum moisture content nor the saturated condition. Data on the remoulded CBR at *in situ* moisture were therefore used in computing the modified structural number of the pavement in the Brazilian data. Factors representing the state of saturation of individual layers and of the full pavement were computed by the ratio of the field or equilibrium moisture content (EMC) to the optimum moisture content (OMC). The moisture condition of individual layers ranged from 0.73 to 1.65 times optimum, with a mean of four percent below the optimum moisture content.

**Climate and pavement moisture**

Apart from the influences of adverse drainage, there is evidence that climate influences the moisture regime in paved road pavements. Early British work in Kenya (O'Reilly, Russam and Williams 1968) found that the Thornthwaite moisture index was a more reliable indicator of the moisture state in subgrade soils than was mean annual rainfall. During the course of the present study, the significance of the Thornthwaite index has been verified in two broad contexts, and extended to all pavement materials by the development of a predictive model incorporating a material property.

Using data from a wide range of climates from semiarid to wet humid (moisture index -50 to 75) in southern Africa, the equilibrium moisture content *in situ*, for a variety of subgrade, subbase and basecourse materials in well-drained pavements, was found to be a function of the moisture index and the percentage of fines in the material, as follows (Emery and Paterson 1983):

\[
PEMC = 0.25 P075 + 0.019 IM + 1.81
\]

(4.8)

with \( r^2 = 0.95; \) coefficient of variation = 12.8 percent; sample = 1,620 observations grouped into 7 classes; and

where PEMC = predicted equilibrium moisture content *in situ*, percent;
P075 = percentage of material finer than 0.075 mm; and
IM = Thornthwaite moisture index.

Annual rainfall, plasticity index, liquid limit, coarser material fractions and layer identification were among the variables found to be not significant.

When the same analysis was applied to the Brazilian data from subgrade, subbase and base layers, the strongest model estimated was very similar to Equation 4.8 above, as follows:

\[
PEMC = 0.27 P075 + 0.018 IM + 2.39
\]

(40.2) (2.6) (5.1)

(4.9)

where \( r^2 = 0.79; \) coefficient of variation = 21.1 percent; sample = 1,314 observations grouped into 438 classes, and t-statistics are given in parentheses. The comparison of observed and predicted values, in Figure 4.9, shows that the model is well-determined. The dispersion appears considerably greater because the observations are not grouped by regional means like the previous analysis but are instead data from individual sections and layers. At this level, the standard error is 3.1 percentage points on a mean EMC of 14.6 percent.
Figure 4.9: Prediction of moisture content in pavement layers: comparison of observed and predicted values in Brazil

The strong similarity of Equations 4.8 and 4.9, derived from such different regions, suggest combining these two relationships. For climates ranging from semiarid to humid (-54 to 100 moisture index), the following general predictive relationship was derived, namely:

\[ PEMC = 0.26 \, P075 + 0.019 \, IM + 2.1. \]  \hspace{1cm} (4.10)

When applied to the Brazilian data, this model had a prediction error of 2.8 percentage points, and a bias of -8 percent which was fairly uniform for all layers. The model has a slight tendency to overestimate at low moisture contents and to underestimate at high moisture contents. The model indicates that moisture contents are from two to four percentage points higher in wet climates than in dry, even under good drainage conditions. Thus it is essential that the in situ moisture condition be used in characterizing pavement strengths.

When the generalized model of Equation 4.10 was applied to a dry-freeze climate, using data from the Arizonan study, the predictions were found to be good in non-freezing areas, having a prediction error of 3 percentage points, a correlation coefficient \( r \) of 0.77 and a bias of -10 percent. For freezing zones, defined by a freezing index greater than 100 or more than 20 freeze-thaw cycles per year, the predicted moisture was about 6 points lower than observed and the prediction error rose to 5 points but the correlation coefficient of 0.71 was still high.

Thus, the model given by Equation 4.10 is reliable over a wide range of moisture conditions for non-freezing climates, but requires modification for freezing climates.
CHAPTER 5
Cracking of Paved Bituminous Surfacings

Bituminous surfacings tend to crack at some stage of their life under the combined actions of traffic and the environment through one or more different mechanisms. The crack is a defect in the surfacing which weakens the pavement and allows water to penetrate and cause further weakening. Once initiated, cracking usually increases in its extent, severity, and intensity, leading eventually to disintegration of the surfacing. Through these effects, the rate of deterioration of the pavement usually accelerates after the appearance of cracking, with particular impact on the rates of rutting and roughness progression.

Cracking therefore has long been an important criterion for maintenance intervention. The accurate predictions of its development and the effectiveness of resurfacing in controlling it are thus key components in predicting the timing and costs of road maintenance. We shall see however that the ability to predict cracking is severely limited by several factors. While mechanistic approaches have quite successfully experimentally modelled the fatigue effects of repeated traffic loading on failure by cracking, field data show large discrepancies of scale and strong effects of weathering. The distinction between traffic-related and weathering-related effects or causes is often difficult. Moreover, the variability of both the properties and behavior of materials under field conditions causes wide variations in the performance of nominally identical pavements.

5.1 MEASURES OF CRACKING

Cracking, like the other forms of surface distress such as ravelling and potholing, is characterized by two distinct phases, as was shown in Figure 4.2. The initiation of cracking is a discrete event in time, which for our purposes we define as the appearance of the crack at the surface, in order to be consistent with condition survey methods in pavement information systems (often in experimental research, initiation is the beginning of a crack anywhere within a bound layer). In the succeeding progression phase, cracking extends progressively over the surface and individual cracks widen.

There is as yet no widely accepted measure for cracking in pavements, and neither is cracking readily measurable by automated instruments. It is an important item however in the pavement condition inventory of a road network. Many different measures have emerged, some of them qualitative rather than quantitative, but no international correlation or standardization has been achieved. There is hope that some consensus might be promoted through major new research initiatives such as the SHRP project in the U.S.A., with international participation, through international consultative groups, and through the current development of automated measuring systems.
5.1.1 Characteristics

The reasons for this situation lie in the number of characteristics required to define cracking, and the variety of assumptions that lie behind attempts to combine these in a composite index. There are four characteristics - type, extent, intensity, and severity - as follows.

The classification of six types of cracking outlined in Table 4.1 - crocodile (or alligator), longitudinal, transverse, map, irregular and block - is one which has a high degree of commonality with many systems. A simpler one based on the broader classification of pattern has three elements, namely:

1. Network cracking - crocodile (alligator) or map cracking, i.e., interconnected polygons;
2. Line cracking - longitudinal or transverse cracking, or line cracks interconnected in rectangular patterns; and
3. Irregular cracking - unconnected cracks, or interconnected with irregular pattern.

Another approach is to define it simply by location, namely wheelpath cracking and non-wheelpath cracking. Yet another is to refer directly to the mechanisms deduced to have caused the cracking, for example: fatigue, shrinkage, reflection, low-temperature, settlement, aging; or alternatively, simply traffic-associated and non-traffic-associated.

Behind these different approaches is the intention to provide information on the probable cause of the cracking, which in turn permits more reliable predictions and provides a rational basis for selecting and designing appropriate maintenance. The greatest objectivity is provided by the first three approaches, which are preferred to the last approaches because the latter require a higher level of interpretative skill. Whether the data are collected manually or automatically, interpretation is better made in the analytical phase than in the data collection phase. The accuracy of classification is improved by having as few, clearly distinct classes as possible, which may be at the expense of detail for the interpretation. Thus information systems for pavement management should have as few as two to four classes, while research programs may have more (preferably as subdivision of the major classes).

Extent (or amount) is the area of the pavement surface covered by cracking (usually the sum of the cracked areas), and is conveniently expressed as a percentage of the surfacing area over a defined unit such as a lane- or pavement-width by a convenient sample length in the range of 100 to 1,000 m. Intensity is expressed either as the total length of cracks in a unit area (m/m²) or as an average spacing of the cracks (considering cracking as a nominally square-grid network) - cracking within a fixed extent can become more intense as individual blocks of the surfacing break down into smaller blocks. Severity is a measure of the width of crack, usually represented by classes. In some classification systems, severity classes include both crack width and intensity.

Without doubt, the handling of four characteristics tends to be cumbersome, and most systems combine two or more for practical reasons. As a minimum, one needs to know the amount of cracking (in which case a rational weighting for
intensity and severity needs to be added to the extent), and preferably also the type (in at least two classes to distinguish traffic- and non-traffic-associated mechanisms). Examples taken from the major empirical studies are described in the next three sections.

5.1.2 Brazil-UNDP Study Classification System

In the Brazil-UNDP road costs study (GEIPOT 1982), cracking was classified by class (severity), area (extent) and type, in a system adapted from the AASHO Road Test (Highways Research Board 1962), and condition survey methods in Texas and South Africa, as follows:

Severity

Class 1: hairline cracks, width 1 mm or less;
Class 2: crack widths 1 to 3 mm;
Class 3: crack widths greater than 3 mm without spalling; and
Class 4: spalled cracks, i.e., fragments of the surfacing adjacent to the crack were lost.

Area

The sum of rectangular areas surrounding individual cracking networks, measured in square meters and eventually reported as a percentage of the subsection area (one lane-width by 320 m length). For linear cracks, the area was defined by a 0.5 m wide strip extending the length of the crack.

Type

Crocodile, irregular, block, transverse or longitudinal.

The intensity of cracking, although defined in the initial stages of the study by crack spacing, was not recorded. The individual locality was recorded, but not included on the computer files.

Conceptually, the initiation and progression phases were characterized by area as shown in Figure 5.1(a) (Texas Research and Development Foundation (TRDF), 1980). Initiation was represented by a discrete event at time }\text{t}_i\text{, and progression by a discrete function of cracking area increasing from 0 to 100 percent. Once initiated, the cracking extends progressively over time eventually covering up to 100 percent of the surfacing area. At the same time, the crack width of the cracking that occurred earlier has been increasing so that some of the area is reclassified as a higher class of cracking (Class 1 becomes Class 2, etc.). The increase of cracking severity therefore causes the area of a lower class of cracking to decrease as the area of a greater class of cracking increases, resulting in the bell-shaped functions of class-area with respect to time shown in Figure 5.1(b). Separate initiation times and progression functions can therefore be defined for each class of cracking. The bell-shaped functions however are awkward for modelling and inappropriate for planning purposes, thus it is preferable to define a cumulative numeric, }\text{CR}_i\text{, which represents the sum of all areas of cracking with a severity of at least class }i\text{, as shown in Figure}
Figure 5.1: General forms of cracking initiation and progression and transformation into summary numerics at four severity levels

(a) Observed development of cracking or ravelling over time

(b) Growth of cracking area by crack class

(c) Continuous form of cracking numeric by level
CRACKING OF BITUMINOUS SURFACINGS

5.1(c), and which was defined by the study as follows:

$$\text{CR}_i = \sum_{j=1}^{4} \text{CL}_j$$  \hspace{1cm} (5.1)

where $\text{CL}_j =$ area cracked of class $j$, $j = 1$ to 4; and $\text{CR}_i =$ cracking area numeric of level $i$.

In the application of the TRDF concept to the current study, $\text{CR}_2$ represents the area of "all cracking" (the sum of areas of classes 2, 3 and 4), and $\text{CR}_4$ represents the area of only class 4 cracking, or "wide cracking". For practical purposes, class 1 (or hairline) cracking is omitted from the modelling because it is difficult to observe (being visible under some conditions and not under others) and has little mechanical impact on pavement behavior. In the original TRDF concept, potholes were included as a fifth class, but as these can develop from other modes of distress, potholing is better treated separately (for example, see Chapter 6).

In the above conceptualization, the initiation of "narrow" (class 2) cracking is synonymous with the initiation of $\text{CR}_2$, (or "all cracking"), and the initiation of "wide" (class 4) cracking is synonymous with the initiation of $\text{CR}_4$. Both $\text{CR}_2$ and $\text{CR}_4$ increase monotonically from 0 to 100 percent and satisfy the condition:

$$\text{CR}_2 \geq \text{CR}_3 \geq \text{CR}_4. \hspace{1cm} (5.2)$$

As the use of separate indices for each severity level of cracking proliferates the number of predictive relationships to be both estimated and applied, an index of cracking, $\text{CRX}$, combining all severities, was defined as follows:

$$\text{CRX} = \sum_{j=1}^{4} (\text{i CL}_j) / 4$$  \hspace{1cm} (5.3)

$$= (\text{CR}_1 + \text{CR}_2 + \text{CR}_3 + \text{CR}_4) / 4$$

where $\text{CRX} =$ area of indexed cracking area, in percent of total surfacing area. In the index, the weighting factor adopted is crack width (equal to $i$ mm), which accords greater importance to the wider cracks in proportion to the area of opening available for water penetration. Whether this or any other nominal weighting is the most rational and relevant to pavement deterioration can only be determined through an empirical study such as this. As a practical device, to further reduce the number of basic cracking numerics needed to two, $\text{CRX}$ was also estimated empirically from $\text{CR}_2$ and $\text{CR}_4$ in the Brazilian data by:

$$\hat{\text{CRX}} = 0.62 \text{CR}_2 + 0.39 \text{CR}_4$$  \hspace{1cm} (5.4)

with standard error = 3.9 percent; $r^2 = 0.99$; and sample = 3,142 observations.

The definitions above have the disadvantage of omitting a measure of cracking intensity. While this is no disadvantage for planning maintenance quantities, it may be a significant consideration in prediction modelling and in estimating the impact of cracking on other modes of distress such as roughness. For example, two pavements progressing to 100 percent of the area cracked, one with sparse cracking at wide spacing and the other with intensive cracking at narrow spacing, may have very different progression rates and very different impacts on rutting and roughness progression.
5.1.3 TRRL (Overseas Unit) Classification System

The system applied in the Kenya road costs study (Hodges, Rolt and Jones, 1975), classified cracking only by an average intensity, without classifying severity and with only an indirect measure of extent.

The cracking was measured in one-meter-square samples of the road surface at 100 m intervals in each wheelpath of the 1,000 m long section. The total length of cracks within the one-meter-square frame were measured and expressed in linear meters per square meter, and the average intensity defined as the average of the 22 measurements on each half-carriageway section. The intensity was thus an open-ended scale. For the computation of maintenance quantities, the area of cracking was defined by the fraction of sampling frames which contain cracking of a certain minimum intensity, such as 5 m/m². For example, 8 out of 22 "frames" represented 36 percent of the area cracked.

Adaptation of the TRRL method to obtain more information on the nature of cracking progression is also possible. For example, areas could be defined for two levels of intensity and a simple two-class severity coding could also be added, using, say, classes 2 and 4 from the Brazil definitions above. The TRRL system has particular advantages in that it provides a clear definition of cracking quantity for the cases of both network cracking and linear cracking, and in defining a sample size and sampling rate. The measurement repeatability and reproducibility appear to be high.

5.1.4 Score Rating and Deduct Systems

Several systems used in pavement management applications combine all features of cracking in a single score rating, either alone or in combination with other modes of distress. An example of the latter, using deduct values to weight severity and quality is the Pavement Condition Index (PCI) (Shahin 1982).

A score rating system for cracking is used in the state of Texas (Lytton and others 1982) in the data base used for the validation exercises in the present study. The system combines the area and the severity of cracking, which are each grouped in four ratings, into a single decimal score as shown in Table 5.1. The sum of the two scores represents the cracking score of the pavement. The conversion between these scores and the indexed cracking parameter of the Brazil-UNDP study developed for the validation exercises was:

\[
\text{CRX} = \text{ATX} \times \text{STX}
\]

where

\[
\text{ATX} = 0.5, 8, 23 \text{ and } 50 \text{ percent area for area numerical ratings of } 0, 1, 2 \text{ and } 3 \text{ respectively;}
\]

\[
\text{STX} = 0, 0.5, 0.75 \text{ and } 1.0 \text{ for severity numerical ratings of } 0, 1, 2 \text{ and } 3 \text{ respectively; and}
\]

\[
\text{CRX} = \text{area of indexed cracking, in percent (as per Equations 5.3 and 5.4)}.
\]
Table 5.1: Decimal score of cracking, combining area and severity ratings in Texas

<table>
<thead>
<tr>
<th>Area</th>
<th>Numerical rating</th>
<th>Decimal score</th>
<th>Severity</th>
<th>Numerical rating</th>
<th>Decimal score</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-1</td>
<td>0</td>
<td>0.005</td>
<td>None</td>
<td>0</td>
<td>0.005</td>
</tr>
<tr>
<td>1-15</td>
<td>1</td>
<td>0.08</td>
<td>Slight</td>
<td>1</td>
<td>0.167</td>
</tr>
<tr>
<td>16-30</td>
<td>2</td>
<td>0.23</td>
<td>Moderate</td>
<td>2</td>
<td>0.333</td>
</tr>
<tr>
<td>&gt; 30</td>
<td>3</td>
<td>0.50</td>
<td>Severe</td>
<td>3</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Note: Cracking score = Area score + Severity score.

Source: Lytton, Michalak and Scullion (1982).

5.2 CRACKING MECHANISMS

5.2.1 Fatigue

Fracture of materials by fatigue results from the cumulative effects of repeated loading cycles. It is characterized by crocodile cracking, usually confined to the wheelpaths, and is primarily associated with traffic. Extensive fundamental research (e.g., Epps and Monismith 1972, Pell 1973) in the past two decades has established well-defined relationships for the fatigue of bituminous materials, which take the general form

$$N_f = K \cdot \varepsilon_t^{-n}$$  \hspace{1cm} (5.6)

where

- $N_f =$ number of repetitions of load in flexure to the initiation of fatigue cracking;
- $\varepsilon_t =$ maximum horizontal tensile strain in the bituminous material under the applied load; and
- $K, n =$ constants depending primarily on material stiffness and binder content.

Laboratory estimates for $K$ and $n$ vary with the loading conditions and material characteristics. Under controlled-strain loading, which generally applies in thin flexible pavements, the fatigue life is in the order of two to three times longer than at comparable strain levels under controlled-stress loading, which generally applies in thick, stiff pavements; intermediate situations can be represented by combination of these behavioral modes using a mode factor (Monismith and Deacon, 1969).

The influence of material characteristics depends on the loading conditions to some degree as summarized in Table 5.2. Raising the binder content of the mixture always increases the fatigue life (except at levels above the optimum for stability), due to the increase in film thickness. Mix stiffness (asphalt stiffness and aggregate gradation), however, increases the life under the
Table 5.2: Factors affecting the stiffness and fatigue life of continuously-graded asphalt paving mixtures

<table>
<thead>
<tr>
<th>Factor</th>
<th>Change in factor</th>
<th>Effect on stiffness</th>
<th>Effect on controlled-stress mode of testing</th>
<th>Effect on controlled-strain mode of testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt viscosity</td>
<td>increase</td>
<td>increase</td>
<td>increase</td>
<td>decrease</td>
</tr>
<tr>
<td>(stiffness)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt content</td>
<td>increase</td>
<td>increase</td>
<td>increase</td>
<td>increase</td>
</tr>
<tr>
<td>Aggregate gradation</td>
<td>open to dense</td>
<td>increase</td>
<td>increase</td>
<td>decrease</td>
</tr>
<tr>
<td>gradation</td>
<td>gradation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Air void content</td>
<td>decrease</td>
<td>increase</td>
<td>increase</td>
<td>increase</td>
</tr>
<tr>
<td>Temperature</td>
<td>decrease</td>
<td>increase</td>
<td>increase</td>
<td>decrease</td>
</tr>
</tbody>
</table>

1/ Reaches optimum at level above that required for stability considerations.
2/ No significant amount of data; conflicting conditions of increase in stiffness and reduction in asphalt strain make this speculative.
3/ Approaches limiting value at temperatures below freezing.
4/ No significant amount of data.

controlled-stress mode (slightly) but decreases the life under the controlled-strain mode. These differences are evident in design criteria based on experimental research. Examples of fatigue life curves for three mixes, and a general nomograph for determining the effect of material characteristics on fatigue life from British research (Brown, Pell and Stock 1977) are shown in Figure 5.2; here, increasing mix stiffness tends to increase the fatigue life slightly because the testing was conducted in the controlled-stress mode. The Shell and Asphalt Institute design curves shown in Figures 5.3 and 5.4 show a strong opposite effect of stiffness, being based primarily on the controlled-strain mode. The Shell method takes material and environmental characteristics (such as binder volume, binder stiffness, air voids and temperature) into account in determining the mix stiffness. A typical model from that research which may be applied to performance predictions is (Shell 1978):

\[
N_L = \left[ (0.86 V_b + 1.08) S_{mix}^{-0.36} \tau_t^{-1} \right]^{1.5} \tag{5.7}
\]

where
- \( V_b \) = volume binder content, in percent of mixture volume (approximately equivalent to 2.4 times the binder content by percentage of mass); and
- \( S_{mix} \) = mix stiffness (which is a function of binder stiffness, air voids and aggregate), in Pa; and
- \( \tau_t \) = tensile strain, in microstrains \( (10^{-6}) \).
Figure 5.2: Fatigue characteristics of bituminous mixtures from British research

(a) *In situ* fatigue lines for typical bituminous mixes

![Graph showing fatigue lines for different mixtures](image)

**Note:** D.B.M. dense bituminous macadam. Hot rolled asphalt: B.S. 594 specification.

(b) Nomograph for predicting *in situ* fatigue performance

![Nomograph for fatigue analysis](image)

**Note:** Both charts include a shift factor of 100 to account for the effects of field performance versus laboratory controlled-stress results, and of rest periods.

**Source:** Brown, Pell and Stock (1977).
Figure 5.3: Fatigue characteristics from Shell Pavement Design Method

(a) Fatigue results for various mixes related to mix stiffness.

(b) Fatigue characteristics for mixes with moderate binder and voids contents.

Note: Mix codes refer to Table 3.1 in source.
Source: Claessen and others (1977).

Figure 5.4: Comparison of Shell and Asphalt Institute design criteria for fatigue of bituminous mixtures

Notes: NCHRP1-10B results based on Finn and others (1977) and Asphalt Institute (1981), calibrated to 45 percent cracking on AASHO Road Test data
1.5 x 10^6 psi = 1 GPa.
In applying these experimental findings, the major factors to be considered are the condition defining "failure" and the relation of the experimental functions to field performance. These include the period for propagation of the initial crack to the pavement surface, the progression period and defined area of cracking, and the effects of rest periods, rate of loading (and traffic speed), mixed loading and aging. Monismith and Witzcak (1982) noted that various researchers applied shift factors of between 2 and 700 to the experimental relationships to "calibrate" them to field performance data. Brown and others (1977) use a factor of 100 in Figure 5.2, for example. The experimental findings are therefore clearly more reliable for the relative influences of different factors that they show than they are for the absolute predictions of fatigue life under real road conditions.

The clear common finding is the strong relationship between the fatigue life and the induced tensile strain, with the life of a given material depending inversely on the third to fifth power of strain in the average case. Reliable predictions of fatigue failure however depend on accurate knowledge of not only the strains but also the appropriate material properties, and it is at this point that the confounding effects, shown in Table 5.2, may cause difficulties for distress modelling. Finally, considerable scatter was common in the experimental results, with lives ranging typically over an order of magnitude, or by a factor of three each side of the mean, even under controlled conditions.

5.2.2 Tensile Strains in Pavements

What then are the structural factors governing the maximum tensile strains developed in a bituminous surfacing? Without attempting to summarize the extensive literature on structural analysis concerning the influences of load, tire contact pressure, layer stiffness and layer configurations, two aspects of particular relevance to the modelling issue are addressed: the measurable pavement properties which affect the induced strains, and the differences between thin and thick pavements.

**Surfacing strains**

The stress and strain profiles of a large sample of pavements in the Brazil-UNDP study were analysed to assess the relative influences of various pavement properties on strains in the surfacing. The profiles were derived for forty-eight paved sections by linear elastic layered-structure analysis, using layer stiffnesses that had been estimated by Queiroz (1981) through an iterative analysis of surface deflections. Several three-parameter relationships between strain, stress, layer thickness and layer stiffness parameters were evaluated to discern useful surrogates for the horizontal tensile strains in the surfacing, and functional forms. Few gave clearly defined relationships however, particularly for peak deflection or strain, but two of interest are shown in Figure 5.5.

In Figure 5.5(a) it can be seen that the tensile strain at the surfacing-base interface increases significantly as the stiffness of the base decreases, and that thin surfacings are more sensitive to the base stiffness than thick surfacings. Thus a soft base, caused by either low strength material, water penetration or inadequate compaction, will induce high tensile strains in the surfacing and cause early fatigue failure. A base stiffness of 300 to 400 MPa is usually required to limit the induced strain to 200 microstrain (10^-6) or less.
Figure 5.5: Factors affecting the maximum horizontal strain in bituminous surfacing

(a) Effect of base stiffness on strain at surfacing interface

(b) Relation between surface and interface strains in surfacing

Notes: Code for thickness: × <25 mm; + 25-35 mm; Δ 35-45 mm; ◇ 45-65 mm; □ 65-100 mm. Curves not statistically fitted.

Source: Linear elastic analysis of Brazil-UNDP study data; stiffness values after Queiroz (1981).
and to achieve a fatigue life in the range of 0.2 to 2 million ESA (based on Figures 5.2 to 5.4, and depending upon the material properties).

In Figure 5.5(b) it can be seen that the maximum tensile strain at the surface, which occurs between dual wheels or outside the tire contact area, exceeds the tensile strain at the bottom of the surfacing when the surfacing thickness is less than about 40 mm. In thin surfacings, therefore, the maximum tensile strain occurs usually at the exposed surface, and not at the bottom interface of the surfacing layer as is often assumed. This point has great importance when the additional effects of aging at the exposed surface are considered, as we shall see in later sections.

Thin versus thick surface layers

These findings and dependence of fatigue characteristics on material properties have very important implications about the relative performance of thin and thick bituminous layers in pavements.

First, research using layered structure analysis, experimental fatigue relationships, and field experiments has indicated that thin surfacings of 50 mm or less, when flexible, have a longer fatigue life than surfacings with a thickness of about 60 to 80 mm due to lower tensile strains (e.g., Freeme and Marais 1973; Queiroz 1981), as shown in Figure 5.5(a). At thicknesses of 100 mm and more, the structural contribution of the surfacing itself becomes significant in reducing the induced tensile strains, and the fatigue behavior shifts from a

Figure 5.6: Relationship of estimated fatigue life to the thickness and stiffness of the surfacing, other layer properties being constant

Source: After Queiroz and Visser (1978).
controlled-strain mode toward a controlled-stress mode, with the result that the fatigue life increases with increasing thickness. This results in convex functions, illustrated in Figure 5.6, in which the fatigue life passes through a minimum at thicknesses in the range of 50 to 100 mm, for asphalt layers of various stiffnesses.

Second, the effect of mix stiffness differs considerably for thin and thick pavements. For thin surfacings, the mix stiffness does not affect the induced strain significantly but reduces the fatigue life (controlled strain behavior), so the net effect of increasing the mix stiffness is to reduce the fatigue life of the pavement, as shown in Figure 5.7(a). In thick bituminous pavements, increasing the mix stiffness both reduces the tensile strain induced in the mix and also, at a given strain level, slightly improves the mix fatigue life (controlled-stress behavior), so the combined effect is a strong improvement in the fatigue life of the pavement, as shown in Figure 5.7(b).

Third, the tensile strain at the underside of thin surfacings is relatively insensitive to the load applied (at constant pressure), but very sensitive to the tire contact pressure. Thus cracking damage in thin surfacings may be relatively independent of axle loading, except insofar as the tire inflation pressures correlate with load. For thick layers, the tensile strains are sensitive to load and relatively insensitive to tire pressures.

A clear conclusion from this is that thin surfacings have long fatigue lives, but they need to be flexible, that is the material should have as low mix stiffness and high binder content as possible, within the limits of the required stability. For surfacings of medium thickness, it is critically important to have high stiffness in the base in order to keep the induced strains to a moderate level for good fatigue life. For thick bituminous layers (over 100 mm) the fatigue performance improves both as the stiffness and as the thickness of the layer increase.

5.2.3 Aging

Through exposure to air, a bituminous binder hardens over time and becomes more susceptible to fracture. If the bituminous binder becomes so brittle that it can no longer sustain the strains associated with daily surface temperature changes, fracture occurs. The rate at which hardening occurs depends on the oxidation-resistance of the binder (which varies with the chemical composition and source of crude), on temperature, and on the film thickness (which determines the length of the oxidation path) (Dickinson 1984). Hardening rates therefore vary with binder source, climate and material design. High binder contents (thick film) and low air voids contents have strongly beneficial effects by lengthening the oxidation path considerably and by excluding air, and thus promoting durability. Dickinson (1984) has also observed that cracking usually occurs when a binder reaches a critical viscosity at which the binder can no longer sustain the low strain levels associated with daily thermal movements and fracture occurs. The critical viscosity is about 5.7 log Pa.s in temperate climates and 6.5 log Pa.s in tropical climates (see Section 6.2.1 and Figure 6.1), which correspond to approximately equivalent viscosities at the road temperatures representative of each climate. By that stage the surfacing is typically nine years old although, depending on the composition of the binder, the age may range between six and fifteen years. This cracking by "aging" usually has the form of irregular or map cracking with a spacing greater than 0.5 meters and, once initiated, is likely to progress rapidly over the full area of surface.
Figure 5.7: Effects of mix stiffness on induced tensile strain and fatigue life for thin and thick bituminous surfacings

(a) Thin surfacings

(b) Thick surfacings

Source: Freeme (1972).
5.2.4 Interaction of Fatigue and Aging

The aging of the bituminous binder, due primarily to oxidation, increases the stiffness of both the binder and surfacing material over time. Consequently the aging strongly reduces the fatigue life of thin to medium thickness surfacings, but probably has little effect on thick bituminous layers because they behave according to controlled-stress conditions. For other than thick pavements therefore, aging advances the time at which fracture might be expected under given traffic flows.

There is therefore significant interaction between the effects of aging and traffic on the timing of fracture through three mechanisms, as illustrated in Figure 5.8. First, the fatigue life of the surfacing material has a time profile. The initial value is related to the material design and the tensile strains induced under the representative mix of axle loadings and tire pressures. Typically, when the maximum strain occurs at the underside of the surfacing, the available fatigue "life" at the surface will be greater than at the underside because of the difference in strain levels. The aging process then reduces available fatigue life at the exposed surface (curve A) more rapidly than at the underside of the surfacing (curve B), resulting in the two profiles shown. Different material designs or pavement designs would result in different relative and absolute positions of these two curves.

Second, the cumulative number of axle load applications from traffic result in fatigue cracking when this equals the available fatigue life in the surfacing material at the intersections of the curves. Under high traffic volumes,

Figure 5.8: Interaction between traffic-related and aging-related fatigue causing cracking in bituminous surfacing

![Figure 5.8: Interaction between traffic-related and aging-related fatigue causing cracking in bituminous surfacing](image)

Source: Author.
shown as curve C, traffic-related fatigue causes cracking first (at 4.5 years in the example) and, depending on the relative disposition of curves A and B, cracking initiation may occur either at the surface (as shown) or at the underside. Crack initiation at the exposed surface as has been noted by a number of observers, e.g., Rolt and others (1986). (Note that the log scale distorts the constant traffic flow shown).

Third, when traffic loadings are very light or the induced strains are low, then curve C may be lower than curve D. In this case, fatigue by the strains due to daily temperature cycles in the surfacing will cause cracking by aging before fatigue occurs due to traffic loading. (Note that, for presentational purposes, the loading cycles and daily cycles have been normalized with respect to strain levels and the fatigue life.)

The relative influences of traffic and aging are therefore likely to vary considerably across pavements of differing surfacings, materials, and traffic, and also across climates, as these factors influence the relative position of the functions shown. Empirical models must represent such interaction and be adaptable, through parameterization or calibration, to such conditions.

5.2.5 Reflection

Reflection cracking occurs when cracking in an underlying layer propagates upwards through the surfacing. Thus it may take the form of any of the main types of cracking. Reflection occurs as a result of stress concentration at the tip of an internal crack or flaw, which reduces the available fatigue life of the surfacing considerably. An engineering rule of thumb estimates the rate of cracking reflection as equivalent to 20 to 50 mm of surfacing thickness per year.

5.2.6 Other Types of Cracking

Cold-temperature cracking results from the combination of thermal contraction and high binder stiffness at very cold temperatures. In essence it is an environmental fatigue process, alike in many respects to the aging-related fatigue mentioned earlier. It takes the form of transverse, longitudinal or map cracking, determined largely by material characteristics and the temperature regime.

Longitudinal and transverse cracking also develop through shrinkage in cemented base materials (including cement- or time-treated materials some slag materials, and naturally-cementing materials such as calcretes, ferricretes, laterites, etc.). Cracks occur at spacings of typically 3 m but varying from 1.5 to 12 m depending on the tensile strength (increases spacing and crack width) and daily or seasonal temperature range (decreases spacing or increases width).

Longitudinal cracking near the pavement edge commonly results from moisture movements through the shoulder; attention to drainage and surface sealing of the shoulder are the best remedies. Settlement of the foundation or embankment may cause either a longitudinal or a long curved, crack.

Given the many variables involved, it is generally not practicable to model these types of cracking for network analysis and they are not considered further here. Examples for transverse and longitudinal cracking are given by Lytton and others (1982).
5.3 MODELLING METHODOLOGY

5.3.1 Previous Model Forms

Combined phases

Most previous prediction models for surfacing distress have combined the initiation and progression phases in a single function, and examples of the general forms adopted follow (the detailed functions are presented in Appendix A).

In the TRRL Kenya study (Hodges and others 1975) the combined areas of cracking and patching were modelled by:

\[(C + P) = a_0 \text{NE} - a_1 [1 + a_2 e^{-a_3 n(\text{NE})}]\] (5.8)

where \(C + P\) = sum of areas of cracking of intensity exceeding 5 m²/m², and patching (of depth surfacing plus basecourse), in m²/km per lane;

\(\text{NE}\) = cumulative equivalent standard axle loads since most recent surfacing, in million ESA;

\(n(\text{NE})\) = arithmetic function of \(\text{NE}\); and

\(a_i\) = coefficients estimated for classes of pavement strength (e.g., \(2.75 < \text{SNC} < 3.25\) and \(3.25 < \text{SNC} < 3.75\)).

In the Texas study (Lytton and others 1982), cracking was modelled by the dimensionless score:

\[S = e^{-(\rho/\text{NE})^\beta}\] (5.9)

where \(S\) = decimal severity score of cracking, ranging in value from 0 at no cracking, to 1 at the maximum defined score; and \(\rho, \beta\) = functions of pavement and environmental parameters, where the value of \(\beta\) determines the shape of the sigmoidal curve (see Figure 5.1) and hence also determines the period elapsing until effective initiation and the rate of progression, simultaneously.

The disadvantage of these model forms is that they constrain the rate of progression by the time to initiation of distress. Typically, for example, the Texas model forces long-surviving pavements to have a rapid rate of progression once cracking initiates. Furthermore, it is not easy to parameterize such forms so that changes in the pavement structure, traffic flow or maintenance treatment will cause the predictions of both initiation and progression to respond correctly.

Separate phases

Other prediction models have comprised separate predictions for the initiation and the rate of progression of cracking, for example as follows (see Appendix A for further details).

In the Brazil-UNDP study (Queiroz 1981, Volume 7 in GEIPOT 1982), cracking initiation was modelled as a function of traffic loading and strength, i.e.:

\[\text{NE}_c = a \text{SNC}^b\] (5.10)
and progression was estimated to be a function of both structural and age parameters, namely:

\[ CR_2 = a + b \, S \log NE' + c \, S \, AGE \, \log NE' \]  

(5.11)

where 
- \( NE_C \) = cumulative number of equivalent standard axles at initiation of class 2 cracking since surfacing, ESA;
- \( CR_2 \) = area of all cracking, in percent of pavement area;
- \( NE' \) = number of equivalent axles passed since initiation of cracking, ESA;
- \( S \) = a pavement strength index (variously Benkelman beam deflection, Dynaflect deflection or structural number in alternative models);
- \( a, b, c \) = estimated coefficients; and
- \( AGE \) = age of pavement since surfacing, years.

In the Arizona ADOT study (Way and Eisenberg 1980), cracking initiation was given by a tabulated value of expected age, and progression was found to be a function of the area of cracking, the previous rate of cracking progression, and the regional climatic factor, namely:

\[ ACR(t) = f(ACR(t-1), CR(t), RG) \]  

(5.12)

where 
- \( ACR(t) \) = increment of cracking area in year \( t \), percent of pavement;
- \( CR(t) \) = area of cracking at beginning of year \( t \), percent of pavement; and
- \( RG \) = a regional climate index.

The advantages of models which separate the predictions of initiation and progression is that these can be estimated independently. This is a form that is readily applicable to pavement management applications, where road condition data that is available from network monitoring can be used to update and improve the predictions, for example by eliminating the initiation prediction if cracking is already present. They are also much more readily adaptable to the prediction of maintenance effects.

### 5.3.2 Current Study: General

The approach adopted for the current study was to model the initiation and progression phases separately, for the reasons just enumerated. Following the concepts presented in Figure 5.1, the continuous numeric \( CR_i \) of cracking area was selected as the progression parameter. Since this could involve separate initiation and progression models for all four levels of severity \( i \), these were rationalized to two (omitting hairline cracking as discussed in Section 5.1.2), namely:

\[ CR_2 = \text{area of all cracking (narrow and wide, classes 2, 3 and 4) in percent of pavement area;} \]
\[ CR_4 = \text{area of wide cracking (class 4), in percent of pavement area.} \]

A further rationalization to a single level of weighted severity is possible through the numeric of "indexed cracking," \( CR_X \), as defined in Equation 5.3. The initiation of \( CR_X \) is coincident with that of \( CR_2 \) and the progression of \( CR_X \) lies numerically between those of \( CR_2 \) and \( CR_4 \), as shown in Figure 5.9. In order to
ensure internal consistency between these relationships the \( CR_4 \) relationships were generally made dependent upon the \( CR_3 \) data; in any event, \( CR_3 \) cannot be less than \( CR_4 \).

In view of the wide differences in fatigue and aging behavior evident for different materials and pavement types, pavements were categorized so as to separate original and maintenance surfacings, hot-mixed asphalt concrete and surface treatments, and flexible and semirigid bases, as detailed in Section 5.3.4.

Secondly, the interactive effects of fatigue and aging were considered to be of potentially dominating importance, given previous experience in pavement performance analysis, and particularly given the predominance of thin- to medium-thickness pavement surfacings in the majority of developing countries of low- to moderate-density population areas. Thus a conventional fatigue model form, such as Equation 5.6, is likely to be inadequate and need to be modified by aging effects. This, coupled with the desire to take full account of the inherent variability of material behavior, evident both in laboratory experiments and field data, led to the search for appropriate model forms and analytical methods for developing probabilistic predictive models.

5.3.3 Probabilistic Failure-time Modelling

The initiation of distress such as cracking is a discrete but highly variable event. That is, cracking will occur at different times at various locations along a nominally homogeneous road. We term the first of these times the initiation of cracking. Another pavement of nominally identical properties and traffic will have initiation at yet a different time. The time, or age of the surfacing, at "failure" (here defined as the appearance of distress) thus varies in the real world, even when given nominally identical conditions. Thus not all
variations observed in experimental data will be accountable to physical parameters and some need to be quantified as stochastic variations. These stochastic variations can be represented by the probability $p$ that cracking will occur at a certain time (or age), given that it has not occurred previously.

The fact that cracking occurs only after a discrete time since construction of the surfacing creates another difficulty in analyzing experimental data. Typically, when a uniform cross-section of different pavements with a range of different ages, strengths and loading conditions are observed over a period of time (or "window"), the actual timing of cracking initiation will not be observed in every case, as illustrated in Figure 5.10. Some of the surfacings may have cracked before the study began (e.g., $T_1$), and some may only crack later after the study has finished (e.g., $T_3$). This is a statistical problem of "censored data". Exclusion of those sections for which the actual event was not observed can bias the analysis one way or another.

The physical mechanisms of fatigue and aging have the combined effect of increasing the chance that failure will occur as time passes, given that failure has not occurred previously. In other words, failure is not a random event but becomes increasingly likely as time and traffic pass.

In order to take account of all these aspects, a special estimation procedure, based on the principles of failure-time analysis, was developed for analysing distress data and generating predictive models. The procedure, originally developed to study the reliability of industrial components, uses a maximum

**Figure 5.10:** The problem of censored data: unobserved events of distress initiation occurring either before or after the observation period

![Graph showing the problem of censored data](image-url)
likelihood estimation so that both observed and censored data can be exploited, and assumes that the stochastic variability of failure times follows a Weibull distribution. The Weibull distribution has the particular virtues that it is flexible in shape (varying in width of dispersion) and that it represents the increasing "hazard" of failure indicated by the physical mechanisms. Details of the concepts, the maximum likelihood estimation model, and the characteristics of the Weibull distribution are presented in Appendix B.

In the probabilistic model estimations which follow, the shape of the probability distribution and survivor function is determined by the parameter $\beta$ as shown in Figure 5.11(a), and the expected (or mean) time to failure, $E(T)$, is expressed in the form:

$$E(T) = B(\beta) e^\gamma$$

(5.13)

where $B(\beta)$ = a factor which is a function of $\beta$ as illustrated in Figure 5.11(b); $eta$ = a parameter estimating the stochastic variation of failure times for nominally identical pavements, from the data; and \( \gamma \) = an estimated function of explanatory variables such as pavement and traffic characteristics $(x)_i$, e.g.: $\gamma = a_0 + a_1 x_1 + ... + a_n x_n$.

The exponential form of the $\gamma$ function was chosen as a convenient nonnegative function, but alternative forms could be adopted provided they were nonnegative.

The probable time to failure $T(p)$, for any probability $p$ that failure will occur by time $t$ given that it has not occurred previously, can be expressed as a function of the expected failure time, as follows:

$$T(p) = K(p) E(T)$$

(5.14)

where $K(p)$ = a factor which is a function of the parameter $\beta$ and $p$, as illustrated in Figure 5.11(c) and defined in Appendix B.

In the case where none of the observations is censored, that is all failure events were observed, the results of the model estimation for the expected life are the same as obtained for the mean life by using least-squares regression analysis.

Evaluating the goodness of fit and accuracy of probabilistic models obtained from censored data is somewhat complicated and also an open research question, as discussed in Appendix B. In this instance, three criteria were used as follows:

1. For the goodness of fit, the average log likelihood (ALL) (the log likelihood (LL) divided by the number of observations), is the best normalized measure of fit for models having a common dependent variable but possibly differing numbers of observations within a common data set. It is not comparable for different dependent variables however.

2. The estimation (or prediction) error is represented by the average confidence intervals (ACI) of an observation, which are the 95th percentile confidence intervals of the dependent variable in the
Figure 5.11: General trends of functions of the Weibull distribution parameter $\beta$ in probabilistic failure-time models of surfacing distress

(a) Survivor functions

(b) Factor $B(\beta)$

(c) Factor $K(p)$ relating probable time of failure, $T(p)$, to expected time, $E(T)$

(d) Semi-interquartile factor, SIQF, indicating dispersion of failure times

Note: $ps = 100(1-p)$; $p =$ probability of failure.

Source: Appendix B, Equations B.9a, B.10b, B.13b, and B.15.
analysis, and expressed in the units of the dependent variable, for example ± 1.2 years. This is an imperfect measure because the value depends on both the scale of the explanatory variables and the errors of estimate of their coefficients.

3. The stochastic variation, or the width of dispersion, of failure times for nominally identical conditions is represented by the "semi-interquartile factor" (SIQF), which is half the difference between the 25th and 75th percentile values of the expected life (as a fraction), i.e.

\[ \text{SIQF} = \frac{[K(0.75) - K(0.25)]}{2}. \]

Thus, for example, when SIQF = 0.30, fifty percent of the observations are likely to be within about 30 percent of the expected time, or a range of approximately 0.70 E(T) to 1.30 E(T). As illustrated in Figure 5.11(d), the dispersion indicated by SIQF generally narrows as \( \beta \) increases.

The objective of the modelling was to maximize the average log likelihood and to minimize the average confidence intervals and semi-interquartile factor. In respect of the SIQF, the minimization ensured that the dispersion included the least possible lack-of-fit from the model and instead was mainly true stochastic variation.

5.3.4 Characteristics and Processing of Data

The primary data base used for estimating predictive models was the Brazil-UNDP study (GEIPOT 1982). The cracking numerics \( \text{CR}_2 \) (all cracking, narrow and wide), \( \text{CR}_4 \) (wide cracking) and \( \text{CRX} \) (indexed cracking) were computed from observed areas of classes 1, 2, 3 and 4 cracking which had been recorded at four to six month intervals during the study.

Type of cracking

The types of cracking included in the numerics were crocodile and irregular cracking because these are the most relevant to the prediction of deterioration under traffic and aging. In practice, it is often difficult to distinguish between traffic-related crocodile cracking and aging-related irregular cracking at the early stages because the crack lengths are small and often not interconnected; this was evident in the Brazilian data when the classifications changed between one survey and the next. For the same reasons, when the classification of block cracking on cemented base pavements was occasionally juxtaposed with crocodile cracking, both types were included in the numerics as being relevant.

Line cracking, both longitudinal and transverse, was uncommon in the study pavements, except for the cemented base pavements. Thus line cracking was excluded from the numerics, except in the cases where it proved later to develop into crocodile cracking, in which case the area was computed as a 0.5 m width times the crack length.
Initiation

Cracking initiation was defined by a cracking area of 0.5 percent for practical reasons. First this helped to ensure that the cracking was not due to a local flaw unrepresentative of the pavement as a whole but was caused by the mechanism of interest. Second, it represents the minimum size (about 5 m² on the standard 320 by 3.5 m subsection) for which a condition survey observation could be expected to be reliably consistent. Because the surveys were made at intervals of four to six months, the first recorded observation was not always 0.5 percent. By convention, the initiation date was regarded as the survey date if the first observation was in the range of 0.5 to 5 percent, and as the previous survey date if the first recorded area was more than 5 percent. Refinement was considered unnecessary since the error in this approach is less than 0.2 years.

Traffic

When considering the time of cracking as the dependent variable in failure-time models, the effects of traffic loading must be expressed through the rate of trafficking, or traffic flow. The flow needs careful definition because of the effects of growth rates. Old pavements typically have current traffic flows that are much greater than existed when they were new, and to use current flow for both new and old pavements would introduce bias, especially given the wide range of ages, flows and growth rates present in the data. Thus a uniform definition of traffic flow was adopted, being the average annual traffic loading over the first eight years of the pavement's life, namely:

\[ YE_4 = \frac{NE_4(8)}{8} \]  

(5.15)

where \( YE_4 \) = indicative annual traffic loading, in millions ESA₄ per lane per year; and \( NE_4(8) \) = cumulative traffic loading until a surfacing age of 8 years, in millions ESA₄ per lane.

Eight years was chosen as broadly representative of the surfacing age at cracking initiation. \( YE_4 \) can be computed conveniently for any situation using the exponential approximation of growth, as follows:

\[ YE(t) = YE(0) e^{gt} \]  

(5.16)

When the initial loading rate \( YE(0) \) is known, then

\[ YE_{4t} = YE(0) (e^{8g} - 1) / (8g) \]  

(5.17)

and when only the current loading rate is known, then

\[ YE_{4t} = YE(t) e^{-g(t-8)} (e^{8g-1}) / (8g) \]  

(5.18)

where \( YE(t), YE(0) \) = annual traffic loading at age \( t \) and \( t = 0 \) years respectively; and \( g \) = average annual traffic growth rate, as a fraction.

Inference space

The six pavement/surfacing categories adopted for the analysis, and the incidence of cracking observed in each category for the sections in the
Brazil-UNDP study, are summarized in Table 5.3. Cracking was evident on about 90 percent of all subsections in each of the asphalt concrete, cemented base, and reseal categories and on about 75 percent of the asphalt overlay subsections. Only 44 percent of the surface treatment subsections showed signs of cracking however, even over the wide age range of up to 21 years, so this provides some prior evidence of good fatigue characteristics.

Table 5.3: Categorization of surfacing and pavement types and general incidence of cracking in the Brazil-UNDP study

<table>
<thead>
<tr>
<th>Pavement category</th>
<th>Code</th>
<th>Number of section - surfacing phases</th>
<th>Number of subsections</th>
<th>Fraction showing cracking</th>
<th>Range of surfacing age</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original Surfacing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt concrete on granular base</td>
<td>AC</td>
<td>30</td>
<td>86/100</td>
<td>1.6 to 23.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(2/2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surface treatment on granular base (chip seal)</td>
<td>ST</td>
<td>36</td>
<td>45/102</td>
<td>2.7 to 21.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SO</td>
<td>10</td>
<td>39/56</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(other)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cemented base with bituminous surfacing</td>
<td>CB</td>
<td>11</td>
<td>35/40</td>
<td>1.6 to 19.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(0/2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maintenance Surfacing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt overlays on granular base (cemented)</td>
<td>OV</td>
<td>23</td>
<td>48/64</td>
<td>0.2 to 15.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8 (cemented)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reseals (chip seal) on granular base</td>
<td>RS</td>
<td>7</td>
<td>11/14</td>
<td>0 to 4.0</td>
<td></td>
</tr>
<tr>
<td>Reseals (slurry seal)</td>
<td>SS</td>
<td>32</td>
<td>60/77</td>
<td>0 to 13.2</td>
<td></td>
</tr>
<tr>
<td>Totals:</td>
<td></td>
<td>149/1492</td>
<td>285/397</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1/ 74 sections had 4 subsections, and 42 sections had 2 subsections, each.
2/ The total number of section-surfacing phases (163) exceeds the total number of study sections (116), because sections fully resurfaced by overlay or reseal during the study have been included under the appropriate category separately for each surfacing phase. The sections "not analyzed" were excluded because the data were inadequate.

The ranges of values of the major explanatory variables are presented in Table 5.4 for each of the six pavement surfacing categories. The range of ages of the surfacings observed was wide for original surfacings, ranging from 2 to over 20 years, but was much narrower for maintenance surfacings, ranging from 0 to generally less than 10 years. The range of traffic flows was also very wide, ranging from 73 to 6,000 veh/day in two directions, and from 100 to 2.3 million ESA/lane/year in terms of equivalent axle loading. Structural properties of the pavement also ranged widely, with Benkelman Beam deflections ranging from 0.20 to 2.0 mm, and the modified structural number from 1.6 to 7.7. Thus the data base has the potential to provide a broad base for predictive modelling in all pavement categories, although the sample sizes are rather small, especially for cemented base pavements and maintenance surfacings.

5.4 INITIATION OF NARROW AND ALL CRACKING

5.4.1 Asphalt Concrete Original Surfacings

The asphalt concrete surfacings in the study were mostly thin surfacings, with thicknesses of typically 50 mm and generally less than 70 mm; only 3 of the 30 sections had thicknesses in the range of 80 to 100 mm (Sections 003, 161 and 162). The basecourse layer comprised either natural gravel, which was usually a slightly plastic fine sandy laterite, or crushed stone, which was usually quartzitic, nonplastic and medium to coarse with maximum stone sizes of 25 to 62 mm. The levels of compaction (95 to 101 percent intermediate compaction, i.e., 1300 kJ/m³) and strength (80 to 130 percent CBR at in situ conditions) were generally high. One section (112) had very weak base material of 50 percent CBR and cracked early; since the surfacing thickness was 63 mm, that section was a perfect example of the early fatigue failure behavior discussed in Section 5.2.2.

The asphalt materials all followed a dense-graded asphalt concrete specification, and laboratory test results were available on 18 of the 20 main study sections but none of the 10 maintenance study sections. In order to assess the adequacy of the binder contents in the asphalt mixes, the binder content data were normalized with respect to the "optimum" binder content (which was not given) using the following approximation (for nonabsorptive aggregates):

\[ BNO = 1 - \left( \frac{BC}{OBC} \right) \]

(5.19)

where
- \( BNO \) = normalized deviation from optimum binder content, fraction;
- \( BC \) = recovered binder content, percent of total mix by mass; and
- \( OBC \) = optimum binder content of the mix conforming to Marshall criteria for dense-graded mixtures, estimated by the approximate algorithm:

\[ OBC \approx 7.8 - 0.1 \times D95 \]

\[ D95 \] = maximum stone size of mix aggregate, in mm.

About 60 percent of the sections with data had binder contents below optimum, with an overall mean \( BNO \) of -0.05, or five percent below optimum and a wide range from -0.50 to +0.58.

The mix stiffness (RMOD) ranged widely from 1,300 to 5,500 MPa. These stiffness values had an unusual, positive correlation with the air void content of the asphalt mix, showing that high void contents (8 to 11 percent) were often
### Table 5.4: Ranges of values of various parameters for the analysis of the initiation of narrow cracking (CR₂) in Brazil Road Costs Study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Variable name</th>
<th>AC</th>
<th>ST</th>
<th>CB</th>
<th>OV</th>
<th>RS</th>
<th>SS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of sections</td>
<td>number</td>
<td></td>
<td>30</td>
<td>36</td>
<td>11</td>
<td>19</td>
<td>7</td>
<td>-</td>
</tr>
<tr>
<td>Number of subsections</td>
<td></td>
<td></td>
<td>100</td>
<td>102</td>
<td>40</td>
<td>64</td>
<td>14</td>
<td>77</td>
</tr>
<tr>
<td>Prior cracking</td>
<td></td>
<td></td>
<td>44</td>
<td>10</td>
<td>14</td>
<td>5</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>Observed cracking</td>
<td></td>
<td></td>
<td>42</td>
<td>35</td>
<td>21</td>
<td>43</td>
<td>11</td>
<td>58</td>
</tr>
<tr>
<td>Future cracking</td>
<td></td>
<td></td>
<td>14</td>
<td>57</td>
<td>5</td>
<td>16</td>
<td>3</td>
<td>17</td>
</tr>
<tr>
<td>Surfacing age</td>
<td>years</td>
<td></td>
<td>1.6-2.0</td>
<td>2.7-21.0</td>
<td>1.6-19.4</td>
<td>0.2-15.0</td>
<td>0-4.0</td>
<td>0-13.2</td>
</tr>
<tr>
<td>Cumulative traffic loading</td>
<td>MESA</td>
<td></td>
<td>0.0001-4.70</td>
<td>0.005-5.16</td>
<td>0.09-1.94</td>
<td>0.03-7.14</td>
<td>0.016-0.75</td>
<td>0.001-1.16</td>
</tr>
<tr>
<td>Traffic loading rate</td>
<td>MESA/lnes/yr</td>
<td></td>
<td>0.0001-0.72</td>
<td>0.0008-0.24</td>
<td>0.004-0.40</td>
<td>0.02-1.59</td>
<td>0.02-1.62</td>
<td>0.002-2.31</td>
</tr>
<tr>
<td>Traffic volume</td>
<td>veh/day</td>
<td></td>
<td>73-4800</td>
<td>100-2300</td>
<td>306-2600</td>
<td>360-6000</td>
<td>450-4500</td>
<td>320-4500</td>
</tr>
<tr>
<td>Surfacing total thickness</td>
<td>mm</td>
<td>Hₐ</td>
<td>20-103</td>
<td>20-50</td>
<td>10-40</td>
<td>37-187</td>
<td>43-75</td>
<td>20-236</td>
</tr>
<tr>
<td>Surfacing added thickness</td>
<td>mm</td>
<td>Hₐ</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>30-180</td>
<td>10-30</td>
<td>3-10</td>
</tr>
<tr>
<td>Deflection (Messenhelm Beam, 80 kN)</td>
<td>mm</td>
<td>DEF</td>
<td>0.27-1.88</td>
<td>0.26-2.02</td>
<td>0.20-1.03</td>
<td>0.21-1.70</td>
<td>0.87-1.90</td>
<td>0.29-1.37</td>
</tr>
<tr>
<td>Modified structural number</td>
<td>mm</td>
<td>SNC</td>
<td>1.55-7.66</td>
<td>2.93-5.15</td>
<td>2.06-4.65</td>
<td>3.75-8.10</td>
<td>3.10-3.49</td>
<td>2.71-7.72</td>
</tr>
<tr>
<td>Resilient modulus of surfacing</td>
<td>GPa</td>
<td>RMOD</td>
<td>1.0-5.5</td>
<td>1.6-2.7</td>
<td>-</td>
<td>0.4-4.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Resilient modulus of cement base</td>
<td>GPa</td>
<td>CMOD</td>
<td>-</td>
<td>-</td>
<td>2.9-25.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CBR of base</td>
<td>%</td>
<td>CBR₂</td>
<td>38-165</td>
<td>32-143</td>
<td>-</td>
<td>48-187</td>
<td>58-165</td>
<td>49-203</td>
</tr>
<tr>
<td>Deflection (Dynaforce)</td>
<td>0.001 in</td>
<td>DMO</td>
<td>0.54-1.31</td>
<td>0.42-1.44</td>
<td>-</td>
<td>0.35-1.60</td>
<td>-</td>
<td>0.58-1.59</td>
</tr>
<tr>
<td>(deviation from optimum)</td>
<td>(13/30)</td>
<td>(13/30)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>(27/58)</td>
</tr>
<tr>
<td>Binder content</td>
<td>fraction</td>
<td>BNO</td>
<td>-0.50-0.58</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(deviation from optimum)</td>
<td>(19/30)</td>
<td>(19/30)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

**Note:** Where some values of a variable were missing on the data files, the number of sections for which values were known is indicated in parentheses below.

**Source:** Brazil-UNDP data.

Related to high stiffness values, shown by the following relationship observed in the data by Queiroz and Visser (1978):

\[
\log_{10} \text{RMOD} = 2.34 + 0.25 V - 0.016 V^2
\]  

(5.20)

where \( V \) = volume of air voids in asphalt mix, percent; and \( \text{RMOD} \) = resilient modulus at 30°C measured by indirect tensile test, in MPa. (Note: The approximate temperature sensitivity of the stiffness was a 3.5-fold increase in stiffness for a negative 10 degrees Celsius temperature change. The constant term has been adjusted from 3.85 to convert stiffness from kgf/cm² to MPa).

This indicates that significant hardening of the binder had occurred in the high-void mixes, because the trend is the opposite of the trend of stiffness with voids in fresh mixes. In many cases (e.g., sections 104, 113, 119 and 168) this hardening was probably due to oxidation, as those surfacings were 9 to 18 years old, but
in other cases (e.g. section 112) where the sections were relatively young, the hardening had probably occurred during construction.

From this preamble it is evident that the effects of aging are strong in the data and may have significant effects on cracking initiation. Indeed this is evident in a simple plot of the observed cracking initiation times against the annual traffic loading, in ESA per lane per year, shown in Figure 5.12, separately for weak pavements (deflection more than 0.6 mm) in chart (a), and for stiff pavements (deflection not more than 0.6 mm) in chart (b).

It is evident that cracking occurred within 10 years of the surfacing construction, in all but two cases, with a mean of 7.1 years; these are relatively short lives compared with usual expectations for asphalt surfacings. More importantly, the effect of traffic loading rate on the initiation time appears to be weak and much less than the strongly negative slope which was to be expected from classical fatigue considerations. Traffic effects become more apparent when the cumulative loadings are related to strength; positively to structural number in (c), and negatively to surface deflection in (d). The correlations are poor, however, probably due to the aging effects.

Estimation of models

Using the basic probabilistic failure-time model of Equation 5.13 and the maximum likelihood estimation procedure, a considerable number of combinations of the explanatory variables in Table 5.4 were evaluated in order to derive predictive models. The dependent variable being predicted (the "failure time") was either real time (the surfacing age in years when cracking initiation appeared), or cumulative traffic (the number of axle loadings in ESA or all axles up to the time of cracking initiation).

A selection of seven models obtained for predicting the expected (or "mean") initiation of cracking is presented in Table 5.5, and others are given in Appendix C. In the case of the real time models, the essential explanatory variables were a strength parameter and the rate of loading (annual traffic), and it was found necessary that these interact directly (as in Equations 5.21 and 5.23) rather than indirectly (as in Equation 5.24) in order for the trends of predictions to be reasonable over the full range of each variable. In the case of the cumulative traffic models, a strength parameter was the only essential variable, but the fit was greatly improved by the inclusion of the rate of aging (in terms of years per axle - again normalized by a strength parameter). The modelling process, in fact, proved to be rather difficult - not so much in achieving a fit,

1/ Since these and subsequent charts include censored data, guidance is needed to interpret them. The initiations actually observed (i.e., \( T_2 \) in Figure 5.10) are shown by the ■ symbol. The censored "prior" events, where cracking had occurred before the study, are shown by the ○ symbol - in this case the time shown is the beginning of the study (\( S_0 \) in Figure 5.10) and the real initiation time (at \( T_1 \) in Figure 5.10) lies somewhere below the symbol on the chart. The censored "future" events, when cracking had not yet occurred, are shown by the + symbol - the time shown is the end of the study (\( S_1 \) in Figure 5.10) and the real initiation time (\( T_3 \) in Figure 5.10) lies somewhere above the symbol on the chart. Thus in the ideal correlation, all ○'s should appear above ■'s, and all the +'s should appear below.
Figure 5.12: Relating observations of the initiation of narrow cracking to traffic and pavement parameters: asphalt concrete original surfacings

(a) Time and loading rate for weak pavements, DEF > 0.6mm
(b) Time and loading rate for stiff pavements, DEF ≤ 0.6mm

(c) Cumulative traffic loadings and modified structural numbers
(d) Cumulative traffic loadings and surface deflection

Note: ■ First crack observed; 0 Cracked earlier than observed; + Not cracked when last observed.
Source: Brazil-UNDP study data.
as in preserving dimensional consistency in both time and axles and in achieving satisfactory trends over the full range of interest.

The size of the data set also proved to be inadequate for more than about three explanatory parameters (there being only 40 fully independent sections among the total 100 subsection observations), and the final models show this in their comparative simplicity. Many of the influencing factors identified earlier in the review (Sections 5.2 and 5.3) were found to have identifiable effects in the data when considered along with the primary variables, but it was not possible from this data set to combine them all together. The effects of surfacing thickness (Equation 5.24) and binder content (Equation 5.23), for example, were quantifiable, but those models do not behave well over the entire range of interest and need to be interpreted with care and with respect to the average properties of the materials and pavements represented in the data.

**Thickness and durability**

The quadratic form of surfacing thickness in Equation 5.24 produces convex curve shapes that are extremely similar to those shown in Figure 5.6, and the minimum life occurs at a thickness of 60.5 mm, which is exactly the same range (60 to 70 mm for the relevant mix stiffness of 2 to 5 GPa) as found in the theoretical research (Section 5.2.2). At extreme thicknesses, and for extreme combinations of SNC and annual traffic loading (\(\text{YE}_4\)), however, the predictions become unreasonable in some cases. The predicted life for a 60 mm thick surfacing with SNC = 4 and traffic of 0.3 million ESA/lane/year is only 2 years by this model. The extreme brevity of the life predicted is due in large part to the fact that two study pavements in this range had very weak base layers (sections 026 and 112 with 55 and 38 percent CBR respectively) and did crack within two to three years. Although the effect of base stiffness is thus also evident in the data, the estimation of a suitable model which included base stiffness as an additional parameter (and which would therefore be more generally applicable) could not be achieved. Thus, while Equation 5.24 does not give suitable predictions for general planning applications, it does give strong empirical evidence in support of the theoretical contention that medium thickness layers are more fragile than either thin or thick layers.

The effect of binder content and film thickness on durability is evident in the model of Equation 5.23. The model indicates for example that a ten percent change of binder content with respect to optimum will result in about twelve percent change in the predicted life. Other models, with different combinations of parameters, indicated still stronger effects on fatigue properties, with the same ten percent change causing thirty to sixty percent change in the predicted number of ESA's carried before cracking initiation. Thus, while the estimation of the size of the effect is rather imperfect due to the omission of other variables and interactions, the beneficial effects of aiming for the highest feasible binder content in the surfacing material are strongly evident both in the data and this particular model (feasible here means consistent with other controls on material properties such as stability, voids, etc.).

**Predicting time of initiation**

The best planning models for predicting the initiation time were Equation 5.21 and 5.22, since these require only SNC and \(\text{YE}_4\) (the annual loading). As illustrated in Figure 5.13(a) and (b) respectively, both show a strong decay of
Table 5.5: Probabilistic predictive relationships estimated for the expected age and traffic at initiation of narrow and all cracking: original asphalt concrete surfacings

<table>
<thead>
<tr>
<th>Equation</th>
<th>Model parameters and estimates(^1)/</th>
<th>Model statistics(^2)/</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ALL</td>
</tr>
<tr>
<td><strong>Surfacing Age at Initiation</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.21 (\text{TY} _\text{cr}^2) = 4.21 \exp(0.139 \text{SNC} - 17.1 \text{YE}_4 / \text{SNC}^2))</td>
<td>(6.9)</td>
<td>(4.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[96]</td>
</tr>
<tr>
<td>5.22 (\text{TY} _\text{cr}^2) = 8.61 \exp(-24.4 \text{YE}_4 / \text{SNC}^2))</td>
<td>(16)</td>
<td>(2.5)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[96]</td>
</tr>
<tr>
<td>5.23 (\text{TY} _\text{cr}^2) = 8.38 \exp(1.21 \text{BNO} - 18.6 \text{YE}_4 / \text{SNC}^2))</td>
<td>(18)</td>
<td>(3.9)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[72]</td>
</tr>
<tr>
<td>5.24 (\text{TY} _\text{cr}^2) = 16.5 \exp(-.098 \text{HS} + 0.00081 \text{HS}^2))</td>
<td>(4.0)</td>
<td>(4.3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[96]</td>
</tr>
<tr>
<td><strong>Cumulative Traffic Loadings at Initiation</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.25 (\text{TE} _\text{cr}^2) = 0.0362 \text{SNC}^{2.65} \text{e}^{-0.143 \text{SY}})</td>
<td>(7.9)</td>
<td>(8.5)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[96]</td>
</tr>
<tr>
<td>5.26 (\text{TE} _\text{cr}^2) = 0.626 \text{DEF}^{-1.92} \text{e}^{-0.028 \text{DY}})</td>
<td>(1.7)</td>
<td>(-4.4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[96]</td>
</tr>
<tr>
<td>5.27 (\text{TE} _\text{cr}^2) = 0.0342 \text{EHM}^{-2.86} \text{e}^{-0.198 \text{EY}})</td>
<td>(4.5)</td>
<td>(-4.6)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[60]</td>
</tr>
</tbody>
</table>

Notes: \(\text{TY} \_\text{cr}^2\) = expected (mean) age of surfacing at initiation of narrow cracking, years;  
\(\text{TE} \_\text{cr}^2\) = expected (mean) cumulative traffic loadings at initiation of narrow cracking, million ESA;  
\text{SNC} = modified structural number;  
\text{DEF} = Benkelman beam deflection under 80 kN single axle load, mm;  
\text{YE}_4 = annual traffic loading, million ESA\_4/lane/year;  
\text{HS} = thickness of bituminous layers, mm;  
\text{BNO} = excess of binder content with respect to optimum, fraction;  
\text{EHM} = maximum tensile strain in surfacing, \(10^{-3}\); and  
\text{SY} = \text{SNC}^4 / (1,000 \text{YE}_4), provided that \(\text{SY} \leq 8\);  
\text{DY} = 1 / (\text{DEF} \text{YE}_4), provided that \(\text{DY} \leq 40\);  
\text{EY} = 1 / (\text{EHM}^4 1,000 \text{YE}_4), provided that \(\text{EY} \leq 6\).  
Method: maximum likelihood estimation procedure, as described in Appendix B.  
1/ Asymptotic t-statistics given in () below estimate.  
2/ ALL = average log likelihood; SIQF = semi-interquartile factor; CI = average confidence interval; \(\beta\) as defined in Section 5.3.3. [ ] = Sample size.  
3/ Failure predictions for any probability \(p\) are obtained by multiplying \(\text{TY} \_\text{cr}^2\) above by the factor \(K(p)\) from Figure 5.11(c) and Equation 5.14.  
Source: Estimates of Equation 5.13 on data from Brazil-UNDP study.
the life as the traffic loading rate increases, or as the pavement strength decreases. The impact of increasing the traffic however depends on the strength of the pavement, with thin (or "weak") pavements being the most affected. For example, doubling the traffic from 0.15 to 0.30 million ESA/lane/year on an SNC 2 pavement is predicted to reduce the life by about 50 to 65 percent, whereas doubling the traffic from 0.3 to 0.6 million ESA/lane/year on an SNC 8 pavement is predicted to reduce the life by only about 8 to 12 percent (or 1 year).

Both models also indicate strong effects of aging because at low traffic loading rates the life predicted is finite and does not tend toward infinity as would be suggested by the purely mechanistic fatigue models of Section 5.2.1. It is here however that differences between the alternative models is apparent. In the first (Equation 5.21), which gave the best statistical fit to the data and is used in the HDM-III model, strong pavements have a longer life than weak pavements, even under extremely light or negligible traffic loadings. This does not agree with the aging/oxidation mechanism quite as well as does the second model (Figure 5.13(b)), which shows a focal point, or common age, at which all surfacings would crack under nil traffic; an age which the theory indicates would depend upon the ambient temperature, binder composition (crude source and viscosity), binder film thickness and air voids in the mix. An advantage of the second form is that calibration of the prediction to other sets of climate and material parameters by applying a multiplying factor is transparently rational. On the other hand, the intercept differences apparent in the first model appear plausibly indicative of the sensitivity of different surfacing thicknesses to aging; for example the weaker pavements (SNC 1.5 to 4) generally have surfacings of 30 to 60 mm thickness which are more susceptible to aging through their full depth than are the thicker surfacings (60 to 100 mm) of the stronger pavements.

In searching for alternative parameters to explain the differences in failure time under light traffic from the first model, however, surfacing thickness alone was not found significant, but the binder content (relative to optimum) was, as shown in Equation 5.23. That third model improved the fit slightly over the second (though on a smaller data base); it is also plausible because it includes a measure of durability (BNO) and is a reasonable alternative predictive model to the first.

The first model, Equation 5.21, was finally selected for use in the HDM-III model, but the second and third models Equations 5.22 and 5.23, are reasonable alternatives and may prove to be more readily transferable to other regimes.

Predicting cumulative traffic to initiation

The three models shown by Equations 5.25 to 5.27 have similar forms that are based primarily on the mechanistic fatigue form of Equation 5.6, with one important difference. The influence of aging is included in the final term, in which the exponent is essentially a measure of the rate of aging (in years per million ESA, normalized by the pavement strength parameter). This has the effect of reducing the total traffic carried up to the time of initiation \( T_{Ecr} \) as the loading rate is diminished, because the aging reduces the available fatigue life.

The strongest model is the first, Equation 5.25, which again uses modified structural number as the pavement strength parameter and is illustrated in Figure 5.14(a). It indicates that doubling the strength (SNC) results in a sixfold increase in the traffic carried before failure; this order of strength effect
Figure 5.13: Predictions of expected age at initiation of narrow cracking: asphalt concrete original surfacings

(a) Model equation 5.21

(b) Model equation 5.22

Note: Semi-interquartile factor 0.320.

Source: (a) Equation 5.21; (b) Equation 5.22.
is fairly consistent throughout the three models and the data (and in other distress modes also). The published literature on fatigue suggests that this is indicative of fairly stiff, brittle materials, however, because laboratory results indicate much larger effects (sixteenfold or more) for flexible materials; in other words, values of $n$ (from Equation 5.6) of 4 or more, rather than the 2.6 to 2.9 estimated from these data. This lower value of $n$ is almost certainly another effect of aging being observed in the data. The aging term itself has the effect of predicting failure within about 9 years (similar of course to the time models) for the mean value.

The third model (Equation 5.27) is very interesting because of its direct relevance to the extensive experimental research on fatigue. It shows a clear relationship to the maximum principal tensile strain in the surfacing, Figure 5.14(b), with a slope of -2.9. The strain parameter, $EHM$, is the maximum of the surface and interface principal strains so that it applies as the "maximum" strain in both the thin and thick surfacing cases. During the analysis it was found that failure had negligible correlation with the interface tensile strain and a fair correlation with the surface strain; thus, as it was the maximum of these two strains that would govern behavior in every case, $EHM$ was defined in this way and proved to be an excellent predictor of cumulative traffic at failure. When the excellent agreement between this empirical model and the experimentally-based mechanistic design methods has been demonstrated (see "Validation"), the importance of this model in permitting the much greater refinements of material and load parameters to be applied to performance modelling will be appreciated. The important difference between this model and experimental fatigue models is the influence of the aging rate parameter, $EY$, which reduces the fatigue life.

Fit, error and stochastic variability

The goodness of fit of the strongest time and traffic models, Equation 5.21 and 5.25, is shown in Figures 5.15(a) and (b) respectively. In the case of the time model, the confidence intervals of the predicted mean are +1.07 years (the mean life in the data was 7.1 years) and the normalized error of the prediction is 7.7 percent. The model therefore fits the data very well and considerably better than the visual impression of the scatter in chart (a) may convey, because the scatter includes both lack of fit and the stochastic variability (probabilistic distribution of failure times). In this instance $SIQF = 0.320$, so that about half the data can be expected to lie within a band between 0.68 and 1.32 times the mean or "expected" life; clearly this accommodates much of the scatter that is evident. For example, for a ten-year expected life:

---

2/ The confidence intervals are the asymptotic 95th percentile values of the residuals. The normalized error is the estimated error of the mean expressed as a percentage of the mean predicted value.

3/ Interpreting scattergrams of these models is complicated because three factors must be considered. 1. The observed events ( ■) should lie close to the line of equality; 2. The censored "prior" events (0) should lie above, and the censored "future" events (+) should lie below, the line of equality; and 3. The remaining scatter is due not only to model error but also to the probabilistic distribution of values about the mean due to the intrinsic variability of material behavior.
Figure 5.14: Predictions of expected cumulative traffic loadings to the initiation of narrow cracking: asphalt concrete original surfacings

(a) Function of modified structural number

(b) Function of maximum tensile strain in surfacing

Note: Aging parameter SY defined in Table 5.5. Semi-interquartile factor = 0.467.

Note: Aging parameter EY defined in Table 5.5. Semi-interquartile factor = 0.545.

Source: (a) Equation 5.25; (b) Equation 5.27.
**Figure 5.15:** Goodness of fit for predictive models of the age and traffic at initiation of narrow cracking: asphalt concrete original surfacings

(a) Surfacing age model

(b) Cumulative traffic loadings model

---

**Note:**
- First crack observed: • Cracked earlier than observed; + Not cracked when last observed.

**Source:** (a) Equation 5.24 and (b) Equation 5.25 applied to Brazil-UNDP study data.

\[ T_{cr} = 10 \text{ years}; \]

The equivalent of "standard error" = 0.55 years (= CI/1.96);

The lower and upper confidence limits = 8.93 and 11.07 years;

25 percent of observations will have \( t < 6.8 \) years; and

25 percent of observations will have \( t > 13.2 \) years.

In the case of the traffic model, Equation 5.25, the confidence intervals are \( \pm 0.237 \) million ESA on a mean observed traffic life of 0.93 million ESA, so the approximate standard error is 0.12 million ESA and the normalized error is 14 percent of the mean. Thus the estimation error is about twice that of the time model. The fit of the model also appears worse from the scattergram in chart (b), because of the wide and skewed dispersion of observations about the predicted expected value. The skewness is characteristic of a log normal distribution and indicates that the dispersion would be nearly symmetrical for log cumulative traffic loading. This is consistent with the fact that a log-log transformation of the models in Equations 5.25 to 5.27 (and Figure 5.14) makes the base part linear, and consistent also with the form of the classical fatigue model. It was also confirmed in earlier models using log loadings as the dependent variable (see Table C.1), which had tight symmetrical dispersions with SIQF of about 0.05, but even that is equivalent to wide dispersions in the natural dimension with quartiles at factors of one-half and two respectively on the expected failure time (or deciles at factors of one-third and three respectively). This is similar to the dispersion found in controlled fatigue experiments so it appears to be a fair measure of the variability intrinsic in material properties and fatigue behavior.

Thus both the fit (in terms of the normalized error) and the uncertainties (as evidenced by the dispersions) are worse for the traffic models than the time models. For example, the lower and upper quartiles are respectively 0.65
E(T) and 1.30 E(T) for the time model, and 0.45 E(T) and 1.39 E(T) for the traffic model. Since planning and management applications must operate with the natural dimensions of time and traffic (and not log traffic), the time models are preferred, both because of the greater certainty in their predictions of the expected failure times and because they predict directly in the planners' dimension of time. For certain design and analytical applications, the traffic models are likely to be preferred because they represent the fatigue mechanisms more explicitly.

5.4.2 Surface Treatment Original Surfacings

The surface treatment surfacings in the study were mostly of double chip seal construction, but 14 of the 50 test sections had other kinds of surface dressings. The "chip seal" double surface treatments generally comprised a 14 to 19 mm stone size in the first seal coat and a 7 to 10 mm stone size in the second seal. The other surface dressings appeared to include a variety of types, including slurry seal, cold mix and sand seal, which could not be classified specifically and which were therefore analyzed separately from the chip seal double surface treatments (DST). The base layer usually comprised natural lateritic gravels, and in 5 of the 36 chip seal DST sections comprised crushed stone materials. The material properties of the crushed stone bases were similar to those in the asphalt surfacing sections, and those of the natural gravel bases were generally adequate but slightly inferior to the crushed stone.

The data observed for the times and traffic loadings to initiation of cracking are compared with the traffic loading rates in Figure 5.16. The range of observed life is great, the longest surviving surface uncracked was 21 years old and the earliest cracking occurred at 2.5 years. At first glance the expected trends are rather more obvious in these data than for the asphalt concrete surfacings (cf. Figure 5.12), but again a significant amount of scatter is present.

Estimation of models

Models were estimated, using the maximum likelihood procedure and the general model form given in Equation 5.13, for various combinations of the variables listed in Tables 5.3 and 5.4 and for various groupings of the data. An indicative selection of these may be found in Appendix C. The initial analysis showed clearly that the surface treatment category of surfacings had to be further subdivided to separate chip seals (conventional double surface treatments, DST) from other types, and that the construction quality classification was highly significant. Without these class definitions, no reasonable correlations were evident; in particular, the scatter about a mean failure time was large and poorly correlated with traffic, which appeared to have negligible damaging effects.

The analysis concentrated on the major group, chip seals (DST), because it was the only group large enough to allow explanatory factors to be determined and because it is the most important group economically. As for asphalt concrete surfacings, the traffic loading rate and pavement strength parameters were essential and had to be interactive in the models to obtain sensible trends. The construction quality indicator was also an essential parameter, but none of the other parameters available were found significant. This, of course, is not to say that other factors such as material properties were not relevant when considering the performance of individual sections, but only that these could not be identified.
Figure 5.16: Relating observations of the initiation of narrow cracking to traffic and pavement parameters: surface treatment original surfacings

(a) Time and loading rate

(b) Cumulative traffic loadings and modified structural number

![Graphs showing data points and trends.]  

**Note:** ■ First crack observed; ● Cracked earlier than observed; + Not cracked when last observed.

Source: Brazil-UNDP study data.

Predicting time and traffic to cracking initiation

The selection of appropriate predictive models is presented in Table 5.6. The time models have similar characteristics and comprise only a strength-traffic-quality interaction term, other parameters not being significant. The predictions, illustrated in Figure 5.17 show a strong decay of life as traffic loading increases or as pavement strength decreases. The modified structural number (Equation 5.28) and surface deflection (Equation 5.29) parameters of strength had similar explanatory powers, with the modified structural number being marginally better. The effect of aging is also prominent, with a maximum expected life under negligible traffic of a little over 13 years. This maximum applies to all pavements because the surfacing is thin and oxidation effects are independent of pavement strength. It is noticeable that the lives in the data, and shown by these models, are generally longer than those expected for asphalt concrete (Figure 5.13), which is empirical evidence of the advantages in durability (through thick binder films) and in flexibility or fatigue life (through the thin layer effect) of surface treatments when compared to asphalt concrete. The balance shifts in favor of asphalt concrete at the higher traffic loadings (more than 0.6 million ESA per lane per year), and flexible pavements surfaced with surface treatments rarely exceed SNC 5 in strength. Also modes of distress other than cracking (e.g., ravelling or bleeding) tend to restrict the satisfactory performance of surface treatments to the lower traffic loadings (the maximum in the Brazil-UNDP study was 0.24 million ESA/lane/year) and to volumes lower than about 4,000 veh/lane/day.
Table 5.6 Probabilistic predictive relationships estimated for the expected age and traffic at initiation of narrow and all cracking: original double surface treatments

<table>
<thead>
<tr>
<th>Equation number</th>
<th>Model parameters and estimates$^{1}$/</th>
<th>Model statistics$^{2}$/</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ALL</td>
</tr>
<tr>
<td><strong>Surfacing Area</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.28 $^{1}$TY$_{cr2} = 13.2 \exp[-24.3 (1 + CQ) \frac{YE_2}{SNC^2}]$</td>
<td>-1.269</td>
<td>0.295</td>
</tr>
<tr>
<td></td>
<td>(29) (2.8) [102] (7.4)</td>
<td></td>
</tr>
<tr>
<td>5.28a $^{1}$TY$_{cr2} = 13.2 \exp[-20.7 (1 + CQ) \frac{YE_4}{SNC^2}]$</td>
<td>-1.269</td>
<td>0.297</td>
</tr>
<tr>
<td></td>
<td>(25) (-2.7) [102] (7.4)</td>
<td></td>
</tr>
<tr>
<td>5.29 $^{1}$TY$_{cr2} = 13.6 \exp[-3.19 (1 + CQ) \frac{YE_2}{DEF}]$</td>
<td>-1.269</td>
<td>0.303</td>
</tr>
<tr>
<td></td>
<td>(27) (-3.5) [102] (7.8)</td>
<td></td>
</tr>
<tr>
<td>5.29a $^{1}$TY$_{cr2} = 13.6 \exp[-2.70(1+CQ)\frac{YE_4}{DEF}]$</td>
<td>-1.274</td>
<td>0.309</td>
</tr>
<tr>
<td></td>
<td>(-3.2) [102] (8.0)</td>
<td></td>
</tr>
<tr>
<td>5.30 $^{1}$TY$_{cr2} = 12.9 - 5.7 CQ$</td>
<td>0.265</td>
<td>1.55</td>
</tr>
</tbody>
</table>

| **Cumulative Traffic Loadings** |                                      |     |      |    |        |
| 5.31 $^{1}$TE$_{cr2} = 0.0072 SNC^{3.97} e^{-0.39 CQ}$ | -0.571 | 0.501 | 0.76 | 1.24 |
|                 | (3.7) (3.9) (-1.7) (25%) (8.1) |                        |
| 5.32 $^{1}$TE$_{cr2} = 1.13 DEF^{-1.06} e^{-0.92 CQ}$ | -0.578 | 0.519 | 0.73 | 1.16 |
|                 | (0.3) (-2.7) (-3.7) (23%) (8.5) |                        |

$^{1}$TY$_{cr2}$, TE$_{cr2}$, SNC, DEF, YE$_4$ are as defined in Table 5.5. CQ = construction quality indicator, where CQ = 0 if construction of the surfacing was not faulty, and CQ = 1 if construction was faulty (poor binder distribution, contaminated stone, early stripping of binder, etc.).

$^{2}$Model statistics are as defined in Table 5.5 and Section 5.3.3. $[\ldots ]$ = number of observations. () = t-statistics.

Source: Estimation of Equation 5.13 from Brazil-UNDP study data using maximum likelihood procedure.

These numbers apply to high quality surfacings, and any construction faults or low quality control results in significantly worse performance; surface treatments are less forgiving in this respect than asphaltic surfacings. The models indicate that the average reduction in life due to poor quality was in the order of 30 to 50 percent in the study data. The simplest model of all, Equation 5.30, indicates average lives of 12.9 and 7.2 year for good and poor quality surface treatments, respectively.

The cumulative traffic models in Equation 5.31 and 5.32 are similar in form to those for asphalt concrete, except that reasonable effects of aging were unable to be estimated. Although the powers of the strength parameters are rather different, the true values in both cases are likely to be intermediate to the extremes, that is, about 3.2 for SNC and -1.6 for DEF. The predictions of the SNC model range from 0.11 million ESA$_4$ for SNC 2 to 4.2 million ESA$_4$ for SNC 5...
Figure 5.17: Predictions of expected age at initiation of narrow and all cracking: double surface treatment original surfacings

(a) Time model with modified structural number

(b) Time model with surface deflection

Note: Semi-interquartile factor 0.297.

Note: Semi-interquartile factor 0.309.

Source: (a) Equation 5.28; (b) Equation 5.29.
pavements of good quality, and forty percent less than these lives for poor quality surfacing construction.

The two versions of each time model, Equations 5.28 and 5.28a (and 5.29 and 5.29a), represent different damaging powers of the axle loading, computing the equivalent standard axle loadings with a power of 2 and 4 respectively. There is little difference between the fits of the two models, although the power of 2 is very slightly better and thus provides slender support for the contention that the damage on thin surfacings is less sensitive to axle loading than indicated by the traditional fourth-power law (this is discussed in depth in Chapter 9). Either can be used satisfactorily for prediction purposes, and typically the fourth-power version (Equation 5.28a) is likely to be preferred because the ESA values are more readily available.

Fit, error and stochastic variability

The goodness of fit of the time and traffic predictive models, shown in Figure 5.18, is similar to that achieved for asphalt concrete. The confidence intervals appear slightly worse because the lives are longer, but the normalized prediction error of 7.5 percent of the mean life (11.6 years predicted) is similar. The variability here is also similar, though slightly less in the time models, with the quartile values being ± 30 percent from the expected life; the dispersion is shown superimposed on the scattergram in Figure 5.18(a) as a means of illustrating how much of the scatter was attributed to variability.

It can be seen that the models generally fit the data well, but it is also clear that the variability of performance is considerable and accounts for most of the remaining scatter. In the time model, the predictions have an upper-bound determined by the predicted age for negligible traffic at 13.2 years; this appears to truncate or distort the scatter in the figure, but this is because the observed values include the variability whereas the predicted mean values (expected lives) do not.

5.4.3 Thin Surfacings on Cemented Base

Cemented base pavements in the study comprised mainly cement-treated lateritic gravel base construction with a wide range of in situ material strength, the resilient modulus ranging from under 3 GPa to 25.5 GPa, and one section of lime-treated base construction. Since the predictive models use mechanical properties of materials, such as the resilient modulus, as explanatory parameters, the models are expected to be broadly representative of all types of base cementation, including both cement and lime-treatment and natural cementation such as occurs in some laterites, calcrites and ferricretes, (noting however that the speed of the natural cementation process may have a significant effect on the time to cracking initiation). Only pavements with original surface treatments were analyzed, but there was some variety in the type of treatment and the surfacing thickness ranged from 10 to 40 mm. The cracking observed ranged from unconnected irregular cracking to strongly-defined crocodile or "ladder-cracking" in the wheelpaths in different cases.

The observed data show strong trends between the time to cracking and the traffic flow (as presented in Figure 5.19), and between the cumulative traffic loading and deflection. The model estimation (examples are given in Appendix C)
Figure 5.18: Goodness of fit of model predicting initiation of all cracking on Brazilian data: double surface treatment original surfacings

The best model estimated for predicting the initiation of crocodile cracking in cemented base pavements was:

\[ T_{cr2} = 1.11 \exp(0.035 \text{HS} + 0.371 \ln \text{CMOD} - 0.418 \ln \text{DEF} - 2.87 \text{YE}_s \text{DEF}) \]  

(5.33)

where \( T_{cr2} \) is the time to cracking, \( \text{HS} \) is the thickness of bituminous surfacing, mm; \( \text{CMOD} \) is the resilient modulus of cemented base, GPa; \( \text{DEF}, \text{YE}_s \) are defined in Table 5.5.

The predictions of the model are depicted in Figure 5.20. These show a decreasing effect of traffic loading rate as the deflection decreases, that is as
Figure 5.19: Relating observations of the initiation of narrow cracking to traffic and pavement parameters: thin surfacings on cemented base

(a) Time and loading rate

(b) Cumulative traffic loadings and surface deflection

Note: ■ First crack observed; 0 Cracked earlier than observed; + Not cracked when last observed.
Source: Brazil-UNDP study data.

The pavement stiffness increases. An increase of base modulus from say 5 to 15 GPa (for example by increasing the cement content) effects an approximately 50 percent increase in the time to cracking for given values of deflection and traffic loading rate. In reality, an increase in the stiffness of the base for a given pavement will also reduce the deflection, so that the real increase in pavement life would be greater than 50 percent and approaching 100 percent in some cases. An increase of 10 mm in the surfacing thickness effects a 35 to 40 percent increase in time to cracking. Surfacing thickness can be expected to attenuate the time to cracking for these pavements because traffic-associated cracking initiates in the base and propagates upward through the surfacing; this is true particularly for surfacings with highly ductile binder and thick binder film.

It should be noted here that the model is not valid when a strain-relieving interlayer is present between the base and surfacing. An interlayer may either occur naturally when a prime coat inhibits cementation in the top of the base and forms a thin separation layer of powder under the surfacing (Paterson and Marais 1980), or be incorporated by design in the form of a membrane at the time of construction (note that the effectiveness of membrane interlayers varies considerably with the type of membrane).

The fit of the model to the data and the prediction error are similar to those achieved for both flexible pavement surfacings. The variability estimated was much less than in the previous cases, with the interquartile range of ±16 percent being only one-half of the previous values; however this result is
Figure 5.20: Prediction of expected age at initiation of narrow and all cracking in semirigid pavements: original thin surfacings on cemented base

(a) Related to annual traffic loading

(b) Related to modulus of cemented base

Note: Semi-interquartile factor = 0.160.
Source: Equation 5.33.
probably an underestimate because the sample size was only one-third of that in the other analyses (i.e., 22 section-traffic combinations). The low variability makes the fit appear to be extremely good in the scattergram comparing observed and predicted values, Figure 5.21.

5.4.4 Asphalt Overlays

Although the Brazil study included 32 asphalt overlay sections (93 subsection observations), and about 75 percent of those had cracked, analysis of the data was limited by the fact that the condition of the underlying, original pavement surface and the overlay thickness were unknown in all but 16 of the 93 subsection observations. As the mechanism of reflection cracking is known to be an important cause of cracking in overlays, this was a severe limitation, and it was handled by testing alternative deductions about the previous condition, from the current age and condition, and deducing the overlay thickness from thickness data. Overlay thicknesses ranged from 50 to 125 mm, and the pavement age at the time of overlay from 4 to 16 years; other ranges were as given in Table 5.4.

Models developed from the data were therefore kept in as simple a form as possible, in order to quantify the major effects of pavement deflections, traffic and overlay thickness (see Appendix C). Of these the two most useful models with reasonable statistics are presented in Table 5.7.

The model in Equation 5.34 indicates a nearly 50 percent reduction in life per million equivalent standard axles per lane per year at a deflection of
0.5 mm, which is about the same sensitivity to traffic as found for asphalt concrete original surfacings and about one-third of the sensitivity found for surface treatments. An increase in deflection of 0.2 mm causes an approximately 30 percent reduction in the time to cracking. The data also showed clearly that wide cracking in the existing pavement prior to overlay also advanced the time to cracking, but the scarcity of the data did not permit reliable determinations; the model in Equation 5.35 was the most representative of the reflection cracking phenomenon but does not include traffic effects, and the confidence intervals on the prediction are poor. It indicates the life of a 100 mm thick overlay to range from 12.6 years for nil previous cracking to 3.1 years for 100 percent previous cracking and the corresponding lives of a 50 mm thick overlay to be about one-half of those values.

These models are considered rather less reliable than those estimated for original surfacings, because of the partial lack of data on the pavement condition prior to overlay and the clear indication that the prior condition is a factor influencing the performance. When the prior condition is good, with little or no wide cracking, then the models for asphalt concrete original surfacings (Table 5.5) can be regarded as applicable also to overlays, based on the similarity of Equation 5.34 to those models. When the previous pavement has wide cracking, the mechanism of reflection cracking advances the time of failure of the overlay; while Equation 5.35 will provide a reasonable first estimate, there are so many engineering characteristics of the overlay and existing pavement which influence the performance that some external adjustment of the predictions to quantify these for the specific situation appears to be warranted.
5.4.5 Reseals

Reseal maintenance surfacings fall into two primary categories: (a) the regular surface treatment (or chip seal) in either single or double coat, and (b) the slurry seal.

Surface treatment reseals

In the Brazil-UNDP study, the surface treatment reseal was applied on only 7 pavement sections (3 asphalt concrete, 4 surface treatment), and as either a single (3 sections) or double (4 sections) seal, so the data were insufficient for rigorous modelling. The life before cracking initiation in these few cases was short, namely:

On surface treatments: \( T_{ycr2} = 2.9 \) years
On asphalt concrete: \( T_{ycr2} = 1.2 \) years

with confidence intervals of \( \pm 1.2 \) years. As most of the reseals had previous cracking in the underlying surface, the failure mechanism was clearly reflection cracking. Experience elsewhere suggests that surface treatment reseals yield longer lives than these, in the order of four to ten years on cracked surface treatments and three to six years on cracked asphalt concrete. The performance thus appears highly dependent on the resealing technology and needs to be determined for local circumstances.

It is recommended that the cracking of reseals be predicted using simply mean lives for particular pavement groups, calibrated to local circumstances, as follows:

<table>
<thead>
<tr>
<th>Underlying surfacing</th>
<th>Predicted mean life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uncracked</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>Table 5.6</td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>Table 5.5</td>
</tr>
</tbody>
</table>

where \( H_s \) is the average thickness of the reseal, in mm.

Slurry seals

In the Brazil-UNDP study, slurry reseals were represented by a large data set comprising 78 observations of full-width slurry seals, which had been applied to certain test sections under the high maintenance policy of the study. These were thin seals, the slurry having a maximum stone size of 5 mm conforming to a "Type II" ISSA specification (International Slurry Seal Association 1986). All slurry seal "skin patching", on local areas were excluded from the analysis because of the uncertainty of when each particular patch started to crack. The time to cracking initiation was very short, and typically in the order of 9 months when previous cracking was present. The following simple prediction model describes the data and their dispersion:
Thin slurry seals are thus generally ineffective in controlling cracking reflection, at least to the initiation of narrow cracking, because they provide a delay of only 0.6 to 1.0 years before cracking reappears from a cracked surface (about 70 percent of the above times). On surfacings that have no previous narrow cracking, the time to cracking initiation is longer, and typically depends on the timing of cracking initiation in the original surfacing. On stiff layers, such as asphalt concrete surfacings or cemented base pavements, the thin slurry seal offers no resistance to cracking reflection because it has negligible ductility and therefore provides only minor attenuation of the initiation of narrow cracking. On flexible surfacings such as surface treatment, the slurry seal is more effective and, if applied as part of the original construction or early in the surfacing's life, increases the durability of the surfacing against oxidation and greatly increases the resistance to ravelling (see Chapter 6).

5.5 INITIATION OF WIDE CRACKING

Prediction of the initiation of wide cracking (that is class 4 cracking at \( \text{CR}_4 = 0.5 \) percent of surfacing area), may be made either independently of, or as a function of, the initiation of narrow cracking. Independent predictions have the disadvantage that wide cracking may be predicted to occur before narrow cracking for some combinations of values of the explanatory variables when all the coefficients and variables are not entirely consistent. Predictions are more reliable, therefore, when expressed as a function of the initiation of narrow cracking. They are also preferable for pavement management applications and other time-series applications where one wishes to predict condition "b", given condition "a".

Models for predicting the time since surfacing construction to the initiation of wide cracking (\( \text{TY}_{\text{cr}} \)) were therefore estimated in the simple linear form:

\[
\text{TY}_{\text{cr}} = a_0 + a_1 \text{TY}_{\text{cr}}
\]  

(5.36)

The estimates of the coefficients and the model statistics are listed in Table 5.8 for each surfacing category. The model form implicitly assumes that the traffic effects which affect narrow cracking also affect wide cracking.

The time interval between the initiation of narrow cracking and its development into wide cracking was typically relatively short. On original asphalt concrete surfacings the interval was typically 2 to 2.5 years and reasonably constant. On surface treatments, the interval diminished with the age of the surfacing from 2.7 years for young surfacings to less than 1.0 year beyond an age of 14 years, presumably as the result of hardening of the binder. The intervals for resels, slurry seals and asphalt overlays were also of the order of 2 years. However, the intervals were generally shorter, about 1.5 years, on cemented base pavements, where the daily thermal movements of the base dominated the cracking mechanisms, and only 0.3 year in open-graded cold-mix asphalt overlays.
### Table 5.8: Estimation statistics of models for predicting the surfacing age at initiation of wide cracking (CR\textsubscript{k4}) from the surfacing age at initiation of all cracking (CR\textsubscript{k2})

<table>
<thead>
<tr>
<th>Pavement type</th>
<th>Expected age ( TY\textsubscript{cr4} = )</th>
<th>t-statistics</th>
<th>Sample size</th>
<th>( r^2 )</th>
<th>S.E. (yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( a_0 ) + ( a_1 ) ( TY\textsubscript{cr2} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>2.46 + 0.93 ( TY\textsubscript{cr2} )</td>
<td>5.2</td>
<td>12</td>
<td>45</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>2.66 + 0.88 ( TY\textsubscript{cr2} )</td>
<td>7.9</td>
<td>22</td>
<td>62</td>
<td>0.89</td>
</tr>
<tr>
<td>Surface treatment(1)</td>
<td>1.16 ( TY\textsubscript{cr2} )</td>
<td>-</td>
<td>46</td>
<td>62</td>
<td>0.84</td>
</tr>
<tr>
<td>Cemented base</td>
<td>1.46 + 0.98 ( TY\textsubscript{cr2} )</td>
<td>4.9</td>
<td>24</td>
<td>43</td>
<td>0.94</td>
</tr>
<tr>
<td>Slurry seals</td>
<td>0.70 + 1.65 ( TY\textsubscript{cr2} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reseals (ST)</td>
<td>1.85 + 1.00 ( TY\textsubscript{cr2} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt overlays</td>
<td>2.04 + 0.98 ( TY\textsubscript{cr2} )</td>
<td>5.5</td>
<td>13</td>
<td>42</td>
<td>0.82</td>
</tr>
<tr>
<td>Open-graded cold-mix</td>
<td>0.26 + 1.44 ( TY\textsubscript{cr2} )</td>
<td>0.7</td>
<td>4.3</td>
<td>14</td>
<td>0.61</td>
</tr>
</tbody>
</table>

Notes: Model form: \( TY\textsubscript{cr4} = a_0 + a_1 TY\textsubscript{cr2} \). Estimated by ordinary least squares regression on only observed events (excluding prior and future events). S.E. = standard error. ST = surface treatment.

1/ The best prediction is given by the maximum of these two functions.

### 5.6 Progression of Cracking

#### 5.6.1 Characteristics of the Data

The progression of cracking, expressed in terms of the numerics CR\textsubscript{k2} and CR\textsubscript{k4} for the areas of all cracking and wide cracking respectively (as defined in Equation 5.1), followed various different trends in the Brazil-UNDP study. Examples, taken from condition survey data at four- to twelve-month intervals, are shown in Figure 5.22 for six pavement subsections. The time delay between the all cracking and wide cracking progressions and their relative slopes are evident. The trends were generally S-shaped, with the rates of progression being slow initially and then increasing as the area of cracking increased. The rate of progression however was often irregular and sometimes reached a plateau at areas in the range of 40 to 80 percent (see (b) and (d) for example) for 1 to 3 years before continuing towards the final asymptote of 100 percent. An effect of pavement strength is apparent in chart (c) which shows that the rate of cracking progression for two pavements carrying identical traffic loading of 0.05 million ESA/lane/year was slower for the stronger pavement; however, the difference may
Figure 5.22: Examples of cracking progression observed in Brazil

(a) Moderately aged overlay

(b) Young asphalt concrete

(c) Light traffic, thin pavements

(d) Old strong asphalt concrete

Note: Subsection 109 SEM SC. Asphalt concrete overlay, 1977 data: traffic 1,750 veh/day, 0.37 million ESA4/lane/yr, SNC 5.4; age 10 yrs. Source: Brazil-UNDP study data.

Note: Subsection 026 SEM SC. Asphalt concrete, 1977 data: traffic 1,500 veh/day, 0.37 million ESA4/lane/yr, SNC 4.6; age 4 yrs. Source: Brazil-UNDP study data.

Note: Subsection 152 COM SC. At 1977: Asphalt concrete, age 4 yrs; traffic 540 veh/day, 0.05 million ESA4/lane/yr; SNC 1.8, DEF 1.1 mm. Source: Brazil-UNDP study data.

Note: Subsection 163 SEM SC. At 1977: surface treatment: age 9 yrs; traffic 1,080 veh/day, 0.03 million ESA4/lane/yr; SNC 3.4, DEF 1.1 mm. Source: Brazil-UNDP study data.

Note: Subsection 119 SC. At 1977: Asphalt concrete: age 19 yrs; traffic 4,000 veh/day, 1.2 million ESA4/lane/yr; SNC 7.2. Source: Brazil-UNDP study data.
also be due to other factors because the deflections of both pavements were about 1.0 mm, the higher rate of progression was for an asphalt concrete surfacing and the lower rate for surface treatment surfacing, and the asphalt surfacing was apparently deficient in binder.

Figure 5.22 also shows that wide cracking has a rate of progression which is initially slower than that of all cracking but later exceeds it so that the CR₂ and CR₄ curves tend to converge at areas in excess of 60 percent. The convergence occurs because the area of class 2 cracking eventually diminishes as the narrow, class 2 cracks develop into wide, class 4 cracks (see chart (b)), at which stage the CR₂ numeric comprises mainly class 4 cracking and less and less class 2 cracking.

The effects of slurry seal maintenance for one asphalt concrete surfacing pavement are shown in chart (d). It shows only a temporary and relatively small retardation of the rate of progression of all cracking, but much stronger retardation of the progression of wide cracking.

The effect of patching maintenance is to reduce the area of cracking observed and the common practice in past studies has been to sum the areas of cracking and patching in order to represent the total damaged area by both repaired and un repaired cracking. When this practice is followed, the cracking area numerics CR₂ and CR₄ can exceed 100 percent when patched areas which have subsequently cracked again are doubled-counted. During processing of the Brazil study data for the cracking progression analyses, difficulties arose with this approach in distinguishing between slurry seal skin patches and full-width slurry seal reseals, patches that affected CR₂ and not CR₄, patches that repaired raveling but not cracking, etc. The analyses were therefore performed only on data from subsections which had received no substantial maintenance in the form of full-width slurry seal or extensive patching: this included nearly all the "minimum maintenance" subsections ("SEM") and those "high maintenance" subsections ("COM") that actually received no maintenance. For these subsections, the amounts of patching were generally nil or less than 5 percent and were ignored. Cracking progression on resurfaced subsections were analyzed separately.

5.6.2 Estimation of Model

Three particular problems had to be addressed in modelling the progression of cracking. First, in the selection of model form, a variety of linear, convex and sigmoidal functional forms were studied, with concern both for the shape of the trends seen in Figure 5.22 and for the ease of subsequent implementation. Second, the irregularity of the trends, which is obvious in Figure 5.22, results in high residual errors if the incremental areas between successive survey records are analysed; however, if the underlying trends are to be enhanced, then the data have to be smoothed using the chosen model form. Third, was the problem of censorship in the data, with some sections having largely complete curves (i.e., those with rapid failure rates), some sections having only partial curves (either the early, middle or late parts), and of course some sections having either no cracking at all or only one or two observations.

Average progression rate methods

The first approach was to smooth the multiple survey data of each subsection by a chosen function of time, and then to estimate a model from the
smoothed, or average, progression rates of the subsections. This approach eliminated most of the measurement errors and effectively enhanced the underlying trends. The two smoothing functions tested were, first, a linear function, i.e.:

\[ CR_i = a + b t \]  \hspace{1cm} (5.35)

where \( t \) is the observation date, in calendar years, \( a \) and \( b \) are estimated coefficients, and \( CR_i \) is the numeric of cracking area for cracking classes \( i \) and greater as defined in Equation 5.1; and second, a logistic (sigmoidally-shaped) function:

\[ (CR_i = 100 e^{a + b t} / (1 + e^{a + b t})) \]  \hspace{1cm} (5.38)

which by transformation of the cracking numeric can be estimated in a linear form as follows:

\[ \log_e [CR_i / (100 - CR_i)] = a + b t. \]  \hspace{1cm} (5.38a)

For these analyses, the data were restricted to subsections with more than three observations and cracking areas in the range of 5 to 95 percent, because the variability of the rate data was very high at the extremes beyond that range. This limited the data to only 46 subsection-traffic combinations out of the 380 total in the study.

The results indicated an average linear rate of progression of 13.6 percent per year for all cracking, and a range from 2.7 to 31 percent per year. As the low rates were mainly associated with low areas of cracking, it was clear that the rate function should preferably be curvilinear. Analysis with the logistic form showed that cracking would progress over the full area (95 percent) within a period of four to twelve years after initiation. Significant effects of traffic loading, pavement strength or pavement type could not be found, and it was apparent that the logistic function was not modelling the curvature of the rate function adequately in every case.

Probabilistic piecewise models

As a lot of data had been excluded from the average rate analysis by virtue of an inadequate range or number of observations, the probabilistic maximum likelihood procedure was applied to the data so as to include the effects of the "censored" data. This was done by using the time or traffic taken for cracking to progress from 0.5 to 30 percent, and from 30 to 60 percent, as dependent variables to be modelled. Where only part of the cracking range was covered, the time (or traffic) was coded as censored.

This approach, when applied to the data separated by surfacing type and cracking class, produced rather inconsistent results with the progression functions often being concave rather than convex for areas up to 50 percent.

Analysis of individual increments

As both previous approaches seemed limited, particularly in respect to the relatively scarce data above 30 percent of cracked area, an approach using linear regression on transformations of all individual observations of incremental area was finally used. This approach permitted the inclusion of data from all
sections that evidenced cracking, providing from 50 to 200 observations per pavement type and cracking class. It also provided a natural weighting of both the number and the range of observations in the statistical estimation procedure.

Five nonlinear, recursive model forms were tested. The recursive form is useful in pavement management and was appropriate because the preliminary analyses had shown that the rate of progression was clearly related to the amount of cracking. In general, the form is:

$$A' = f(A, X)$$

where $A$ is the amount of cracking, $A'$ is the rate of progression (say, in percent per year) and $X$ represents a set of explanatory parameters. The models and the transforms that eliminate time (or traffic) from the explanatory function were:

1. $A = 1 - e^{-t}^b$, for $a < 0$, $b < 0$.
   $$A' = a b (A + 1) \frac{\ln (A + 1)}{a}(1-1/b)$$

2. $A = a e^{-b/t}$
   $$A' = A \left(\ln A\right)^2 / b$$

3. $A = e^{a + b t} / (1 + e^{a + b t})$

4. $A' = a A^b$

5. $SA' = a SA^b$

where $A$, $A'$ are here expressed as a decimal fraction; $t$ represents time or traffic; $SA = \text{minimum of } (A, 1 - A)$, $SA'$ is the rate of change of $SA$ with respect to $t$; and $a$ and $b$ are coefficients to be estimated ($a$ as a function of pavement type, strength and traffic, and $b$ preferably as a constant).

The first two forms, which yield unsymmetrical trends, and the third form which is symmetrical, produced reasonable results, but the curvature constraints were such that the early rate of progression was greatly underestimated by them, and the model fits tended to be inferior to those for the fourth and fifth forms. The simplicity of the fourth form (which is used by Arizona) was appealing but it is not sigmoidal. The fifth form was selected for the analyses because of its symmetry and flexibility of curvature; in reality the estimates of $a$ and $b$ are almost identical in the two cases so that estimates from form 5 can be used in a model of form 4 with confidence.

Redefining $b$ in the general derivative form of model 5, as follows:

$$dSCR_{it} = a SCR_{it} \frac{1-b}{dt}, \quad (5.39)$$

the area of cracking at time $t$, $CR_{it}$, is derived by integration, i.e.:

$$CR_{it} = (1 - z) 50 + z \left[ z a b t_{ci} + z 0.5+b + (1 - z) 50^{b} \right]^{1/b}. \quad (5.40)$$
The incremental area of cracking during the period $\Delta t$, $\Delta CR_{it}$, is given by:

$$
\Delta CR_{it} = zz \{ [zz \ a \ b \ \Delta t + SCR_{it}^{-b}]^{1/b} - SCR_{it} \} \tag{5.41}
$$

and the time taken to reach area $CR_{it}$ is given by:

$$
t_{ci} = [(1 - zz) 50^b + zz SCR_{it}^{-b} - 0.5^b] / a \ b \tag{5.42}
$$

where $CR_{it}$, $\Delta CR_{it}$, $a$ and $b$ are defined as above;

$$
SCR_{it} = \text{minimum} \left[ CR_{it}' - 100 - CR_{it} \right];
$$

$$
t_{ci} = \text{time since initiation of } CR_{i} \text{ cracking (years) in time-base models, or traffic loadings since initiation (million ESA) in traffic-base models};
$$

$$
\Delta t = \text{increment of time (years) in time-base models, or increment of traffic loading (million ESA) in traffic-base models};
$$

$$
z = 1 \text{ when } t_{ci} \leq t_{s0} \text{ and } z = -1 \text{ otherwise};
$$

$$
t_{s0} = (50^b - 0.5^b)/a \ b; \text{ i.e., the time to } 50\% \text{ area cracked}; \text{ and}
$$

$$
zz = 1 \text{ when } CR_{it} \leq 50; \text{ otherwise } zz = -1.
$$

5.6.3 Predictive Models for All Surfacings

Two sets of models were derived, time-base models for cracking progression as a function of time and independent of strength or traffic variables, and traffic-base models as a function of traffic loading and strength. There was usually only a small difference in the goodness of fit between these two dimensions, though the traffic-base models were generally superior, especially in the coefficient of variation. Traffic-base models were not applicable for some surface types, and the time-base models are the most generally applicable.

Time-base models

The estimates of coefficients $a$ and $b$ for each surface type and cracking class are given in Table 5.9, and Figure 5.23 illustrates the progression functions for both all cracking and wide cracking, inclusive of the time delay before the initiation of wide cracking. The models are used by applying the values of $a$ and $b$ into Equation 5.40 or 5.41. From the table it can be seen that the fit of the models is fair with $r^2$ values in the range of 0.3 to 0.4 and coefficients of variation in the order of 40 to 60 percent - much of the residual error comes from measurement errors which cause high variance when successive differencing of the data is made to determine the incremental values.

The values of $(1-b)$, the exponent in Equation 5.39, are generally in the range of 0.55 to 0.80, which implies a strong curvature in the progression functions, as can be seen in Figure 5.23. The rates of progression of wide cracking are generally faster than the rates of all cracking, but, as the area of wide cracking can never exceed the area of all cracking, the all cracking function governs.
Table 5.9: Time-base models for predicting cracking progression as a function of incremental time

<table>
<thead>
<tr>
<th>Cracking class and surfacing</th>
<th>Model estimates(^1)</th>
<th>Model statistics</th>
<th>Period (years) between 0.5% cracking and areas of 30% and 95%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>b</td>
<td>CV (%)</td>
</tr>
<tr>
<td>All cracking</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>1.84</td>
<td>0.45 (9.0)</td>
<td>54</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>1.76</td>
<td>0.32 (10.0)</td>
<td>58</td>
</tr>
<tr>
<td>Cemented base</td>
<td>2.13</td>
<td>0.36 (6.6)</td>
<td>48</td>
</tr>
<tr>
<td>Asphalt overlays</td>
<td>1.07</td>
<td>0.28 (8.8)</td>
<td>78</td>
</tr>
<tr>
<td>Reseals and slurry seals</td>
<td>2.41</td>
<td>0.34 (11)</td>
<td>38</td>
</tr>
<tr>
<td>Wide cracking</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>2.94</td>
<td>0.56 (4.2)</td>
<td>53</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>2.50</td>
<td>0.25 (5.2)</td>
<td>48</td>
</tr>
<tr>
<td>Cemented base</td>
<td>3.67</td>
<td>0.38 (5.0)</td>
<td>35</td>
</tr>
<tr>
<td>Asphalt overlays</td>
<td>2.58</td>
<td>0.45 (4.9)</td>
<td>52</td>
</tr>
<tr>
<td>Reseals</td>
<td>3.4</td>
<td>0.35 (4.0)</td>
<td>37</td>
</tr>
</tbody>
</table>

Note: The 15th and 85th percentile confidence intervals are about 0.6 and 1.6 times the predicted value.

\(^1\) = t-statistic of b estimate.

Source: Estimate of Equation 5.39 in form: \(\ln (ACR_i/\Delta t) = \ln a + (1 - b) \ln SCR\).

Strong similarities are evident among groups of surface types, for example, between asphalt concrete and asphalt overlays which take twelve to fourteen years for full cracking progression, and between surface treatments on either granular or cemented base and reseals which take seven to eight years for full progression.

In general no other explanatory parameters were found to be significant; traffic and pavement strength became significant only when included as an interaction term with the correct time dimension, and this is best seen in the traffic-increment models. It was considered that the rate of progression might be the stochastic sequence of cracking initiations occurring on all elements of the section and therefore related to the age of the surfacing at initiation; however, significant correlation was not found.

Simple linear progression rate models will be preferable in a number of pavement management applications. Table 5.10 presents the average values for each surface type and cracking class, for two segments 0.5 to 30, and 30 to 95 percent (which introduces some "curvature" effect), and for the full range 0.5 to 95 percent. Once again, when the all cracking and wide cracking rates are used jointly, the area of all cracking so predicted should govern and not be exceeded by the area of wide cracking.
Figure 5.23: Predicted cracking progression for original and maintenance surfacings: time–base models

(a) Original surfacings

(b) Maintenance surfacings

Source: Equation 5.42 and Table 5.9.
Table 5.10: Average linear cracking progression rates for various surfacings, cracking classes and ranges

<table>
<thead>
<tr>
<th>Surfacing</th>
<th>All cracking (%/year)</th>
<th>Wide cracking (%/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.5-30</td>
<td>30-95</td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>6.3</td>
<td>8.7</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>7.6</td>
<td>11.4</td>
</tr>
<tr>
<td>Cemented base</td>
<td>8.7</td>
<td>12.5</td>
</tr>
<tr>
<td>Asphalt overlay</td>
<td>5.0</td>
<td>7.6</td>
</tr>
<tr>
<td>Reseals</td>
<td>10.2</td>
<td>31.0</td>
</tr>
</tbody>
</table>

Source: Derived from Table 5.9, data from Brazil-UNDP study.

Traffic-base models

The model estimates for each surface type and cracking class, including indexed cracking, are given in Table 5.11 and illustrated in Figure 5.24. In this case, the model can be applied either by substituting the traffic increment $\Delta N E_n$ for $\Delta t$ in Equation 5.42, or, as is better suited to management applications in which the time increment is the primary dimension, by substituting $Y E_n \Delta t$ for $\Delta t$ (where $Y E_n$ is the annual traffic loading in million ESA/lane/year). Similarly, $t_{ci}$ would be replaced by $N E_{ci}$, the cumulative traffic since initiation, in Equation 5.40, which thus becomes:

$$ CR_{it} = (1 - z) 50 + z (a b N E_{ci} + z 0.5^b + (1 - z) 50^b)^{1/b} \quad (5.43) $$

Models are shown for two traffic increments, $\Delta N E_2$ and $\Delta N E_4$. These represent different formulations of equivalent standard axle loading, the first computed with a relative load damage power $n$ of 2 and the second with a power of 4. The reason for this was to test the supposition, mentioned earlier, that cracking in thin surfacings was less sensitive to axle loading than a fourth power would suggest, as studied in detail in Chapter 9. In fact, powers of 0, 2, 4 and 6 were applied (see Table 9.8), but only those for 2 and 4 are reported here. A scan of Table 5.11 shows that an $n$-value of 2 gives a better fit than 4 in virtually all cases, and was generally the optimum value, thus demonstrating the supposition to be valid. As most planning and management programs operate with a relative load damage power of 4, the $\Delta N E_4$ models are the ones expected to be applied when the traffic-base models are preferred and are the curves shown in the figure.

Strong effects of pavement strength on the rate are evident. These are in the same order as the effects on cracking initiation (Section 5.4), being a power of two to four on the modified structural number and approximately linear on surface deflection (powers ranging from 0.6 to 1.5). The modified structural number was the strongest explanatory variable for asphalt concrete surfacings, surface deflection was marginally better than modified structural number for surface treatments, and both deflection and base-modulus were strong explanatory variables for cemented base pavements. Wide cracking progressed 30 to 60 percent faster than all cracking in surface treatments, but at approximately similar rates
Table 5.11: Traffic-base models for predicting the rate of cracking progression as a function of incremental traffic

<table>
<thead>
<tr>
<th>Traffic increment</th>
<th>Parameter estimates</th>
<th>Model statistics&lt;sup&gt;1&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of all cracking CRₐ (%)</td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>1. <strong>ANEₐ</strong></td>
<td>586 SNC².17</td>
<td>0.63</td>
</tr>
<tr>
<td>2. <strong>ANEₐ</strong></td>
<td>450 SNC⁻².27</td>
<td>0.65</td>
</tr>
<tr>
<td>Surface treatment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. <strong>ANEₐ</strong></td>
<td>338 SNC⁻¹.91</td>
<td>0.29</td>
</tr>
<tr>
<td>4. <strong>ANEₐ</strong></td>
<td>1,760 SNC⁻³.23</td>
<td>0.28</td>
</tr>
<tr>
<td>5. <strong>ANEₐ</strong></td>
<td>42 DEF⁻⁰.821</td>
<td>0.30</td>
</tr>
<tr>
<td>6. <strong>ANEₐ</strong></td>
<td>44 DEF⁻¹.01</td>
<td>0.27</td>
</tr>
<tr>
<td>Thin surfacing on cemented base</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7. <strong>ANEₐ</strong></td>
<td>3.33 DEF⁻⁰.46 CMOD⁻⁰.78</td>
<td>0.43</td>
</tr>
<tr>
<td>8. <strong>ANEₐ</strong></td>
<td>2.43 DEF⁻⁰.64 CMOD⁻⁰.90</td>
<td>0.41</td>
</tr>
<tr>
<td>Area of wide cracking, CRₐ (%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9. <strong>ANEₐ</strong></td>
<td>586 SNC⁻².37</td>
<td>0.70</td>
</tr>
<tr>
<td>10. <strong>ANEₐ</strong></td>
<td>718 SNC⁻².52</td>
<td>0.72</td>
</tr>
<tr>
<td>Surface treatment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11. <strong>ANEₐ</strong></td>
<td>497 SNC⁻¹.58</td>
<td>0.35</td>
</tr>
<tr>
<td>12. <strong>ANEₐ</strong></td>
<td>4,520 SNC⁻³.19</td>
<td>0.39</td>
</tr>
<tr>
<td>13. <strong>ANEₐ</strong></td>
<td>143 DEF⁻¹.48</td>
<td>0.45</td>
</tr>
<tr>
<td>14. <strong>ANEₐ</strong></td>
<td>160 DEF⁻¹.48</td>
<td>0.45</td>
</tr>
<tr>
<td>Thin surfacing on cemented base</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15. <strong>ANEₐ</strong></td>
<td>5.19 DEF⁻⁰.43 CMOD⁻⁰.65</td>
<td>0.32</td>
</tr>
<tr>
<td>16. <strong>ANEₐ</strong></td>
<td>3.93 DEF⁻⁰.59 CMOD⁻⁰.74</td>
<td>0.30</td>
</tr>
<tr>
<td>Indexed cracking, CRX (percent)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17. <strong>ANEₐ</strong></td>
<td>1,560 SNC⁻³.70</td>
<td>0.25</td>
</tr>
<tr>
<td>18. <strong>ANEₐ</strong></td>
<td>3,330 SNC⁻⁴.25</td>
<td>0.25</td>
</tr>
<tr>
<td>Surface treatment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19. <strong>ANEₐ</strong></td>
<td>73.6 DEF⁻¹.36</td>
<td>0.41</td>
</tr>
<tr>
<td>20. <strong>ANEₐ</strong></td>
<td>81.0 DEF⁻¹.57</td>
<td>0.41</td>
</tr>
<tr>
<td>Thin surfacing on cemented base</td>
<td></td>
<td></td>
</tr>
<tr>
<td>21. <strong>ANEₐ</strong></td>
<td>0.178 CMOD⁻⁰.45</td>
<td>0.35</td>
</tr>
<tr>
<td>22. <strong>ANEₐ</strong></td>
<td>0.074 CMOD⁻⁰.51</td>
<td>0.29</td>
</tr>
</tbody>
</table>

<sup>1</sup> Models of Equation 5.42 estimated by linear regression on the log transform, i.e.:

\[
\ln (\text{ACR/ANEₐ}) = \ln a + (1 - b) \ln \text{SCR}
\]

S.E. = standard error; C.V. = standard error/mean (%); n = sample size.

Source: Analysis of Brazil-UNDP study data.
Figure 5.24: Predicted cracking progression for original surfacings: traffic-base models

(a) Asphalt concrete

(b) Double surface treatment

(c) Surfacing on cemented base: base modulus 6 GPa

(d) Surfacing on cemented base: Base modulus 20 GPa

Note: The origin for the wide cracking progression curves has not been offset from the all-cracking curves.

Source: Equation 5.42 with coefficient a and b from Table 5.11 as follows: (a) Models 2 and 10; (b) Models 4 and 12; (c) and (d) Models 8 and 16, respectively, for all and wide cracking.
in asphalt concrete surfacings. In semirigid pavements, the rate of cracking progression was almost directly proportional to the stiffness of the base under comparable deflections; for the example shown in charts (c) and (d), a threefold increase in base modulus increases the rate under 0.3 mm deflection from about 14 percent area per million ESA to 45 percent area per million ESA. Usually an increase in base modulus would be offset by a decrease in deflection, so the difference in progression rates would be somewhat smaller in practice, perhaps twofold instead of threefold in the example above.

Although surface treatments tended to have longer lives before cracking initiation than asphalt concretes, cracking progression in surface treatments tended to be two to four times faster than in asphalt concrete surfacings, for pavements of comparable strength. This behavior, and the fact that the rate of wide cracking progression tends to be much faster than narrow/all cracking, demonstrate why preventive maintenance or early corrective maintenance (resealing at the first signs of cracking) is so important for surface treatment/granular base pavements and semirigid pavements.

For cemented base pavements and reseals, the time-base models are better than the traffic-base models, because the cracking progression is being dominated by reflection cracking and brittle behavior.

Comment

The time-base models will satisfy most demands of a management system, in either the sigmoidal or linear (and segmented) forms because they represent a good average for typical standards of pavement strength and loading. However, when the effects of different loading rates or of pavement strength on cracking are to be evaluated, then the traffic-base models are preferred (where available). It should be noted that experience with the traffic-base models has shown that sometimes progression rates outside the range of 2.7 to 31 percent per year may be generated by the models; unless there is local experience to the contrary, it is recommended that the effective time rates be limited to within the range of about 2 to 40 percent per year.

5.7 VALIDATION

The validity of the predictive models has been evaluated against published fatigue cracking data and against independent field data from performance studies in Arizona, Texas, Illinois and Tunisia.

5.7.1 Fatigue Cracking Behavior

For independent data on which to assess the validity of the models predicting cracking initiation, we first utilize the various published fatigue criteria that have been based on both experimental research and field performance and are now incorporated in major pavement design methods. The inclusion of field performance is important as it was noted that shift factors in the order of 10 to 100 are applied to most experimental relationships in order to have them fit field performance data (Section 5.2.1). The three sources chosen are the Nottingham method (Brown, Pell and Stock 1977) shown in Figure 5.2, the Shell method (Shell International Petroleum Co. 1978) shown in Figure 5.3(b), and the American NCHRP study (Finn and others 1977), shown in Figure 5.4 (as adapted by the Asphalt Institute (1981) with calibration to 45 percent area of cracking on data from the
These all estimate the cumulative number of applications of a given tensile strain causing the initiation of (or a specified amount of) cracking, without allowance for aging. The NCHRP-Asphalt Institute prediction is here adjusted from 45 to 0.5 percent cracking by applying a factor of 0.6, based on the source documents.

The only strain-based empirical model is Equation 5.27, and predictions from this model are compared with those of the independent sources in Table 5.12. The predictions are made for four levels of strain, firstly for a constant value of the aging parameter, \( E_Y \), and secondly for a common life of ten years.

Comparing the prediction and estimates firstly for a common level of 200 microstrain \( (10^{-6}) \), there is excellent agreement, with the prediction of 1.3 million ESA being virtually identical to the estimates of the Nottingham and NCHRP methods (1.0 and 1.2 million ESA) and somewhat less than the 3 million ESA estimate of the Shell method. Considering the wide dispersion commonly associated with fatigue predictions (the quartiles of the empirical prediction are 0.3 and 2.6 million ESA, and the design method estimates typically range by a factor of three each side of the mean), the Shell method's estimate can also be regarded as similar.

When the influence of strain level is considered, some differences appear due to differences in the exponent of the strain parameter in the various models. The empirical predictions from the Brazilian data have a trend which is very similar to that of the NCHRP-Asphalt Institute studies (the exponents are -2.9 and -3.3 respectively), when the aging parameter, \( E_Y \), is held constant: the model's predicted range is 9.2 to 0.09 million ESA and the NCHRP-estimated range is 12 to 0.06 million ESA, for strain levels of 100 to 500 microstrain. The Nottingham estimate shows a greater range (22 to 0.02 million ESA) and the Shell method a much greater range (200 to 0.02 million ESA). Since there were strong indications in the Brazilian data that the relative load damaging power (which is nominally equivalent to the exponent of the strain parameter) was in the order of 2 to 4 for all models of cracking in asphalt surfacings, there is at least internal confirmation that a power value of 3 is of the correct order. The higher values of 4.5 for the Nottingham method and 5.5 for the Shell method do not appear to be applicable for the range of materials observed in Brazil and the United States of America; generally such values apply to high stiffness materials in thick layers (i.e., in the stress-controlled mode of behavior under load).

The trend of predictions from the empirical model for a target life of ten years, shown in the lower part of the table, appears overly suppressed with a range of only 5 to 0.2 million ESA for strains from 100 to 500 microstrain. This implies that the impact of the aging parameter (strain-traffic flow interaction) is perhaps too strong outside a fairly narrow range of the parameter \( E_Y \) (\( E_Y \) ranges from 8 down to 0.75 for strains of 100 to 500 microstrain, respectively).

A valuable inference can be drawn from this validation against fatigue models. From the NCHRP and Shell methods it is apparent that the reason for the apparently short life of asphalt surfacings in Brazil is the high stiffness of the bituminous mixtures in relation to the strain induced in the surfacing; thus longer lives would be achieved with more flexible, lower stiffness mixtures, or alternatively with reduced strain levels brought about through a stiffer base or stiffer pavement. The mix stiffnesses in Brazil, ranging in the study from 1 to 5.5 GPa at 30°C, with a mean of 2.5 GPa, are considered high and should be lowered.
Table 5.12: Comparison of strain-based predictions for cracking initiation with published fatigue criteria in major design methods

<table>
<thead>
<tr>
<th>Maximum tensile strain (10^-6)</th>
<th>Traffic loading (MESA/yr)</th>
<th>Model¹/ prediction (MESA)</th>
<th>Design method estimates (MESA)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nottingham²/ Shell³/ NCHRP 1-10B⁴/</td>
</tr>
<tr>
<td>Aging parameter KV = 5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>2.00</td>
<td>9.21</td>
<td>22</td>
</tr>
<tr>
<td>200</td>
<td>0.125</td>
<td>1.27</td>
<td>1.0</td>
</tr>
<tr>
<td>300</td>
<td>0.025</td>
<td>0.40</td>
<td>0.18</td>
</tr>
<tr>
<td>500</td>
<td>0.003</td>
<td>0.09</td>
<td>0.02</td>
</tr>
<tr>
<td>Life of 10 years</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>0.51</td>
<td>5.10</td>
<td>22</td>
</tr>
<tr>
<td>200</td>
<td>0.125</td>
<td>1.27</td>
<td>1.0</td>
</tr>
<tr>
<td>300</td>
<td>0.073</td>
<td>0.76</td>
<td>0.18</td>
</tr>
<tr>
<td>500</td>
<td>0.021</td>
<td>0.21</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Notes: Asphalt mix stiffness 2,500 MPa at 30°C.
1/ Equation 5.27.
2/ Figure 5.2 (Brown Pell and Stock 1977): V_b = 13%, softening point = 50°C.
3/ Figure 5.3(b) (Shell International Petroleum Co. 1978).
4/ Figure 5.4 (Adapted from Finn and others 1977 by Asphalt Institute (AI) 1981).

Sources: As noted.

to improve the surfacing life. The situation, in fact, may be an effect of aging, as evidenced by the correlation with the air void content of the mix given in Equation 5.20, in which case an improvement in life would also come from improving the control of the manufacture and compaction of the materials during construction.

5.7.2 Data from Independent Studies

The predictive models for initiation time and rate of progression for all pavement types were applied to pavement data from the Arizona, Texas, Kenya and Illinois (AASHO Road Test) studies of pavement performance and the predictions compared with the observed initiation and progression of cracking. Small calibration studies performed for Tunisia and England were also included. The results are presented in Table 5.13 for the initiation, and Table 5.14 for the progression, of all cracking.

The initiation of cracking was very well predicted in terms of the mean values in every study except the AASHO Road Test. The differences between the observed and predicted means were typically less than one year, or about three to ten percent of the mean time, which is highly acceptable accuracy. The correlation coefficients were also reasonable, and of similar order to the original model fits, so the variance is approximately equivalent to the stochastic variation of
Table 5.13: Validation of the prediction of surfacing age at initiation of all cracking: data from independent studies

<table>
<thead>
<tr>
<th>Study and initiation time</th>
<th>Mean Age (yrs)</th>
<th>Ratio of Correlation means</th>
<th>Standard deviation (yrs)</th>
<th>Minimum Age (yrs)</th>
<th>Maximum Age (yrs)</th>
<th>Sample Age (yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arizona</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>8.92</td>
<td>1.02</td>
<td>0.36</td>
<td>5.49</td>
<td>2</td>
<td>25</td>
</tr>
<tr>
<td>Predicted</td>
<td>8.71</td>
<td>(0.42)</td>
<td>-</td>
<td>6.4</td>
<td>13</td>
<td>51</td>
</tr>
<tr>
<td>Texas</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>15.8</td>
<td>1.10</td>
<td>0.12</td>
<td>4.89</td>
<td>1.3</td>
<td>46</td>
</tr>
<tr>
<td>Predicted</td>
<td>14.4</td>
<td>(.)</td>
<td>-</td>
<td>2.3</td>
<td>94</td>
<td>211</td>
</tr>
<tr>
<td>Illinois (AASHTO)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>0.6</td>
<td>0.14</td>
<td>-0.30</td>
<td>0.5</td>
<td>0.0</td>
<td>1.7</td>
</tr>
<tr>
<td>Predicted</td>
<td>4.3</td>
<td>(.)</td>
<td>-</td>
<td>3.9</td>
<td>0.1</td>
<td>22</td>
</tr>
<tr>
<td>Tunisia</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>12.0</td>
<td>1.05</td>
<td>0.40</td>
<td>2.9</td>
<td>4.5</td>
<td>25</td>
</tr>
<tr>
<td>Predicted</td>
<td>11.4</td>
<td>(.)</td>
<td>-</td>
<td>1.2</td>
<td>6.0</td>
<td>13</td>
</tr>
<tr>
<td>England</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>5.8</td>
<td>1.03</td>
<td>0.9</td>
<td>-</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>Predicted</td>
<td>5.7</td>
<td>(.)</td>
<td>-</td>
<td>-</td>
<td>4</td>
<td>10</td>
</tr>
<tr>
<td>Kenya</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>1.2</td>
<td>0.6</td>
<td>0.46</td>
<td>0.2</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td>Predicted</td>
<td>2.1</td>
<td>(1.3)</td>
<td>-</td>
<td>0.6</td>
<td>1.2</td>
<td>2.4</td>
</tr>
</tbody>
</table>

- Not applicable.
. Not available
() Prediction error

Source: Author, with R.J. Tomlinson. Observed data from study sources (Chapter 4) for Arizona, Texas, Illinois and Kenya; Tunisia (Paterson in Newbery and others 1988); England (Wyley and others 1986).

The progression of cracking was predicted only moderately by the time-base models with the observed rates being 10 to 50 percent slower than predicted, and the correlation coefficients being generally low. The reasons for this have not been fully resolved, but probable explanations include: the difficulty in converting the data metrics, the traffic-base models may predict better than the time-base models, and the high stiffness of asphalt mixes in the study may have
Table 5.14: Validation of prediction for progression of all cracking on data from independent studies: time-base models

<table>
<thead>
<tr>
<th>Study and progression period</th>
<th>Mean(^1) time of means</th>
<th>Correlation coefficient</th>
<th>Standard deviation</th>
<th>Minimum time</th>
<th>Maximum time</th>
<th>Sample Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arizona</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>6.45 (1.75)</td>
<td>0.18</td>
<td>1.41 (1.53)</td>
<td>4</td>
<td>9</td>
<td>22</td>
</tr>
<tr>
<td>Predicted</td>
<td>3.68 (3.35)</td>
<td></td>
<td>1.53 (1.40)</td>
<td>7.6</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>Texas</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>3.05 (1.19)</td>
<td>0.04</td>
<td>1.76 (1.18)</td>
<td>1</td>
<td>7</td>
<td>961</td>
</tr>
<tr>
<td>Predicted</td>
<td>2.56 (2.14)</td>
<td></td>
<td>1.18 (0.30)</td>
<td>4.5</td>
<td>961</td>
<td></td>
</tr>
<tr>
<td>Illinois (AASHTO)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>5.61 (2.18)</td>
<td>0.23</td>
<td>8.91 (1.38)</td>
<td>0.0</td>
<td>56</td>
<td>733</td>
</tr>
<tr>
<td>Predicted</td>
<td>2.57 (.)</td>
<td></td>
<td>1.38 (0.50)</td>
<td>5.7</td>
<td>733</td>
<td></td>
</tr>
<tr>
<td>Kenya(^2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>4.0 (1.09)</td>
<td>0.63</td>
<td>-</td>
<td>2.4</td>
<td>5.2</td>
<td>5</td>
</tr>
<tr>
<td>Predicted</td>
<td>3.6 (1.3)</td>
<td>-</td>
<td>1.0</td>
<td>5.2</td>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>

1/ Period taken for progression between initial and final observations if cracking, years.

2/ Traffic-base model.

Source: Equation 5.41 and Table 5.9 applied to data from study sources (Chapter 4).

yielded fast progression rates. In France the progression rate for thick bituminous pavements is cited as two to four percent per year (SETRA 1987), which is less than half the values predicted by the time-base model for thin to medium thickness surfacings, but agrees with the predictions of the traffic-base models.

5.8 CONCLUSION

The interaction of the mechanisms of fatigue and aging in causing cracking in bituminous surfacings has been demonstrated strongly from mechanistic principles, from experimental research, and through the predictive models derived empirically from the data of the Brazil-UNDP study. Aging has the effect of limiting the life of surfacing materials to about 8 to 20 years from the time of construction to the initiation of cracking, through the process of oxidation which proceeds at a rate dependent on the film thickness and chemical composition of the binder, the climate (ambient temperature and sunshine), and the air void content or exposure of the material to air. Models predicting the age at cracking initiation tended to predict more strongly and with less uncertainty than cumulative traffic loading models.

The empirical strain-based model was strongly validated against the fatigue criteria of major design methods, and the time model of initiation was
generally well-validated in a wide range of conditions, with the exception of freezing climates and (possibly) under accelerated trafficking. While the predictive models for initiation were generally well validated, calibration of the predictions by a factor is advised, so as to take into account the climatic and material characteristics specific to a region. This is achieved by multiplying the predicted time ($T_{CR}$) or traffic ($TE_{CR}$) by a factor, determined from comparing the mean age at failure of a large sample of lightly trafficked pavements with the predicted expected age at initiation. The progression predictions by time-base models appear to need suppression in most of the few cases studied so far, and the calibration factor to be applied to the amount of cracking is likely to be in the range of 0.5 to 0.8. The few applications of the traffic-base progression models so far has been encouraging.

Clear, interacting effects of traffic loading and pavement strength on both the initiation and progression of cracking have been identified empirically. The modified structural number was the best strength parameter for original and overlay asphalt surfacings, and for surface treatments. The surface deflection (by Benkelman beam) was also good for surface treatments and was strongest for cemented base pavements. For reseals by surface treatment or slurry seal, cracking occurred mainly through reflection from previous cracking and was independent of pavement strength and traffic in the small sample available. The relative damaging effects of different axle loadings were found to relate to the second or third power of the axle load; this was especially clear for thin surfacings but higher powers of nearer four were found for asphalt concrete surfacings.
Disintegration and Wear of Paved Road Surfacings

This chapter addresses two modes of pavement distress originating almost solely in the surfacing. Disintegration results in the actual or potential loss of material and this affects both the structural and functional integrity of the pavement. It includes the loss of stone particles from the surface (ravelling), and the loss of fragments of the surfacing (potholing, peeling and edgebreak). The wear of surface texture affects the functional performance, primarily in respect of vehicle safety through the skid resistance and aquaplaning potential of the surface. It includes the loss of microtexture through the abrasion of stone particles (polishing) and the loss of macrotexture through stone embedment, bleeding and, in some cases, through ravelling. Two mechanisms are involved, namely disintegration and viscous flow.

6.1 TYPES OF DISTRESS AND SIGNIFICANCE

The economic and physical significance of disintegration and texture changes with respect to user costs, user acceptability and their impact on other modes of road deterioration varies considerably with the type and severity of disintegration, and consequently so do the types of maintenance and the decision criteria for intervention. Ravelling has negligible effect on roughness (see Chapter 8), has serious structural implications only when the surfacing is thin and thus liable to potholing, but may increase road noise and flying stone hazards. It thus usually triggers maintenance (patching or resurfacing) mainly as a preventive measure against consequent, more serious types of distress such as potholes and loss of skid resistance, and against surface moisture penetration to the lower layers. Potholes, peeling and edgebreaks should, and usually do, evoke an immediate routine maintenance response of patching because they can cause exceptionally sharp increases in user costs and accident risks\(^1\). They also have the physical impact of accelerating pavement distress and, left unrepaired, typically result in the need for full resurfacing or reconstruction at a greatly increased cost.

Texture wear though polishing, embedment and bleeding have a functional and economic impact because they create hazards to road safety, but have a generally minor impact on structural performance. The impacts on safety receive relatively light treatment here, since skid resistance measurements were not incorporated in the empirical studies. Accident costs in developing countries are typically a very small fraction of user costs though they may however be large in absolute amounts (see Chesher and Harrison 1987). The physical impact, for example, of bleeding, may even appear beneficial to the structural performance

\(^1\) Note however that comparatively little attention has been devoted to quantifying these impacts reliably, partly because of their diversity (See for example Hide, Morosuk and Abaynayaka 1983). Physical studies on the influence of surface discontinuities on road safety are reported in Transportation Research Board (1984) and Zimmer and Ivey (1983).
because it is likely to retard, or even prevent, cracking. Thus balance between functional and structural criteria is required in any economic evaluation of these distress modes.

The difficulty with modelling these types of distress for prediction is that considerable variations of performance are found in practice due to variations of material design or specification, construction quality, and construction practice, that are simply impracticable to quantify and thus model individually. The differences tend to be particularly large between regions or countries where specifications and construction practices may differ markedly. For example in some countries, the ravelling of surface treatments is very rare but bleeding occurs occasionally, and yet in others the reverse is true (as was the case in the Brazil study area). To be of value to planners therefore any prediction models of disintegration will require adaptation to reflect local practice and materials, and should be probabilistic to reflect the uncertainty of when distress would occur.

To make the problem tractable, these types of distress were reduced to three, representing the major effects above, i.e., ravelling, potholing and polishing, and models for these are discussed in the following subsections. In the Brazil study, time-series data were collected on ravelling and, to a variable degree, on potholing, but not on polishing or skid resistance. An alternative approach, adopted in some systems, is to avoid distinctions of the types of distress, and instead to estimate the "life" of the surfacing and maintenance from survivor curves or directly from the observed intervals between resurfacing or resealings under either existing or "satisfactory" maintenance practice. Typical intervals of 6 to 15 years are common for chip seal surface treatments under traffic volumes of typically less than 2,000 veh/day, but with good construction technology carrying up to 6,000 veh/day, and for asphalt concrete surfacings under traffic volumes of more than 2,000 veh/day.

6.2 RAVELLING

6.2.1 Mechanisms of Ravelling

Ravelling is here defined to include loss of stone from the surfacing either by mechanical fracture of the binder film or by loss of adhesion between binder and stone (which, in the presence of water, is also known as "stripping"). While detailed inspection would reveal which mechanism applied (from stripping the loose stone is clean; from ravelling it remains coated), such a distinction is not usually practicable in condition surveys.

Mechanical fracture of the binder film around a stone particle occurs when the binder has become too brittle or the film is too thin to sustain the stresses imposed through the tire contact area of a moving vehicle. The published literature on asphalt durability is extensive, but the following demonstration from Australian research serves to illustrate the mechanisms. Whether the binder is hardened through the evaporation of plasticizing oils by overheating during construction, or through long-term thermal oxidation (see Section 5.2), observations have shown that fracture is likely to occur when the binder viscosity reaches about 5.7 log Pa s in temperate climates (average daily air temperature, minimum 6°C, maximum 20°C) or 6.5 log Pa s in tropical climates (minimum 11°C, maximum 30°C) (Dickinson 1982). (Note: The viscosity cited is the apparent
viscosity at 45°C of extracted bitumen at a shear rate of 0.005 s⁻¹.) The value for the tropical climate is higher because the pavement temperatures are higher, but at the pavement temperatures representative of each climate the viscosities at fracture would be similar. As the rate of hardening by reaction of the bitumen with oxygen increases with temperature, so hardening tends to be more rapid in tropical than temperate climates and the time taken to reach the critical viscosity is about equal in both climates, see Figure 6.1. The age of the surfacing at this stage is approximately 9 years, but depending on the chemical resistance of the bitumen to oxidation it may be as short as 6 years or as long as 15 years. The rate of aging thus depends also on the crude and refinery sources of the bitumen. A thick binder film has a long diffusion-path so that the unexposed parts of the film remain ductile for much longer than the exposed surface, but thin binder films tend to oxidize rapidly. Thus the effect of aging on raveling can be expected to be much greater on thin films (which are associated with small stone sizes, less than 12 mm) than on thick films (which are associated with large stone sizes, more than 12 mm).

Initial hardening of the binder film can occur during construction when the bitumen is heated to spraying or mixing temperatures in the order of 135 to 160°C. If the binder is overheated or held at the higher temperatures for long periods, some of the lighter volatile fractions of the bitumen evaporate resulting in a higher initial or "laydown" viscosity that effectively reduces the life expectancy of the binder. The tires of moving traffic provide the mechanical

![Figure 6.1: Effects of climate on rate of binder hardening and appearance of distress in surface treatments](image)

Source: Dickinson (1982).
forces which can cause fracture of the binder film, and which pluck the stone out of the bituminous matrix by a combination of horizontal stresses in the tire contact area and suction following its passage. Thus while the contact pressure, size and design of a tire may affect the rate of ravelling, it is unlikely that wheel load will affect it.

Loss of adhesion between the binder film and the stone particle is usually due either to the presence of water (stripping) or to contamination. Adhesion between bitumen and stone develops through molecular bonding (see for example, Peterson and others 1974, Petersen and others 1982, Dickinson 1984), and as most aggregates have slightly negative charges they tend to attract water in preference to bitumen which has a roughly neutral charge. Hydrophilic aggregates (highly siliceous aggregates fall in this category) are particularly susceptible to disbonding in the presence of water. The use of binder additives such as hydrated lime (which as an alkali salt raises the pH at the bonding surface), and chemical amines, has been shown to improve adhesion under some circumstances (e.g., Petersen and others 1974, Taylor and Khosla 1983, Lay 1984). Mechanical loss of adhesion occurs if dust on the stone prevents a full bond from developing between the binder and the stone particle - the use of clean stone is a vital element of specifications for surface treatment construction. Loss of adhesion is thus usually controlled either through the construction specifications in respect of the selection of stone, cleaning of the stone, and protection from water or rain until full bonding and firm embedment of the stone have occurred, or through the use of binder additives.

6.2.2 Empirical Modelling of Initiation and Progression

The data base for empirical modelling was again taken from the Brazil-UNDP paved road deterioration study. Ravelling was recorded during the four to six-monthly visual condition surveys, and quantified by the sum of areas of ravelling on a test subsection; i.e., in the same way as areas of cracking were measured (Chapter 5). As ravelling was not considered a problem on asphalt concrete surfaces, data were recorded only for surface treatment surfacings. There was no definition of severity of ravelling included.

From a review of the data and inspection of the test sections, it was apparent that various phenomena were being included under the category of ravelling, as follows:

1. Stone loss by mechanical fracture (true ravelling);
2. Stone loss by loss of adhesion (stripping or contamination);
3. Scabbing - loss of a fragment of surfacing, such as of slurry seal, exposing the underlying bituminous surface;
4. Stone loss through lack of binder - where narrow longitudinal strips of basecourse or underlying surfacing had become exposed, attributable to faulty binder distribution at the time of construction; and
5. Ravelling of either the top or bottom layer of surfacing, without distinction.
As it was clear that some of these subcategories of "ravelling" could quite confidently be attributed to problems that manifestly had occurred during construction and had resulted in premature ravelling distress, a construction quality code (CQ) was defined as follows and assigned based on field inspection:

\[
CQ = 1 \text{ In the cases where the seal appeared to be streaky due to faulty binder distribution, or 100 percent loss of stone occurred within one to three years due apparently to loss of adhesion; and}
\]

\[
CQ = 0 \text{ In the absence of identifiable surfacing construction problems.}
\]

Careful attention was given to the classification of surface types as original or reseal, and by subcategory of surface treatment (chip seal, slurry seal, cold mix, etc.).

The approach to modelling the initiation and progression of ravelling was similar in most respects to the approach used for cracking, and simplified by the fact that there was only one class of severity.

Ravelling initiation

For the initiation of ravelling, the probabilistic failure-time method of analysis was again applied because all the conditions that made it appropriate for cracking initiation were considered valid also for ravelling. Once more the data included censored observations, those events that were not directly observed during the study period. Also the various elements of the surfacing can be regarded as independent, all elements being subject to homogeneous conditions of traffic and environment and the materials being essentially homogeneous, though with stochastic variations of material properties. Thus the assumptions applying to the use of a Weibull distribution, and the distribution of minima, remained valid for ravelling initiation.

The ravelling data were processed in the same way as for cracking. Initiation was defined to occur when 0.5 percent of the area was classified as ravelled, and the initiation events were coded separately by "window code" indicating whether initiation occurred prior to, at, or after the observation date.

Those events observed during the study were deemed to occur on the survey date if the first positive area of ravelling observed was between 0.5 and 5 percent, and on the previous survey data if the first area exceeded 5 percent. The data included both original surfacings and reseals on granular base pavements. While the base type does not affect ravelling per se, cemented bases were omitted from the analysis because that data may have included effects of crack spalling. The scope of the data base is summarized in Table 6.1.

Models were estimated for many combinations of surface type, traffic parameter (equivalent axle loadings with varying load damage powers from 0 to 6), pavement strength parameter (deflection, structural number), and interactive combinations of these terms. No strength parameters were found to have a significant influence on ravelling initiation. Traffic flow was found to have a significant
Table 6.1: Range of data on ravelling of surface treatments (chip seal), slurry seal and cold open-graded asphalt mixes in Brazil road costs study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Unit</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Minimum</th>
<th>Maximum</th>
<th>No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age at initiation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- prior to study</td>
<td>TY_rav</td>
<td>years</td>
<td>5.88</td>
<td>3.99</td>
<td>0.19</td>
<td>17.74</td>
<td>228</td>
</tr>
<tr>
<td>- observed</td>
<td></td>
<td>years</td>
<td>13.79</td>
<td>2.63</td>
<td>10.97</td>
<td>15.22</td>
<td>5</td>
</tr>
<tr>
<td>- later than study</td>
<td></td>
<td>years</td>
<td>6.83</td>
<td>3.55</td>
<td>1.13</td>
<td>17.74</td>
<td>93</td>
</tr>
<tr>
<td>Surface type</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- double surface treatment</td>
<td>DST</td>
<td>no.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>116</td>
</tr>
<tr>
<td>- slurry seal reseal</td>
<td>SLU</td>
<td>no.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>96</td>
</tr>
<tr>
<td>- cold mix (open-graded)</td>
<td>CMIX</td>
<td>no.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>16</td>
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<tr>
<td>- asphalt concrete</td>
<td>AC</td>
<td>no.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Traffic</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- average daily traffic</td>
<td>YAX</td>
<td>veh/day</td>
<td>1,700</td>
<td>1,800</td>
<td>100</td>
<td>4,500</td>
<td>114</td>
</tr>
<tr>
<td>- all axles</td>
<td>YAX</td>
<td>M/la.yr</td>
<td>0.645</td>
<td>0.895</td>
<td>0.033</td>
<td>3.821</td>
<td>114</td>
</tr>
<tr>
<td>- equivalent axle loads</td>
<td>YB4</td>
<td>MESA_4/</td>
<td>0.194</td>
<td>0.404</td>
<td>0.007</td>
<td>2.810</td>
<td>228</td>
</tr>
<tr>
<td>Pavement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- construction quality</td>
<td>CQ</td>
<td>-</td>
<td>0.162</td>
<td>-</td>
<td>0(191)</td>
<td>1(37)</td>
<td></td>
</tr>
<tr>
<td>- deflection</td>
<td>DEF</td>
<td>mm</td>
<td>0.79</td>
<td>0.37</td>
<td>0.26</td>
<td>2.02</td>
<td>228</td>
</tr>
<tr>
<td>- structural number</td>
<td>SNC</td>
<td>-</td>
<td>3.86</td>
<td>0.88</td>
<td>2.71</td>
<td>7.72</td>
<td>228</td>
</tr>
</tbody>
</table>

1/ The age data for "prior" initiation events represent the surfacing ages at the beginning of the study, and for "later" initiation events, the surfacing ages at the end of the study.

Source: Analysis of Brazil-UNDP study data.

and reasonable effect only when the data were distinguished by surface type and by the group of pavements that had manifest surface construction quality problems. The final model was very simple, using only three explanatory parameters, as follows:

\[ TY_{rav}(sp) = K_{sp} \cdot s \cdot \exp(-0.655 \cdot CQ - 0.156 \cdot YAX) \]  \hspace{1cm} (6.1)

where \( TY_{rav}(sp) \) = predicted age of surface treatments at the initiation of ravelling, with probability of survival \( sp \), in years;

\( CQ \) = construction quality (0 if no faults, 1 if faulty);

\( YAX \) = annual flow of all vehicle axles, millions/lane/year;
\[ a_s = \text{constant related to surfacing type, e.g.,} \]
\[ \text{chip seal} : a_s = 10.5; \]
\[ \text{slurry seal} : a_s = 14.1; \]
\[ \text{cold-mix} : a_s = 8.0; \text{ and} \]
\[ K_{sp} = \text{factor depending on probability of survival, } sp, \text{ as defined in Table B.1 for } \beta = 2.472 \text{ (also see later Figure 6.4(b)). } K_{sp} = 1 \text{ for mean value.} \]

The statistics of the model estimation are given in Table 6.2, for the total of 228 observations and wide range of surfacing ages from less than 1 year up to nearly 18 years. The goodness of fit of the predictions on the Brazil data base is shown in Figure 6.2 in which the symbols indicate the prior, observed or future status of the observation of failure, in the same way as for the cracking initiation predictions.

Considerable scatter is evident in the goodness of fit, which is not surprising given the stochastic nature of the ravelling mechanism and the large variation to be expected, even within surface type, due to construction practice and material specification effects. It is noticeable that the prior and future events are generally well explained by the model (lying generally above and below the line of equality respectively) and have evidently constrained the estimation effectively. It should also be noted that a certain amount of bunching is evident in the predictions; for example, all chip seal surface treatment predictions tend to be bunched about 10 years although the observed ages at initiation ranged

**Figure 6.2: Goodness of fit of probabilistic prediction model for initiation of ravelling on Brazil data**

![Graph showing goodness of fit](image)

*Note:*  
- ••• Distress occurred before first observed age  
- ■■■ Distress occurred at observed age  
- +++ Distress yet to occur, after last observed age

*Source:* Equation 6.1 on data from Brazil-UNDP study.
Table 6.2: Initiation of ravelling: estimation of probabilistic model for various surface treatments (various)

\[
TY_{rav} = B(\beta) \exp (2.108 - 0.279 \text{CMIX} + 0.293 \text{SLU} - 0.655 \text{CQ} - 0.156 \text{YAX})
\]

Double surface treatment mean \( (TY_{rav}) = 10.5 \exp(-0.655 \text{CQ} - 0.156 \text{YAX}) \)
Slurry seal: mean \( (TY_{rav}) = 14.1 \exp(-0.655 \text{CQ} - 0.156 \text{YAX}) \)
Cold mix: mean \( (TY_{rav}) = 7.97 \exp(-0.655 \text{CQ} - 0.156 \text{YAX}) \)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimate</th>
<th>t-statistic</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Units</th>
</tr>
</thead>
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<tr>
<td>Dependent</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TYRAV _rav</td>
<td>1.00</td>
<td>-</td>
<td>6.00</td>
<td>4.07</td>
<td>0.19</td>
<td>17.74</td>
<td>yr</td>
</tr>
<tr>
<td>Independent</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \beta )</td>
<td>2.472</td>
<td>12.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B(\beta)</td>
<td>1.279</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Constant</td>
<td>2.108</td>
<td>35.5</td>
<td></td>
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<td>CMIX</td>
<td>-0.279</td>
<td>-1.32</td>
<td>0.07</td>
<td>(n=16)</td>
<td>0</td>
<td>1</td>
<td>-</td>
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<tr>
<td>SLU</td>
<td>0.293</td>
<td>2.39</td>
<td>0.395</td>
<td>(n=96)</td>
<td>0</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>CQ</td>
<td>-0.655</td>
<td>-7.11</td>
<td>0.162</td>
<td>(n=37)</td>
<td>0</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>YAX</td>
<td>-0.156</td>
<td>-2.80</td>
<td>0.722</td>
<td>1.232</td>
<td>0.033</td>
<td>3.8</td>
<td>M axles</td>
</tr>
<tr>
<td>DEF</td>
<td>-</td>
<td>n.s.</td>
<td>0.788</td>
<td>0.373</td>
<td>0.261</td>
<td>2.02</td>
<td>mm</td>
</tr>
<tr>
<td>SNC</td>
<td>-</td>
<td>n.s.</td>
<td>3.86</td>
<td>0.88</td>
<td>2.71</td>
<td>7.72</td>
<td>mm</td>
</tr>
</tbody>
</table>

No. Observations: 228
Average log likelihood = 1.212
Quartile probabilities = 0.68 \( TY_{rav} \), 1.29 \( TY_{rav} \)
\[ \pm 1.08 \text{ yr (DST)} \]
Average 95\% confidence intervals = \[ \pm 2.81 \text{ yr (SLU)} \]
\[ \pm 2.62 \text{ yr (CMIX)} \]

Correlation Coefficients

<table>
<thead>
<tr>
<th>Variable</th>
<th>TY_rav</th>
<th>CMIX</th>
<th>SLU</th>
<th>CQ</th>
<th>YAX</th>
<th>DEF</th>
</tr>
</thead>
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<tr>
<td>1. TY_rav</td>
<td>1.000</td>
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<td></td>
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<tr>
<td>2. CMIX</td>
<td>-0.160</td>
<td>1.000</td>
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<td>3. SLU</td>
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<td>1.000</td>
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<tr>
<td>4. CQ</td>
<td>-0.135</td>
<td>0.065</td>
<td>-0.209</td>
<td>1.000</td>
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</tr>
<tr>
<td>5. YAX</td>
<td>-0.310</td>
<td>-0.020</td>
<td>0.228</td>
<td>-0.074</td>
<td>1.000</td>
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<tr>
<td>6. DEF</td>
<td>-0.067</td>
<td>-0.013</td>
<td>-0.079</td>
<td>-0.289</td>
<td>0.111</td>
<td>1.000</td>
</tr>
<tr>
<td>7. SNC</td>
<td>-0.026</td>
<td>0.041</td>
<td>0.315</td>
<td>-0.105</td>
<td>0.376</td>
<td>0.042</td>
</tr>
</tbody>
</table>

Note: Method - Maximum likelihood estimation, Weibull distribution (see Appendix B). CMIX, SLU = constants for open-graded cold mix and slurry seal, respectively.
CQ, YAX defined in Equation 6.1. DEF, SNC defined in Chapter 4.
Source: Analysis of Brazil-UNDP study data.
generally between 4 and 14 years. These are variations that were not explainable by any of the available parameters - traffic has a relatively minor effect on the predicted age, and all remaining variation is due to undefined material property and construction quality variations.

Ravelling progression

As in the case of cracking progression, it was found that the time-series data of ravelled area was best represented by a sigmoidal (S-shaped) function, the area being normalized as a percentage of the test subsection area. The rate of ravelling progression was computed as the difference in areas of ravelling observed between consecutive condition survey dates divided by the period of time between the surveys. It was estimated from the data by linear regression on log-transformed data with correction for the log-mean bias, and confirmed also by nonlinear regression, resulting in the following model expressed in derivative form:

\[
d\text{ARAV}_t = 4.42 \text{ SRAV}_t^{0.648} \quad dt
\]

where \(d\text{ARAV}_t\) = increment of ravelling area, percent of carriageway area; \(dt\) = incremental time, years; and \(\text{SRAV}_t\) = sigmoidal function of ravelling area, \(\text{ARAV}_t\) at time \(t\), \(\text{min}(\text{ARAV}_t, 100 - \text{ARAV}_t)\).

This when integrated yields the following expressions for the absolute area of ravelling at time \(t\) since initiation:

\[
\text{ARAV}_t = (1-z) 50 + z \left[ a b t + z 0.5^b + (1-z) 50^b \right]^{1/b}
\]

where \(a = 4.42\)
\(b = 0.352\)
\(z = 1\) if \(t < T_c\); and \(z = -1\) otherwise; and
\(T_c = (50^b - 0.5^b)/a b\); i.e. the time to 50% area ravelled
\(t\) = time since initiation, in years.

and for the time to reach a given area, since initiation:

\[
t = \left[ (1-z) 50^b + z \text{SRAV}_t^b - 0.5^b \right] / a b
\]

where \(z = 1\) if \(\text{ARAV}_t < 50\); and \(z = -1\) otherwise.

The statistics of the model estimation and the range of values of the explanatory variables tested are given in Table 6.3, and the scatter diagram of the observed and predicted values is given in Figure 6.3. Again a high degree of scatter is evident due to the many unquantifiable variables that affect the ravelling mechanism, but the model explains 41 percent of the data, which is a satisfactory result for this mode of distress. The rate of progression could be related only to time and not to traffic or to equivalent axle loading (a small effect of traffic was found in a contrary sense, increasing the time-rate of ravelling as traffic flow decreased, but this was insignificant and considered spurious). Insufficient data (27 of 338 observations) were available for surface types other than original double surface treatment (chip seal) so all data were pooled.
Table 6.3: Progression of ravelling: surface treatments (various)

\[ \ln(\Delta RAV/\Delta T) = 0.990 + 0.648 \ln MSRAV \]
\[ \Delta RAV = 4.42 SRAV^{0.648} \Delta T \]

Range of values in analysis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimate</th>
<th>t-statistic</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dependent</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(\ln(\Delta RAV/\Delta T))</td>
<td>1.00</td>
<td>-</td>
<td>2.414</td>
<td>1.290</td>
<td>-0.948</td>
<td>5.672</td>
<td></td>
</tr>
<tr>
<td>Independent</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intercept</td>
<td>0.990</td>
<td>9.15</td>
<td>2.198</td>
<td>1.270</td>
<td>-0.598</td>
<td>3.911</td>
<td></td>
</tr>
<tr>
<td>(\ln SRAV)</td>
<td>0.648</td>
<td>15.19</td>
<td>(15.6)</td>
<td>(15.3)</td>
<td>(0.5)</td>
<td>(100)</td>
<td>%</td>
</tr>
<tr>
<td>Others tested</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AGES</td>
<td>-</td>
<td>n.s.</td>
<td>8.38</td>
<td>4.88</td>
<td>0.50</td>
<td>20.73</td>
<td>yr</td>
</tr>
<tr>
<td>DEF</td>
<td>-</td>
<td>n.s.</td>
<td>0.63</td>
<td>0.29</td>
<td>0.15</td>
<td>1.63</td>
<td>mm</td>
</tr>
<tr>
<td>YAX</td>
<td>-</td>
<td>n.s.</td>
<td>0.62</td>
<td>0.36</td>
<td>0.04</td>
<td>1.50</td>
<td>M.axles/yr</td>
</tr>
<tr>
<td>HS</td>
<td>-</td>
<td>n.s.</td>
<td>34.2</td>
<td>19.2</td>
<td>20.0</td>
<td>118.0</td>
<td>mm</td>
</tr>
</tbody>
</table>

No. Observations: 338  \(r^2 = 0.407\)  Standard error (\(\ln(\Delta RAV/\Delta T)\)) = 0.995
Transformation correction factor = 1.641  Coefficient of variation = 41.2%

Correlation Coefficients

<table>
<thead>
<tr>
<th>Variable</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. (\ln(\Delta RAV/\Delta T))</td>
<td>1.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. (\ln SRAV)</td>
<td>0.638</td>
<td>1.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. (\ln AGES)</td>
<td>0.117</td>
<td>0.254</td>
<td>1.000</td>
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<td></td>
</tr>
<tr>
<td>4. (\ln DEF)</td>
<td>-0.097</td>
<td>-0.057</td>
<td>0.028</td>
<td>1.000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5. (\ln YAX)</td>
<td>-0.124</td>
<td>0.068</td>
<td>0.111</td>
<td>-0.001</td>
<td>1.000</td>
<td></td>
</tr>
<tr>
<td>6. (\ln HS)</td>
<td>-0.009</td>
<td>-0.081</td>
<td>-0.172</td>
<td>-0.041</td>
<td>0.042</td>
<td>1.000</td>
</tr>
</tbody>
</table>

Note: The values in parentheses below the natural logarithm variables are range statistics of the variable before log transformation. Method: Ordinary least squares linear regression on logarithmic transformation. AGES = age of surfacing, years. HS = surfacing thickness, mm.

Source: Analysis of Brazil-UNDP study data.
6.2.3 Predictions and Engineering Interpretation

The general form of the model predictions is shown in Figure 6.4(a) and the survivor function representing variability of life about the predicted mean in Figure 6.4(b). The expected life before ravelling is seen to diminish with increasing traffic flow. The life however is not highly sensitive to traffic flow, decreasing by approximately 14 percent per million vehicle axles per lane, or by about 5.4 percent per 1,000 vehicles per day in two directions (ADT). In reality it is possible that the kneading action of traffic might produce a beneficial effect at low traffic volumes, through the binder film being raised around the particles as the stones are embedded and thus countering the aging or oxidation effects. Such a beneficial effect would show as a slightly convex parabolic asymptotic shape at low traffic volumes but, given the high variability in the data due to other unquantified factors, such refinements were impossible to estimate. In fact experience has dictated the common practice that the designed rate of application of binder be increased slightly as the expected traffic volume decreases for just this reason, while at the same time avoiding an excessive application that might lead to the opposite problem of bleeding.

The variability evidenced by the data is considerable, as shown in the survivor function Figure 6.4(b). It indicates that about 80 percent of the individual observed lives for a given surface type and traffic flow spanned a
Figure 6.4: Model predictions for the time to raveling initiation of various surface treatment types as a function of traffic flow and construction quality

(a) Predicted Mean Time to Initiation

(b) Variability of Lives about Predicted Mean Life

Note: $K_{ps}$ is defined by Equation 8.13 for $\beta = 2.472$ and $ps = 100 (1-p)$.

Source: Equation 6.1.
range from about 45 to 157 percent of the mean life observed for those same conditions. This is similar to the variability determined for cracking initiation in original surfacings of flexible pavements, see Section 5.4.

Different curves are shown in Figure 6.4(a) for the three main surfacing types included in the study. It is apparent that in the Brazil data, slurry seal were more durable in respect of ravelling than chip seals (double surface treatment), which in turn were more durable than the open-graded cold-mixed asphalt. This ranking of durability is consistent with the binder film thicknesses and void contents of these surfacings which determine the relative susceptibilities of these materials to oxidation. For example, an open-graded mixture has a very short diffusion path through the binder film (unless it is thick) and much of the film is exposed to air through the interconnected voids.

The relationship for the "chip seal" or conventional double surface treatment is probably dominated by the behavior of the top layer of fine, void-filling aggregate. In the Brazil study area, the top layer was usually a 10 mm stone and the lower layer a 19 or 16 mm stone, and the application rates were such that the resulting surface comprised variegated sizes. The ravelling observed on the study sections was often the loss of the fine stones of that top layer. This type of surface treatment is to be distinguished from the single stone size appearance of surfaces common for example in Australia and New Zealand, in which 14 or 16 mm stone sizes are employed in the top layer, largely in order to achieve a thicker binder film and thus a more durable surfacing than is possible with smaller stone sizes. Such surfacings may be expected to perform better than indicated by the model, even to the extent that ravelling may hardly ever be observed, for example as reported by Clouston (1984) (in which cases other types of distress develop as discussed later).

In the case of slurry seals, the performance data are for a 5 mm maximum size graded aggregate slurry (conforming approximately with Type II of specification A105, International Slurry Seal Association (ISSA) (1978), and British Standard BS 434: Part 2: 1973), applied as a maintenance seal to either asphalt concrete or double surface treatments. Generally, ravelling under traffic can be controlled through the mix design process by varying the volume of binder in the mix and testing by a wet-track abrasion test (ISSA Bulletin No. TB-100). In the Brazil study the loss of aggregate from the slurry matrix was not a common problem: in fact only 15 of the 96 subsections of slurry were observed to "ravel" and in most of those cases the distress was actually delamination or "scabbing". In other cases where a surface patch of slurry was worn away by abrasion under traffic this was not recorded as ravelling (for example, sections 022 and 030) - in some of these cases the slurry appeared to be a fine 2.36 mm mix (ISSA Type I) used usually for crack filling. The relationship for slurry seal is therefore not very strongly based and perhaps understates the effect of traffic, but it is considered reasonable for current applications.

The effects of traffic were not found to differ significantly for the various surfacing types. The effect of loading, estimated through various equivalent 80 kN single axle load factors, was found to be insignificant (see Section 9.5), which is consistent with the mechanism of distress discussed earlier. Structural strength parameters of the pavement were insignificant, as had been expected.

The construction quality class parameter (CQ) turned out to be highly significant and indicated that the expected life in respect of ravelling was
halved for those sections that appeared on inspection to be faulty (37 of 228). Most but not all 228 sections were inspected so it is conceivable that some of the other early failures might have been similarly classed. The class parameter is a qualitative rather than quantitative parameter because of the subjective element that was necessarily involved, and to some extent it is naturally biased towards those sections considered to ravel prematurely. The primary purpose of its inclusion, however, is to provide at least some quantitative estimate that would permit an economic evaluation of the value and effect of quality control on pavement performance.

The raveling progression predicted by the model of Table 6.3 is illustrated in Figure 6.5. This shows a single curve independent of traffic flow and pavement characteristics, applying to all the surfacing types studied, that is double surface treatment, thin slurry seals and open-graded cold-mix asphalt. It shows that raveling extends over 100 percent of the pavement area in an average of 4.1 years from the time of initiation. The raw data (see Table 6.3) indicated an average rate of progression of 23.4 percent per year. The range however was considerable, with more than 10 percent of the rates being less than 1 percent per year (these were mainly at the initial stages of raveling) and some 10 percent of the rates exceeding 50 percent per year, with a maximum of 100 percent in 4 months.

The observed rates of raveling progression are therefore highly variable, with the magnitude of the time taken for raveling to extend over the whole

**Figure 6.5: Model prediction for the progression of raveling area for all surface treatments over time**

![Graph showing raveling progression over time](source: Equation 6.3)
pavement surface ranging from 1.5 years at the 15th percentile to 11 years at the 85th percentile. Once again, as for cracking progression, the rate seems to be reflecting that the variability of failure times within a section (of all the segments of the surface, which is the area progression) is similar to the variability of failure times (ravelling initiation) between like sections; in this case the average of 4.1 years for progression compares with an average of about 6 years for initiation.

For engineering prediction purposes the only practical approach is to use the average progression rate given here by the single function in Equations 6.2-6.4.

6.2.4 Validation of Prediction Model

Direct validation of the ravelling models has not yet been undertaken through a lack of independent data sets. Instead, they have been validated indirectly by comparing the predicted time for a level of 50 percent ravelling to be reached, which is representative of typical criteria triggering resurfacing or other maintenance to be undertaken, with observed intervals between reseals or resurfacings. These predictions may be summarized as follows:

<table>
<thead>
<tr>
<th>Surface type</th>
<th>Average daily traffic (veh/day)</th>
<th>Predicted age (years) at 50 percent area ravelled resurfacing criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>500</td>
<td>5,000</td>
</tr>
<tr>
<td>Double surface treatment (chip seal)</td>
<td>12.2</td>
<td>9.8</td>
</tr>
<tr>
<td>Slurry seal (on chip seal)</td>
<td>15.6</td>
<td>12.4</td>
</tr>
<tr>
<td>Cold-mix (open-graded)</td>
<td>9.8</td>
<td>7.9</td>
</tr>
</tbody>
</table>

These predictions are generally consistent with observed reseal frequencies in the order of 8 years (or less) on moderately high volume roads and 12 years (or more) on low volume roads, for surface treatments. These are typical also of reseal frequencies in Australia, New Zealand, South Africa, and other countries. Some variation exists with age and seal type: in Tunisia for example, the life of coarse (19 mm stone size) surface treatments in the southern, arid regions (less than 400 mm precipitation per year) is about 15 to 17 years, which is longer than predicted because of the large stone size and thick binder film, but on more recent reseals, the life has often been much shorter, in the order of 6 to 10 years (SETEC in Paterson 1985a). It is thought probable that changes in the chemical composition of the bitumen has caused a change in performance, an issue which is receiving increasing attention elsewhere (see Petersen 1984).

The particular combination of slurry seal applied on top of a surface treatment, or chip seal (often known as a Cape Seal when the surfacing is designed in this way rather than occurring as a result of maintenance) is known to be very durable because of the strong interlocking action of the slurry void-filling coat and its high resistance to oxidation. The excellent performance of the Cape Seal is demonstrated by the following data on life before resealing, under traffic of 200 to 4,000 vehicles per day in two directions (Biesenbach and Alexander 1979):
The predictions above are therefore of the correct order — slightly conservative, perhaps, in underestimating the lives of high quality surfacings — and appropriate for the general level of surfacing technology in a wide range of countries. Adjustment to specific local technology can be effected by applying a multiplier to the prediction of \( T_{Y_{rav}} \) to calibrate the age (binder-oxidation) component, namely:

\[
T_{Y_{rav}} = K_{rv} K_{sp} a \exp (-0.655 CQ - 0.156 YAX)
\]

where \( a \), \( T_{Y_{rav}} \) are as defined as above, \( K_{rv} \) = adjustment factor for oxidation-aging susceptibility for local binder and construction technology; and \( K_{sp} \) = factor depending on desired probability of survival, shown in Figure 6.4(b).

### 6.3 POTHOLES

Potholes are the most visible and severe form of pavement distress. Extreme costs can be incurred by the user through tire blowout, or damage to the wheels or suspension system through the high impact forces occurring when a vehicle strikes a pothole. Vehicle speeds are reduced significantly in order to avoid potholes or to minimize their dynamic impact. For these reasons, routine maintenance to patch potholes is virtually unquestioned as essential to the most minimum serviceability of a pavement. The influence of potholes on safety however is negligible for holes of less than 80 mm depth and 1000 mm length, according to Zimmer and Ivey (1984), but becomes significant when drivers take hazardous evasive action (see also Transportation Research Board 1984).

Despite such maintenance being "essential", and despite potholing being perhaps the least predictable form of distress, it is necessary to include some prediction of potholing and its impact on vehicles in an economic evaluation model such as HDM to serve as the economic penalty of deferred or neglected maintenance. Quantification of these predictions is the aim of this section.

#### 6.3.1 Potholing Mechanism

Potholing results from the disintegration and loss of surfacing material and, subsequently, base material. In order to distinguish this from ravelling, a pothole is defined as follows:

"A pothole is a cavity in the road surface which is 150 mm or more in average diameter and 25 mm or more in depth."

The dimensions are the minimum that affect the motion of a car wheel and measured roughness significantly, and are consistent with Hide and Keith (1979). Shallower depths are typical of ravelling.
In surface treatments, potholes may develop either from ravelling which has exposed the base, or from wide cracking which has spalled or reached such intensity (typically 50 to 100 mm spacing) that fragments are easily removed. In asphalt surfacings, potholes develop from when wide cracking becomes intense (say 50 to 300 mm spacing) or shows spalling, which typically must be severe before interlocking effects within the relatively thick surfacing are overcome.

The bond of the surfacing to the base and the nature of the base strongly influence the initiation, shape and size of a pothole. An adequate tack or prime coat with good penetration and bond to the base significantly retards potholing after cracking. An interlayer of fines at the top of the base, such as remains sometimes at construction after the slushing of a crushed stone base or retarded cementation of a cemented material, effects easy separation of the surfacing after cracking and thus promotes both the initiation and the rate of progression of potholing. Untreated base materials, excepting highly compacted densely-graded crushed stone or self-cemented gravels (e.g., laterites, calcretes and ferricretes), disintegrate rapidly under the action of all vehicular traffic, particularly in the presence of water. Bituminous base materials rarely disintegrate, except when they are susceptible to stripping or when the binder film is brittle and thin; the latter cause is typical for many penetration macadam pavements built in developing countries.

The diameter of the pothole depends on the condition of the surrounding surfacing and its ability to withstand attrition. In asphalt surfacings, it will be confined to the area of wide, intense and severely spalled cracking, and may be sharp-edged. In thin, brittle surface treatments, it progresses rapidly to 400 m or 1000 mm diameter and is usually dish-shaped. The depth may remain as shallow as 25 to 50 mm when the base is strongly resistant to disintegration. Otherwise, the depth is largely dependent on the volume of traffic striking the pothole and the presence of water, which greatly increases the scouring action. Plastic gravels are particularly susceptible to water action even though they often perform excellently when the base is well-sealed or remains dry. Typically the depth is unlikely to exceed one quarter of the diameter of the pothole, and rarely exceeds 100 mm.

Potholing is therefore usually the ultimate form of distress consequential upon deferred maintenance of severe cracking or ravelling, and is likely to be most severe in wet climates and low-standard base materials. Random potholing in otherwise undistressed pavements does also occur occasionally and can usually be attributed to a random defect in materials, for example organic matter in asphalt surfacing or a clayey "soft spot" in a base.

6.3.2 Empirical Prediction of Initiation and Progression of Potholes

As the factors affecting both the initiation and progression of potholing are diverse and highly dependent upon specific material and construction properties, it is not practicable, and in most instances not useful, to develop a model including such properties. Most appropriate would be a statistical estimation of the likelihood of occurrence given the surface condition, base type, traffic and climate. The data available however are insufficient to estimate such a model statistically, and therefore simple models were constructed based on data collected in the Caribbean (Hide and Keith 1979), Ghana (Roberts 1983), Kenya (Jones and Rolt 1983) and Brazil (GEIPOT 1982 as presented in World Bank 1985) as follows.
A study on St. Vincent in the Caribbean (Hide and Keith 1979) collected data on severely deteriorated low-volume roads in urgent need of rehabilitation. On some roads, extensive cracking and an incidence of up to one pothole per linear meter of road length were commonplace, and in many cases the whole surfacing had disintegrated causing vehicles to drive on the exposed boulder base. The pavements were thin, constructed with a grouted bitumen penetration macadam surfacing of 25 mm stones blinded with 13 mm stone and 6 mm dust, ranging in age from 8 to 26 years and carrying 110 to 1,540 vehicles per day. Patching repair of the potholes was found to deteriorate rapidly as soon as water penetrated, and a surface dressing program was undertaken using labor-intensive sand-sealing. Data from six lengths of road are summarized in Table 6.4, giving the average volume and area of patching (which are equated to the volume and area of potholing) deduced from patching costs; the road roughness measured before patching, after patching and after sand-sealing (Bump Integrator roughness values have been converted to International Roughness Index units (m/km IRI), see Table 2.1); and the average daily traffic. The progression of potholing in the worst case was equivalent to 7 percent of pavement area per year under traffic of 1,540 veh/day. The data indicate that potholing increased at the rate of approximately one percent area per year per 200 veh/day ADT. This is applicable to thin pavements with penetration macadam surfacings and a pavement modified structural number of less than 3.

Data from a study of surface treatment pavements in Ghana (Roberts 1982) are summarized in Table 6.5. The incidence of potholing observed ranged from 0 to 680 m²/km on roads ranging in width from 5.5 to 7.4 m under traffic volumes of 23 to 414 commercial vehicles per day. Analysis of the data indicates that the rate of potholing progression ranged from 0.1 to 9 percent of pavement area per year. Potholing appeared to be associated with average cracking intensities in excess of 0.5 m/m², and the rate of potholing progression was higher in the regions of moderately high rainfall (1600 - 1900 mm/year) than in those of lower rainfall (900 - 1000 mm/year).

Table 6.4: Pothole and patching data from thin bitumen penetration macadam pavements on St. Vincent, Caribbean

<table>
<thead>
<tr>
<th>Units</th>
<th>Poorest 3 roads</th>
<th>Best 3 roads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annual patching cost</td>
<td>ECS$/km</td>
<td>6,200</td>
</tr>
<tr>
<td>Annual volume of patching²</td>
<td>m³/km</td>
<td>40 - 50</td>
</tr>
<tr>
<td>Pothole frequency</td>
<td>per km</td>
<td>700 - 1,000</td>
</tr>
<tr>
<td>Annual area of patching²</td>
<td>m²</td>
<td>7 - 12</td>
</tr>
<tr>
<td>Roughness²</td>
<td>m/km IRI</td>
<td>10.4</td>
</tr>
<tr>
<td>Before patching</td>
<td>6.23</td>
<td>5.82</td>
</tr>
<tr>
<td>After patching</td>
<td>5.81</td>
<td>5.25</td>
</tr>
<tr>
<td>Change to due patching</td>
<td>-4.2</td>
<td>-3.0</td>
</tr>
<tr>
<td>Average daily traffic (ADT)</td>
<td>veh/day</td>
<td>760</td>
</tr>
<tr>
<td>Range of ADT</td>
<td>180 - 1,540</td>
<td>110 - 250</td>
</tr>
</tbody>
</table>

1/ Costs in 1979 Eastern Caribbean dollars (ECS), where ECS2.7 = US$1.00.
2/ Deduced from text and cited unit costs of approximately ECS120/m³.
3/ Assuming a depth:diameter ratio of one-sixth, and given road widths of 4 to 5 m.
4/ Converted from Bump Integrator (BI) values by IRI = 0.0032 BI¹.89.

Source: Adapted from Hide and Keith (1979).
Table 6.5: Initiation and progression of potholing observed on surface treatment pavements in Ghana

<table>
<thead>
<tr>
<th>Section number</th>
<th>Traffic ADH³/</th>
<th>Age²/</th>
<th>Rainfall mm/yr</th>
<th>Initial potholes area %</th>
<th>Pothole progression %/yr</th>
<th>Cracking intensity³/ m/m²</th>
<th>MAPOT (YHX/SNC)³/</th>
<th>YHX million heavy vehicle axles/lane/year (≈ 0.00048 ADH); SNC modified structural number</th>
</tr>
</thead>
<tbody>
<tr>
<td>1B1</td>
<td>146 N</td>
<td>1020</td>
<td>0.01</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>3B1</td>
<td>189 0</td>
<td>&quot;</td>
<td>3.2</td>
<td>0.4</td>
<td>0.1-0.8</td>
<td>10</td>
<td></td>
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</tr>
<tr>
<td>4B1</td>
<td>80 0</td>
<td>&quot;</td>
<td>0.01</td>
<td>0</td>
<td>0.1-0.5</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2B2</td>
<td>23 N</td>
<td>1600</td>
<td>0.9</td>
<td>0.15</td>
<td>0.1-0.8</td>
<td>27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3B2</td>
<td>179 0</td>
<td>&quot;</td>
<td>0.3-4.5</td>
<td>0.1-0.8</td>
<td>0-1.0</td>
<td>6+</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4B2</td>
<td>36 0</td>
<td>&quot;</td>
<td>1.0-5.0</td>
<td>0.6-9.0</td>
<td>0.2-0.6</td>
<td>60+</td>
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</tr>
<tr>
<td>5B2</td>
<td>154 0</td>
<td>&quot;</td>
<td>0.2-1.0</td>
<td>0.1-0.3</td>
<td>0-0.35</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1B3</td>
<td>68 N</td>
<td>900</td>
<td>0.01</td>
<td>0.1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>2B3</td>
<td>62 0</td>
<td>&quot;</td>
<td>0.01</td>
<td>0.1</td>
<td>0.1</td>
<td>7</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>3B3</td>
<td>180 0</td>
<td>&quot;</td>
<td>0.1</td>
<td>0.2</td>
<td>0-0.9</td>
<td>5</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>1B4</td>
<td>414 0</td>
<td>1900</td>
<td>0.2-4.5</td>
<td>0.3</td>
<td>0-0.7</td>
<td>5</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>2B3</td>
<td>60 0</td>
<td>&quot;</td>
<td>0.01</td>
<td>0.1</td>
<td>0</td>
<td>7</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>5B4</td>
<td>77 N</td>
<td>&quot;</td>
<td>0.01</td>
<td>0-1.2</td>
<td>0-0.4</td>
<td>30</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>6B4</td>
<td>40 N</td>
<td>&quot;</td>
<td>0-2.5</td>
<td>0.6-5.0</td>
<td>0-0.8</td>
<td>60+</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

1/ Average daily heavy (commercial vehicle) traffic in two directions (veh/day).
2/ Construction dates not known: N(new) = less than 5 years old; O(old).
3/ Ten samples 1 m square per km.
4/ AAPOT = annual increment of potholing area (col. 6);
YHX = million heavy vehicle axles/lane/year (≈ 0.00048 ADH);
SNC = modified structural number.

Source: Adapted from Roberts (1982).

In Kenya (Jones and Rolt 1983), one of the sections of the 1975 road costs study, a cement-treated base pavement (Section OB19) on the Nairobi-Mombasa road, was given only routine patching maintenance over a three-year period (1975 to 1978) after cracking had reached severe levels (covering 100 percent of the area with intensity exceeding 3 m/m²). Potholing was not recorded separately but was estimated to cover 5 percent of the pavement area.

In the Brazil study (GEIPOT 1982), although the null maintenance policy required the immediate patching of all potholes and the roughness measurements were required to avoid potholes in order to prevent damage to the equipment, a few observations were recorded (see World Bank 1985). Isolated potholing of less than 0.2 percent of pavement area was recorded on 25 of the 380 subsections when the combined areas of cracking and ravelling were less than 10 percent. On at least another 8 subsections, larger areas of potholing developed when either the cracking or ravelling covered more than 60 percent of the area.

These data show considerable variability, with the initiation of potholing sometimes preceding cracking or ravelling and sometimes not occurring even after wide cracking or ravelling reach 100 percent. In general however it appears reasonable to state that the probability of potholes developing becomes significant when the area of wide cracking exceeds 20 percent or the area of ravelling exceeds 30 percent. As the thickness of the surfacing and the volume of traffic are likely to affect the timing of potholing and are important to an economic
evaluation, these conditions were incorporated together in an algorithm as follows.

The time \( T_{\text{MIN}} \) from the initiation of triggering distress (wide cracking or ravelling) to the initiation of potholing is expressed by:

\[
T_{\text{MIN}} = \begin{cases} 
\max [2 + 0.04 \text{HS} - 0.5 \text{YAX}; 2] & \text{if base is not cemented} \\
\max [6 - \text{YAX}; 2] & \text{if base is cemented}
\end{cases}
\] (6.6)

and because maintenance may have been undertaken during this time (and presumably to the most severe distress), the initiation of potholing at \( T_{\text{MIN}} \) is made conditional upon the time since initiation and area of distress satisfying the following:

1. Wide cracking:
   \( \text{AGES} - \text{TY}_{\text{cr4}} > T_{\text{MIN}} \) and \( \text{CR}_{4} > 20 \); or

2. Ravelling:
   \( \text{AGES} - \text{TY}_{\text{rav}} > T_{\text{MIN}} \) and \( \text{ARAV} \geq 30 \); or

3. Potholing: If potholing has been observed on the pavement at any previous time during the current surfacing period, then the probability of its recurring are considered high regardless of conditions (a) and (b) above, so that \( T_{\text{MIN}} \) would automatically apply.

where

\( T_{\text{MIN}} = \) predicted time between the initiation of either wide cracking or ravelling, whichever occurs earliest, and the probable initiation of potholing, years;

\( \text{HS} = \) total thickness of bituminous surfacing, mm;

\( \text{YAX} = \) annual number of vehicle axles, million axles per lane per year;

\( \text{AGES} = \) age of most recent surfacing, years;

\( \text{TY}_{\text{cr4}}, \text{TY}_{\text{rav}} = \) predicted surfacing ages for initiation of wide cracking and ravelling respectively, years; and

\( \text{CR}_{4}, \text{ARAV} = \) areas of wide cracking and ravelling respectively, percent of pavement area.

The devised algorithm is illustrated in Figure 6.6 where it can be seen that the delay before potholing occurs increases by one year per 25 mm thickness of surfacing and by one year per reduction of 2 million axles per lane per year. The chances of potholing occurring on thick bituminous-base pavements are much smaller than on a thin single surface treatment.

The progression of potholing area observed in the various studies ranged from 0.1 to 9 percent area per year. Using the available data as a guide and relating these to mechanistic parameters such as traffic flow, surfacing thickness and base quality, a group of algorithms were constructed in which the incremental area of potholing could derive from one of three sources, namely from wide cracking, ravelling or from the enlargement of existing potholes, as follows:

\[
\Delta \text{AAPOT} = \min \{\Delta \text{AAPOT}^{\text{cr}} + \Delta \text{AAPOT}^{\text{rv}} + \Delta \text{AAPOT}^{\text{pe}}; 10\}
\] (6.7)
Figure 6.6: Predicted minimum time between wide cracking and potholing initiations

![Diagram showing time to pothole initiation vs traffic volume and bituminous layer thickness]

Source: Equation 6.6.

where

\( \Delta APOT \) = the predicted change in the total area of potholes during the analysis year due to road deterioration (limited to a maximum of 10 percent per year), in percent;

\( \Delta APOT_{cr} \) = the predicted change in the area of potholes during the analysis year due to cracking;

\( \Delta APOT_{rv} \) = the predicted change in the area of potholes during the analysis year due to ravelling; and

\( \Delta APOT_{pe} \) = the predicted change in the equivalent area of potholes during the analysis year due to pothole enlargement.

As the area of potholing has to be related to the volume of potholing, both to estimate patching quantities and because roughness is linearly related to potholing volume (as shown next in Section 6.3.3), a standard conversion between them was adopted, assuming an average pothole depth of 80 mm:

\[
\Delta VPOT = 0.80 W_1 \Delta APOT \quad (6.8a)
\]

or simply (assuming an average lane width of 3.4 m):

\[
\Delta VPOT = 2.7 \Delta APOT \quad (6.8b)
\]

where

\( \Delta VPOT \) = volume of open potholes, \( m^3/\text{lane/km} \); and

\( W_1 \) = effective lane width (i.e., road width divided by the effective number of lanes), m.

The rates of progression due to cracking and ravelling were then assumed to take the form:

\[
\Delta APOT_{cr} = \min (2 CR_4 U; 6) \quad (6.9a)
\]
\[ \Delta A P O T_{rv} = \min (0.4 \text{ ARAV } U; 6), \] (6.9b)

where \[ U = \frac{(1 + CQ)}{(YAX/SNC)} \times 2.7 \text{ HS} \]

This is illustrated in Figure 6.7(a) for the cracking-induced and ravelling-induced components. The rate of progression is thus modelled to increase linearly as the total area of distress increases, and as the traffic volume increases. Basecourse and surfacing quality are important factors but difficult to quantify, thus the construction quality factor (CQ) and modified structural number (SNC) are incorporated as simple surrogates of quality. (Note: This is not intended to imply that the rate of potholing is physically related to pavement strength, which is unlikely; SNC is used simply as a convenient surrogate of base quality that is typically higher for high standard pavements.) As with initiation, thick surfacings are modelled to be less sensitive to pothole progression than thin surfacings.

The component for enlargement of the potholes represents the scouring action of water and traffic removing material from a pothole, and the erosion of the surfacing by spalling. In another algorithm devised to represent the effects in a sensible way, this is expressed as follows:

\[ \Delta A P O T_{pe} = \min \{ \Delta A P O T \left[ K_{\text{base}} \text{ YAX } (\text{MMP} + 0.1) \right]; 10 \} \] (6.10)

where \[ K_{\text{base}} = \begin{cases} 0.6 & \text{if base is cemented} \\ 0.3 & \text{if base is bituminous} \end{cases} \]

\[ \text{MMP} = \text{mean monthly precipitation, in meters.} \]

The algorithm is illustrated in Figure 6.7(b) for a granular base pavement. This component of pothole progression provides an accelerating rate of progression as a function of the area of potholing, that is in the absence of routine maintenance.

6.3.3 Effect of Potholes on Roughness

The economic penalty of potholing deterioration has to be derived through the impact of road roughness on vehicle operating costs. There is some evidence from the Caribbean study to suggest that operating costs on a severely deteriorated paved road (with extensive potholing and patching) may be higher than costs on an unpaved road at the same roughness level (Hide, Morosiuk and Abaynayaka 1983). This suggests that the roughness index may understate the extreme costs associated with sharp impacts, and whether this is a problem only with the Bump Integrator roughness scale used in that study (which is highly sensitive to short wavelength roughness, see Chapter 2) or with other scales as well, is undetermined at present. The trend suggests that the understating of costs might be even stronger for a roughness index such as IRI, which is a more attenuated scale than BI (Chapter 2).

As the Brazil study specifically excluded pothole effects from roughness measurements, and as the Caribbean study provides data on only one type of pothole (shallow and wide), a simulation study was undertaken to quantify the effects of pothole size and frequency on the ride response of a typical vehicle. Simulation
Figure 6.7: Prediction of annual progression of potholing area by components due to wide cracking, ravelling and enlargement

(a) Increment due to Wide Cracking and Ravelling

(b) Increment due to Enlargement

Note: Traffic Volume, YAX, in million axles/lane/year. HS as above.
Source: Equation 6.9.
is the most effective method of analysis, because the high peak suspension velocities caused by potholes exceed the capacity of most sensors and the limit of travel on suspension systems in vehicle-mounted response-type roughness measuring systems and create high risks of instrument and vehicle damage.

The method adopted was to apply a mathematical simulation of a moving vehicle on elevation data of various real road profiles, superimposed on which were potholes of various dimensions located at various intervals along the wheel-path.

The simulation selected was the quarter-car model of the International Roughness Index (Sayers, Gillespie and Paterson 1986) for reasons of standardization and because this model is the most widely accepted representation of typical passenger car response to roughness. As vehicle speeds are typically slower on surfaces with a significant incidence of potholing, the reference index \( \text{RARS}_0 \) at a standard simulation speed of 50 km/h was selected for the study instead of the IRI (which is \( \text{RARS}_0 \) at its standard speed of 80 km/h). This choice influences the waveband of roughness that will be sensed and includes roughness in the

| Table 6.6: Summary of dimensions and incidence of potholes and base road roughness used in roughness simulation study |
|---|---|---|
| **Parameter** | **Units** | **Matrix level** |
| **Dimensions of pothole** | | 1 | 2 | 3 |
| Size | mm | 30 | 70 | 50-100-50 |
| Depth | mm | 300 | 500 | 750 |
| Diameter | % lane | 0.0022 | 0.0059 | 0.0132 |
| Area | m² | 0.00141 | 0.0092 | 0.0295 |
| Volume | m³ | | | |
| **Incidence** | | 10 | 50 | 100 |
| Frequency | km⁻¹ | 100 | 20 | 10 |
| Spacing | m | | | |
| **Base road roughness** | | AC | DST | DST |
| Surfacing type | | | | |
| Roughness, IRI | m/km | 2.2 | 3.4 | 5.5 |
| Roughness, \( \text{RARS}_0 \) | m/km | 2.54 | 4.55 | 7.07 |
| Roughness, \( Q_{	ext{m}} \) | ct/km | 29 | 42 | 72 |

**Notes:** AC = Asphalt concrete; DST = Double surface treatment; IRI = International Roughness Index; \( \text{RARS}_0 \) = Reference average rectified slope statistic of the profile for a vehicle simulation speed of 50 km/h; \( Q_{	ext{m}} \) = Quarter-car Index of roughness as used in Brazil-UNDP study (see Chapter 2).

**Source:** This study.

\(^2/\) See Sayers, Gillespie and Queiroz (1986) for mathematical definition.
range of 0.8 to 20 m wavelength content, compared with the IRI range of 1.6 to 40 m wavelength content (Note: this range does not exclude potholes, which are typically less than 1 m in diameter, because the perturbation effect of potholes is attenuated through the vehicle speed and the oscillations are sensed as will be seen in the study results).

The study was designed to cover four sizes and shapes of pothole (including the null case), three pothole frequencies and three levels of base roughness (a total of 36 combinations) in order to establish the relationships between these primary parameters, as summarized in Table 6.6.

The three dimensions of pothole were selected from six options to cover the range of primary interest. The nominal surfacing areas were computed as circular areas and the volumes as half-obloids. The superposition of the potholes on the road profile data is shown in Figure 6.8. It should be noted that the simula-

Figure 6.8: Physical dimensions and disposition of potholes superimposed on real profile elevations for pothole simulation study

(a) BASE ROAD PROFILE

(b) 300mm DIAMETER POT-HOLE

(c) 500mm DIAMETER POT-HOLE

(d) 750mm DIAMETER POT-HOLE

Profile Elevations @ 250mm Spacing
Distance Between Potholes (m) = 1000/Pothole Frequency

Note: Vertical Scale Distorted
tion follows only the elevation data points and does not "see" the profile between these discrete points. As the simulation includes a smoothing or averaging of the road slope over a 250 mm long tire contact patch of the wheel, data at smaller intervals tend to be highly attenuated anyway. Variants of the shape of a pothole edge are therefore omitted: although these would affect the distortion of the tire carcass (and thus tire wear), they do not influence road roughness as it is defined herein.

Regular frequencies of 10, 50 and 100 potholes per kilometer were based on the studies reported earlier. As a two-wheelpath simulation was employed, the potholes were located in only one wheelpath, so the net displacement at the axle centre of the simulated vehicle was one-half of the vertical displacements given in Figure 6.8.

The three road profiles selected as base profiles were taken from data collected in Costa Rica (see Delgado 1984) comprising elevations at 250 mm intervals in two wheelpaths measured by rod and level survey. The 160 m long sections were extended by replication to the 320 m length required for the simulation.

Example results of the simulation are shown in Figure 6.9, which depicts the simulated vehicle response (average rectified slope, ARS, in m/km $R_{ARS_{50}}$) along the length of the section of medium roughness (3.4 m/km IRI) for the null condition and three pothole sizes, separately for each frequency of potholing. The response shown is the cumulative average ARS response of the vehicle and the sharp vertical rises represent the immediate effect of each pothole. The net effect of potholes on the ARS response of a vehicle is represented by the asymptote, or steady-state, value of the lower-bound envelope of the saw-tooth curves, which occurs towards the end of the section.

These asymptotic values are listed in Table 6.7 and represent the roughness "seen" by the vehicle when travelling on a potholed road surface in the conditions given by the various combinations of pothole size and frequency, and base road roughness. On the right-hand side of the table, the increase in roughness ($AR_{ARS_{50}}$), which is attributable to the potholes, is listed. A plot of the results in Figure 6.10 shows that the sensed roughness increases almost linearly with potholing frequency and increases with pothole size, but that the rates of increase appear to be independent of the base road roughness. When this was evaluated in terms of pothole size, surface area and volume, it was found that the effective increase in sensed roughness was a linear function of the total volume of potholing, as shown in Figure 6.11. By linear regression the relationship was:

$$AR_{ARS_{50}} = 7.45 V_{POT_{w}}$$  \hspace{1cm} (6.11)

where $AR_{ARS_{50}}$ = increase in the (simulated) roughness response of a vehicle (average rectified slope of axle-body displacement at 50 km/h) due to the presence of potholes, in m/km; and

$V_{POT_{w}}$ = total volume of potholes in a vehicle wheelpath, in m$^3$/lane/km.

This is an extremely useful result because it permits a simple and direct link between the volume of potholing, the volume of patching necessary for repair and their effects on the average roughness level sensed by traffic. Alternative
Figure 6.9: Evolution of simulated vehicle response (RARS$_{50}$) along road section of moderate roughness (3.4 m/km IRI) for various sizes and frequencies of potholing.

(a) Low Frequency: 10 Potholes/Lane km
- No Potholes.
- 300 mm diameter potholes.
- 500 mm diameter potholes.
- 750 mm diameter potholes.

(b) Medium Frequency: 50 Potholes/Lane km

(c) High Frequency: 100 Potholes/Lane km

Note: Initial roughness 3.4 m/km IRI. Holes equally spaced from 40m. Method Quarter-Car Simulation for 50 km/h speed from Sayers, Gillespie and Queiroz (1986).

Source: Author, and road profile data from Costa Rica (Delgado, 1984).
measures of the pothole size such as area, depth and diameter did not produce unique relationships over all conditions.

As the relationship above represents only those potholes directly in a wheelpath, calibration to real conditions that include avoided potholes is required. The Caribbean data cited in Table 6.4 indicates the following ratios between roughness change and patching volume per lane:

1. Poorest roads: \( k_{\text{pvol}} = \frac{\Delta R_{\text{patch}}}{\Delta V_{\text{patch}}(W/W_{l})} \)
   \( = 4.2/45 \ (3.3/4.2) \approx 0.13; \) and 

2. Other roads: \( k_{\text{pvol}} = 3.0/20 \ (3.3/4.2) \approx 0.3 \)

where \( k_{\text{pvol}} \) = measured ratio of increase in roughness to volume of potholes, in m/km IRI per m³/lane/km; 
\( \Delta R_{\text{patch}} \) = observed decrease in roughness due to patching, m/km IRI; 
\( \Delta V_{\text{patch}} \) = volume of patching, in m³/lane/km; and 
\( W, W_{l} \) = widths of pavement and lane respectively, in m.

Table 6.7: Roughness values and increments caused by various frequencies and sizes of potholes for three road profiles: results of simulation study

<table>
<thead>
<tr>
<th>Pothole size</th>
<th>( RARS_{s_{0}} ) (m/km) at frequency per km</th>
<th>( \Delta RARS_{s_{0}} ) (m/km) at frequency per km</th>
<th>Average ( \Delta RARS_{s_{0}} )</th>
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<tbody>
<tr>
<td>10</td>
<td>7.065</td>
<td>7.065</td>
<td>0</td>
</tr>
<tr>
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<td>7.065</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>7.223</td>
<td>7.835</td>
<td>8.434</td>
</tr>
<tr>
<td>4</td>
<td>8.126</td>
<td>11.791</td>
<td>16.409</td>
</tr>
<tr>
<td>5</td>
<td>9.338</td>
<td>17.244</td>
<td>27.433</td>
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</tr>
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</tr>
<tr>
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<td>4.788</td>
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</tr>
<tr>
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<td>5.175</td>
<td>9.375</td>
<td>14.325</td>
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<tr>
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<td>6.189</td>
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<tr>
<td>5</td>
<td>6.189</td>
<td>14.849</td>
<td>25.534</td>
</tr>
</tbody>
</table>

Notes: Pothole sizes:
- 0 = null condition;
- 1 - 30 mm depth, 300 mm dia., 0.00141 m³;
- 4 - 70 mm depth, 500 mm dia., 0.0092 m³;
- 5 - 100 mm depth, 750 mm dia., 0.0295 m³.
Profile data from sections in Costa Rica designated CR02, CR12 and CR09 for low, moderate and high roughness levels respectively.

Source: Author's analysis.
Figure 6.10: Simulated roughness as function of pothole frequency for three pothole sizes

![Graph showing simulated roughness as function of pothole frequency for three pothole sizes.]

Source: Table 6.7.

Figure 6.11: Increase in roughness as direct function of total potholing volume derived from simulation study

![Graph showing increase in roughness as direct function of total potholing volume.]

Note: Values of potholing volume represent combinations of pothole size and frequency.

Source: Table 6.7.
Adjusting the simulation study RARS\textsubscript{e}, index result to IRI roughness units by the factor 0.8 (see Table 2.1), we have:

3. Simulation result: \( kp_{vol} = 6.0 \text{ m/km IRI per m}^3/\text{lane/km} \)

The effect measured in the Caribbean was thus only about one-thirtieth of the simulated effect of pothole volume on roughness. In order to evaluate this large difference, we must first note an important difference between the two studies. In the Caribbean, the volume of patching that was equated to pothole volume was not concentrated in the wheelpaths and, because of the nature of the penetration macadam base which crumbles into a shallow depression with relatively gentle rather than sharp edges, we can expect many of these depressions to have been connected. Indeed, a volume of 45 m\(^3\)/km over a road width of 4.2 m indicates an average of only 11 mm depth of patching for the entire area; this compares with a maximum total pothole volume of 3 m\(^3\)/lane km in the simulation study. We can expect a factor down to about one-eighth or one-tenth for holes that are avoided by the measuring instrument (i.e., average hole diameter 0.5 m on a 4.2 m width), and the remaining factor of one-fourth is attributed to the shallow, interconnected shape of the depressions.

Which value therefore is most appropriate for use in predicting deterioration and maintenance effects? It is possible that the high coefficient of 6 m/km IRI per m\(^3\)/lane km of potholes applies to the discrete, sharply-edged, holes when they are struck by the vehicle, and the low coefficient to very widespread dish-shaped disintegration. For modelling purposes, the coefficient \( kp_{vol} \) should be between 0.16 and 0.75 m/km IRI per m\(^3\)/lane km of potholing so as to make up to one-eighth allowance for the avoidance of holes in a 3.3-3.6 m lane. Empirical data from the roughest paved section in the Brazil study, Section 112, largely confirms this range -- the roughness residual due to potholing was 2.1 m/km IRI and the area of holes encountered during measurement was of the order of 1 to 5 percent, bounding probably on the lower side, which thus favors a coefficient in order of 0.4 m/km IRI per m\(^3\)/lane km, or 1.0 m/km IRI per percent of lane area.

In the HDM-III model, the lowest coefficient, i.e. 0.16, was adopted as a conservative estimate and because the value was determined empirically. The form of the relationship was based on the simulation study, i.e.

\[
\Delta R_{pot} = 0.16 \Delta V_{POT} = 0.42 \Delta A_{POT}
\]

where \( \Delta R_{pot} \) = the change in roughness due to a change in potholing, m/km IRI; \( \Delta V_{POT} \) = the change in volume of potholes, m\(^3\)/lane/km; and \( \Delta A_{POT} \) = the change in area of potholing, % pavement area.

6.3.4 Comment

The prediction relationships presented in this section for the development of potholing distress constitute a rational attempt to quantify this ultimate form of distress which has such severe economic consequences. From necessity, they have had to be based on small data sets and to adopt surrogates for the unquantifiable factors which affect them. The relationships are not claimed to be
accurate, but they are considered to give ballpark estimates and a suitable ranking of the influences of various pavement, traffic and environmental parameters suitable for the purposes of economic evaluation.

The critical reader may be likely to challenge the predictive algorithms proposed for the development of potholing on the grounds that they are more complex than the data warrant. Indeed it is apparent from the discussion that considerable engineering judgement and supposition have been used in developing the effects of various factors on the occurrence and growth of potholing. Moreover, practical application of these algorithms has resulted in the adjustment of some of the coefficients, notably reduction of the first coefficients in Equations 6.9a and b to the current values, in order to make them realistic. The motivation for the algorithms has come of course from the need to adequately represent the deterioration and maintenance effects in such a way that the impact of different policies (particularly timing) can be compared and quantified. There is a clear case for strong calibration of these algorithms to represent the effects for local conditions in individual cases, but they are considered to be a reasonable starting point.

In particular the linear relationship of the roughness sensed by vehicles to the total volume of potholing can be regarded as fairly indicative of the effect of discrete potholes and one that is particularly useful for its direct relationship to the volume of patching maintenance. Further empirical study is needed however to study the rather large difference in the effect on roughness found between the simulation and empirical studies. The current explanation is that the range of 0.16 to 0.75 IRI per m³/lane km of potholing represents the difference between the larger impact of discrete sharply-edged potholes on roughness when compared to that of long shallow depressions. A reasonable average value to assume is 0.40 m/km IRI per m³/lane km of potholing. For the HDM-III model however the lower value of 0.16 m/km IRI per m³/lane km was adopted. Implicit in both cases is the allowance that the vehicles encounter in the wheelpaths only about one-eighth of the total potholing area through avoidance.

6.4 TEXTURE AND SKID RESISTANCE

Adequate friction between the tire and the road surface depends on both the macrotexture depth, which is the average protrusion height of surfacing aggregate that affects the drainage of water from the tire contact area and the risk of hydroplaning, and on the microtexture of the aggregate, which tends to polish under traffic at a wear rate that depends on the mineral type. Neither texture nor skid resistance were measured systematically in the major road costs studies so the review of this distress mode is therefore brief.

The loss of macrotexture can occur through either bleeding or stone embedment. Bleeding is caused either by excessive binder content or by inadequate viscosity for the temperature conditions, and tends to be a greater hazard for surface treatments than asphalt concrete. Even in welldesigned treatments, stone embedment will occur under traffic even if neither ravelling nor bleeding develops. No separate models are presented for these modes of distress (see Lytton and others (1982) for a prediction of bleeding). However, the ravelling model appears to be a reasonable substitute for any of the three related modes because it gives a satisfactory estimate of the expected life of a surface treatment for non-cracking distress, whichever form that might take. It results in reseal
cycles of 9.5 years for traffic volumes of 1,000 veh/lane/day and 6 years for volumes of 5,000 veh/lane/day.

Polishing of the aggregate microtexture occurs in both surface treatment and asphalt concretes. Using data and model forms from the few published sources available, the FHWA study (Rauhut and others 1984) developed a prediction model for the reduction of skid resistance as a function of all vehicle passes for two major classes of aggregate, i.e., typical limestones and nonpolishing aggregates, as follows:

**Typical limestones:**

$$g(N) = 0.582 N^{0.574}$$  \hspace{1cm} (6.11)

**Nonpolishing aggregates:**

$$g(N) = 0.354 N^{0.383}$$  \hspace{1cm} (6.12)

where \( g(N) \) = damage function of skid resistance defined from the "skid number" (SK) and a terminal condition (i.e., \( g(N) = 1 \)) of \( SK = 35 \), namely:

$$SK \text{ (initial)} = \frac{SK (initial) - SK(N)}{SK (initial) - 35}$$

SK = skid number, the measure of skid resistance at 64 km/h (40 miles/h); and

N = cumulative number of vehicle passes, in millions per lane.

Note that a Skid Number of 35 is approximately equivalent to a sideways force coefficient (sfc) of 0.5 - 0.6.

These functions are compared with the prediction of terminal ravelling (criterion of 50 percent area) based on Equations 6.1 and 6.3, in Figure 6.12. For surface treatments of good nonpolishing aggregate, it can be seen that ravelling is likely to be the maintenance trigger at traffic volumes of up to 5000 veh/lane/day, and that skid-resistance is likely to be the trigger at higher volumes. The skid-resistance constraint also applies to asphalt concrete. The comparable life for surfacings of limestone aggregates is only one-sixth that of other aggregates, so that safety through skid-resistance is critical for traffic volumes that exceed only 600-1,000 veh/lane/day.

### 6.5 GENERAL COMMENT

It is evident from this chapter that the disintegration mode of distress is largely influenced by material characteristics, which are generally impractical for use as parameters in network-level predictive models and which are often difficult to quantify. Typically the material design and construction specifications are intended to adapt the materials to suit the environment and traffic of the specific application, although this is not always achieved for various reasons and test data quantifying the construction compliance are rarely available.

Consequently, the empirical models must rely on only major measurable effects such as traffic, and combine other factors into observable classes such as surface type, construction quality and environment. Thus also, a high variability of individual observations is to be expected due to the variations of characteris-
The model predicting the initiation of raveling utilizes surface type classes, traffic volume and a construction quality class. The coarseness of the classes, three for surface type and two for quality, and the inherently stochastic variability of the failure phenomenon, combine to produce quite high levels of uncertainty in the predictions; typically, some 10 percent of observed lives may be less than 45 percent of the predicted mean life and some 10 percent may be more than 65 percent above the mean. In the raveling progression model, no traffic or class effects were significant, and it is apparent that the rate of progression is inherently a random event best described by a time model, as given by Equation 6.3. The effects of aging and weathering are strong in the raveling initiation model and implicit in the progression model. As these effects are highly dependent on the microclimate of the pavement surface, and on the viscosity, volatiles content and adhesion properties of the binderaggregate matrix, adjustment of the predicted mean life and rate of progression, by means of a multiplicative factor, may be required to calibrate the model to a local situation. Predictions by the model are generally sensible and realistic however so that the adjustments are likely to be minor in many cases.

The models for the initiation and progression of potholing are algorithms derived intuitively from three modest data bases. They use only factors which may be generally available for a network-level analysis, namely surfacing thickness, traffic volume, construction quality class, and lastly structural...
number (as a general surrogate parameter for material quality). While individual observations of potholing behavior may be expected to vary very widely about the predicted means, high accuracy is not required in these particular models and they are considered to give satisfactory general indications for the purposes of network-level analysis. For project-level applications the greatest improvement would come from quantifying or classifying the relevant material properties of the basecourse and the base-surfacing bond, were such information available. For either application, calibration to local conditions is advisable whenever reliable representative data exist.

The model for predicting loss of skid resistance uses just two parameters, traffic volume and class of the surfacing aggregate (limestone or non-polishing). Once again individual variability may be high, but the general trends lead to realistic predictions of the timing for maintenance, especially when combined with the ravelling models.

The approach to modelling the impact of potholes on the level of road roughness was distinctly different. Mathematical simulation was feasible and produced the extremely useful result that the impact on roughness is linearly related to the volume of potholing in the vehicle wheelpaths, and thus also to the volume of patching required for maintenance. After making allowances for avoided potholes, however, a significant discrepancy remains with respect to the available set of field data, amounting to a predicted impact of 3 to 6 times greater than observed. Whereas the field conditions in that instance probably represented interconnected, shallow dish-shaped potholes, the simulation prediction seems to give the more realistic impact of deep discrete potholes. Until further field work resolves the issue, the more conservative, i.e., lower, impact on roughness has been recommended.
Permanent or unrecovered deformations in pavements include rutting, shoving, heave, small ("birdbath") depressions, edge depressions, and large ("bathtub") depressions. These constitute distress because they directly increase road roughness, with concomitant effects on user benefits, dynamic loads, riding quality and safety, and because they can cause ponding of water and thus create safety hazards.

In this chapter we concentrate mainly on rut depth progression, since rutting is historically a primary criterion of structural performance in many pavement design methods. The other types of deformation are generally much less tractable for direct modelling because they depend to a large degree on material properties, their local variations, and their interactions with the pavement's microclimate; typically these deformations are controlled through construction and material specifications to be negligible, and their evolution is thus modelled indirectly through roughness progression, as detailed in Chapter 8.

Edge depressions, which typically indicate either inadequate pavement and shoulder width or inadequate drainage, and bathtub depressions, which typically occur through the subsidence of formation or on expansive clay subgrades, are two categories of deformation that sometimes have significance in planning and management issues. However, although they influence the benefits to be gained from raising the construction standards of formation width and height, they are not addressed in depth because the modelling of their evolution is difficult on account of the number of parameters involved. Allowance for their effects on roughness needs to be made explicitly when necessary.

### 7.1 MECHANISMS OF DEFORMATION

The causes of permanent deformation can be classified into traffic-associated and non-traffic-associated causes, as summarized in Table 7.1. Traffic loading causes deformation when the stresses induced in the pavement materials are sufficient to cause shear displacements within the materials. Thus single loads or a few excessive loads or tire pressures, causing stresses that exceed the shear strengths of the materials, can cause plastic flow, resulting in depressions under the load and often heave alongside the loaded area. Repeated loadings at lesser load and tire pressure levels cause smaller deformations which accumulate over time and become manifest as a rut if the loadings are channelized into wheel-paths. Finally, static or long-term loadings can cause creep, or time-dependent, deformation in viscous materials such as bituminous materials, and in soils.

#### 7.1.1 Densification and Plastic Flow

There are two mechanisms of traffic-associated deformation of importance to modelling efforts. Densification involves volume change in the material,
Table 7.1: Summary of causes and types of permanent deformation

<table>
<thead>
<tr>
<th>General cause</th>
<th>Specific causative factor</th>
<th>Example of distress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic-associated</td>
<td>Single or comparatively few excessive loads</td>
<td>Plastic flow (shear distortion)</td>
</tr>
<tr>
<td></td>
<td>Long-term (or static) load</td>
<td>Creep (time-dependent) deformation</td>
</tr>
<tr>
<td></td>
<td>Repetitive traffic loading (generally a large number of repetitions)</td>
<td>Rutting (resulting from accumulation of the small permanent deformation associated with passage of wheel loads)</td>
</tr>
<tr>
<td>Non-traffic-associated</td>
<td>Expansive subgrade soil</td>
<td>Swell or shrinkage</td>
</tr>
<tr>
<td></td>
<td>Compressible material underlying pavement structure</td>
<td>Consolidation settlement</td>
</tr>
<tr>
<td></td>
<td>Frost-susceptible material</td>
<td>Heave (particularly differential amounts)</td>
</tr>
</tbody>
</table>


resulting from tighter packing of the material particles and sometimes also the degradation of the particles into smaller sizes. Typically, densification in pavements is controlled through the compaction specifications at the time of construction, in which the density specifications to be achieved have been selected in accordance with the expected loadings and pavement type. The densities required are typically lowest in subgrade layers, where the stresses induced under traffic loadings are least, and highest in the upper layers of the base and surfacing where the induced stresses are highest. The more tightly packed particle structure at high densities has higher shear strength and thus deforms less than at low densities; it also is less permeable and thus less susceptible to moisture ingress.

Typical compaction specifications range from 85–95 percent of standard compaction (e.g., AASHTO T99, equivalent to 600 kJ/m³) for subgrade and selected layers, to 95–100 percent of modified compaction (e.g., AASHTO T180, equivalent to 2,700 kJ/m³) for basecourse layers, and 98–100 percent of Marshall density for bituminous surface layers. The influence of compactive effort can be great especially at lower levels; Barksdale (1972) showed that reducing compaction to 95 percent instead of 100 percent of modified compaction increased the plastic strain in granular basecourse materials by 185 percent, whereas an increase to 105 percent caused a reduction in plastic strain of only 10 percent.

Consequently, pavements which were constructed many years ago to comparatively low compaction standards, may manifest significant rutting when subjected to greater increased axle loadings or tire pressures, due to the further densification which occurs. By corollary also, however, old pavements which have been densified under many years of traffic are often found to have exceptionally strong
subgrades because of the high densities eventually achieved, and this has often led to enhanced structural capacity and performance. The typical deformation trend for densification is thus an initial rise until the density and strength balance the applied loadings, followed by essentially little change, unless either the imposed loading increases or the strength is decreased by saturation, for example.

Plastic flow is the second mechanism. Flow involves essentially no volume change and gives rise to the shear displacements in which both depression and heave are usually manifest. It occurs when the induced stresses exceed the shear strength of the material or are sufficient to induce creep. This is controlled in pavement design by the selection of materials according to a measure of shear strength (for example, the California Bearing Ratio, CBR, for soils, Marshall or Hveem stability for bituminous materials, and so forth). Thus most design methods require higher strength materials in the upper regime of the pavement, and indicate combinations of layer thickness and stiffness sufficient to attenuate the applied stresses to levels at which the accumulated shear deformations through the full pavement depth are within a specified rut depth criterion. Hence for modelling purposes, some technique must be found for combining these dual gradients of material (shear) strengths on the one hand, and induced stresses (which depend on loads, layer thicknesses and material stiffnesses) on the other hand, both of which generally tend to diminish with depth in the pavement.

It is evident, for example, that the major part of total deformation may be located in one particular layer of the pavement if the material there is too weak for the loadings being applied, while in other cases the same total deformation may be more evenly distributed through the depth of the pavement. Thus similar levels of rut depth can result from very different depth-profiles of deformation. Early design approaches focussed on deformation of the subgrade layers (comprising the weakest materials), and subsequent experimental approaches to modelling have often addressed the problem by evaluating individual layers. An intermediate approach, which summarizes the deformation potential for the full pavement depth in terms of one or two summary indices, seems to be needed for developing a model suitable for planning and management purposes. One possible approach is the stress factor proposed by Chou (1983) which weights the deviator stresses at various depths by the layer thickness.

7.1.2 Prediction Approaches

Two types of approach have been used in pavement design methods. In the first, excessive deformation beyond a specified "failure" limit is prevented through applying criteria derived from empirical correlation to pavement performance. These criteria are either empirical or mechanistic. The commonest empirical criterion is the CBR test used in the method of the U.S. Corps of Engineers (1957) (see also Yoder and Witczak 1975) to relate the required cover thickness to the material shear strength and the design wheel load coverages. Mechanistic criteria are commonly based on limiting the vertical compressive strain in the subgrade (estimated by elastic theory) to a level dependent upon the traffic loading (see, for example, reviews by Monismith (1976) and Barker and others (1977)). In general, however, these "limiting criteria" approaches are not useful to performance modelling because of the need to predict not a limit but the trend of accumulated deformation during the life of the pavement, identifying the response to the actions of traffic, environment and maintenance.
The second type of approach predicts the trend of deformation under repeated loading either mechanistically, based on laboratory material characterization and theoretical structural analysis of the stresses and strains induced in each layer under the traffic loading, or empirically, by correlation between field data of rut depth trends and explanatory parameters representing the pavement and loading.

Our approach has to be empirical, because we must use summary data of pavement and traffic characteristics. So next we review the principles emerging from the mechanistic approach in order to identify suitable model forms, parameters and surrogates of mechanistic parameters from which to develop an appropriate empirical model.

7.1.3 Mechanistic and Experimental Models

The mechanistic approach estimates the plastic or permanent strain in each layer (or sublayer) of the pavement and sums the deformations over the full depth to estimate the total deformation. The estimation of the plastic strain however is complex and many forms have been published (see for example the review by Monismith 1976). All mechanistic models of deformation trends are in fact partly empirical because they relate the plastic strains observed experimentally under controlled loading conditions to mechanistic parameters of the stress/strain regime and to material or pavement characteristics. Repeated loading tests commonly reveal the following general form:

\[ \varepsilon_p = a N^b \]  

(7.1)

where \( \varepsilon_p \) = permanent or plastic, unrecovered strain;  
\( N \) = number of stress applications;  
\( a, b \) = estimated coefficients, functions of applied stress, moisture content, dry density and material characteristics;

and where the permanent vertical strain at a point in the pavement can be estimated from the local stresses by a function of the form:

\[ \varepsilon_{pz} = R \left[ \sigma_z - 0.5 (\sigma_x + \sigma_y) \right] \]  

(7.2)

where \( \sigma_x, \sigma_y \) and \( \sigma_z \) = stresses in radial, tangential and vertical directions respectively; and  
\( R \) = ratio of effective strain and stress estimated as a function of material properties and number of stress applications.

The estimates of the coefficients \( a, b \) and \( R \) are functions of applied stresses and material properties that vary widely among researchers, depending on the materials involved (bituminous, granular and fine-grained soil materials all differ significantly) and the degree to which non-axial stresses and rolling-wheel effects are taken into account (Monismith 1976). Comprehensive models are given by Brown and Pappin (1982), Ullidtz and Larsen (1983), Kenis and others (1982). Under typical highway conditions the value of the exponent \( b \) is less than unity, so that the rate of deformation decreases over time with cumulative load applications, giving a concave trend of deformation.

In a sensitivity analysis of the Shell and Asphalt Institute models using laboratory data, Akhter and Witczak (1985) showed that deformation was most
sensitive to the asphalt thickness and stiffness, and that in three-layer systems, the subgrade stiffness, and the thickness and stiffness of the intermediate layer were also important. Others agree for thick asphalt pavements, often including also parameters of the applied stresses and loading time (van de Loo 1976, Meyer and others 1976, Kenis and Sharma 1976).

In an empirical model derived from AASHO Road Test data that combines most of these effects, Saraf (1982), amending an earlier model, showed the rate of rut depth progression per axle passage to be a function of pavement deflection, applied stress in the asphalt and number of load repetitions, as follows, for flexible pavements with less than 150 mm asphalt thickness:

\[
\log RR = -3.781 + 4.343 \log D - 0.167 \log NE_4 - 1.12 \log \sigma_c
\]  

where \( RR \) = change of rut depth, in 25 mm/million ESA; \( D \) = surface deflection under 80 kN axle load, in 0.025 mm; \( \sigma_c \) = vertical compressive stress in asphalt, in 6.9 kPa; and \( NE_4 \) = cumulative number of equivalent 80 kN single axle loads.

A densification component accounted for 78 percent of the permanent deformation in asphalt in a study example by Abdulshafi and Majidzadeh (1984).

For subgrade soils, and other fine-grained materials, Majidzadeh and others (1976) have shown that the coefficient \( a \) in Equation 7.1 is a function of the dynamic stress and dynamic modulus, and that the coefficient \( b \) is virtually constant for soils with dynamic modulus greater than 40 MPa, across a wide range of densities, moisture contents, soil types and applied stresses. Values of their parameter range from 0.82 to 0.95, which correspond to \( b \)-values of 0.05 to 0.18.

Although the "limiting criterion" approaches are not directly applicable to the prediction of rut depth progression, they can be transformed to provide useful information on the magnitude of deformation rates. The criteria typically take the form:

\[
\log \varepsilon_{vi} \leq c + d \log N_p
\]  

where \( \varepsilon_{vi} \) = maximum vertical compressive strain induced in layer \( i \) under load \( p \); \( N_p \) = cumulative number of applications of load \( p \) causing the rut depth to reach a specified limit; and \( c, d \) = coefficients depending upon the material \( i \) and load \( p \).

Examples from Chou (1976), Monismith (1976) and Barker and others (1977) indicate that the slope coefficient \( d \) above is of the order of 0.23 for many soils and bituminous materials, and slightly higher (about 0.35) for some coarse granular materials. If one approximates the deformation rate to be linear with load applications, these criteria yield relationships of the following type:

\[
RDN = c' \varepsilon_{vi}^{4.3}
\]  

where \( RDN \) = rate of rut depth progression, say, in mm/million ESA. The exponent of vertical strain (equal to \( 1/d \)) thus has a typical value of about 4.3 for many materials (or 3 for coarse materials). As the vertical strain is almost directly proportional to the applied load, these relationships indicate that the rate of
rutting per load application increases with approximately the fourth power of axle loading. The coefficient c' seems to range widely from 100 to 30,000 mm/million ESA when strain is in $10^{-3}$ mm/mm.

7.1.4 Deformation under Accelerated Loading

A number of pavement trafficking studies have gathered field data on rut depth trends, including the AASHO Road Test, Brampton Road Test, many road tests in the United Kingdom, and various circular test track studies. Of particular interest are results from accelerated trafficking tests on in-service highways with controlled loading using the Heavy Vehicle Simulator (Maree and others 1982), because these show deformation trends for pavements with relatively thin asphalt surfacings (i.e., less than 100 mm), which are more common in developing countries than thick asphalt pavements.

Trafficking tests on five pavements, ranging in strength from about 3 to 6.5 modified structural number, under dual wheel loads of 40, 70 and 100 kN, yielded data of the rate of deformation before and after surface cracking, and with and without water ingress. The results, normalized to equivalent 80 kN single axle loads in Figure 7.1, show very strong effects of cracking and water ingress on the rate of deformation. On road P157/1, for example, the rate increased from 0.35 mm/million ESA before cracking, to 1.25 mm/million ESA after cracking and surface water ingress. On pavement N3, where the rate of deformation before cracking occurred had begun to slow down from about 4 mm to 0.8 mm per million ESA, the rate after cracking and wetting returned to the higher previous values. These trends were found to be roughly linear after log-transformation,

Figure 7.1: Permanent deformation of five pavements under controlled heavy wheel loading showing effects of cracking and soaking

Source: Maree and others (1982).
which confirms the general form of Equation 7.1. The data for the rate of deformation indicate values of $b$ that range from 0.1 to 0.3 on pavements before soaking, to 0.5 to 1.20 on pavements after cracking and saturation.

Similar trends can be seen for asphalt layers and thick asphalt pavements in the results of other trafficking studies. For example, Lister (1981) reports the relationship

$$D = W^{1.0} (T + 5)^{2.5} N^{0.28} \tag{7.6}$$

in which the trend of deformation ($D$) is generally concave with respect to the number of load applications ($N$) (the $b$-value is 0.28), but the rate increases with wheel load ($W$, in kN) and asphalt temperature ($T$, in °C). Paterson (1972) and others have shown that the rate increases also with tire pressures. Huber and Helm (1987) reported in-service performance data, which generally corroborates the Akhter and Witczak (1985) study, showing that the rate correlates with asphalt mix characteristics such as air voids, binder content and stability; they propose threshold values of these and other factors that would limit the plastic flow to acceptable levels, for current loading and material design conditions in Saskatchewan, Canada.

7.1.5 Summary

Traffic-associated permanent deformation, and rutting in particular, thus results from a rather complex combination of densification and plastic flow mechanisms, in which material characteristics, induced stresses and strains (which are themselves a function of wheelloads, tire pressures and pavement stiffness), and environmental factors (moisture and temperature) each have significant influences. Typically, densification is controlled through compaction specifications at the time of construction and plastic flow is controlled through the structural and material design specifications of the pavement. The relative influences of densification and flow thus depend on the adequacy of those various specifications for the conditions applying, and may change for example if loads or tire pressures increase or if the pavement is weakened by cracking and saturation.

Typical deformation trends are summarized in Figure 7.2. Curve A represents a pavement with generally adequate structural and material design in which the deformation occurs primarily through densification in a concave trend and tends asymptotically towards a limit; the limit is likely to be a function of the compaction specifications relative to the level of applied loading, and the $b$-value of Equation 7.1 is likely to be much less than one.

When the structural or material design specifications are inadequate, the deformation will tend to follow curve B, in which plastic flow dominates and the $b$-value is high; this would be typical of either overloading conditions, or soft asphalt in thick asphalt pavements, for example.

When cracking and water ingress occur on a sound pavement of Type A, without prompt maintenance, the rate of deformation is likely to increase as illustrated by curve C, with a slope and curvature dependent upon the degree of weakening incurred in the pavement; in these cases, the $b$-value may increase significantly, and in the worst case may tend towards one or greater as in Type D. After maintenance the slope and $b$-value would decrease again. Finally, pavements
which suffer seasonal extremes of either temperature or moisture may exhibit a combination of these trends as shown by curve E; this type, for example, was the trend observed during the AASHO Road Test as a result of the freeze-thaw-summer seasonal cycles.

The model form for permanent deformation therefore should be similar to Equation 7.1, but probably in two phases of before and after cracking. The parameters which will be important in determining the coefficients a and b include pavement layer thicknesses and stiffnesses (particularly of the asphalt layer and subgrade), applied stresses, cracking, and water ingress. In addition we can expect that compaction standards in the initial construction would influence how much densification occurs, and that this pre-densification should be reflected in commensurately lower deformations in overlaid or rehabilitated pavements.

7.2 ESTIMATION OF EMPIRICAL MODEL

7.2.1. Data Characteristics and Analysis

As the diversity and detail of the parameters required to utilize the reported models were generally impractical for applications in a network-level model, an empirical estimation of rut depth progression was made from the Brazil-UNDP study data. These data are considered to provide a sound basis for evaluating modern pavement design standards, but it will become evident that this choice restricts the validity of the model in respect of certain extreme conditions.

Rut depth in the Brazil study was measured manually with a wooden frame gauge based on the design of the AASHO gauge with a base length of 1.2 m. Readings were taken at four locations at 80 m-intervals in each wheeltrack of a subsection. For analytical purposes these data were reduced to a mean and standard deviation by subsection, and the ratio of the external and internal wheelpath-mean values. The 95th percentile of mean rut depths is sometimes used as a criterion.
for maintenance intervention, and the variation of rut depth, being a measure of the longitudinal variation of surface profile, has a geometric relationship to roughness. The ratio of rut depths in the external and internal wheelpaths indicates the adequacy of pavement shoulder support in relation to drainage and moisture penetration, and gives rise to vehicle roll.

Rut depth values were generally low in the Brazil-UNDP study on account of the modern design standards of the existing pavements, and predominantly thin asphalt surfacing construction (thicknesses of generally less than 100 mm). About 95 percent of the rut depth values were less than 8 mm during the study period and, on the six pavements with higher rut depths, the maximum values of the mean and standard deviation ranged up to only 16 mm and 15 mm respectively, as seen in Table 7.2. Thus, while these data are rather typical of road networks designed to modern design codes, which will be the main range of application, the comparative lack of severely underdesigned pavements may somewhat restrict the validity of any models for cases of extreme behavior.

The variability between successive observations of rut depth was moderately strong in the data, relative to the generally small absolute values, on account of measurement error. For example, individual observations had an average standard deviation of 1.0 mm from the trend of rut depth over the study period (for subsection-mean values), which was equivalent to 50 to 100 percent of the average increment between observations. Also, after the end of the formal study a slight negative trend of 10 percent per year was evident in the data, due perhaps to a change in the measurement operation; this was compensated by adopting an average value for that latter stage. These variations are due largely to the manual method of measurement adopted, and would not be significant were the general levels of rut depth much higher. Automatic measuring techniques, are essential in future research of this type.

### 7.2.2 Compaction Index

As the compaction achieved in a pavement at the time of construction was expected to influence the densification occurring under traffic, it was necessary to define a reference compaction standard. The reference must be a profile of compaction effort decreasing with depth, in order to be compatible with specification practice. For example, Brazilian specifications used an intermediate Proctor compaction (1,300 kJ/m$^3$) standard for base and subbase layers, and standard Proctor (or AASHTO) compaction (600 kJ/m$^3$) for selected and subgrade layers, with the target density for each layer expressed as a percentage of the relevant standard.

A reference profile of nominal compaction ($C_{nom}$) was therefore defined by:

$$C_{nom, i} = 1.02 - 0.14 z_i$$

and the relative compaction achieved for each layer $i$ ($RC_i$) was defined by

$$RC_i = \min [1, C_i/C_{nom, i}]$$

where

$C_i =$ the compaction of layer $i$ defined by $C_i = DD_i/MDD_i$;

$DD_i =$ in situ dry density of layer $i$;
Table 7.2: Summary statistics of inference space of rut depth data in Brazil - UNDP road costs study

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Units</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Sample size</th>
</tr>
</thead>
<tbody>
<tr>
<td>All sections</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>RDM</td>
<td>mm</td>
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<td>0</td>
<td>15.8</td>
<td>2,546</td>
</tr>
<tr>
<td>RDS</td>
<td>mm</td>
<td>2.23</td>
<td>1.46</td>
<td>0</td>
<td>14.6</td>
<td>2,546</td>
</tr>
<tr>
<td>SNC</td>
<td>mm</td>
<td>4.37</td>
<td>1.17</td>
<td>2.06</td>
<td>7.74</td>
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</tr>
<tr>
<td>CRX</td>
<td>%</td>
<td>11.9</td>
<td>23.6</td>
<td>0</td>
<td>100</td>
<td>2,546</td>
</tr>
<tr>
<td>COMP</td>
<td>-</td>
<td>0.972</td>
<td>0.027</td>
<td>0.879</td>
<td>1.00</td>
<td>2,546</td>
</tr>
<tr>
<td>NE₄ x 10⁻⁶</td>
<td>ESA</td>
<td>1.43</td>
<td>3.33</td>
<td>0.0006</td>
<td>34.1</td>
<td>2,546</td>
</tr>
<tr>
<td>DEF</td>
<td>mm</td>
<td>0.67</td>
<td>0.30</td>
<td>0.19</td>
<td>2.0</td>
<td>2,546</td>
</tr>
<tr>
<td>MMP</td>
<td>m/mo</td>
<td>0.123</td>
<td>0.012</td>
<td>0.086</td>
<td>0.149</td>
<td>2,546</td>
</tr>
<tr>
<td>AGER</td>
<td>years</td>
<td>7.66</td>
<td>4.53</td>
<td>0.05</td>
<td>23.1</td>
<td>2,546</td>
</tr>
<tr>
<td>H₁</td>
<td>mm</td>
<td>52.3</td>
<td>40.5</td>
<td>13</td>
<td>233</td>
<td>2,546</td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RDM</td>
<td>mm</td>
<td>3.98</td>
<td>2.81</td>
<td>0</td>
<td>15.8</td>
<td>797</td>
</tr>
<tr>
<td>RDS</td>
<td>mm</td>
<td>2.38</td>
<td>1.67</td>
<td>0</td>
<td>13.1</td>
<td>797</td>
</tr>
<tr>
<td>SNC</td>
<td>-</td>
<td>4.50</td>
<td>1.23</td>
<td>2.06</td>
<td>8.74</td>
<td>797</td>
</tr>
<tr>
<td>H₁</td>
<td>mm</td>
<td>52.3</td>
<td>23.6</td>
<td>20</td>
<td>103</td>
<td>797</td>
</tr>
<tr>
<td>MCRX</td>
<td>%</td>
<td>21.6</td>
<td>29.2</td>
<td>0</td>
<td>100.</td>
<td>797</td>
</tr>
<tr>
<td>AGER</td>
<td>year</td>
<td>8.20</td>
<td>4.78</td>
<td>0.40</td>
<td>23.1</td>
<td>797</td>
</tr>
<tr>
<td>Surface treatment</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RDM</td>
<td>mm</td>
<td>3.50</td>
<td>1.82</td>
<td>0</td>
<td>14.6</td>
<td>1,215</td>
</tr>
<tr>
<td>RDS</td>
<td>mm</td>
<td>2.47</td>
<td>1.34</td>
<td>0</td>
<td>14.6</td>
<td>1,215</td>
</tr>
<tr>
<td>SNC</td>
<td>-</td>
<td>3.85</td>
<td>0.62</td>
<td>2.71</td>
<td>5.90</td>
<td>1,215</td>
</tr>
<tr>
<td>CRX</td>
<td>%</td>
<td>8.2</td>
<td>20.2</td>
<td>0</td>
<td>100</td>
<td>1,215</td>
</tr>
<tr>
<td>AGER</td>
<td>year</td>
<td>8.31</td>
<td>4.35</td>
<td>0.11</td>
<td>21.0</td>
<td>1,215</td>
</tr>
</tbody>
</table>

Notes: RDM = mean rut depth of both wheelpaths, in mm; RDS = standard deviation of rut depth of both wheelpaths, in mm; COMP = compaction index of pavement relative to a standard (Equation 7.7), fraction; AGER = age of the pavement since latest overlay or construction, in years; DEF = mean peak Benkelman beam deflection under 80 kN standard axle load of both wheelpaths, in mm; SNC = modified structural number of the pavement; NE₄ = cumulative number of equivalent 80 kN standard axles (ESA) (computed with relative damage power of four), in ESA; MMP = mean monthly precipitation, in m/month; CRX = area of indexed cracking, in percent; and H₁ = thickness of bituminous layers, in mm. - Dimensionless.

Source: Analysis of Brazil-UNDP study data.
MDD<sub>i</sub> = maximum dry density of material in layer i determined in the laboratory to the relevant compaction standard;

C<sub>nom,i</sub> = the nominal specification of compaction to be achieved in layer i with respect to the relevant standard, as a fraction;

R<sub>C,i</sub> = relative compaction, i.e., the ratio of the compaction measured in the field to the nominal compaction, as a fraction;

z<sub>i</sub> = depth at bottom of layer i, in meters, where z ≤ 1.

As illustrated in Figure 7.3, this nominal "specification" allows for 100 percent of intermediate compaction in a 150 mm base, down to 88 percent of standard compaction in a subgrade at 1 m depth. The relative compaction index for the full pavement (COMP) was then defined as the average relative compaction weighted by layer thickness, over a 1 m depth, as follows:

\[
\text{COMP} = \sum_{i=2}^{n} \frac{\text{RC}_i \left(\frac{H_i}{\sum_{i=2}^{n} H_i}\right)}{\sum_{i=2}^{n} H_i},
\]

where H<sub>i</sub> = thickness of layer, in mm. For this analysis, the computation was applied only to layers of untreated materials because these were the likely sources of densification. In principle it could be extended to include bituminous materials with a separate definition of nominal compaction.

Many modern construction specifications utilize the higher standard of the modified AASHTO compaction (AASHTO T180) (2,700 kJ/m<sup>3</sup>) as the reference for the upper pavement layers. Allowance was made for this in the definition of C<sub>nom,i</sub> by fixing rather higher levels than usually specified in Brazil. Thus, for example, the common specification for base materials, calling for densities meeting 98 percent of modified AASHTO compaction, is represented by 1.00 in the

**Figure 7.3:** Definition of nominal compaction reference as a profile of depth in the pavement

<table>
<thead>
<tr>
<th>Nominal Compaction</th>
<th>Base and Subbase: Fraction of Intermediate Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>0.9</td>
</tr>
<tr>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

Note: Standard compaction 600 kJ/m<sup>3</sup>; intermediate compaction 1,300 kJ/m<sup>3</sup> modified compaction 2,700 kJ/m<sup>3</sup>. For nominal compaction equivalent to a modified compaction standard, deduct 0.02 from the nominal compaction shown (for base and subbase only).

Source: Author.
Given the dimensionless ratio form of the compaction index, values greater than unity are possible when very high compaction efforts are applied. The range observed in the Brazil study was from 0.88 to 1.00 over the whole sample of low and high class pavements.

7.2.3 Analysis

Two model forms were analyzed, incremental and cumulative, for both the mean and standard deviation of rut depth as follows:

(a) Incremental

\[
\begin{align*}
\Delta RDM_{it} &= \beta_0 \Delta NE_{it} + u \\
\Delta RDS_{it} &= \beta_1 \Delta RDM_{it} + w
\end{align*}
\]

(b) Cumulative

\[
\begin{align*}
RDM_{it} &= \beta_2 \Delta \text{AGE}_{it} \Delta \text{NE}_{it}^\gamma + u \\
RDS_{it} &= \beta_3 \Delta RDM_{it} \Delta \text{NE}_{it}^\gamma + w
\end{align*}
\]

where

- \( RDM_{it} \) = mean rut depth for section i at time t;
- \( RDS_{it} \) = standard deviation of rut depth for section i at time t;
- \( \text{NE}_{it} \) = cumulative equivalent standard axle loading for section i at time t;
- \( \text{AGE}_{it} \) = pavement age since construction or most recent rehabilitation for section i at time t;
- \( \Delta RDM_{it}, \Delta RDS_{it}, \Delta \text{NE}_{it} \) = increments in RDM, RDS and NE respectively over period t to t+1;
- \( \alpha, \beta, \gamma, \delta \) = functions to be estimated;
- \( u, w \) = measurement errors in the mean and standard deviation of rut depth, respectively.

The incremental models proved to be weak when estimated from observation-intervals in the time-series data because of the magnitude of the measurement error with respect to the generally very small rut depth values. As the average increments over the study period were also small or sometimes negative because of the above mentioned systematic error in some of the data, a period-increment analysis was not satisfactory either. Thus estimations were made on the absolute values of the rut depth parameters using the cumulative model forms above, similar to Equation 7.1, and in this way the measurement errors had much less impact on the model estimation.

The form of the cumulative models is exponential with shapes usually of the types A and B in Figure 7.2 such that

\[ 0 < \alpha < 1; \ 0 < \gamma < 1; \ 0 < \delta < 1. \]
The selection of variables to be included in the functions \( \beta \) and \( \gamma \) were based on mechanistic considerations, as discussed below, and the exponents \( x \) and \( \delta \) of rut depth and age respectively were considered constants. The parameters considered in the structural functions included modified structural number, Benkelman beam deflection, rehabilitation age, rehabilitation state, compaction index, amount and severity of cracking, precipitation, deflection-path-ratio, rate of trafficking, variations of the structural number and deflection within a section, surfacing thickness, pavement type, and base material type.

The relative compaction index was expected to influence both the initial rate of rut depth progression and the magnitude of rut depth attained through densification. The modified structural number, surface deflection, or response characteristics such as vertical compressive strain below the subgrade interface or deviator stresses, should be the primary parameters determining the rut depth under given traffic loading, and thus one or more of these were mandatory in the \( \beta \) function. The deflection-path-ratio parameter, indicating transverse non-uniformity in the pavement, could influence either the \( \delta \) or \( \gamma \) functions, but was not found to be significant in the models. The effects of cracking and rate of precipitation, and their interaction, were expected to influence the rate of change of deformation rate, i.e., parameter \( \gamma \).

### 7.2.4 Prediction Models

The models, estimated first by linear regression on logarithmic transforms of the variables and finally by non-linear multiple regression on 2,546 time-series observations of the subsection-mean and subsection-standard-deviation of rut depth, were as follows:

**Mean rut depth:**

\[
RDM = 1.0 \text{AGER}^{0.166} \text{SNC}^{-0.502} \text{COMP}^{-2.30} \text{ERM}^{0.0166} \text{NE}^{-0.4} \text{ERS}^{(-7.98)}
\]

with 2,546 observations, standard error = 1.71 mm, \( r^2 = 0.42 \), t-statistics as given in parentheses; and

where \( \text{ERM} = 0.0902 + 0.0384 \text{DEF} - 0.009 \text{RH} + 0.00158 \text{MMP} \text{CRX} \)

\[
(12.5) (19.0) (-3.61) (8.85)
\]

**Standard deviation of rut depth:**

\[
RDS = 2.063 \text{RDM}^{0.532} \text{SNC}^{-0.422} \text{COMP}^{-1.664} \text{ERM}^{0.0166} \text{NE}^{-4} \text{ERS}^{(-5.67)}
\]

with 2,546 observations; standard error = 1.02 mm; \( r^2 = 0.51 \); and

where \( \text{ERS} = -0.009 \text{RH} + 0.00116 \text{MMP} \text{CRX} \)

\[
(3.43) (6.96)
\]

\( \text{RH} \) = the rehabilitation states where \( \text{RH} = 1 \) for overlaid pavements and \( \text{RH} = 0 \) for original pavements; and

other variables are as defined in Table 7.2. The scatter diagrams comparing observed and predicted values are shown in Figure 7.4 for the mean (a), and standard deviation (b), of rut depth, respectively.
**Figure 7.4:** Goodness of fit of predictive models for the mean and standard deviation of rut depth on data from the Brazil-UNDP study

(a) **Mean Rut Depth**

![Graph showing the goodness of fit for mean rut depth with a line of equality and observed versus predicted values.](image)

- Line of Equality
- $r^2 = 0.42$

Note: A = 1 obs., B = 2 obs., etc.

465 observations hidden

(b) **Standard Deviation of Rut Depth**

![Graph showing the goodness of fit for standard deviation of rut depth with a line of equality and observed versus predicted values.](image)

- Line of Equality
- $r^2 = 0.51$

Note: A = 1 obs., B = 2 obs., etc.

1,058 observations hidden

**Source:** (a) Equation 7.12 and (b) Equation 7.13 on data from Brazil-UNDP study.
These diagrams and the statistics illustrate that the models explain about one-half the variances of the data. This somewhat mediocre fit is due largely to the effects of measurement errors and the variations of behavior between like pavements. Some is also due to parameters missing from the models, and in particular a bias causing the underprediction of mean rut depth at high rut depth values can be noted in Figure 7.4(a). This is an imperfection of the model that is yet to be improved - study of the residuals indicated that little improvement was to be gained from any of the primary structural variables and only little improvement was to be gained from the cracking and precipitation interaction. Further work may focus on a model form that permits a stronger sensitivity of the rate of change to the levels and changes of cracking and moisture, because the experimental data suggest a sharper transition point between the uncracked and cracked states. This may be a function of the traffic and time since cracking, for example, whereas the current model averages (and thus probably dilutes) the cracking effect over the whole time since new. The bias is less in the standard deviation model but is present nevertheless, as seen in Figure 7.4(b).

7.2.5 Engineering Interpretation of Predictions

The predictions given by the models for the mean and standard deviation of rut depth are illustrated in Figure 7.5 for a traffic loading of 500,000 ESA/lane/year on two pavements of 2 and 6 modified structural number respectively. The prediction for mean rut depths shows a very strongly concave function in which the rate of rut depth progression diminishes over time as a function of the cumulative equivalent axle loadings and the various structural parameters. The initial rates are high, indicating a degree of early densification under traffic which causes small rut depths of the order of 3 to 5 mm within a period of about one year, but subsequently the rates drop to less than 1 mm per year.

The influence of densification in the model is moderately strong; for example, if the compaction at the time of construction was 10 percent below the nominal specifications (i.e., COMP = 0.9) this would cause a 27 percent increase in the mean rut depth. This effect permits an evaluation of some of the benefits of either enhancing or enforcing compaction specifications for construction.

The power term applied to axle loadings, ERM, which is the b-value of Equation 7.1, has a range of values for uncracked pavements from about 0.09 to 0.13, and for fully cracked pavements from about 0.11 in arid climates (250 mm annual precipitation) to 0.20 in wet climates (5000 mm annual precipitation). This range of values is very close to the values 0.05 to 0.18 found experimentally by Majidzadeh and others (1976) for fine-grained soils over a wide range of moisture and stress conditions. However the impact of cracking and water ingress in the model is much less than observed in the full-scale accelerated loading studies of Maree and others (1982), which indicated higher values of 0.4 to 1.20 on pavements with saturated base and subgrade. At this stage it is not certain to what degree those higher values can be attributed to the very high wheel loading used in their studies (mainly 70 and 100 kN, see the discussion in Section 9.3.2), or to deformation within the unbound granular base, or to the short rest period between loadings. Under normal service conditions, rainfall is intermittent and the loadings are not all concentrated during the period when the materials are critically or partially saturated, so the deformation effects tend to be attenuated. This was probably particularly true in the Brazil study area where the subtropical rainfall tended to be of short duration and high intensity. Thus while the marginal effect of the loading-moisture-cracking interaction may produce
Figure 7.5: Predictions of the progression of the mean and standard deviation of rut depth

(a) Mean Rut Depth

(b) Standard Deviation of Rut Depth

b-values (ERM in Equation 7.12) greater than 0.40, these were not observed in the in-service conditions of the Brazil study.

The structural effects in the mean rut depth model are represented by the modified structural number (SNC) and the deflection. Because the value of ERM is of the order of 0.11, the power of SNC relative to the cumulative equivalent axle loading is about 4.6; this value compares very closely with the values in the order of 5.0 obtained in the roughness prediction model (Chapter 8) but is lower than the values in the order of 7 to 9 found in many pavement design codes (see the discussion in Section 8.6.2). The rate of rut depth progression increases as the pavement strength decreases through two effects, namely the decrease in SNC and the increase of deflection in the power applied to the axle loading (ERM). The net effect can be seen in Figure 7.5(a).

According to the model, the rut depth ultimately tends towards a maximum value that is reached within about ten years, except in the case of strong water and cracking effects. The predictions, which mirror the empirical data, show in effect that when the pavement deflection is less than 1 mm or the modified structural number is greater than 3, the maximum rut depths are not likely to exceed 10 mm even under heavy traffic of up to 1 or 2 million ESA/lane/yr. Figure 7.6 shows this threshold effect for a range of traffic loadings from 50,000 to 2 million ESA/lane/yr.

Figure 7.6: Relation of the ultimate mean rut depth to pavement strength and traffic loading as predicted by the model: an apparent threshold effect

Note: Computed with Equation 7.12 for pavement age of 10 years and no cracking.

Source: Author.
Table 7.3: A tentative guide to minimum pavement strengths required to limit ultimate rut depth to 10 mm maximum as a function of maximum single axle loading

<table>
<thead>
<tr>
<th>Pavement strength required</th>
<th>Maximum single axle loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Benkelman beam deflection</td>
<td>Modified structural number</td>
</tr>
<tr>
<td>&lt; 1.2 mm and ≥ 1.5</td>
<td>≤ 120 kN &lt; 10</td>
</tr>
<tr>
<td>&lt; 0.9 mm and ≥ 2.0</td>
<td>≤ 130 kN &lt; 20</td>
</tr>
<tr>
<td>&lt; 0.7 mm and ≥ 3.0</td>
<td>≤ 145 kN &lt; 30</td>
</tr>
</tbody>
</table>

Note: Conditional on the assumption that individual materials are not subject to significant plastic flow by reason of saturation, inadequate shear strength or inadequate binder stiffness.

Source: Derived by author from paved road performance data, Brazil-UNDP study.

This "strength threshold" is likely to depend upon the maximum axle loadings present in the traffic, since these and the tire pressures determine the stresses induced in the pavement. A preliminary study of the spectra of loadings in the study yielded a tentative guide in terms of the 99th percentile axle load, as given in Table 7.3. Note that these applied to pavements maintained in a generally uncracked, well-drained condition, and with traffic loadings of up to 1.5 million ESA/lane/yr.

In sections where estimates of the vertical compressive strain in the subgrade were available, the thresholds above corresponded to strains under an 80 kN single axle load in the range of 600 to 1,000 microstrain.

The variation of rut depth, defined here by the standard deviation, was found in the study to depend primarily on the mean rut depth as shown in Figure 7.5(b). Other factors entering the model, such as cumulative traffic, have comparatively minor effects. It had been expected that the standard deviation of rut depth would be well correlated with parameters describing the variability of pavement strength within the section, such as the variation of deflection or structural number, but no statistically significant relation could be identified.

7.3 VALIDATION

To date, validation has been undertaken on data from the Arizona study (see Chapter 4 and Appendix A), consultant studies in England and Canada, and comparison with three published empirical relationships. In each case only the prediction of mean rut depth could be evaluated.

On the Arizona data, the increment of mean rut depth over the monitoring periods of 4 to 10 years was predicted moderately well, as shown in Figure 7.7(a) and by the statistics in Table 7.4. The prediction error was 1.1 mm on a mean
Figure 7.7: Validation of rut depth prediction model: comparison of observed and predicted values from Arizona data for an average 8.2 year period

(a) Incremental Rut Depth

(b) Trends of Mean Rut Depth

Source: Equation 7.12 on data from Way and Eisenberg (1980).
Table 7.4: Validation of rut depth prediction model: evaluation against data from Arizona and AASHO Road Test

<table>
<thead>
<tr>
<th>Parameter and statistic</th>
<th>Mean</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arizona network (51 observations: 10 years)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Absolute rut depth (last)(mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>5.16</td>
<td>1.69</td>
<td>12.98</td>
</tr>
<tr>
<td>Predicted</td>
<td>4.98</td>
<td>2.21</td>
<td>14.03</td>
</tr>
<tr>
<td>Residual</td>
<td>0.18</td>
<td>-11.53</td>
<td>9.61</td>
</tr>
<tr>
<td>Prediction error</td>
<td>4.02</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Bias correction</td>
<td>1.03</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Correlation coefficient, r</td>
<td>0.01</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Incremental rut depth (mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>1.72</td>
<td>0</td>
<td>4.23</td>
</tr>
<tr>
<td>Predicted</td>
<td>1.38</td>
<td>0.21</td>
<td>9.81</td>
</tr>
<tr>
<td>Residual</td>
<td>0.62</td>
<td>-1.93</td>
<td>3.08</td>
</tr>
<tr>
<td>Prediction error</td>
<td>1.14</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Bias correction</td>
<td>1.25</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Correlation coefficient, r</td>
<td>0.41</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Rate of rutting (mm/year)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>0.207</td>
<td>0</td>
<td>0.508</td>
</tr>
<tr>
<td>Predicted</td>
<td>0.124</td>
<td>0.05</td>
<td>0.31</td>
</tr>
<tr>
<td>AASHO Road Test (1883 observations, 2 years)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Absolute rut depth (period 1)(mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>4.23</td>
<td>-1.74</td>
<td>28.3</td>
</tr>
<tr>
<td>Predicted</td>
<td>1.89</td>
<td>0</td>
<td>5.1</td>
</tr>
<tr>
<td>Bias correction</td>
<td>2.23</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Correlation coefficient, r</td>
<td>0.66</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Absolute rut depth (period 2)(mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>9.71</td>
<td>0</td>
<td>38.9</td>
</tr>
<tr>
<td>Predicted</td>
<td>2.74</td>
<td>0</td>
<td>6.34</td>
</tr>
<tr>
<td>Bias correction</td>
<td>3.54</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Correlation coefficient, r</td>
<td>0.59</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Incremental rut depth (mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>5.48</td>
<td>0</td>
<td>37.2</td>
</tr>
<tr>
<td>Predicted</td>
<td>0.85</td>
<td>0</td>
<td>3.8</td>
</tr>
<tr>
<td>Residual</td>
<td>4.63</td>
<td>-2.5</td>
<td>36.3</td>
</tr>
<tr>
<td>Correlation coefficient, r</td>
<td>0.49</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Notes: Bias correction = observed mean/predicted mean. -- Not applicable.
1/ Observed data from Way and Eisenberg (1980).
2/ Observed data from AASHO Road Test Computer files, by courtesy of Asphalt Institute, College Park, Maryland, U.S.A.
Source: Author, with R.J. Tomlinson.
observed increment of 1.7 mm in an average period of 8.2 years and the correlation coefficient was 0.41. On predicting the absolute rut depth levels, the means of the predictions and observations compared very closely (5.2 mm and 5.0 mm, respectively), but the variance was considerable, giving an overall prediction error of approximately 4 mm which is in the same order as the mean observed value. Inspection of the data shown in Figure 7.7(b) and statistics revealed that the predictions were poorest for the pavements having medium or thick asphalt surfacings. With few exceptions, the model underpredicted the rut depth for that group, which was mostly beyond the empirical thickness range of the model, so that it seems likely that plastic flow within the thick asphalt layers under the high summer temperatures of the region was poorly represented by the model.

Saraf's model (1982), derived from the AASHO Road Test data, has very different characteristics with a nearly linear dependence on cumulative axle loading (b-value 0.83); it predicts 3.5 mm rut depth after 20 years for the SNC = 6 case in Figure 7.5, which compares very well, but 1,400 mm for the SNC = 2 case! In contrast, his earlier model predicts well for the SNC = 2 case and very poorly for the SNC = 6 case. Saraf's models thus seem to be inappropriate to mixed traffic applications.

In a direct evaluation against AASHO Road Test data in this study, results from which are reported in Table 7.4, high correlations of 0.59 to 0.69 were obtained in predicting the absolute rut depth, and also for the incremental rut depth \( r = 0.49 \). A strong bias toward underestimation is evident, however, with a bias correction factor in the order of 3. Preliminary examination of the results showed that the bias was probably related to the spring-thaw effects on pavement strength which were not accommodated in the predictions (for example, that bias would be accounted for by a 40 percent reduction in strength for half the year). The high correlations are strong evidence that the primary structural and traffic effects which covered an extremely wide range at the road test, have been well represented in the prediction model.

A detailed study of six sites in Saskatchewan, Canada (N.D. Lea and Associates 1987), covering a range of traffic from 0.08 to 1.8 million ESA (cumulative) and ages from 2 to 14 years, compared the observed rut depths with predictions from the model. The results tabulated in Table 7.5 show that the predictions compared very closely to the observations, with an average bias of only 9 percent and a prediction error of 2.3 mm for a range of observed rut depths from 2 to 11.5 mm. On five other sites, at which the asphalt characteristics did not meet the threshold criteria set out by Huber and Heiman (1987), the observed rut depths were generally higher (6 to 15 mm) for similar traffic levels; from this it can be deduced that plastic flow due to asphalt softness was adding about 100 percent to the predicted effects in those cases.

In England, a study (Wyley and others 1986) to evaluate and calibrate the relationship for British conditions found that the predictions of rut depth followed the correct trend (a high correlation coefficient of 0.98) but needed to be increased three- or four-fold, as seen in Table 7.5. Once again it appears that the discrepancy may be due to plastic flow in the bituminous layers, but further verification remains to be done. By way of comparison, the TRRL Whole Life Cost Model (WLCM) using the deformation model of Nunn (1986) and performance data from experimental road sections, also shows higher rates of rutting, but the rates are independent of both asphalt thickness and deflection. This seems to confirm that plastic flow characteristics may be dominating the English perfor-
Table 7.5: Validation studies of rut depth prediction model: on data from Saskatchewan and England

<table>
<thead>
<tr>
<th>Parameter and statistic</th>
<th>Mean</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saskatchewan, Canada (6 observations, one-time)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean rut depth (mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td>5.25</td>
<td>2.0</td>
<td>11.5</td>
</tr>
<tr>
<td>Predicted</td>
<td>4.80</td>
<td>3.0</td>
<td>6.3</td>
</tr>
<tr>
<td>Residual</td>
<td>0.45</td>
<td>0.4</td>
<td>5.2</td>
</tr>
<tr>
<td>Prediction error</td>
<td>2.31</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Bias correction</td>
<td>1.09</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Correlation, r</td>
<td>0.80</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

| England (5 observations, 4-10 years) | | | |
| Rut depth (last)(mm) | | | |
| Observed | 12.4 | 0 | 25.0 |
| Predicted | 2.9 | 0 | 7.0 |
| Prediction error | 9.5 | 0 | 18.0 |
| Bias correction | 4.3 | - | - |
| Correlation | 0.93 | - | - |

Sources: Based on consultants' preliminary studies for Saskatchewan by N.D. Lea and Associates (1987); for England by Wyley and others (1986).

It is apparent from these validation studies that the models apply best to thin-surfaced pavements (asphalt thicknesses less than about 100 mm), that the trend and average rate of progression in relation to mixed traffic loading is generally well-validated, but that the variance of the absolute rut depth about the predicted mean may be high. For thick asphalt pavements and in wet-freeze climates, the model tends to underpredict the mean rut depth, requiring bias corrections of up to three or four-fold. The trend of deformation in relation to traffic loading appears to become more nearly linear under conditions in which the applied stress is high in relation to the material stiffness (e.g., of the asphalt surfacing in thick asphalt pavements, and of partially saturated materials when cracking and water ingress occur in a thin-surfaced pavement).

These findings indicate that two types of correction may need to be made to the prediction model when applying it to certain situations where the material and climatic characteristics differ strongly from the Brazilian empirical base, as follows:

1. For wet-nonfreezing, or wet-freezing climates, seasonal effects must be taken into account by weighting the pavement strength (SNC) accordingly (see Section 8.6.4, for example), or alternatively by applying a general correction factor to the predictions in the order of 2 to 4; and
2. For situations where plastic flow is occurring within thick asphalt layers (soft asphalt at high road temperatures), the power term $E_R M$ in Equation 7.12 needs to be increased by a factor of about 1.5 to 2.5 (by preference, a term including asphalt properties, thickness and temperature might be added, but this requires further research analysis).

7.4 SUMMARY COMMENT

Traffic-associated permanent deformation, manifest mainly by wheelpath rutting, is caused by densification and plastic flow. The amount due to densification depends strongly on the compaction standards applied at construction; laboratory research has shown that a 5 percent reduction in the density achieved can increase plastic strains by over 100 percent in individual layers, and the empirical model developed from field data in this study indicates that the same reduction increases rut depths by 15 percent or more.

Following a general form emerging from experimental and theoretical research, the empirical model shows a generally diminishing rate of rut depth progression with increasing cumulative traffic. When stresses induced in the pavement are within the elastic range of the materials so that plastic flow is negligible, then the rut depth tends towards an ultimate value determined largely by the amount of densification and the pavement strength. When plastic flow occurs, for example through weakening due to cracking, softening of layers by saturation, softness of asphalt under high temperatures, or escalation of axle loading and tire pressures, then the trend of deformation becomes more nearly linear with traffic and the rate increases.

Validation of the empirical model shows that the general form is sound with generally high correlations to independent field data, but that correction factors in the order of two to four need to be applied to the predictions when significant amounts of plastic flow are evident, such as in the situations above. This is likely to be necessary wherever cracking occurs in pavements of moisture-sensitive materials in wet climates or seasons, or where moderately thick asphalt layers are used in warm and hot climates. There is thus scope for enhancing the model to accommodate the plastic flow phase more fully.
CHAPTER 8

Predicting Roughness Progression for Paved Roads

This chapter is a major focus of the prediction of paved road deterioration because it draws together the impacts of all other distress modes (cracking, disintegration and deformation) and maintenance on road roughness, which is the dominant criterion of pavement performance in relation to both economics and quality of service. The key issues lie in quantifying the relative impacts of traffic loading, pavement strength and type, aging, environment, and secondary distress (or alternatively, the maintenance standards). Previous modelling efforts have dealt with only subsets of these, for example, loading and strength, or age alone, or distress alone, resulting in incomplete models because of limitations variously in the data base, the model form, or the correlation method.

The incremental model developed here from field data incorporates all these factors in what is found to be a powerful model. It shows for example the differing roles of the strength-loading interaction and aging, and the response to maintenance. As this primary model requires information on secondary distress, which is no disadvantage for either pavement management or simulation models like HDM-III, it is too complex for certain general applications such as road pricing and taxation studies. Thus another simpler model is also developed which aggregates all the influences in terms of only four major parameters, namely strength, loading, age and environmental class. Extensive validation is made on independent data sets with climates ranging from arid to wet-freezing, and results are also compared against major pavement design methods.

8.1 PREDICTION REQUIREMENTS

Predicting the trend of roughness, or roughness progression, over the life-cycle of a paved road pavement, is undoubtedly the most critical of the various pavement performance predictions. Since the vehicle operation components of user costs for a road on given alignment depend to a large degree on roughness, the optimum timing of maintenance intervention and the economic benefits associated with it depend very largely on the prediction of roughness progression. We saw in Chapter 2, for example, that vehicle operating costs typically increase by between 2 and 4 percent for each one m/km IRI in roughness over the range of good to poor conditions (2 to 10 m/km IRI) of paved roads. The roughness prediction also dominates in those pavement management systems and pavement design methods that utilize a generalized index, such as the AASHTO Present Serviceability Index (PSI), as the performance criterion because typically the serviceability correlates very highly with, and in many cases is computed directly from, roughness. Riding quality, as determined subjectively by panel ratings, also correlates highly with roughness (See Chapter 2, and Janoff and others 1985).

In order to appreciate the accuracy of prediction required, one should realize that over, say, a nominal 15-year interval between major maintenance
activities undertaken at optimum timing, roughness will increase by only about 0.1 m/km IRI (or 3 to 5 percent) per year on high-volume roads (say over 1000 veh/day), and by about 0.2 m/km IRI (or 5 to 10 percent) per year on low-volume roads. Given the difficulties of monitoring roughness that were discussed in Chapter 2, where the levels of precision were seen to be somewhat inferior to these typical annual increments, it is clear that establishing an adequate empirical data base and developing reliable prediction models is no straightforward task. It is also clear that if the prediction errors are of a similar magnitude, which we shall see to be the case, then the impact on the accuracy of long-range predictions of maintenance timing can be severe, with a typical error being ± 5 years. Fortunately, in economic life-cycle cost analysis, the influence of such effects is greatly reduced by the discounting of future costs, and in pavement management, the accuracy of predictions is continually enhanced because knowledge of the current condition is updated by condition monitoring surveys.

For road pricing and regulation studies, it is not only the magnitude of roughness changes which is important but also the attribution of that damage to various classes of users and to non-user sources such as environmental factors. Once again because roughness is the predominant mode of distress in determining the major component of road maintenance costs, to be recovered from road users in both the economic total cost and the user-serviceability approaches, the attribution of roughness changes to traffic, loadings, climate and other factors is crucial in cost allocations. This requirement is a particularly urgent need in current policy issues.

Finally, we are aware that road roughness itself is a multi-faceted pavement characteristic, with properties differing according to viewpoint. The definition adopted in this study (Chapter 2) combines the short, medium and long wavelength bands of roughness in the way they are "seen" by a moving vehicle. It is clear however that different combinations of amplitudes in these wavebands can give the same roughness number, so that some pavements with predominantly short wavelength roughness may have identical "roughness" (IRI) to other pavements with say predominantly medium or long-wavelength roughness. From the engineering perspective however, the roughness developing in each of these wavebands is likely to derive from widely different mechanisms. The choice for prediction modelling therefore lies between the prediction of a single roughness index on the one hand, and the prediction of a set of indices (for example the various waveband-indices utilized by the French and Belgians) on the other hand. In the one case, there is the knowledge that more than one mechanism is in effect and that the relative impacts of those mechanisms may not be distinguishable. In the other case, the size and scope of the data base required for estimating the trends of multiple indices are considerable, and the indices are usually combined into one summary statistic eventually anyway, for convenience in both application and reporting. At present the choice is completely constrained to the single index, because it is only for that that we have comprehensive data on both road deterioration and user costs available for analysis.

8.2 PREVIOUS STUDY AND MODEL FORMS

8.2.1 Structural Effects

Strong effects of pavement strength and traffic loading, or in other words structural factors, on roughness progression were first quantified
comprehensively under controlled experimental conditions at the AASHO Road Test in Illinois (AASHO 1962). Deterioration rates in the test were accelerated by a factor of about ten, with most failure levels of distress being reached within the two-year trafficking phase of the test, and a maximum number of 1.1 million applications of one level of axle loading in each test loop. Thus the test data provided very little information on long-term environmental effects and no direct information on the behavior under mixed traffic. Nevertheless, by comparing the performance of a test pavement across lanes, each lane being associated with one axle loading, the now-well-known law relating the number of applications causing unit damage to the approximately fourth power of the axle load was derived, and has been used almost universally since then for reducing mixed traffic loadings to equivalent standard axle load (80 kN) applications (ESA). Most importantly the Test also defined the structural number (SN) as an index of pavement strength, representing a composite layered structure as an equivalent pavement thickness of uniform properties. These two aggregating indices, ESA and SN, made it possible to develop the overall performance equation relating the trend of serviceability to traffic loading and pavement strength.

The AASHO performance equation expresses progression in the dimensionless damage parameter, $g_t$, as the fractional loss of serviceability with respect to a selected criterion of terminal serviceability, as follows:

$$g_t = \frac{P_o - P_t}{P_o - P_r} = \left(\frac{N_t}{\rho}\right)^\beta$$  \hspace{1cm} (8.1)

where

- $P_o$ = serviceability index at time $t = 0$;
- $P_t$ = serviceability index at time $t$, where $P_t = f$ (slope variance, mean rut depth, cracking and patching) as defined in Equation 4.1, and slope variance is a measure of roughness;
- $P_r$ = the terminal serviceability criterion, at which rehabilitation or reconstruction is indicated;
- $N_t$ = cumulative number of equivalent 80 kN standard axle loads to time $t$;
- $\rho$, $\beta$ = functions of axle type, axle load and pavement strength parameters, including the structural number and later (1972) a soil support parameter; and
- $g_t$ = dimensionless damage parameter defining the functional loss of serviceability incurred prior to time $t$ (Note that when $P_t = P_r$, $g_t = 1$).

As so many pavement design methods have been based on the AASHTO relationship (detailed in Appendix A and AASHTO 1982), it is an important reference for roughness progression, and typical prediction curves are presented in Figure 8.1, using the conversion between PSI and IRI from Chapter 2.

The explicit relation of roughness to pavement strength and traffic loading was reasserted by the findings of the TRRL study on in-service roads in Kenya (Hodges, Rolt and Jones 1975, and updated in Parsley and Robinson 1980). The important differences from the AASHO Road Test included the direct use of roughness instead of serviceability, observations under mixed traffic loading (reduced to ESA), a variety of pavement types on various subgrade strengths (not a single subgrade or set of materials as at AASHO), and a variety of pavement ages.
giving windows on different stages of the performance trend. The relationship took the following simple linear form:

\[ R_t = R_0 + s(S) N_t \]  

(8.2)

where \( s(S) \) = function of modified structural number of pavement strength; \( R_0 \), \( R_t \) = roughness at times \( t = 0 \) and \( t \) respectively; and \( N_t \) is as defined for Equation 8.1. Initially the pavement strength function \( s(S) \) was expressed as a constant for each of two ranges of modified structural number, in part because the overall range was not great (2.5 to 5.1, Table 4.2) and because of the scatter in the data. In the cubic parametric function incorporated in the updated model (RTIM2, see Appendix A), the effect of SNC at values lower than 3 became exaggerated, implying excessive benefits for the strengthening of very weak pavements under light traffic. Another concern was that 80 percent of the pavements in the Kenya study were of cement-treated base construction which, under moderate and high traffic loadings, cracked within the first year and suggests that the rate of roughness progression may have been dominated by the block-cracking and disintegration process.

Figure 8.1: Roughness predictions given by the AASHTO Interim Pavement Design Guide (1981) and the TRRL RTIM2 Road Transport Investment Model (1975 Kenyan Study)

Source: Equation A.4, AASHTO model (AASHTO 1981); Equations A.9, A.10, RTIM2 model (Parsley and Robinson 1982).
However, Figure 8.1 shows that the RTIM2 model predicts considerably slower rates of roughness progression than the AASHTO model, with an effect equivalent to a 60% rise in structural number. Modification of the structural number for subgrade strength has been included already, so the difference is believed to be due to the environmental difference between the harsh cold (AASHO) and warm arid (Kenya) climates. We return to this point when considering validation. Another point to notice is that each model assumes a constant value for initial roughness, but the two values differ considerably, being about 1 m/km IRI for the AASHTO model and 3.5 m/km IRI for the RTIM2 model. Conceptually through the model form, the AASHTO model can handle different initial roughness values and suggests that the rate of progression would be influenced slightly. The RTIM2 model however indicates that the rate is independent of initial roughness.

8.2.2 Time-related Effects

Other studies conducted on in-service pavements however have been quite unable to identify any structural effects of pavement strength or traffic loading, and have related the roughness progression directly to time and pavement age. Way and Eisenberg (1980), in an analysis of a ten-year series of roughness data on 51 pavements in Arizona, USA developed the incremental recursive expression:

\[ \Delta R_t = a R_t \Delta t - b \]  \hspace{1cm} (8.3)

where \( \Delta t \) is an increment in time, and \( a, b \) = constant parameters which were related to environmental parameters of rainfall, elevation, freeze-thaw cycles, temperature, etc. (see Appendix A for values). Potter (1982), analysing data from Victoria and Queensland in Australia with many different model forms, was unable to improve upon the following function of age (with \( r^2 \) in the range of 0.13 to 0.46):

\[ R_t = R_o + a t^b \]  \hspace{1cm} (8.4)

where \( t \) = pavement age, in years; and \( a, b \) = coefficients determined for each data set (\( b \) ranged from 0.9 to 3.6).

Other studies reporting time-related roughness progression include, for example, 7 or more percent/year in Canada (Cheetham and Christison 1981), 7 percent/year in Spain and 20 to 30 percent/year in Belgium (Lucas and Viano 1979), which are much higher than the average of about 2 percent/year in Australia and the 2 to 8 percent/year range in Arizona. The Arizona study related its range of progression rates to the environmental effects expressed through the coefficients in Equation 8.3, and this may also explain the wider differences in the other studies.

The Arizonan and Australian studies, and others, highlight an important difficulty of the empirical approach to this problem. A high degree of collinearity typically exists between pavement strength and traffic loading on in-service pavements, brought about through the engineering design process and consistent application of a design code with a fairly uniform design period. As a result, the causal strength-loading damage relationship may be obscured by a strong damage-time correlation. While statistical methods can cope with this, both studies were hampered by a lack in either the strength or loading data. Strong cross-sectional ranges are required in the sample of pavements to be analysed, including particularly high-strength low-load, low-strength high-load combinations.
Thus while simple correlative relationships, such as those in Equations 8.3 and 8.4, may be adequate for pavement management predictions when confined to local applications, they are completely inadequate for generalized technical or economic evaluation of the interactions of structural and environmental factors on roughness, particularly over a broad range of conditions.

8.2.3 Interactive Effects and Trend Shape

Some indication of interaction between time and structural-related effects was found by Queiroz (1981) in the first analyses conducted on the Brazil Road Costs Study, as detailed in Appendix A.3.1. These are expressed in absolute terms (i.e. levels rather than slopes) of roughness, cumulative traffic and pavement age as follows:

\[ R_t = f(R_0, N_t, t, S) \]  \hspace{1cm} (8.5)

where \( S \) is one of various pavement strength parameters used in alternative models, and the other parameters are as defined earlier.

A different approach in which a normalized rate of deterioration has been related to the level of surface distress is being developed in a current TRRL study (Jordan, Ferne and Cooper 1987). Data collected on 400 road and motorway bituminous pavements in Britain over a period of six years have been normalized to indicate the proportional changes of profile unevenness over 2-year periods. These proportional changes in unit time periods were found to be strongly related to the level of surfacing distress, as shown in Figure 8.2 where distress is depicted in four categories from 1 for a sound pavement to 4 for a pavement with a moderate amount of wide cracking and spalling. The changes range from the order of -16 to 34 percent per year in sound pavements, to 84 to 140 percent per year in severely-distressed pavements. The direct equivalences in roughness terms (since the profile unevenness relates to the squared deviations) are averages of 4 (-8 to 16) percent per year for sound pavements, and 46 (36 to 55) percent per year for severely cracked pavements. These rates of change are very high, but they represent strongly localized rates of deterioration within lengths of 1 to 5 m from laser-sensed data at very small intervals. The averaging effect of the tire contact area and the averaging over long section lengths, which are both implicit in the definition of roughness, would reduce these amounts so that the real changes in roughness, as defined here, would be less, perhaps in the order of 2.5 to 30 percent per year. These rates result in roughness levels that are in the order of four times the initial level by the time the severest distress category is reached. The British observations can be generalized by the following expression:

\[ \frac{\Delta R_t}{R_t} = \max(a \mathrm{CX}^b; c) \Delta t \]  \hspace{1cm} (8.6)

where \( \mathrm{CX} \) is the amount and level of cracking distress, and \( a, b \) and \( c \) are constants. A noticeable feature is that the independent variables include only time and distress and exclude traffic loading. In an earlier approach to the independent variables, the researchers had utilized a dimensionless fraction of expired pavement life, where life was expressed in ESA and was evaluated from surface deflection measurements, but for the final model they preferred the more easily-defined surface distress categories.
Figure 8.2: Proportional change in uneveness (PU,) in a constant period as related to surface condition in British study.

2 year changes from 1981 to 83, 82 to 84 and 83 to 85

<table>
<thead>
<tr>
<th>Distress category</th>
<th>Bituminous roadbase</th>
<th>Lean concrete roadbase</th>
<th>Average rut depth over 20m (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>-0.4</td>
<td>-0.4</td>
<td>-0.4</td>
</tr>
<tr>
<td>3</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>4</td>
<td>2.4</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td>&gt;20</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Distress category: 1. sound; 2. fine cracking; 3. wide cracking; 4. wide cracking and spalling.
Unevenness coefficient PU3 is defined in Table 2.1.

In sharp contrast to that shape of roughness progression function is the sigmoidal-shape proposed by Lytton and others (1982) which modifies the AASHTO serviceability function for Texas and FHWA studies as follows:

$$g_t' = \exp \left[-\frac{\rho}{N_t}N\right]$$  \hspace{1cm} (8.7)

where $g_t'$ = modified damage parameter, and all other variables are defined as for Equation 8.1. This function implies that the serviceability tends to an asymptotic boundary defined by $g_t = 1$, with the shape of the S-curve being determined by $\beta$ and the scaling, in terms of equivalent standard axle loadings, by the parameter $\rho$, as shown in Figure 4.4. Such a plateau-effect for roughness would be indicative of permanent deformation trends, like those demonstrated for rut depth progression in the previous chapter. The trend would therefore only apply under the conditions where the induced shear stresses (relating to the applied axle loading and tire pressure) do not exceed the shear strength threshold, that is, under mode-I deformation conditions. Under other circumstances, the shape may simply reflect proximity to the lower boundary of the serviceability scale. Support for the S-shape in the Texas data base appears somewhat equivocal, but some evidence appears after maintenance in the latter stages of the first TRRL Kenya study (as discussed under Validation).
8.2.4 Theoretical Models

The theoretical approach to roughness progression has been indirect because the mechanistic models of pavement response and behaviour include subsystems for cracking and rutting, but not roughness itself. The indirect approach, that has been adopted for example by Rauhut and others (1976), Paterson (1978), Ullidtz (1979) and Uzan and Lytton (1982), utilizes the variance or the standard deviation of the rut depth as a correlate of roughness. In the US Federal Highway Administration's (FHWA) VESYS IIM model, slope variance (SV) is related to the stochastic variation of rut depth, var(RD), through a spatial variation function (Rauhut and others 1976). Uzan and Lytton avoided the transformation through SV by deriving a new expression to define serviceability which incorporates the variance of rut depth explicitly, namely:

\[ P_t = 4.436 - 1.686 \log_{10} \left[ 1 + 350 \var(RD) \right] - 0.881 \text{RD}^{2.8} \]

where
- \( P_t \) = mean rut depth (inches);
- \( C \) = cracking area (ft\(^2\)/1000 ft\(^2\));
- \( P \) = patching area (ft\(^2\)/1000 ft\(^2\)); and
- \( \var(RD) \) = variance of rut depth (inches\(^2\)).

In general however, information is scant for quantifying the relationship between rut depth and variation of rut depth, and thence to roughness. Conceptually the relationship is strong when roughness is considered to be induced by variations in the pavement deformation occurring under vehicle loading. However, strong empirical data are required to establish the relationship.

8.2.5 General Comments

A practical drawback of many models such as those of Equations 8.1, 8.2, 8.4, 8.5, 8.7 and 8.8 is the need to know the initial roughness \( R_0 \) in order to apply the model to prediction. Commonly in applications for network policy valuation or pavement management, the initial roughness immediately after construction is not known and the assumption of a standard initial value can lead to highly invalid conclusions where the construction standards differ from those assumed (e.g., typical ranges are 1.0 to 2.5 m/km IRI for new asphalt pavements, and 1.5 to 4.0 m/km IRI for new surface treatment pavements). To overcome the problem, a shift function must be applied for the models to coincide with observed data at a point in time and this raises the question of whether the rate of progression depends upon the initial roughness. In some cases a derivative function may be derived successfully. Thus models which originally have a derivative or incremental form, such as Equations 8.3 and 8.6, are inherently the most attractive for simulation modelling and pricing studies.

In summary, it is readily apparent that wide ranges exist within both the structurally-related and time-related models reported for predicting roughness progression, and particularly that little success has been achieved in the identification of joint effects of traffic loading, pavement strength, time and environment. Roughness progression rates range from less than 2 percent to more than 30 percent per year in the studies, often without clear quantification by explanatory parameters that would permit the transfer and application of models to conditions outside the original base.
There is reasonably consistent agreement that the trend of roughness is generally convex over time or traffic, with the rate of progression increasing toward the end of the pavement life or as the roughness level increases; the degree of convexity however varies greatly across the studies from nil (linear model) to high for those reporting high percentage rates of increase. In some cases the high rates appear to be associated with high levels of surface distress. Reduction of the progression rate at the end of the pavement life as indicated by the S-shaped function, Equation 8.7, appears to be a special case applying only when traffic loadings are low enough to permit rutting to stabilize and when surface distress remains minor.

8.3 New Approach and Empirical Base

8.3.1 Objectives

With few exceptions, the earlier models for predicting roughness progression treat roughness as an independent mode of distress, attempting to correlate it directly to primary factors such as traffic loading and pavement strength, or age, throughout the pavement's life. What is lacking in them is a clear mechanistic association between roughness and the other modes of distress such as cracking, potholing and rutting which themselves give rise to some of the changes in roughness. Implicitly in some of the relationships, though not expressed explicitly through distress parameters, the acceleration of roughness progression that is observed towards the end of the pavement life is due to the occurrence and growing severity of surface distress. While there is a need for aggregate models that simply relate roughness (or a performance index such as serviceability) to primary factors, such models are inadequate for policy evaluation and management in two important respects.

First, is the need to evaluate maintenance effects. Many maintenance activities repair or modify surface defects such as cracking, ravelling, potholes and depressions through the means of surface seals, filling materials and patching which have a negligible impact on pavement strength and an indeterminate net effect on pavement age (in respect of the remaining life). Thus aggregate models provide no explicit mechanisms by which the effects of such maintenance upon roughness can be evaluated, especially in the short term. In the long term, the only approach that could be adopted for aggregate models would be to develop separate curves for specific generalized levels of maintenance, assuming certain standards of timeliness and activity (see for example a very recent Indian study, CRRI 1986). Preferably therefore we desire a roughness model that makes the effects of surface defects and repairs explicit in both short- and long-term predictions.

Second, is the recognition of variations in the behavior of road pavements, arising from both the mechanistic differences of pavements that fall within one strength group, and also from the inherently stochastic nature of properties and behavior within one pavement and across similar pavements. We are aware for example that cracking develops at a time dependent upon the surface type and aging effects, as well as upon traffic loading and strength factors which differ fundamentally from those influencing deformation. This, together with stochastic variations, means that two pavements in the same general strength group and under similar traffic may probably crack at different times and, if the cracking influences roughness, then the roughness progression rates would differ.
Next we recognize a need to incorporate the concurrent effect of structural factors and environment-age factors in a model of roughness progression, in cognizance that the apparent attribution of damage to either one or the other in most previous models was in fact partly true in each case but not universally true.

These needs have been addressed by developing two empirical models, differing in their levels of complexity and accuracy, and suiting different applications.

The first, described next, is a comprehensive and fairly sophisticated model intended for the purposes of life-cycle simulation of discrete construction and maintenance activities, and for pavement management applications. For these purposes the model needs to have an incremental (or essentially, derivative) form to predict changes of roughness given the current state and imposed conditions, the history (if significant) and the chosen interventions of maintenance. It represents the interacting effects of individual modes of distress, maintenance, traffic, pavement age and strength, and environment on roughness progression. Note here that this should not be confused or compared in any way with the AASHTO Serviceability formula in Equation 4.1 (or the adaptation in Equation 8.8) which does not indicate the interactions between distress types and roughness progression but indicates only the relative weights that the various distress types have in the perceived need for rehabilitation intervention, i.e., the serviceability index.

The second, described in Section 8.5, is a simple aggregate roughness model that predicts the trend of the absolute level of roughness as a function of only four primary parameters, namely the cumulative traffic loading in ESA, pavement age, pavement strength, and a generalized environmental coefficient. Any individual effects of surface distress and minor maintenance on roughness are implicit in the model. It was developed for use in the more general performance model applications such as road transport pricing and cost allocation studies, in which the more detailed simulation of individual distress types and maintenance effects are both cumbersome and unwarranted. In this respect the model parallels the AASHTO serviceability-trend model, but differs in that it incorporates mixed traffic and time-related environmental effects, and predicts roughness rather than serviceability.

### 8.3.2 Empirical Base

The empirical base chosen for developing the statistical models was again the Brazil-UNDP Road Costs Study because it incorporates the most comprehensive sets available of parallel time-series data on roughness, cracking, raveling, rut depth, maintenance, traffic loading and rainfall for a broad, experimentally-designed factorial of flexible pavement types and traffic volumes, as outlined in Chapter 4. In particular it is known that the roughness measurements were all calibrated to a reliable profile reference so that the trends over the five-year study period had no long-term systematic bias (see Section 2.4).

Before going on to introduce the data it is necessary to comment upon how the random and short-term systematic errors in the roughness data were handled in the analyses (these errors, with specific reference to the Brazil study, were discussed in Section 2.4 and a sample of data was shown in Figure 2.13). First,
during the data processing phase, some averaging of consecutive roughness measurements was necessary in order to make the time intervals coincide with those of all other condition measures; this had the effect of reducing the random error from about 14 percent to about 10 percent.

Second, it was found, when analysing the data for every observation interval (a total of 3,149 observations) for the comprehensive incremental model, that that 10 percent random error in the absolute roughness measurement resulted in an error in the roughness increment that was about four times greater than the average roughness increment (0.31 and 0.08 m/km IRI respectively). For this reason the capacity of the statistical analysis to discriminate between the influences of the various parameters was severely limited, and the detailed interval-approach had to be abandoned. This experience also led to the recommendation made in Chapter 2 that profilometric methods should be preferred to response methods in future deterioration research in order to reduce the size of the random measurement error.

Thus third, it was decided necessary to apply curve-smoothing techniques to the roughness data and to analyse the gross change of roughness over the study period in order to dramatically improve the "signal-to-noise" ratio. There being 8 to 10 measurements on each section over 3 to 5 years, this improved the ratio ten-fold from 0.27 to 3.0. The final use of this technique was solely to determine accurate values of the first and last observations of roughness and the difference between them, and in this way any influence that the choice of smoothing function might have had on the form of the final model function was avoided. Because of interest in the shape of the trends however several smoothing techniques were evaluated including linear, piecewise linear-exponential, quadratic, piecewise linear-quadratic, and exponential, of roughness on time (time being the most accurate as well as most convenient base). Of these the exponential form generally gave the best fit for most study sections, as follows:

\[ \ln R_{jt} = a_j + b_j \ \text{AGE}_{jt} + w_{jt} \]  

(8.9)

where

- \( R_{jt} \) = roughness of the pavement subsection \( j \) at time \( t \) determined from the calibrated response-type meter (Maysmeter);
- \( \text{AGE}_{jt} \) = age of the pavement at time \( t \), in years;
- \( a_j, b_j \) = coefficients to be estimated for each subsection \( j \) and rehabilitation phase; and
- \( w_{jt} \) = roughness measurement error (generally linear with the logarithm of roughness, see Section 2.4).

These smoothed functions were also used to estimate the initial roughness, \( R_0 \), at the time of construction or most recent overlay, for use in the aggregate model analysis. In the case of the few sections in which the observed roughness trend appeared to be slightly negative on account of measurement error, the initial roughness was set equal to the mean observed roughness.

The characteristics and scope of the data available are illustrated in Figure 8.3 and Table 8.1. These show the changes of roughness on the 380 subsections of the Brazil-UNDP study, as determined from 3,149 measurements and aggregated by the smoothing technique just described. A number of important characteris-
tics are evident from the figure in which, for the sake of clarity, the data represent just a 30 percent sample (one from each fully independent pavement) and the trends are simplified to straight lines. Firstly, the initial roughness was clearly not a constant value for the study pavements as had been adopted in the AASHO Road Test and Kenya formulations, but varied between about 1.0 and 3.5 m/km IRI for asphalt pavements and between 1.3 and 7.3 m/km IRI for surface treatment pavements. Secondly, the rate of increase of roughness was not a unique function of age in the sample; some young pavements showed early rapid deterioration, while others showed negligible deterioration rates even at ages of 12 to 20 years, and still others showed a late rapid deterioration that is reminiscent of the trend shown in Figure 8.2.

The average annual rates of roughness progression, as indicated in the table, were 5.2 percent for asphalt pavements, 4.8 percent for surface treatments and 6.7 percent for cemented-base pavements. The rates ranged from nil to maxima in the order of 22 to 29 percent per year, which are less than the maximum rates of about 46 percent per year noted in the UK study, shown in Figure 8.2. The higher rates tended to be associated with high levels of cracking, as shown in Figure 8.4 and Table 8.2. For uncracked pavements the rate averaged about 4 percent per year and was similar for all pavement types, while for cracked pavements the rates tended to be higher for those pavements in which the cracked layer was thicker, especially for cemented-base pavements. It is evident that the study encompassed a broad range of circumstances including, for example, a range of

Figure 8.3: Example of roughness trends observed over 5-year period in the Brazil-UNDP Road Costs Study

![Graph showing roughness trends](image-url)

Note: Trends simplified to linear approximation for clarity. Sample shown is one subsection from each of 116 independent sections.
Source: Brazil-UNDP study data.
Table 8.1: Ranges of roughness progression data and associated parameters observed in Brazil-UNDP Road Costs Study for three pavement types

<table>
<thead>
<tr>
<th>Observed Parameter Unit</th>
<th>Asphalt and asphalt overlays</th>
<th>Surface treatment, granular base</th>
<th>Cemented base</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>Min</td>
<td>Q25</td>
</tr>
<tr>
<td>Roughness progression</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- rate</td>
<td>5.16</td>
<td>0</td>
<td>0.99</td>
</tr>
<tr>
<td>- increment</td>
<td>IRI</td>
<td>-0.50</td>
<td>0.07</td>
</tr>
<tr>
<td>- average</td>
<td>IRI</td>
<td>2.67</td>
<td>1.13</td>
</tr>
<tr>
<td>- initial</td>
<td>IRI</td>
<td>1.80</td>
<td>1.02</td>
</tr>
<tr>
<td>- final</td>
<td>IRI</td>
<td>2.98</td>
<td>0.93</td>
</tr>
<tr>
<td>Modified structural number</td>
<td>-</td>
<td>4.82</td>
<td>2.08</td>
</tr>
<tr>
<td>Benkelman deflection</td>
<td>mm</td>
<td>0.68</td>
<td>0.19</td>
</tr>
<tr>
<td>Age (average)</td>
<td>yr</td>
<td>7.09</td>
<td>2.15</td>
</tr>
<tr>
<td>Traffic loading</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- annual per lane</td>
<td>MESA/yr</td>
<td>0.35</td>
<td>.0001</td>
</tr>
<tr>
<td>- increment</td>
<td>MESA</td>
<td>1.40</td>
<td>.0006</td>
</tr>
<tr>
<td>- avg. cumulative</td>
<td>MESA</td>
<td>1.78</td>
<td>.0009</td>
</tr>
<tr>
<td>Cracking - average</td>
<td>%</td>
<td>15.1</td>
<td>0</td>
</tr>
<tr>
<td>- increment</td>
<td>%</td>
<td>20.3</td>
<td>-22.5</td>
</tr>
<tr>
<td>Patching - increment</td>
<td>%</td>
<td>4.3</td>
<td>-3.3</td>
</tr>
<tr>
<td>Rut depth - increment</td>
<td>mm</td>
<td>0.90</td>
<td>-0.22</td>
</tr>
<tr>
<td>of standard deviation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of subsections</td>
<td></td>
<td>156</td>
<td>145</td>
</tr>
</tbody>
</table>

Note: Min = Minimum, Q25 = 25% quartile, Q75 = 75% quartile, Max = maximum.

Source: Author's analysis of data from Brazil-UNDP study.
Figure 8.4: Influence of the amount of cracking on the rate of roughness progression observed in Brazil

![Graph showing influence of amount of cracking on rate of roughness progression](image)

Note: Indexed amount of cracking weights wide cracking twice as much as narrow cracking.
Data are group-means of 30 groups representing 4-year trends of 361 subsections.
Source: Brazil-UNDP study data.

Table 8.2: Observed normalized rate of roughness progression in relation to pavement type and amount of cracking: Brazil-UNDP study

<table>
<thead>
<tr>
<th>Pavement type</th>
<th>Statistic</th>
<th>% Roughness increase per year by amount of cracking (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0-9</td>
</tr>
<tr>
<td>Surface treatment on granular base</td>
<td>Average</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td>Range</td>
<td>3.0-4.4</td>
</tr>
<tr>
<td></td>
<td>Sample</td>
<td>110</td>
</tr>
<tr>
<td>Asphalt concrete on granular base</td>
<td>Average</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>Range</td>
<td>0.6-10.1</td>
</tr>
<tr>
<td></td>
<td>Sample</td>
<td>94</td>
</tr>
<tr>
<td>AC or DST on cemented base</td>
<td>Average</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td>Range</td>
<td>4.1-4.7</td>
</tr>
<tr>
<td></td>
<td>Sample</td>
<td>27</td>
</tr>
<tr>
<td>All pavements</td>
<td>Average</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>Total sample</td>
<td>231</td>
</tr>
</tbody>
</table>

Note: "Range" applies to the averages of the 30 pavement strength groups.
Sample indicates number of pavement subsections.
Source: Author's analysis of Brazil-UNDP Study data.
pavement age from new to 23 years, of traffic loading from 100 to 1.7 million ESA per lane per year and up to a maximum of 17 million cumulative ESA, and of pavement modified structural number from 2.1 to 8.7. The range is somewhat lacking only in the extreme levels of roughness; although the maximum observed was nearly 10 m/km IRI (130 QI), only seven subsections reached a roughness above 7 m/km IRI (90 QI) because nearly all potholes were patched promptly under the minimum maintenance policy applied throughout the study. From this and other studies it has been found that roughness levels over 8 m/km IRI are almost always associated with either open potholes, or else extremely frequent and poor-quality patching.

A comment needs to be made about the apparently negative roughness trends evident for some of the subsections. In some cases the physical explanation of the reduction in roughness was found in the patching maintenance applied during the study. Where there was no physical explanation, the negativeness of the trend was not statistically significant, and instead was simply a coincidence of measurement errors being relatively positive at the start, and relatively negative at the end, of the study for those particular measurements. Since such errors are a natural part of any empirical data set, and were here considered to be randomly distributed across the observations, it was important not to correct the negative trends to zero since that would have introduced a bias into the data. Instead the errors were treated statistically in the regression analyses, as discussed in the following sections.

8.4 COMPONENT INCREMENTAL MODEL

8.4.1 Principles

The basic hypothesis used in developing the comprehensive roughness progression prediction model was that the various mechanisms giving rise to roughness changes should be represented by separate components within the model. In broad terms it was considered that these fell into three groups, according approximately to the parameters involved, the source depth within the pavement, and the resulting waveband of roughness, as follows.

Structural deformation, resulting from plastic deformation in the pavement materials under the shear stresses imposed by traffic loading, commonly appears as rutting in the wheelpaths. Usually this deformation is considered to occur primarily in the subgrade layers, though plastic deformation may also be large in other layers such as the base and subbase when the material shear strength is inadequate (either through saturation or through increased axle loadings and tire pressures). This category thus includes the effects of environmental factors on material strength and rutting behavior under loads. However rut depth alone will not give rise to roughness if the depth is uniform; instead it is the variation of rut depth which relates to roughness as deviations in the longitudinal profile. These variations will therefore be a function of the uniformity of construction and environment of the pavement layers, and particularly of the subgrade. Typically these variations are likely to have medium wavelengths in the range of 2 m to 10 m, but shorter in the case of base deformation.

Superficial defects such as potholes, patches, ravelling, cracking, or shoving, humps, and localized depressions are generally associated with shallow-seated distress originating in either the surfacing or base of the pavement.
These defects typically range in size from less than 0.3 m up to about 2 meters in diameter, with a corresponding waveband of about 0 to 5 m wavelengths. The inclusion of cracking in this group may seem strange, because the crack openings are narrow and easily bridged by the tire so that the vehicle does not "see" the crack as roughness unless there is a significant step or faulting across the crack. Its inclusion comes instead from the local or "birdbath" depressions that often develop in a cracked area, and from the effects of spalling of wide cracks (the precursor of a pothole) such as are particularly evident on cemented base pavements.

The environmental factors, which influence roughness through non-structural effects, include primarily temperature and moisture fluctuations, but also foundation movements such as subsidence, which cause volume changes or distortions in the pavement. Daily thermal expansion and contraction movements are a function of the diurnal temperature range, which is often large in desert climates; particularly after cracking, these movements lead to faulting and spalling. The effects of seasonal moisture movements depend upon the effectiveness of drainage and the shrinkage properties of the material; in the extreme case of expansive soils without moisture control, the volume changes may be large (Rauhut and Lytton (1984) describe a useful model of this). In freezing climates, the combined volume/roughness effects of temperature and moisture are particularly severe. Thus there are various factors, not directly related to traffic or pavement strength, which influence roughness progression and which appear potentially difficult to quantify. The effects will be evident in any of the wavebands of roughness, that is long, medium or short wavelengths, depending on the mechanism involved.

The model was therefore structured as follows:

\[ \Delta R_t = f_1 (\text{strength, condition, Atraffic, environment}) \]
\[ + f_2 (\text{Asurface condition, Amaintenance}) \]
\[ + f_3 (\text{condition, environment, Atime}) \]
\[ + \text{measurement error}. \quad (8.10) \]

This shows an additive combination of the three major components in the increment of roughness, and provides at the same time for such interactions as may prove significant. During the course of analysis, many formulations of the terms comprising each component were tested, and the final choice was determined with respect to three criteria, namely that

- the terms and the magnitudes of their coefficients met the requirements of engineering reasonableness;
- the parameters included in the terms were statistically significant at the 0.005 level; and
- the function could be integrated to be broadly consistent with the aggregate model.

### 8.4.2 Empirical Model and Accuracy

After preliminary evaluation using linear regression techniques, the final statistical development of the model was made using a nonlinear least-squares regression technique (SAS 1979). While the basic form of the model became evident in the early stages, many variants were examined in order to avoid adverse effects of correlations between the explanatory parameters, to review alternatives.
to the parameters, and finally to ensure that the model was integratable and fundamentally consistent with the aggregate levels model of roughness progression. The model, as estimated on the Brazilian data was as follows:

\[
\Delta R_{It} = 134 e^{0.023 t} SNC^{5.0} + 0.114 \Delta RDS + 0.0066 \Delta CRX + 0.010 \Delta PAT + Z_{pot} + 0.023 R_{It} \Delta t \tag{8.11}
\]

where
- \(\Delta R_{It}\) = increase in roughness over time period \(t\), m/km IRI;
- \(R_{It}\) = roughness at time \(t\), m/km IRI;
- \(\Delta RDS\) = increase in rut depth standard deviation of both wheelpaths, mm;
- \(\Delta CRX\) = increase in indexed area of cracking, \(\%\);
- \(\Delta PAT\) = increase in area of surface patching, \(\%\);
- \(\Delta t\) = incremental time period of analysis, years;
- \(\Delta NE_4\) = incremental number of equivalent axle loads in period \(\Delta t\), million ESA/lane;
- \(SNC\) = modified structural number of pavement strength;
- \(t\) = age of pavement or overlay (yrs);
- \(H\) = thickness of cracked layer, mm; and
- \(CRX\) = area of cracking, \(\%\);
- \(Z_{pot}\) = dummy intercepts estimated for sections with potholing.

The model fitted all pavement types without significant class differences, and detailed statistics of the parameter estimates and goodness of fit of the model are given in Table 8.3 under model A(2). The three versions of model A, representing various constraints on the power of SNC, were evaluated with the aggregate levels model of the next Section. A(1) optimized the slope model, A(3) the levels model, and A(2) gave the best fit for both slope and levels models. The robustness of the formulation is evident from the generally strong significance of the individual coefficients. It is also evident from the relative contributions made by the individual components to the overall goodness of fit of the model, as presented in Table 8.4. With the model simplified to its underlying five-component form, the fit (by linear regression) improves to \(r^2 = 0.75\), whereas the original fit of \(r^2 = 0.59\) represented the variances due to all eleven parameters involved.

The goodness of fit that was achieved is shown as a scattergram of predicted and observed values in Figure 8.5. This shows that the model fits the data well, over the wide range of roughness increments observed up to 7 m/km IRI, and that the prediction error of about 0.5 m/km IRI is rather uniform throughout the range. Thus small increments are predicted as accurately as large increments,

\[\text{Statistical note: In this particular formulation of the model, the statistical estimation of the exponent was hindered firstly by the small sample of sections having strong cross-sectional effects (shown by the fair t-statistic 2.8 for the coefficient 134) and secondly by the correlation between the estimates of the coefficient } b \text{ and exponent } y. \text{ The optimum obtained by interactive constraints gave values of 38 and 4.1 for the coefficient and exponent respectively, but the value of the exponent in the final model was constrained to 5.0 (with negligible loss of fit) so as to match the strongly-determined optimum estimated in the aggregate levels model (described next).}\]
Table 8.3: Estimation of component incremental roughness prediction models

**Model A:** $\Delta RI_t = b \cdot e^{m \cdot t} \cdot (1 + SNCK)^{-y} \cdot \Delta NE_t + \sum a_i \cdot \Delta SD_{it} + m \cdot RI_t \cdot At$;

and $SNCK = SNC - c \cdot H \cdot CRX$.

**Model B:** $\Delta RI_t = \sum a_i \cdot \Delta SD_{it} + m \cdot RI_t \cdot At$.

<table>
<thead>
<tr>
<th>Parameter coefficient</th>
<th>A(1) unconstrained</th>
<th>A(2) constrained$^2$</th>
<th>A(3) constrained$^3$</th>
<th>B unconstrained</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m$</td>
<td>0.0227 (6.4)</td>
<td>0.0230 (6.5)</td>
<td>0.0237 (6.8)</td>
<td>0.0284 (8.4)</td>
</tr>
<tr>
<td>$b$</td>
<td>37.7 (2.8)</td>
<td>134 (2.8)</td>
<td>1160 (2.5)</td>
<td>-</td>
</tr>
<tr>
<td>$y$</td>
<td>4.11 (16.9)</td>
<td>5.0 (--)</td>
<td>6.65 (--)</td>
<td>-</td>
</tr>
<tr>
<td>$c \cdot 10^3$</td>
<td>0.0887 (7.0)</td>
<td>0.0758 (6.5)</td>
<td>0.0628 (5.9)</td>
<td>-</td>
</tr>
<tr>
<td>$a(ARDS)$</td>
<td>0.114 (4.4)</td>
<td>0.114 (4.4)</td>
<td>0.114 (4.3)</td>
<td>0.129 (4.8)</td>
</tr>
<tr>
<td>$a(ACRX)$</td>
<td>0.0066 (6.1)</td>
<td>0.0066 (6.1)</td>
<td>0.0066 (6.1)</td>
<td>0.0057 (5.2)</td>
</tr>
<tr>
<td>$a(APAT)$</td>
<td>0.0100 (3.9)</td>
<td>0.0099 (3.9)</td>
<td>0.0099 (3.9)</td>
<td>0.0117 (4.4)</td>
</tr>
<tr>
<td>Standard error</td>
<td>0.5141</td>
<td>0.5145</td>
<td>0.5166</td>
<td>0.5385</td>
</tr>
<tr>
<td>$r^2$</td>
<td>0.589</td>
<td>0.588</td>
<td>0.588</td>
<td>0.551</td>
</tr>
<tr>
<td>Number of observations</td>
<td>361</td>
<td>361</td>
<td>361</td>
<td>361</td>
</tr>
</tbody>
</table>

1/ $t$-statistics are given in parentheses; (--) indicates constraint of the parameter; - indicates not included in the model.

2/ This constraint equated the coefficient $y$ to the value of the unconstrained estimate for the aggregate level model, i.e., $y = 5.0$

3/ This constraint on the value of $y$ optimized the estimation in Table 8.4 (later).

**Note:** $\Delta SD_{it}$ comprised $ARDS_i$, $ACRX_i$, and $APAT_i$ as the only significant distress variables.

**Source:** Author's computations from data of Brazil-UNDP Study. Method nonlinear least-squares regression.
Table 8.4: Relative contributions and significance of individual components in the component incremental roughness

<table>
<thead>
<tr>
<th>Model component</th>
<th>Model terms</th>
<th>Sequential effects</th>
<th>Individual effects</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Type I</td>
<td>F-value</td>
</tr>
<tr>
<td>Structural</td>
<td>e^mt, SNCX^5, ANE^4</td>
<td>40</td>
<td>401</td>
</tr>
<tr>
<td>Age-environment</td>
<td>mRAt</td>
<td>41</td>
<td>435</td>
</tr>
<tr>
<td>Rut depth s.d.</td>
<td>ARDSD</td>
<td>3</td>
<td>36</td>
</tr>
<tr>
<td>Cracking</td>
<td>ACRX</td>
<td>6</td>
<td>59</td>
</tr>
<tr>
<td>Patching</td>
<td>APAT</td>
<td>5</td>
<td>59</td>
</tr>
<tr>
<td>Potholing</td>
<td>Zpot</td>
<td>5</td>
<td>30</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

Notes: Determined by general linear least-squares regression of the component terms above, based on Equation 8.11 where m = 0.023. Parameters are defined with Equation 8.11. Type I SS (sums of squares) is the incremental improvement in error SS for consecutive additions of further terms, expressed here as a percentage of the model SS(276.6). Type II SS are the reduction of error SS due to adding the relevant term into the model after all others have been included; it is independent of sequence and is expressed as a percentage of the error SS(93.3). The fit for the linear combination of components is \( r^2 = 0.748 \).

Source: Author, on data from Brazil-UNDP study.

Figure 8.5: The goodness of fit of the multicomponent incremental model predicting roughness progression on the Brazil-UNDP Road Costs Study data

![Graph showing the goodness of fit](image)

Source: Estimation of Equation 8.11 on Brazil-UNDP study data.
in absolute terms. This corresponds to an error in an annual prediction of roughness that is in the order of only 0.12 m/km IRI (1.6 QI, 100 mm/km BI, or 0.05 PSI), which is a highly acceptable result, especially given the diverse nature of the conditions studied.

Although the prediction error is small in absolute terms, it is apparent from Figure 8.5 that errors for the smaller increments can be large in relative terms and of a similar order to the increments being observed. This is not entirely due to a shortcoming of the model formulation however, because about one half of the apparent error derives from the observation measurement errors, a fact readily appreciated from the number of observed increments falling below the zero ordinate. In fact the measurement error remaining in the data even after the curve-smoothing and differencing exercise was still of the order of 0.2 to 0.3 m/km IRI. Thus the error component due to lack-of-fit of the model formulation itself was only about one-half of the values cited in the previous paragraph. This means that the model formulation was remarkably strong in representing the wide variety of conditions in the data base; its validity with respect to other databases is evaluated later in Section 8.7.

Examples of the fit of model predictions to the observed roughness trends are shown in Figure 8.6 and 8.7 for a diverse range of sections, roughness levels, and ages. In each case, both traffic lanes, CS and SC, are depicted for one pair of subsections; the solid lines represent the prediction using condition and traffic data for each observation date, and the broken lines represent the observed roughness trend without smoothing. As the model estimation was based on the increment over the whole period, the predicted and observed trends should coincide at each end and, if the averaging of all incremented effects over the period was valid, then the intermediate points should agree also. The trends in Figure 8.6, which represent moderately rough pavements, are predicted well and the effects of measurement errors, while obvious, are relatively small. In chart (c), the trend for direction CS fitted well for most of the period, but a large roughness change at the end was apparently not well-explained by the models; here the observed data seem inconsistent because deep patching and slurry seal maintenance were undertaken at that time and are accounted for in the model and in the condition survey, but not in the roughness measurements. There is also slight underprediction for the cemented-base pavement in chart (d), which was extensively cracked and may have had unrecorded potholes that would have accounted for the difference. For the sample of low-roughness pavements shown in Figure 8.7, the trends are also well predicted although the error effects appear relatively larger. One exception is an overlay pavement chart in (b), which was one of the "negative" trend sections, where it appears that the effect of cracking on roughness is slightly overestimated. Overall, it is readily apparent that the model fits the data of observed trends very well for a variety of flexible and semi-rigid pavements, and that the model prediction error, net of any measurement errors, is really very small, amounting to only about 0.06 m/km IRI per year.

8.4.3 Engineering Implications

The various components of the incremental model, which are presented in Equation 8.11 in a layout similar to the general one of Equation 8.10, make differing contributions to the total roughness change predicted under different situations. In the statistical estimation, they all made generally similar contributions to the model fit, though with slightly less coming from the rut
Figure 8.6: Examples of predicted roughness trends compared with observed trends on pavements having moderately high roughness level

(a) Section 113

Observed
Predicted
△ Subsection SEM CS
□ Subsection SEM SC

Note: Asphalt concrete on gravel base. 1977 traffic
0.12 M ESA/lane/yr; SNC 5.0; cracking CRX 22-84%.
Source: Equation 8.11 and Brazil-UNDP study data.

(b) Section 009

Observed
Predicted
△ Subsection COM CS
□ Subsection COM SC

Note: Asphalt overlay on surface treatment. 1977 traffic
0.89 M ESA/lane/yr; SNC 4.5; cracking CRX 13% (CS), 1-9% (SC), and maintenance.
Source: Equation 8.11 and Brazil-UNDP study data.

(c) Section 123

Observed
Predicted
△ Subsection COM CS
□ Subsection COM SC

Note: Surface treatment on gravel base. 1977 traffic
0.07 M ESA/lane/yr; SNC 3.5; cracking CRX 0-100%.
Source: Equation 8.11 and Brazil-UNDP study data.

(d) Section 170

Observed
Predicted
△ Subsection COM CS
□ Subsection COM SC

Note: Surface treatment on cemented base. 1977 traffic
0.18 M ESA/lane/yr; SNC 3.1; cracking CRX 11-100%.
Source: Equation 8.11 and Brazil-UNDP study data.
Figure 8.7: Examples of predicted roughness trends compared with observed trends on pavements having low roughness levels

(a) Section 021

(b) Section 006

(c) Section 002

(d) Section 126

Note: Asphalt concrete on gravel base, 1977 traffic
0.13 M ESA/lane/yr; SNC 5.2; cracking CRX 0-14%.
Source: Equation 8.11 and Brazil-UNDP study data.

Note: Asphalt overlay on asphalt, 1977 traffic
0.76 M ESA/lane/yr; SNC 5.3; cracking CRX 0.1-10%
Source: Equation 8.11 and Brazil-UNDP study data.

Note: Surface treatment on gravel base, 1977 traffic
0.04 M ESA/lane/yr; SNC 4.3; cracking CRX 0%
Source: Equation 8.11 and Brazil-UNDP study data.

Note: Surface treatment on cemented base, 1977 traffic
0.04 M ESA/lane/yr; SNC 4.4; cracking CRX 0-1%
Source: Equation 8.11 and Brazil-UNDP study data.
The factors that were found to have statistically significant impacts on roughness progression included rut depth variation, pavement strength, cracking, and traffic loading in the structural deformation component; cracking, patching and potholing in the surface defects component; and roughness and time in the environment-age component. Amongst the variables that were not significant were mean rut depth, age, and deflection in the structural component; ravelling and narrow cracking in the surface defects; and pavement strength, age, and rainfall in the non-traffic or environmentally-related term. What are the engineering implications?

The primary structural deformation term is conceptually based upon the AASHTO performance model, incorporating traffic loading, pavement strength and interaction with the environment-age variables. Cracking is seen to accelerate the roughness progression by causing a drop in the apparent strength (SNCK), which is the most severe for pavements in which the cracked layer(s) is thick and constitute a major portion of the pavement's structural capacity. Thus this term distinguishes between the performances of two pavements that have similar modified structural number and traffic loading but different thickness of bound layer. The effects of pavement strength and traffic loading on pavement performance are determined by the exponent of the net pavement strength parameter, SNCK. The y-value of 5.0 is very similar to the values found in the mean rut depth model (Equation 7.11), and the AASHO and TRRL-Kenya performance studies, as discussed later in Section 8.6.2. This deformation term has a strong impact on predictions especially when the traffic loading is very heavy relative to the structural number, and when cracking significantly reduces the structural capacity. In the data base, relatively few sections (about 6 percent) fell into this category but these were important in determining the interaction of pavement strength and traffic loading on roughness. In other cases, most of the roughness change was explained by the remaining model components. The important implication of this, discussed further in the next section, is that roughness, in pavements that are strong enough to resist deformation under traffic loadings, may yet develop through the other components of surface distress and environment effects.

The second term of the structural component, the relation to rut depth variation, is important. While the conceptual link with roughness is clear, this model provides a strong empirical quantification of the effect. The coefficient is statistically well-determined and robust, varying little in value over a range of model variants, including those in which the other structural term was omitted. Other forms of rut depth parameters, including a quadratic function and mean value, were significantly inferior to the linear, standard deviation parameter. The effect is strong, despite also the rather large amount of measurement error present in the data due to the manual method of rut depth measurement. Strong and independent corroboration of the coefficient's value (0.114 m/km IRI per mm) comes from a value of 0.14 m/km IRI per mm of rut depth variation obtained from a series of old, thin pavements in southern Africa, in which deformed sections with rut depths up to 80 mm and undeformed sections were compared with measured roughness levels (Paterson and Netterberg 1983). The value deduced from AASHO Road Test data is of the same order but slightly higher, being about 0.15 to 0.25 m/km per mm of rut depth variation. The strength of the relationship has particularly important implications for the mechanistic modelling of roughness because it confirms that the rut depth variation, which can be predicted mechanistically, can be used to estimate the roughness progression. Further evidence improving and confirming the relationship may be expected to come from the copious data now being collected by automated condition survey devices in various studies.
The cracking contributes a small but significant amount of roughness progression in the additive term, which supplements the effects found in the rut depth variation and structural deformation terms. It is included independently of patching and becomes negative when patching is applied to repair cracking. The term comprises the fractional extent of cracking, weighted for severity so that wide, spalling cracks dominate the effect. The mechanisms inducing roughness here are the effects of spalling and unevenness generated across cracked blocks of surfacing, and the birdbath-type of depression that often results from localized deformation in the base as a result of surface cracking. The amount of cracking, measured in crack length per unit area (e.g. m/m²) may have been a better correlate with roughness progression than severity, but was not available in the data set. A 60% area of cracking, which is equivalent to full cracking in both wheel-paths, contributes about 0.4 m/km IRI increase in roughness. Worse consequences result when the cracking is unrepaired and leads to potholing.

The patching term in the model refers to surface patching which, in the study, comprised either replacement of a distressed area of thin surfacing by cold bituminous mix or a superficial patch of fine slurry seal (5 mm max. size aggregate). Although the procedure for pothole patching on the high-maintenance subsections nominally included better preparation techniques (squaring and excavating before patching) than on the minimal maintenance subsections, no distinction could be made in the model estimation because the data on the primary condition survey files tended to be irregular in the classification of patching type and the measurement of area. The coefficient of 0.0099 indicates that surface patching increased the roughness by 0.01 m/km IRI per percentage of area patching which, after deducting the decrease due to repaired cracking (0.0066 m/km IRI), indicates a small residual of 0.0033 m/km IRI added roughness. Using the volume-to-roughness relationship of potholing as a guide (0.002 m/km IRI per mm depth per percent of lane area, Section 6.3.3), the residual is equivalent to the effect of an average protrusion (either positive or negative) of 2 to 5 mm, which is in the same order as the height of the patches. Independent data from a Kenya network survey (Section 8.7.2) indicates a coefficient of 0.08 m/km IRI per percentage area for patch protrusions of 15 to 25 mm. In general, the coefficient could thus be replaced by 0.003 H_p, where H_p is the average patch protrusion, in mm.

The final surface distress component represents pothole and other major surface profile deviations. As potholes were usually repaired immediately during the study on both high- and low-maintenance sections, and as open potholes were avoided in the roughness measurement when they were present, direct statistical estimation of the effect of potholes on roughness was not possible. In the model estimation, dummy intercept terms were estimated for five subsections which had significant defects, amounting to about 2.1 m/km IRI on the four subsections of section 112 which was cited as having "100% wide cracking, potholes and patches", and about 1.2 m/km IRI on subsection 022 SEM CS for a shoving effect in an over-filled soft asphalt mix. Using the roughness-pothole effect derived by simulation in Section 6.3, we substitute the following for this component:

\[
Z_{pot} = 0.16 \Delta V_{POT} \text{ (m/km IRI)} = 2.00 \Delta V_{POT} \text{ (counts/km QI)} \tag{8.12}
\]

where \( \Delta V_{POT} \) = increment in volume of open potholes, in m³/lane/km. As discussed in Section 6.3, some evidence suggests that the coefficient may be three or more times higher than 0.16 for sharp, deep, holes.
The final component in the model, referred to as the environment-age component, represents a uniform annual percentage increase in roughness independent of traffic loading. The component indicates that an average of 2.3 percent annual increase in roughness was estimated to occur that could not be attributed to traffic, either as the equivalent axle loading or as the number of all vehicle axles. The rate amounts to a total roughness increase of 22% over 10 years or 50% over 20 years. The coefficient is well-determined, and its value increases if the roughness increments are constrained to be non-negative, or if the structural deformation function is omitted, as shown in Table 8.3. Considerable effort was made to find other factors influencing the value of the coefficient. It was postulated for example that the coefficient might decrease as the pavement aged so that the roughness might reach an asymptotic value, all other components being negligible under good maintenance and for high pavement strength, but that effect was not significant. It was postulated also that the rate might be lower for asphalt concrete pavements and for strong pavements (that is, decreasing as SNC increases), but these were not substantiated either. Neither was the value a function of traffic volume (in either vehicles or axles) which is collinear with time within a section. The value is almost certainly influenced by the pavement environment, but no significant, sensible effect of climate could be determined within the Brazil study area, using either the Thornthwaite Moisture Index which ranged from 10 to 100 in the study region, or the mean annual precipitation, which ranged from 1040 to 1790 mm per year. As we shall see in Section 8.6.4, further work at the macroclimatic level, applying the model to data in widely different climates, has established that the coefficient does vary with climate over a range of about 0.005 to 0.10.

Finally, a few remarks about the modelling process and in particular the parameter interactions. During the course of the model development, considerable difficulty was encountered in achieving any sensible estimation of traditional model forms, such as the AASHTO performance model and others summarized at the beginning of this chapter, from the empirical base. This was due mainly to the shortcomings of the traditional forms as discussed earlier, and in part to the inherent collinearity that exists between various parameters, for example between traffic loading and pavement strength, roughness, age and surface distress, and so forth. Many interactions were investigated, in combinations and powers of parameters and in substitutions, with negligible improvement on the final model presented in Equation 8.11. An earlier version for example included roughness raised to a power of 1.1 in the nonlinear structural deformation term, but that had the two effects of obscuring the contribution of cracking and of making the integrated, levels form of the model unstable. The other case in point was the estimation of the power of the pavement strength parameter (SNC) in that same term as discussed in the footnote earlier.

The Brazilian model given by Equation 8.11 was then generalized in the light of this discussion, to give the following version:

\[
ARI_t = 134 \cdot e^{SNC - 5.0} \cdot ANE_t + 0.114 \cdot ARDS + 0.0066 \cdot ACRX + 0.003 \cdot HAPAT + 0.16 \cdot AVPOT + m \cdot RI_t \cdot At \tag{8.13}
\]

where \( m = \) environmental coefficient, estimated for Brazil study area as \( m = 0.023 \) and further evaluated in Section 8.6.4;
AVPOT = increment in volume of open potholes, m³/lane/km;
Hp = average rectified protrusion of patch repairs above or below surrounding surface, in mm;

and other parameters are as defined for Equation 8.8. The parameter coefficients are expected to transfer well to other circumstances, with the possible exception of the linear cracking and patching terms which may transfer weakly to other regimes where either the cracking/environment interaction or the patching methodology and quality differ significantly from the Brazil empirical base. Even in these cases, it will be found that the effects are both small and able to be calibrated, so that the primary attention for calibration should focus on the environmental coefficient, m.

8.4.4 Predictions and Damage Causes

Examples of the predictions of roughness progression given by the incremental model in Equation 8.13, using distress data generated by the empirical models developed in previous chapters, are presented in Figure 8.8. Two pavement strengths are shown, with six levels of traffic loading on each and minimal maintenance of patching all potholes. The curves show clearly differing trends that reflect the impacts of the different traffic loadings and surface distress on roughness. At extremely low, negligible traffic levels, roughness nevertheless increases due to the effects represented in the environment-age component; the rate of increase depends on the environment coefficient m and the initial roughness level. At higher traffic levels, the rates of roughness progression are both higher and also changing more rapidly due to the impact of surface distress.

All the curves have a generally convex shape with the rate of roughness progression increasing as the levels of roughness and surface distress increase. The rates reach about 10 percent per year in the case of normal design (e.g., 100,000 ESA per year for SNC 3) and 20 percent per year for the overloading/underdesign case (e.g., five times more traffic), which are plausible in relation to most other reported studies.

The differing contributions made by the various causes of roughness are illustrated in Figure 8.9 for one pavement under light, medium, and heavy traffic loadings (representing the overdesigned, normal design, and underdesigned cases respectively). For this example, no maintenance is being applied so that the effects of potholing are evident. Under the light loading case in chart(a), very little damage derives from the deformation or distress components. In the normal design case in chart(b), the contributions from deformation, surface distress and, environment are more or less similar. In the case of overloading shown in chart (c), the deformation component dominates the performance. In each case, the roughness in the first several years of the pavement's life increases slowly and almost linearly at a rate that depends on the design standard and the environment. The rate increases after cracking begins, and becomes very rapid if potholing is allowed to progress in the absence of maintenance. As discussed in Section 6.3, the effects of potholing may even be understated here by a factor of 3 or more, especially for sharp, deep potholes. Thus the risk of underdesign or overloading can be seen to be a very rapid disintegration in the later phase of the pavement's life, an almost L-shaped function, but one which is controllable by timely maintenance.
Figure 8.8: Roughness progression prediction curves given by the multicomponent incremental model for a maintenance policy of patching all potholes: two pavement strengths and six constant annual traffic loadings.

(a) Asphalt Concrete Pavement Modified Structural Number 3

(b) Asphalt Concrete Pavement Modified Structural Number 5

Note: Maintenance comprised patching of all potholes in the year in which they appeared.
Source: Equation 8.13 applied through Road Deterioration and Maintenance Submodel of HDM-III, using surface distress generated by models from this study in Chapters 5, 6 and 7.
Figure 8.9: Illustration of component sources of roughness damage for three levels of loading, as predicted by component incremental model.

(a) Light Loading, or Overdesigned Pavement

(b) Medium Loading, or Normal Pavement Design

(c) Heavy Loading, or Underdesigned Pavement

Note: Surface treatment on granular base pavement, SNC 3, with traffic loading of (a) 0.02, (b) 0.10, and (c) 0.50, million ESA/lane/year, respectively.

Source: Equation 8.13 applied through Road Deterioration and Maintenance submodel of HDM-III.
8.5 AGGREGATE ROUGHNESS TREND MODEL

8.5.1 Principles

The component incremental model, which models the manner in which roughness evolves from surface distress, is too complex for many applications to policy issues because it involves the iterative simulation of each type of distress and requires a computer-algorithm (such as the HDM III model) for computation. When the need is solely to predict roughness and the preference is for a closed-form solution, such as in regular pavement design or the evaluation of road damage costs and user charges, then a simple form is required.

Typically the requirement is to predict the general trend of roughness over time as a function of pavement strength and traffic loading, with a view to determining the life of the pavement before major maintenance (rehabilitation), the damage attributable to different axle loadings, and so forth. Thus the model should aggregate all subsidiary effects and mechanisms within one general function which is comprised of only the major parameters, i.e., time \( t \), roughness \( R(t) \), cumulative traffic \( X(t) \), pavement strength \( S \), environment \( E \), and preferably maintenance level \( M \). The trend of roughness level over time and traffic would be given by

\[
R(t) = F(t, X(t), S, E, M, R_0), \quad (8.14)
\]

and the change in roughness would come from the derivative form:

\[
dR = R'(t) \, dt. \quad (8.15)
\]

For simplicity these two forms will be referred to as an "aggregate level model" and "aggregate slopes model" respectively.

It is important for many analytical applications that the slopes model be integratable to the levels form, thus finite increments of roughness could be computed from first differences and all forms would be compatible. The effect of traffic in particular is important for applications to the attribution of damage.

8.5.2 Analytical Approach

The saga chronicled in this section demonstrates that the development of a "simple" model was by no means a simple task. The task proved to be highly salutary however because in the course of finding the solution some important lessons were learned that lend considerable strength to previous arguments.

In the search for a suitable model form, the models were based initially on previous forms such as the traffic-dependent forms of the nonlinear AASHTO (Equation 8.1) and Texas (Equation 8.7) models and the linear TRRL Kenya model (Equation 8.2)/. The AASHTO and Texas forms can be converted to roughness dimensions and simplified as:

\[
2/ \text{Since these required the initial roughness of the pavement in new condition (} R_0 \text{) as a parameter, } R_0 \text{ was estimated by backward extrapolation using the time-series fitted curve of Equation 8.9, constrained only when necessary to ensure that the value lay within the reasonable bounds of construction quality.} 
\]
GR(t) = \left[ \alpha S^{-Y} X(t) \right]^B \quad (8.16)

and

GR(t) = \exp \left[ - (\alpha S^{-Y} X(t)^{-1})^B \right] \quad (8.17)

where GR(t) is a form of the serviceability damage function (Equation 8.1) that can be expressed either as:

\[
GR(t) = \frac{(R(t) - R_0)}{(R_t - R_0)}
\]

in which R_t is the terminal roughness for rehabilitation, or more simply as:

\[
GR(t) = R(t) - R_0
\]

in which the \((R_t-R_0)\) term is absorbed into the constant \(\alpha\).

Two approaches were used for estimating the models in the levels form. In the first, a nonlinear least-squares regression method (SAS 1979) was applied to the data with each section being represented by the first and last observations (i.e., a total of 714 observations) and the parameters \(\beta\) and \(\gamma\) were estimated directly. In the second, which was similar to that used in the AASHO Road Test analysis, the numerical values of \(\beta\) and \(\gamma\), which is the \(\alpha SY\) term, were computed for each section directly from the time-series data and functions of strength and age parameters were estimated for each by linear regression.

Considerable difficulty was experienced in achieving stable estimates for these model forms, particularly the Texas form, and a large number of variants were also tried (as detailed in Appendix D). The variants included the linear form and also pavement age, since the time dimension had proved to be so important in the incremental model development. Two disturbing features were common to most of these models. The first was that the coefficient \(\beta\) was significantly smaller than one, which implied that the roughness-traffic or roughness-time trends were concave, with the rate of roughness progression decreasing as the pavement aged, i.e., highly contrary to both the time-series data of individual sections and to most previous evidence. The second feature was that the \(\alpha\)-coefficients of the pavement strength term (modified structural number, SNC) were generally small, because the power coefficient \((\gamma\), in Equations 8.16 and 8.17) was in the order of 2 to 3, which is considerably less than the range of 5 to 9 found in previous models (Equations 8.1 to 8.2, for example).

One explanation for the difficulties was found to be the exclusion of the surface distress and roughness-age parameters that were important in the incremental model. There are two forms which, in first differences, are similar to that model, the closest being exponential and the other linear, as follows:

\[
R(t) = R_0 \exp[\alpha S^{-Y} X(t) + m t] + \sum_i a_i SD_i(t) \quad (8.18)
\]

\[
R(t) = [R_0 + \alpha S^{-Y} X(t)] e^{m t} + \sum_i a_i SD_i(t) \quad (8.19)
\]

where \(m\) and \(t\) are as defined for the component incremental model, \(SD_i(t)\) represent various types of distress \(i\) at time \(t\), and other parameters are as defined earlier. When these forms were estimated in first difference (slopes) form the \(r^2\) of both models improved dramatically from about 0.03 to about 0.43 when the surface
distress components were included. Large differences in the values of the coefficients were found however depending upon whether the model was estimated in the levels or slopes form. Also, models suitable for one form were not satisfactory for the other.

Finally it was deduced that the primary explanations for these unsatisfactory results were certain characteristics of the empirical data. First, it was a statistical "error-in-variables" problem, in which not all the effects of pavement strength were being represented in the structural number for example, nor were all the effects of traffic represented in cumulative ESA, and so forth. Second, the analysis was essentially using cross-sectional data (one pair of observations per section) to generate a "within-section" trend of roughness over traffic or time, and thus was combining early-life data from some sections with late-life data from others, and so forth, and resulting in concave instead of convex trends.

The solution to the first was to group the data so that the effects of the main parameters were balanced and underlying effects would be accentuated. The data were grouped by modified structural number (SNC), and annual traffic loading (in orders of magnitude of ESA) and amount of cracking (three levels: 0 to 9, 10 to 39 and 40 to 100 percent) into a total of thirty groups. This revealed firstly that the (marginal) rate of damage per equivalent axle for a given level of pavement strength in the data varied with the rate of trafficking, being greatest at low volumes and least at high volumes, the elasticity (-\(y\)) being of the order of 3. In practice, if pavements were constructed according to design, light pavements would carry light traffic and strong pavements would carry heavy traffic loading, and across that "diagonal" of conditions, the value of the \(y\)-power of SNC was about 5, which is substantially the same as found in the AASHO and TRRL-Kenya studies. Secondly, the group analysis revealed that the marginal rate of damage per equivalent axle was greatest for old, cracked pavements, so that pavement age and surface condition were two omitted variables that were influential.

The third revelation of the grouped-data analysis, and the one that led to the solution for the second problem, was that the marginal rate of damage per equivalent axle was least for large increments of traffic loading (\(\Delta NE\)). All sections had an essentially constant time-window of about four years, but an extremely wide traffic-window of equivalent axle loading. The result of this in the cross-sectional analysis was that the sections of high loading rates were receiving much less statistical weighting, per ESA, in the analysis than those of low loading rates. This resulted in the apparently concave roughness trend estimated for the aggregate levels model.

Thus the solution to the problems was to seek a more-balanced cross-section in the data with respect to axle loadings and time by statistical weighting. This was achieved by generating additional data for the sections of higher traffic loading by interpolation at constant intervals of 200,000 ESA, which had the effect of balancing the constant time-window and constant traffic-window weightings in the statistical regressions.

8.5.3 The Aggregate Levels Model and Predictions

The model estimated from the "expanded" data set followed the linear approximation of the component model, as given in Equation 8.18 but omitting the surface distress terms, \(SD_i\). This generally fitted better than the exponential
model, and nonlinear least-squares regression gave the following relationship for flexible pavements without extensive cracking:

\[
RI(t) = [RI_0 + 725 (1 + SNC)^{-4.99} NE_4(t)] e^{0.0153 t}
\]  

(8.20)

where  
- \(RI(t), RI_0\) = roughness at times \(t\) and \(t = 0\) respectively, in \(m/km\) IRI;  
- \(NE_4(t)\) = cumulative equivalent standard axle loadings until time \(t\), in million ESA/lane;  
- \(t\) = age of the pavement since overlay or construction; and  
- \(SNC\) = modified structural number.

The statistics of the model, given in Table 8.5, indicate a standard error of only 0.48 \(m/km\) IRI and a good \(r^2\) of 0.75. Most of this apparently good fit however derives from the data being in levels form since \(RI(t)\) correlates well with \(RI_0\), for when the model is tested in "slope" prediction (as shown in the lower part of the table), the fit degrades considerably, with the standard error rising to 0.77 \(m/km\) IRI (which is 50 percent higher than for the component model) and the \(r^2\) dropping to 0.15. General evaluation of the predictions come in the next section, and comment on model features follows here.

The most important features of the model to note are that the value of the y-power of modified structural number is about 5, and that the shape of the function is convex, as shown by the prediction curves in Figure 8.10. In both respects the model is generally consistent with prior knowledge, thus clearly capturing the primary underlying effects in the data. The curvature of the function is very flat however and almost linear, which is similar to the AASHTO and RTIM2 functions pictured in Figure 8.1, but very different from the strong curvatures evident in the British data (Figure 8.2) and the full component incremental model (Figure 8.8) which both reflect a pronounced effect of cracking on the rate of roughness progression (albeit the former showing a stronger effect than the latter).

Another important feature to note is that the parallel effects of age and cumulative traffic are both present in the model, thus overcoming the deficiencies of the previous traffic-only and time-only models reviewed in Section 8.2. The impact of the time effect is evident at the extremely low traffic loading shown in the prediction curves of Figure 8.10, for which the roughness trend is distinctly positive.

Using this aggregate levels form, it was also possible to derive a satisfactory model having the Benkelman Beam deflection (DEF) as the pavement strength variable in lieu of the modified structural number, as follows:

\[
RI(t) = [RI_0 + 0.0129 DEF^{0.883} NE_4(t)] e^{0.0196 t}
\]  

(8.21)

The model fit however was distinctly weaker than the structural number version, as seen from the statistics given in Table 8.5. No stable estimation could be achieved using the Dynaflect deflections.

Some final comments on other features of the model development are warranted. Firstly, no significant and reasonable differences were found between major pavement types for the values of the coefficients, so those presented above apply to flexible pavements with either asphalt concrete or surface treatment surfacings on untreated granular base, but exclude semi-rigid pavements with
Table 8.5: Estimation of Aggregate Levels Models and evaluation of slope predictions

\[ R_{It} = (R_{I0} + b S^\gamma N_{E}^\beta) e^{mt} \]

Parameter estimates for given strength parameter (S)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Modified structural number</th>
<th>Constrained</th>
<th>Unconstrained</th>
<th>Deflection (mm)</th>
<th>unconstrained</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td></td>
<td>0.0169 (20.3)</td>
<td>0.0153 (17.9)</td>
<td>0.0196 (25.5)</td>
<td></td>
</tr>
<tr>
<td>b</td>
<td></td>
<td>945 (16.6)</td>
<td>725 (2.36)</td>
<td>0.0129 (8.2)</td>
<td></td>
</tr>
<tr>
<td>\gamma</td>
<td></td>
<td>-6.65 (fixed)</td>
<td>-4.99 (19.7)</td>
<td>0.833 (4.3)</td>
<td></td>
</tr>
<tr>
<td>\beta</td>
<td></td>
<td>1.00 (fixed)</td>
<td>1.00 (fixed)</td>
<td>1.00 (fixed)</td>
<td></td>
</tr>
<tr>
<td>Standard error (IRI)</td>
<td></td>
<td>0.487</td>
<td>0.475</td>
<td>0.509</td>
<td></td>
</tr>
<tr>
<td>r^2</td>
<td></td>
<td>0.737</td>
<td>0.751</td>
<td>0.714</td>
<td></td>
</tr>
<tr>
<td>No. observations</td>
<td></td>
<td>1124</td>
<td>1124</td>
<td>1124</td>
<td></td>
</tr>
</tbody>
</table>

Slope Predictions (ARI_t over 4-year study period)

<table>
<thead>
<tr>
<th>Bias (pred/obs)</th>
<th>Standard error (IRI)</th>
<th>r^2</th>
<th>No. observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.759</td>
<td>0.769</td>
<td>0.210</td>
<td>361</td>
</tr>
<tr>
<td>0.654</td>
<td>0.773</td>
<td>0.150</td>
<td>361</td>
</tr>
</tbody>
</table>

Notes: t-statistics are given in parentheses after the parameter estimates. Strength parameter S; for modified structural number S = 1 + SNC; for deflection, S = DEF, Benkelman beam deflection, in mm. Statistical method was nonlinear least-squares regression. Roughness in m/km IRI.

Source: Author's computations from Brazil-UNDP Study data.

Figure 8.10: Predictions of roughness progression using the aggregate levels prediction model for flexible pavements

(a) Modified Structural Number 3
(b) Modified Structural Number 5

Notes: Initial roughness 2.0 m/km IRI; environmental coefficient, m = 0.023.
Source: Equation 8.20.
cemented-base. Although the subset of surface treatment pavements initially seemed to yield significantly lower coefficients for both $y$ and $m$ than asphalt concrete, it was found that the range of structural number in the subset was inadequate for the estimation so that any differences were judged spurious. Likewise, the value of $\beta$, the power applied to cumulative traffic, $X(t)$, was not significantly different from 1 and thus was fixed as 1.0.

Secondly, the value of the environmental coefficient, $m$, is about 30 percent lower than in the component incremental model which implies that the specifications for traffic and time are not directly comparable in the models. The inclusion of extensively-cracked pavements in the data redressed this difference and increased the curvature of the predictions beneficially, but had other detrimental effects that prevented a more satisfactory model from being obtained.

Thirdly, the statistics in Table 8.5 indicate that the fit of the model when applied to slope prediction was improved by constraining the structural number power $y$ to the larger value of 6.65, with negligible degradation of the fit in the levels form. The value of 6.65 was obtained by heuristic optimization and subsequently was applied also to the component model as shown in the third column of Table 8.3. In the end, the best heuristic solution was determined to be the models as presented in Equations 8.19 and 8.20 respectively, with a $y$-power of 5 and only slight constraint being applied to the component model as discussed in the footnote to Equation 8.19, and no constraint to the aggregate levels model.

8.6 INFLUENCES OF TRAFFIC, TIME, STRENGTH AND ENVIRONMENT

8.6.1 Traffic Loading and Pavement Age

Much has been made of the parallel effects of traffic and time throughout this chapter and the satisfactory statistical estimation of alternative empirical models that include both effects has been presented in the two preceding sections. Here, we present the empirical evidence of those effects and discuss their implications.

Figure 8.11 presents all the roughness trends observed in the Brazil-UNDP Study for two groups of pavements, each falling in a common flexible pavement strength category; the examples shown are asphalt concrete pavements with modified structural numbers rounding to 5 in (a), and rounding to 7 in (b). The roughness trends are presented as the increase of roughness since construction or rehabilitation, which normalizes the data across different pavements. The trends are shown against cumulative equivalent standard axle loadings in the left-hand charts, and against age in the right-hand charts.

If roughness progression were a function of only traffic loading and pavement strength, as indicated by the traffic-related models in Section 8.2.1, then all the trends in the cumulative loading charts would tend to be coincident within a reasonably narrow band. Instead we note that there is a broad scatter, with some extremely rapid rates of progression at low levels of cumulative ESA and some extremely low rates at high levels of cumulative ESA. By way of comparison, the AASHTO predictions fall at about the center of the scatter giving rates of 1 m/km IRI roughness increment per 1.3 million ESA and per 8 million ESA respectively for modified structural numbers of 5 and 7.

On the other hand, the same trends appear much more as common families when depicted in relation to pavement age, as seen in the age charts. The trends
Figure 8.11: Comparing the observed effects of cumulative traffic loading and of aging on the roughness trends for two groups of asphalt concrete flexible pavements

(a) Pavements with Modified Structural Number 3

(b) Pavements with Modified Structural Number 7

Note: Normalized roughness = R(t) - R(0). Trends shown as straight lines for clarity. Each group comprises pavements with SNC values rounding to the nominal value, e.g., 2.5 to 3.4 for SNC 3.

Source: Brazil-UNDP study data.
Figure 6.12: Comparing the observed effects of cumulative traffic loading and of aging on the roughness trends for two groups of surface treatment flexible pavements

(a) Pavements with Modified Structural Number 4, Uncracked

(b) Pavements with Modified Structural Number 6, Uncracked

Note: Normalized roughness = RI(t) - RI(0). Trends shown as straight lines for clarity. Each group comprises pavements with SNC values rounding to the nominal value, e.g., 3.5 to 4.4 for SNC 4.

Source: Brazil-UNDP study data.
cover a range of about 1 to 3 percent growth per year and can be compared with the statistically-estimated overall trend of 2.3 percent per year shown on the charts. The correlation is not perfect because both traffic and age are influencing the roughness, but the improvement with respect to the correlation with traffic is clear.

The evidence is thus compelling that aging effects were strong in the data and explain a sizeable proportion of the roughness trends observed. Similar effects were evident on surface treatment pavements, although the patterns were less pronounced, as shown in Figure 8.12. Thus it is imperative that predictive models include both traffic and aging effects if they are to be reliable and applicable for the wide range of circumstances usually extant in a road network. In the case of the data illustrated in Figure 8.11, for example, the range of traffic flows in the SNC 7 pavement category was from as low as 1,000 ESA per year to as high as 2 million ESA per year.

Another important issue with respect to traffic loading effects is whether the equivalent standard axle load (ESA) transformation of mixed traffic is valid. The annual \( (Y_{EA}) \) and cumulative \( (N_{EA}) \) loading parameters used in the analysis represented the ESA computed with a uniform relative load damage power of 4, in line with the widely adopted "fourth-power damage law" originating from the AASHO Road Test. If this damage law were not applicable to all the pavements in the study, then the relative scaling, and possibly the correlations, of the roughness trends with respect to traffic loading would be altered. A detailed evaluation of the relative load damage effects evident in the data, however, confirmed that the fourth-power of load was appropriate to roughness progression, as described in the next chapter.

8.6.2 Pavement Strength Effects

The influence of pavement strength on performance is vitally important to any evaluation of pavement design standards or of rehabilitation policies, and has been the primary facet of all major design methodologies. A mutual importance therefore exists between empirical studies such as this and major design methodologies. On the one hand, empirical models, when properly formulated, provide valuable feedback because the field performance data include all the imponderables omitted from design methodologies. On the other hand, major design methodologies embrace both the results of other empirical and theoretical studies and also many years' experience of widespread prooftesting (some more successfully than others!). With empirical roughness models, the two issues involved are the influence of strength on the predicted roughness, and the choice of strength parameter.

In the models of Table 8.3 and Equations 8.16 to 8.19, the coefficient \( \gamma \), which is the power on the strength term (modified structural number), determines the marginal influence (or in economics parlance, the elasticity) of pavement strength with respect to traffic loading, and together with the coefficient \( \alpha \) determines the influence of pavement strength on roughness. Recalling the discussion on the development of the empirical component and aggregate roughness models, it was noted that the value of \( \gamma \) was not easily determined. This was due in the main to a limited cross-sectional range of strength-loading combinations, and in part to collinearity between the strength, loading, roughness and cracking variables, so that the values of \( \alpha \) and \( \gamma \) were critically dependent upon the parameter-specification of the model. In the earliest releases, the \( \gamma \)-value was 2.8,
but after improvement of the specification, the $y$-value rose to 5.0. This value implies that a 50% increase in strength would increase the ESA-life of a pavement by nearly 8-fold.

In the AASHTO (1981) performance model, the strength influence is represented by $(1+SN)^y$ in both the $\beta$- and $\rho$-functions of Equation 8.1, as detailed in Appendix A. In the $\beta$-function the $y$-power is 5.19 and in the $\rho$-function it is 9.36. The net result from the two functions is that the effective $y$, as specified in Equation 8.16, varies as a function of structural number, the damage index $g(t)$, and the axle load. The overall effect, shown in Figure 8.13(a), is one of high $y$-values of greater than 8 for low maintenance standards with a terminal serviceability lower than 2.5 PSI (i.e., roughness above 3.8 m/km IRI), and low, highly variable $y$-values of 1 to 5 for extremely high maintenance standards with a terminal serviceability higher than 3.5 PSI (i.e., roughness less than 2.0 m/km IRI).

In an analysis of various design charts, Cox and Roit (1986) noted that many methodologies showed a $y$-value greater than the 6.3 average value of the AASHTO model. In the British Road Note 31 design method, the $y$-value varies from about 10 for low traffic to about 7.5 for high traffic.

Comparison of the effects for the four methods (the component incremental model combined with distress models in the HDM-III model, the aggregate levels

Figure 8.13: Influences on the marginal roughness strength-loading relationship expressed through the power ($y$) of modified structural number

(c) AASHTO Performance Model

(b) Comparison of Empirical Models with AASHTO and RTIM2 Models


Source: Author.
model, the AASHTO model, and the RTIM2 model), is made in chart (b) of Figure 8.13. The overall $y$-value in the HDM-III model is about the same level as in the AASHTO model at low pavement strength, but diverges at higher strengths; the drop in the case of the HDM-III model is on account of a low $y$-power of only 2 in the cracking relationships (Chapter 5), and the rise in the case of the AASHTO model possibly reflects the extra resistance to a freeze-thaw environment afforded by greater thicknesses of asphalt concrete. The RTIM2 model shows similar effects to the HDM-III and aggregate levels model in the strength range above 3.5 SNC, but much greater sensitivity of life to strength in the range below 3.5 (which was the primary range of validity of that model); perhaps that greater sensitivity was on account of brittle behavior in the cemented-base pavements under extremely heavy axle loads that predominated in that study.

On balance, it is considered that the $y$-value of 5 in the component incremental and aggregate levels models is both reasonable and valid in relation to prior knowledge. The value is constant, and no alternative function (such as the AASHTO version) was found, nor is it likely to be possible from such empirical modelling from in-service data, particularly on recognition that the structural number is a general, but not necessarily sufficient, parameter of strength.

8.6.3 Strength Parameter

The modified structural number was clearly the strongest predictor of the strength parameters tested in the roughness progression models. The Benkelman beam surface deflection, which is moderately correlated with the modified structural number, was not statistically significant in the deformation term of the component incremental model, but did yield a fair alternative to SNC in the aggregate levels model. In the levels model, however, the $y$-power of 0.88, indicating the influence of deflection on roughness, was only about one-third of that expected from the structural number correlation, and the fit of the roughness prediction was inferior to that of the SNC version, particularly in the slopes version, as seen previously in Table 8.5.

The inference drawn here is that surface deflection, while being an excellent indicator of relative strength along a nominally homogeneous pavement, is apparently not sufficient as a strength comparator to provide satisfactory predictions of roughness across different pavements and conditions. The reason for this is that material stiffness is not a sufficient indicator of deformation potential across different materials, although it is of course a good indicator for any one material, because deformation depends on the shear strength of the material as well as the induced stresses. Thus surface deflection, which aggregates the material stiffness effects under essentially common stress levels, is inferior to a parameter such as structural number which accounts for both shear strength and induced stress level through the material-layer coefficients utilized in its computation.

The Dynaflect deflection indices of maximum deflection and curvature proved to be yet weaker indicators of roughness progression, although having fair correlations to both the Benkelman Beam deflection and structural number, and no statistical models for roughness progression could be found. Apparently, the stiffness derived under the low stress levels induced by this method are even further removed from the deformation behavior of the pavements than those under the heavier loading of the Benkelman Beam method.
What do these findings imply for the deflection-based pavement evaluation methods that are in widespread use? Firstly, it implies that the prime application of deflection surveys should be to measuring the uniformity of nominally homogeneous pavements and identifying the weaker sections, for application especially in rehabilitation evaluation and design. Secondly, for application to roughness prediction on different pavements, deflection data should preferably be used to estimate the modified structural number and hence the roughness; to achieve this effectively however the deflection-structural number relationships of Equations 4.5 and 4.6 could probably be improved for families of various pavement types. Finally, they do not detract from the value of deflection as a nondestructive method of structural evaluation for pavement rehabilitation design, because when a particular type of deflection is an integral part of a design method, deflection is a reasonable relative indicator of the improvement in strength achieved in the new pavements.

Neither however do the findings imply that the modified structural number is a sufficient indicator of pavement strength. The lack-of-fit remaining in the empirical models is due in good part to material behavior and strength effects which are not embraced by the modified structural number. Mechanistic theory and analytical studies have shown many shortcomings of the structural number formulation, including its lack of account for the influences of layer thickness on stress and strain distributions (and hence material stiffness and behavior), or for the sequence of layers and interactions between them (such as the influence of subgrade stiffness on the effective stiffness of a granular subbase or base, and of a cemented subbase on the stiffness and behavior of either a granular base or a bituminous base). Attempts to develop and estimate an improved formulation of the modified structural number from the Brazil-UNDP data base were unsuccessful, mainly on account of the many coefficients to be estimated, the measurement errors inherent in the data, and the difficulty of grouping material and layer data into a small enough number of parameters for estimation to be practicable. Limited attempts to find a satisfactory mechanistic parameter alternative to structural number also were unsuccessful. More work is warranted on finding an appropriate or improved formulation of a pavement strength parameter that embraces all these effects.

8.6.4 Environment

Most climatic effects in the roughness models have been incorporated through the various components; the modified structural number includes effects of drainage, rainfall and cracking, since the layer strengths (e.g., in terms of CBR) are determined for in situ conditions (not for the "worst" conditions often used for design purposes, such as the soaked CBR). For prediction applications, therefore, the modified structural number should be computed either on a seasonal basis, or weighted by season as follows:

$$S_{\text{NC}} = \left[ t_a S_{\text{NC}}^{-\gamma} + t_b S_{\text{NC}}^{-\gamma} \right]^{-1/\gamma} \quad (8.22)$$

the subscripts a, b denote seasons a and b respectively, $t_a$ and $t_b$ are the fractions of time applicable to the respective seasons, and $\gamma = 5$ according to Equations 8.11 an 8.20.

The remaining and perhaps most important effect is in the coefficient $m$ of the environment-age component in Equation 8.13. It represents an annual average effect of all non-traffic-related environmental factors, including daily
temperature changes, seasonal and drainage-related moisture variations, freeze-thaw effects, foundation movements, and so forth. Mention was made earlier that the value of \( m \) could not be explained by microclimatic factors (such as local rainfall) in the course of the model estimation. However, application of the roughness model to performance data from widely differing environments in countries other than Brazil has provided strong evidence that the coefficient can be related to environmental factors at a macroclimatic level.

Preliminary evaluation of the environmental coefficient \( m \) has been made in a total of five environments to date, viz.:

1. Arizona, USA - arid to semiarid, hot to freezing;
2. Tunisia - arid to subhumid, warm nonfreezing;
3. Kenya - semiarid, warm nonfreezing;
4. Colorado, USA - subhumid, warm to freezing; and
5. Brazil - subhumid to humid, warm nonfreezing.

The Arizona (Way and Eisenberg 1980) and Kenya (O'Connell and Jones 1984, Jones, O'Connell and Howitt 1985) studies comprise time-series data similar to the Brazil reference study. The Tunisia study (Part II in Newbery and others 1988) was a one-time survey of road characteristics and condition which permitted a back-analysis of overall roughness trends. The Colorado study was the Ordway Experimental Base Project (Shock and Kallas 1982), which was a satellite experiment of the AASHO Road Test; the data available were the initial and final conditions from a 13-year monitoring period of 20 pavements located on a common road. Key data statistics from the studies, given in Table 8.6, show that they covered wide ranges of pavement strength, traffic loading, and pavement age (up to 34 years), and that the time-series data covered periods from 3 to 13 years (with the exception of the one-time Tunisia survey).

The method of evaluation assumed that the environmental class-effect was entirely represented in the coefficient \( m \) and that the coefficients and formulation of all other terms in the component incremental model of Equation 8.13 were essentially correct. The model predictions, applied using the strength, traffic, age, and condition data from each study, were then compared with the observed changes of roughness over the study period. The value of \( m \) compensating for any lack-of-fit between the observed and predicted roughness increments was considered the estimate of \( m \) for that environment. In the absence of time-series in the Tunisia study, the back-analysis method adopted reasonable trial values of initial roughness by class of road (namely 2.7, 3.1 and 3.5 m/km IRI) and solved for the mean roughness and \( m \) by trial and error. In each case, the validity of the assumption concerning the other roughness components was checked against any correlation between prediction residuals and pavement strength.

The results shown in the lower part of Table 8.6 indicate very strong and reasonable influences of climate on the coefficient. For example, the Arizona study covered a variety of regions from desert to mountainous that were represented by a local regional factor, \( RG \), i.e.:

\[
RG = 0.1 \left[ \text{Elevation (1000 ft)} + \text{Rainfall (inch/year)} + \text{Temperature/freeze-cycle zone (1 to 9)} \right]
\]  

(8.23)

The coefficient \( m = 0.012 \ RG \), estimated by statistical regression, yields \( m \)-values from about 0.006 for semiarid desert conditions (\( RG = 0.5 \) to 1.5) ranging
Table 8.6: Evaluation of environmental coefficient of roughness progression in various regions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Arizona USA</th>
<th>Tunisia (1984)</th>
<th>Kenya Network</th>
<th>Kenya USA</th>
<th>Colorado Study</th>
<th>Brazil Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of analysis</td>
<td>Time-series</td>
<td>Back-analysis</td>
<td>Time-series</td>
<td>Time-series</td>
<td>Time-series</td>
<td>Time-series</td>
</tr>
<tr>
<td>Number of sections</td>
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<td>43/34</td>
<td>49</td>
<td>34/20</td>
<td>20</td>
<td>361</td>
</tr>
<tr>
<td>Age (at end) (yr)</td>
<td>10-34</td>
<td>6-29</td>
<td>4-18</td>
<td>4-26</td>
<td>13.3</td>
<td>4-28</td>
</tr>
<tr>
<td>Period of study (yr)</td>
<td>8.7</td>
<td>one-time</td>
<td>4.9</td>
<td>7</td>
<td>13.3</td>
<td>3-5</td>
</tr>
<tr>
<td>SNC</td>
<td>0.8-8.7</td>
<td>1.7-5.5</td>
<td>2.5-5.1</td>
<td>2.5-5.1</td>
<td>5.2-8</td>
<td>1.5-7.0</td>
</tr>
<tr>
<td>Benkelman deflection (mm)</td>
<td>0.24-3.08</td>
<td>-</td>
<td>0.15-0.46</td>
<td>-</td>
<td>-</td>
<td>0.13-2.0</td>
</tr>
<tr>
<td>Annual traffic (MESA)</td>
<td>0.003-0.50</td>
<td>0.01-2.4</td>
<td>0.012-3.6</td>
<td>0.008-0.6</td>
<td>0.01</td>
<td>0.000-1.8</td>
</tr>
<tr>
<td>Initial roughness (IRI)</td>
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<td>1.2-3.7</td>
<td>n.a.</td>
<td>1.4-4.9</td>
<td>0.7-1.7</td>
<td>n.a.</td>
</tr>
<tr>
<td>Roughness increment (IRI)</td>
<td>-0.2-3.2</td>
<td>0.6-2.8</td>
<td>0.3-1.7</td>
<td>-0.4-1.70</td>
<td>1.6-3.1</td>
<td>0-4.9</td>
</tr>
<tr>
<td>∆R deformation 1/</td>
<td>0-1.1</td>
<td>0.03-1.8</td>
<td>0.0-0.4</td>
<td>-</td>
<td>0.2-0.9</td>
<td>0-2.8</td>
</tr>
<tr>
<td>∆R surface distress 1/</td>
<td>0-0.4</td>
<td>0-0.2</td>
<td>0.0-1.3</td>
<td>-</td>
<td>0.02-0.1</td>
<td>0-3.1</td>
</tr>
<tr>
<td>∆R environment-age 1/</td>
<td>0.1-1.0</td>
<td>0.4-2.6</td>
<td>0.1-0.3</td>
<td>-</td>
<td>1.1-2.5</td>
<td>0.1-0.8</td>
</tr>
<tr>
<td>m-range</td>
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<td>0.004-0.012</td>
<td>-</td>
<td>-</td>
<td>0.037-0.13</td>
<td>n.a.</td>
</tr>
<tr>
<td>m-average 1/</td>
<td>0.012 RG 3/</td>
<td>0.011</td>
<td>0.014</td>
<td>0.007</td>
<td>0.065</td>
<td>0.023</td>
</tr>
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<td>Climate type</td>
<td>Arid to arid to semiarid</td>
<td>Arid to semiarid</td>
<td>Semiarid</td>
<td>Arid to semiarid</td>
<td>Cold subhumid</td>
<td>Humid</td>
</tr>
<tr>
<td>Climate class 2/</td>
<td>DF, INF</td>
<td>INF</td>
<td>INF</td>
<td>.INF</td>
<td>INF</td>
<td>WFN</td>
</tr>
<tr>
<td>Rainfall (mm/yr)</td>
<td>100-630</td>
<td>100-800</td>
<td>400-800</td>
<td>200-800</td>
<td>900</td>
<td>1200-2000</td>
</tr>
<tr>
<td>Thornthwaite Index</td>
<td>-80, -20</td>
<td>-90, -10</td>
<td>-80, -60</td>
<td>-80, -60</td>
<td>-40</td>
<td>10, 100</td>
</tr>
</tbody>
</table>

n.a. Not applicable.
- Not available.
1/ Component as defined by Equations 8.10 and 8.13.
2/ D=dry, W=wet, F=freeze, NF=nonfreeze (Rauhut and others 1984).
3/ RG=regional factor of Arizona Department of Transportation, depending on rainfall, elevation, and temperature zone as defined in Appendix A (range 0.5 to 9).
4/ Sample from 1,000 km of network.
5/ Sample from 2,000 km of surveyed network.
Source: Author's computations on reference material.
up to 0.05 (RG = 4) for freezing mountainous conditions, thus straddling the value 0.023 estimated in Brazil for a warm humid climate.

Under arid to semiarid climates, the values obtained were 0.011 for Tunisia and 0.014 for the first Kenya study (the original RTIM2 base) which agree well with the 0.01 value determined for Arizona. The strongest determination comes from the second Kenya study in which an independent estimation of Equation 8.11 was made on data from 7 years of roughness measurements resulting in an m-value of 0.0071 (Morosiuk and others 1987).

Under freezing climates, the data indicate that the coefficient rises to about 0.035 in dry-freeze conditions (Arizona) and 0.065 in moist-freeze conditions (Colorado). This range of 3.5 to 6.5 percent annual roughness increase compares well with the order of 7 percent observed in Canada (Cheetham and Christison 1981). Still higher values of 10 to 13 percent for extreme freezing conditions have been inferred informally from discussions. A direct quantification has been made from the AASHO Road Test, where Loop 1 pavements were not trafficked during the test. The fractional rate of increase of roughness for those pavements, illustrated in Figure 8.14, averaged 0.23 or 23 percent per year. The rate was seen to be virtually independent of surfacing thickness and structural number, except that very thin pavements with SN less than 1 seemed to deteriorate even more rapidly, through frost heave. This rate is probably an upper limit for wet-freezing conditions. While one may question whether the same rate would also hold when the pavements are under trafficking, the validation study (described in the next section) appears to confirm it.

Figure 8.14: Roughness progression in a wet-freeze climate in the absence of trafficking: Loop 1 pavements at the AASHO Road Test

Note: 142 time intervals from 48 sections. Key: A = 1 obs., B = 2 obs., etc. Source: AASHO Road Test data.
No quantified estimates are available yet for warm wet, subhumid to humid climates (the value for the Brazil region, a summer-rainfall climate, may differ from the value for a continually wet, or higher rainfall climate); currently a value of 0.025 to 0.030 is suggested.

Recommended values to be assigned to the m-coefficient for various climates are given in Table 8.7. While this is based on relatively few evaluations so far, the facts that the values were both reasonably consistent across widely different countries and regions, and also fall into a pattern which appears eminently reasonable, lend strong credibility to the general level of effect. Evaluation of the values over a broader spectrum is clearly desirable as more studies are made in the future.

### 8.7 EVALUATION AND VALIDATION

Considerable effort was applied during the modelling phase to make the prediction model transferable to other situations, using a mechanistic approach to the model formulation so as to avoid a simple correlation model or "fingerprint" of a specific locality. Care is needed in validating the model however in order to avoid false conclusions. Here a distinction is made between evaluating the model against current design methodologies which themselves, being based on empirical data, are subject to the limitations of data range and models specification, and validating the model by applying it to independent data sets of road performance data and assessing the prediction error.

#### 8.7.1 Evaluation against Design Methods

Certain elements of the prediction models have been compared with those of major design models during the preceding discussion. Here we pursue the axiom

<table>
<thead>
<tr>
<th>Moisture classification</th>
<th>Moisture classification index</th>
<th>Tropical nonfreezing</th>
<th>Subtropical nonfreezing</th>
<th>Temperate freezing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arid</td>
<td>-100 to 61</td>
<td>0.005</td>
<td>0.010</td>
<td>0.025</td>
</tr>
<tr>
<td>Semiarid</td>
<td>-60 to -21</td>
<td>0.010</td>
<td>0.016</td>
<td>0.035</td>
</tr>
<tr>
<td>Subhumid</td>
<td>-20 to +19</td>
<td>0.020</td>
<td>0.030</td>
<td>0.065</td>
</tr>
<tr>
<td>Humid, wet</td>
<td>20 to 100</td>
<td>0.025</td>
<td>0.040</td>
<td>0.10-0.23</td>
</tr>
</tbody>
</table>

1/ After Thornthwaite (1955).
2/ Definition of these classes is as yet uncertain: Tropical means warm temperatures 15 to 40°C and small range; Subtropical includes warm, high range (5 to 50°C) and cool, moderate range (-5 to 30°C); Temperate freezing includes climates with annual pavement freezing (this last class may require either subdivision, or revision of the model).

Source: Author's recommendations.
of mutual feedback between empirical models and design models (Section 8.6.2) by comparing the life predictions of the models developed here with those of the AASHTO (1981) design method and other empirical models. The component incremental model here is referred to as HDM-III since the combination of distress relationships from this book and the incremental roughness model are incorporated in the Highway Design and Maintenance computer model HDM-III (Watanatada and others 1987a).

Figure 8.15 presents the predictions of pavement life, defined for this example by the number of cumulative equivalent standard axles (ESA) causing the roughness to rise from an initial value of 2 m/km IRI (3.5 PSI, 1400 mm/km BRI) to a threshold of 6 m/km IRI (1.7 PSI, 4,700 mm/km BRI). For the predictive models having time-traffic interaction, the life period is taken as 15 years.

In the intermediate range of pavement strengths from about 2.5 to 4.0 SNC, the lives predicted by the HDM-III and aggregate roughness models are of a similar order to, but generally lower than, the predictions of the RTIM2 empirical model. Without calibration to Kenyan conditions, the HDM-III model predicts lives of 50 to 60 percent less than the RTIM2 predictions: adjustment of the environmental coefficient m to the Kenyan climate accounts for one-third of the difference, and adjustment of the analysis period from 15 to the order of 5 years (in respect of the high rate of trafficking on the Nairobi-Mombasa road in the Kenyan study) accounts for the remainder. These models are thus consistent with each other (though RTIM2 represents conditions somewhat specific to Kenya), and

Figure 8.15: Pavement life in cumulative equivalent standard axles as predicted by various models for common performance limits of 2 to 6 m/km IRI

Note: Roughness conversions by Figure 2.15. Limits 2 to 6 m/km IRI correspond to 26-78 Q1: 1400-4,700 mm/km BRI, 3.5-1.7 PSI.

Sources: AASHTO Equation A.4; RTIM2 Equations A.9 and A.10; Queiroz-GEIPOT Equation A.16. Component model Equation 8.13 plus recommended distress equations (see Chapter 10) applied in submodel of HDM III (Watanatada and others, 1987a); Aggregate model Equation 8.20.
especially in comparison to the predictions of the AASHTO design method which are about a factor of 10 smaller. The predictions of the correlative model developed by Queiroz (1981) in the early analysis of the Brazil-UNDP study data are clearly only applicable over a very narrow range of 2.5 to 3.0 SNC, and that life-strength (SNC) relation is distinctly atypical of the other models; this illustrates the shortcomings that can occur when the range of data available is limited as it was for that early analysis.

At the extremes of pavement strength the comparisons are different. For very weak pavements, the slope of the life-strength relationship steepens for all models except the aggregate levels model, and the lives become shorter than 30,000 ESA for all but the HDM-III and aggregate levels models. For very strong pavements, the HDM-III and AASHTO predictions converge, but are still somewhat smaller than the RTIM2 predictions.

The primary explanation for the enormous difference between the AASHTO model and other model predictions is believed to be environmental effects. Adjustment of the environment coefficient in the HDM-III model accounts for a large portion of the differences, adjusting the prediction upwards about 50 percent to match the semiarid climate of the Kenya data base of RTIM2, and adjusting the prediction downwards by about 60 percent to match the freeze-thaw climate of the AASHTO empirical base. It appears however that the freezing climate of the AASHTO base may have had a more severe effect on the life of thin pavements than of strong pavements; (it is thought that the strong pavements gained their strength from large thicknesses of asphalt and were thus relatively less susceptible to thaw conditions than the weak pavements, in which the behavior of the unbound layers probably dominated the performance). Thus the environmental effect should apparently not be a constant factor applied to the ESA-life, as is done in the AASHTO procedure, but instead vary with respect to the pavement strength and the period. The environment coefficient m and cracking predictions in the HDM-III model set of relationships together have that effect, which results in a slight rotation of the life-strength relationship about a focus at the upper strength extreme.

The aggregate levels model of roughness gives slightly higher predictions of life (by 10 to 20 percent) than the component incremental model HDM-III. This is due to the fact that the estimation of the aggregate levels model was made on only uncracked pavements, so the predictions have an optimistic bias in those cases where cracking develops before the critical roughness is reached.

The component incremental model, incorporated in HDM-III, thus compares extremely well in general with the reference methods over their valid ranges, has the advantages of an empirically-based interaction between environment, age, distress, and traffic for explaining cross effects, and also behaves more rationally over the full range than any of the alternative models.

8.7.2 Validation

The validity of the component incremental roughness prediction model was studied by applying the model to performance data obtained in published independent studies conducted elsewhere. Six major data sets, comprising time-series data of road condition, traffic, and pavement characteristics, were used from the Arizona, Kenya (two) and Colorado studies referred to earlier, the Texas study over a 7-year period (Texas Transportation Institute 1986), and the AASHO Road
Test (AASHO 1962) using original study data. Two further data sets, from Tunisia and southern Africa permitted historical back-analysis from current condition for extremes of pavement age and strength.

Methodology

These data bases cover a wide range of conditions and an extremely large amount of data, mostly on computer files. The data processing included transformation of all roughness and serviceability values into the International Roughness Index, all other pavement condition data into parameters compatible with the prediction model, and the conversion of traffic and pavement construction data into equivalent standard axle loadings and modified structural number, respectively. For the AASHO Road Test data, the structural number was adjusted for the strongly seasonal effects of freeze-thaw-summer, using the same seasonal deflection data as were used for the Road Test correction but applying it through Equation 8.22 - this resulted in a factor of 0.908 being applied to the structural number to obtain the relevant weighted average for the model.

The analytical approach was to predict the roughness increment for individual time-periods within a study and for each section by applying the model to the independent variables (condition, pavement and traffic parameters), and then to evaluate the predicted increments against the observed roughness increments. Since all the studies evidenced high variability in the roughness time-series data on each section, like that in the Brazil study, the roughness trends were smoothed within each section so as to reduce the within-section variances, and obtain a common basis on which to compare the results across all studies.

The identification of erroneous or bad data in an exercise such as this is difficult and several instances were suspected. It was not possible to examine each case in detail however, so the only data eliminated from the analysis were those in which major maintenance was deduced from large negative roughness increments (more than 1 m/km IRI loss after within-section averaging). The variance of roughness data in relation to the trend was particularly high in the Texas data where the rate of roughness progression was generally small (0.06 m/km IRI per year) and maintenance activities had often been carried out; thus the residual variance after averaging within the sections still remained high. In the first Kenya (1984) data set, the roughness trends were disturbed by spuriously high values soon after surface treatment reseals had been applied, followed by reversion toward the original roughness levels after a couple of years' bedding-down. A few extremely thin pavements in the Arizona and Kenya studies yielded outlying results, where it was thought that insufficient allowance may have been made for the in situ subgrade strength, especially after many years of consolidation.

The predictive accuracy was evaluated through the prediction error, the mean prediction bias, and correlation statistics. The mean prediction bias measures the accuracy of the predictions at an aggregate level, being defined as the ratio of the predicted and observed mean roughness increments, with a value of 1 being perfect, and a value of less than one representing underprediction. The prediction error is a measure (in roughness units) of the accuracy of individual predictions, being the equivalent of a standard error of the absolute differences between predicted and observed values, including both the bias and the variance; it should be evaluated in relation to the mean increment. The correlation coefficient and bias represent how well the predictions correlated with the observations, distinguishing between different cases and indicating whether there was any
tendency for larger increments to be predicted differently from smaller increments. Both these correlation statistics may appear to be poor when the range of observed increments is small relative to the measurement errors.

Component incremental model on time-series data

The results of the analysis on all six time-series databases, together with the original Brazil analysis for comparison, are presented in Table 8.8. For the upper half of the table no adjustments were made to the prediction by Equation 8.11, but for the lower half, the environmental coefficient $m$ was adjusted to represent the climate relevant to the data base. In all cases, except for Texas where that part of the analysis was not completed, the values for $m$ were taken from the analyses reported for Table 8.6 and Figure 8.14. The scattergrams comparing the observed and predicted roughness increments for all six cases are shown in Figure 8.16. In most cases similar scales have been used for ease of comparison but the range is largest for the AASHO Road Test.

Comparing firstly the results before environmental adjustment, we see that there was a fairly substantial range of prediction bias, ranging from over-prediction of the rate of roughness progression in Kenya and Texas by factors of 1.6 to 2.0, to underpredictions in Arizona, Illinois and Colorado by factors of 0.8 to 0.45. These relate very obviously to the climatic differences, the first group being generally dry, non-freezing climates and the second dry to wet freezing climates. The prediction errors were similar in most cases, being in the order of 60 to 120 percent of the observed increment, except in the case of Texas (230 percent) where the mean observed increment was only 0.2 m/km IRI over 3 to 4 years.

With the environmental (i.e., nontraffic) adjustments, the prediction bias is almost entirely eliminated, as seen in the lower part of Table 8.8 and in Figure 8.16. The bias correction values, ranging from 0.83 to 1.03, imply that the mean predictions are accurate to within just a few percent of the mean observed roughness increments. The prediction errors have also been considerably reduced, to levels which range from about 26 to 95 percent of the observed increments and are thus comparable to that of the source estimation of the model itself (0.5 m/km IRI in four years, or 78 percent). Recalling that at least one-half of that original error was attributable to measurement error (Section 8.4.2), we deduce that the prediction error for the model itself is only in the order of 40 percent of the true roughness increment. This is remarkably accurate across such diverse conditions, and an extremely satisfying result.

Although the scatter of individual observations about the line of equality appears substantial (and in the same order as in the Brazilian empirical base), the correlation coefficients are generally high, ranging from 0.5 to 0.9. Only in the Texan case was the correlation poor, due mainly to the large number of apparently negative increments, and seemingly this was a data problem, rather than a prediction problem. The high correlations imply that the model is explaining the case differences very well, distinguishing between deterioration processes that are predominantly related to structural factors, those that are predominantly related to surface distress and environment, and those that are mixtures of all factors. The contributions of the five main components of the model were particularly robust in this respect, because the prediction residuals were seen to be independent of any of the components and of any of the major parameters (pavement strength, traffic loading and age), with only minor exceptions as noted below.
Table 8.8: Validation of roughness progression prediction model on independent data from studies in Arizona, Kenya, Texas, Illinois and Colorado

<table>
<thead>
<tr>
<th>Roughness parameter (m/km IRI)</th>
<th>Empirical data source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample and period</td>
<td>389 sections x one 4-year period.</td>
</tr>
<tr>
<td>Observed mean</td>
<td>2.67 (1.1, 8.9)</td>
</tr>
<tr>
<td>Average annual rate</td>
<td>0.16</td>
</tr>
<tr>
<td>Observed increment</td>
<td>0.65 (-0.5, 7.4)</td>
</tr>
<tr>
<td>Before adjustment of environmental coefficient m</td>
<td></td>
</tr>
<tr>
<td>Predicted increment</td>
<td>0.64 (0.1, 5.3)</td>
</tr>
<tr>
<td>Residual</td>
<td>-0.01 (-2.5, 1.8)</td>
</tr>
<tr>
<td>Prediction bias 3/</td>
<td>0.69</td>
</tr>
<tr>
<td>Prediction error 4/</td>
<td>0.51</td>
</tr>
<tr>
<td>Correlation coefficient, r (bias)</td>
<td>0.77 (1.0)</td>
</tr>
<tr>
<td>After adjustment of environmental coefficient m</td>
<td></td>
</tr>
<tr>
<td>Coefficient m</td>
<td>-</td>
</tr>
<tr>
<td>Predicted increment</td>
<td>-</td>
</tr>
<tr>
<td>Residual</td>
<td>-</td>
</tr>
<tr>
<td>Prediction bias 3/</td>
<td>-</td>
</tr>
<tr>
<td>Prediction error 4/</td>
<td>-</td>
</tr>
<tr>
<td>Determination r², (bias)</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: All roughness units are m/km IRI. Statistics are net of within-section variations. Range given in ( ).
1/ 37 sections having ARI < -1.0, and no reported maintenance, were excluded. Values are within-section means of mixed time periods of up to 7 years and averaging 3 years.
2/ Main-factorial study only; data analysed in mixed time segments.
3/ Bias = prediction/observation slope determined by linear regression, and standard error in parentheses.
4/ Approximate equivalent for 4-year period given in ( ) where appropriate.
Source: Author's computations on data files from respective studies.
Figure 8.16: Validation of incremental roughness predictive model on independent data sets: comparison of observed and predicted increments

(a) Kenya Costs Study (1984)

- Observed Increment vs Predicted Increment plot
- Prediction Bias: 1.01
- Error: 0.44 IRI
- $m = 0.007$

(b) Kenya Network Sample

- Observed Increment vs Predicted Increment plot
- Prediction Bias: 0.99
- Error: 0.26 IRI
- $m = 0.007$

(c) Arizona Network Sample

- Observed Increment vs Predicted Increment plot
- Prediction Bias: 0.83
- Error: 0.54 IRI
- $m = 0.012$

(d) Texas Network Sample

- Observed Increment vs Predicted Increment plot
- Prediction Bias: 1.67
- Error: 0.43 IRI
- $m = 0.023$

(Figure continues on next page)
In most cases, there was no bias apparent in the prediction residuals. Only in the cases of the Kenya 1984 Study and the AASHO Road Test was it found that large roughness increments were generally tending to be underpredicted, and small increments to be overpredicted, by the model. When the prediction residuals were studied in each case to determine whether any particular component or parameter in the model was at fault, no strong discrepancies were evident. Generally there was no residual correlation with modified structural number, traffic loading, rut depth variation or cracking, which meant that structural and surface distress mechanisms were soundly represented in the model. On the AASHO data (where the increments were three times larger than elsewhere), residual correlation indicated that cracking and patching had stronger effects than predicted by the model, which is possibly due to the freezing environment and to acceleration of the trafficking and deterioration rates. On the Kenya 1984 Study data, pavements after resealing had lower increments than predicted and some badly-cracked semi-rigid pavements had higher increments than predicted. On the Arizona data, there was slight residual correlation with structural effects for very thin pavements (modified structural number less than 1.5). In the case of the Kenya network sample, some adjustment for patch protrusion \((H_p = 15 \text{ mm})\) had already been included based on available field data. The only consistent residual correlation in most cases was with the roughness increment itself, and sometimes with the mean roughness level. The conclusion is therefore that, while the model may be underestimating the convexity (curvature) of the roughness progression trend over time by up to about 35 percent, the average predictions of roughness increment are extremely accurate, within the bounds of reasonable error.

This is a remarkable finding. It means that the prediction model given by Equation 8.13, together with the environmental coefficients of Table 8.7, provides valid predictions of roughness progression over a very wide range of
Table 8.9: Comparing the validity of several roughness prediction models on databases from Brazil, Arizona and Kenya

<table>
<thead>
<tr>
<th>Roughness prediction model</th>
<th>Brazil (m/km IRI)</th>
<th>Arizona (m/km IRI)</th>
<th>Kenya (m/km IRI)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Component incremental model (HDM-III)</td>
<td>0.51</td>
<td>0.56</td>
<td>0.60</td>
</tr>
<tr>
<td>Arizona DOT</td>
<td>2.20</td>
<td>0.72</td>
<td>0.82</td>
</tr>
<tr>
<td>RTIM2</td>
<td>0.95</td>
<td>1.35</td>
<td>1.15</td>
</tr>
<tr>
<td>Texas TTI</td>
<td>2.90</td>
<td></td>
<td>2.8</td>
</tr>
</tbody>
</table>

1/ Equation 8.13.
2/ Way and Eisenberg (1980), see Appendix A.
3/ Parsley and Robinson (1982), see Appendix A.
4/ Lytton and others (1982), see Appendix A.

Source: Author.

Pavement conditions, pavement strengths, traffic loadings, climates, and maintenance treatments. Moreover, the model is capable of making the predictions for widely different regimes, with an accuracy which is as good as can be achieved within any one regime alone.

Other models on time-series data

The predictive capability of the component incremental model can be compared with that of other predictive models when applied to a similar diverse range of independent data sets. Table 8.9 shows the results of a small study applying the Arizona DOT, RTIM2 and Texas TTI models (detailed in Appendix A) to the Brazil, Arizona and Kenya study data sets, using the same approach as above. In every case, the component incremental model predicts the roughness progression very much better than any other model. This included even the correlation models developed specifically from the same data bases; for example, the models from the Arizona and Kenya studies had prediction errors more than 30 percent worse than given by the component incremental model on their respective data sets. From the evaluation discussed in Section 8.7.1, it is apparent that the AASHTO pavement performance model would also have inferior predictive capability, on at least the Brazil data set.

Validation for extremes by back-analysis

In order to assess the model's validity at extremes, a combination of historical back-analysis from current condition and time-series analyses were used. Table 8.10 summarizes the results of analyses on the Tunisian data mentioned earlier, and South African data comprising eleven strong, lightly-trafficked pavements and five old, weak pavements. The mean predictions fitted the data very well. Also, the prediction errors of individual observations were generally small, not exceeding 0.53 m/km IRI nor exceeding 50 percent of the observed increment, which are similar in magnitude to those on the original Brazilian base. Figure 8.17 shows the strong correlation obtained for the absolute roughness levels on the strong and weak pavements, covering a range of 2.4 to 5.8 modified structural number and ages up to 40 years.
Table 8.10: Validation of component incremental model on Tunisian and South African network sample data: average values

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Tunisia</th>
<th>Strong pavements</th>
<th>Weak pavements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prediction error</td>
<td>IRI</td>
<td>0.53</td>
<td>0.06</td>
<td>0.47</td>
</tr>
<tr>
<td>Bias correction</td>
<td></td>
<td>0.85</td>
<td>0.92</td>
<td>1.18</td>
</tr>
<tr>
<td>Observed increment</td>
<td>IRI</td>
<td>0.98</td>
<td>0.12</td>
<td>0.86</td>
</tr>
<tr>
<td>Predicted increment</td>
<td>IRI</td>
<td>0.78</td>
<td>0.13</td>
<td>0.73</td>
</tr>
<tr>
<td>Observed roughness</td>
<td>IRI</td>
<td>4.0</td>
<td>2.53</td>
<td>4.85</td>
</tr>
<tr>
<td>Pavement age</td>
<td>yr</td>
<td>14.5</td>
<td>12.8</td>
<td>40</td>
</tr>
<tr>
<td>Modified structural</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>number</td>
<td>yr</td>
<td>2.95</td>
<td>5.55</td>
<td>3.35</td>
</tr>
<tr>
<td>Number of sections</td>
<td></td>
<td>33</td>
<td>11</td>
<td>5</td>
</tr>
<tr>
<td>Period</td>
<td></td>
<td>point</td>
<td>4.1</td>
<td>point</td>
</tr>
</tbody>
</table>

**Sources:** Tunisia - data from Newbery and others (1988); South Africa - in collaboration with A.T. Visser and F. Netterberg, data by courtesy of National Institute for Transport and Road Research, Pretoria.

Figure 8.17: Validation of incremental roughness model on southern African data

8.8 CONCLUSION

The model developed here (Equation 8.13) for predicting roughness progression has a radically different form from traditional performance and pavement design models which attribute roughness changes only to structural factors, and from correlative models which have often been unable to distinguish any causative factors other than age. The model predicts incremental roughness through three groups of components, dealing with structural, surface distress, and environment-age-condition factors respectively. The basic premise that road roughness develops through multiple mechanisms has been clearly demonstrated from field data, and the parameters and coefficients of the model estimated from the Brazil-UNDP study data base were consistent with the premise. The data and the model show that significant deterioration can occur even in the absence of structural weakness.

Roughness progression follows a generally convex trend, with the rate of progression depending initially upon the level of traffic loading relative to the pavement strength and on the environmental coefficient, and then rising more rapidly once surface defects such as cracking, potholing, and patching occur.

Macroclimatic effects, represented through the environmental coefficient m, have been generally well-established through applying the model form and the model itself to independent data sets from widely differing climates. The values range from about half of one percent in arid warm regions to over seven percent in wet freezing regions for the annual rate of roughness progression not related to trafficking effects. Apart from these broad trends, the factors influencing the coefficient's value are ill-defined as yet - refinement of the climatic classes especially in freezing climates and to include high rainfall and hot conditions, the effects of drainage and soil type, and so forth, are expected to be important.

The strong validation of the model across eight major independent data sets from widely differing climates, ranging from arid nonfreezing to wet freezing, is a remarkable finding that demonstrates the fundamental plausability of the model and its transferability. Further study seems necessary on an apparent slight bias in the model, which tends to slightly underestimate the convex curvature of the roughness progression, resulting in overestimation of small changes and underestimation of large roughness changes. The overall prediction error however is very small, amounting to only about 0.5 m/km IRI over a 4 year period in each case study, or less than 50 to 80 percent of the observed increment, which is a factor of 1.4 to 6 times better than any alternative tested.

The aggregate model (Equation 8.20) for predicting the absolute roughness from four major parameters without the need for separate distress components, is a suitable, simple alternative to the component model for general pavement life estimates and taxation studies. It utilizes traffic, strength, age and environmental parameters, but the empirical evidence suggests that the terminal level of surface distress or some surrogate is needed to enhance the predictive accuracy to a level similar to that of the component model.
CHAPTER 9
Relative Damaging Effects

The relative attributions of pavement damage to traffic- and non-traffic associated effects, and the allocation of traffic-associated damage costs amongst vehicle classes, are important issues in the pricing of highway use. The traditional approach of using a fourth-power law for comparing the relative damaging effects of different axle loadings is reviewed here, firstly in the light of controlled experiments, and then from the fresh perspective of in-service conditions, which combine the real world factors of mixed traffic and long-term environmental effects. In the new approach, damage is evaluated in separate distress modes, firstly for extant effects through correlation, and then more comprehensively through the empirical distress models developed in earlier chapters. The findings give the first confirmation of the fourth-power law for roughness under in-service conditions. They also go further in defining other relations for other distress modes and particularly in identifying sizeable environmental effects. The implications for cost allocation and pavement design and maintenance standards are then explored, using a convenient classification of axle loading spectra.

9.1 PROBLEM AND CONCEPTS

Of all issues spurring a renewed interest in research on road deterioration, those concerning the relative damaging power of different axle loads and configurations on one hand, and the relative attribution of damage to traffic and environmental, or traffic-independent, effects on the other hand, are among the keenest. Both issues impinge on the allocation of the costs of road damage, of use-related costs amongst vehicle classes and of non use-related costs, which are key considerations in the taxation of road users. On the question of the regulation of axle loadings, and specifically of evaluating economically-optimal limits on axle loading, the issue of the relative damaging power of different axle loads is crucial, particularly because the damage is generally believed to relate to the fourth power of axle loading. Considerable attention was therefore devoted to these aspects during the course of the research, in order to bring as much new information to bear on these issues as possible, and especially to quantify the effects under in-service conditions of mixed traffic and natural environment.

The problem of establishing relative damaging effects arises when confronting the need to represent mixed traffic of different vehicle classes, different axle configurations and loadings, and different tire sizes and pressures, in a composite traffic variable that properly represents the pavement damage associated with loading. The problem arises also when evaluating the damage arising from environmental effects over time, since these range from the continuous effects of oxidation to intermittent effects of rainfall that depend on the levels of distress, maintenance and material susceptibility. Thus, although traffic and time are readily distinguishable as concurrent dimensions which can enter an
empirical relationship either independently or interactively (as illustrated in the empirical models developed earlier), each encompasses a variety of factors that is difficult to consolidate into a single dimension.

9.1.1 Traffic Loading

Traffic loading for example is such a heterogeneous mixture of many factors, varying from road to road, over time with traffic growth and changes in technology, and across countries, that we are compelled to satisfy two extreme demands, namely:

1. To define a practical and relevant summary statistic representing the average effects of traffic loading for use in design and modelling methodologies; and

2. To determine the relative damaging effects arising from individual axle loadings, axle configurations, tire sizes, types and pressures, and from the dynamic effects associated with vehicle speed and individual suspension-types, in such a way that the impact on the marginal costs of road damage and the optimum vehicle design and loading regulations can be evaluated for each factor.

Apart from early concerns on the relative effects of pneumatic and steel tires, most approaches have concentrated on either axle or wheel loadings, including for example the separate design curves for different nominal axle loads in the CBR pavement design procedure, the representation of all axle loadings in an equivalent number of standard 80 kN (18,000 lbf) single axle loads (ESA) arising from the AASHO Road Test, and the Equivalent Single Wheel Load (ESWL) concept developed by the U.S. Corps of Engineers for heavy duty pavements.

The common approach now for road pavements is to reduce mixed traffic loadings to the single unit of equivalent standard axle loadings (ESA), which is the number of passages of a standard axle load that cause the same amount of damage as the mixed traffic. The formal methodology was derived from the AASHO Road Test and is described later in Section 9.3, but for the present we set out the simple concept which is the basis used in this chapter, and which we will see is suitable for most practical purposes. The number of passages (Nₙ) of the standard axle load (Pₛ) which cause the same damage as a number of passages (Nₚ) of any given load (P) is represented in the simplest form by:

\[
\frac{Nₙ}{Nₚ} = \left(\frac{P}{Pₛ}\right)^n = f_{n'}
\]  

where the value of n is approximately four, and the standard axle load Pₛ is usually adopted as 80 kN (18,000 lbf) for dual-tired single axles (and other values for different axle types), based on the AASHO Road Test. The number of equivalent standard axle loads (ESA) causing the same damage as one passage of load P is thus given approximately by the fourth power of the load P relative to the standard load Pₛ, and is often called an equivalent axle load factor, shown by fₙ in the equation. The coefficient n is the relative load damage power. The adoption of the 80kN single axle as a standard reference has historical ties to North American practice since the 1960s, and approximated the legal limits prevailing at that time. Although higher loadings of the order of 100 to 130 kN are now prevalent especially in Europe and some developing countries, the retention of the 80 kN standard makes sense at present because of the wealth of
empirical design methodologies based on it and the simplicity of mathematical transformations for any given load.

9.1.2 Environment

While time (and thus age) is a universal dimension, the environmental factors which influence non-traffic-associated damage are not. And although the effects of individual factors such as rainfall, temperature, oxidation, materials and freezing can be identified conceptually, and to a very limited extent theoretically, the overall impact is so complicated by the interactions and combined effects that empirical distinctions are impracticable and regional averaging is inevitable. Rauhut and others (1984) in the US FHWA cost allocation study reduced the effects to four zonal combinations of dry/wet and freeze/nonfreeze conditions. Ambient temperatures have been best characterized for theoretical studies by the weighted mean annual average temperature (w-MAAT) by Claassen and others (1977). In the present study, the non-traffic-associated effects have been quantified in coefficients, and these are being related to environmental factors through comparative studies with independent data bases from other climates. The approach of environmental classification appears to be the most practical approach since it captures the major effects and makes empirical validation feasible.

9.1.3 Damage

The definition of damage is crucial to any quantification of relative damaging effects. Prior to 1960, damage was usually conceived in relation to what engineers perceived as "failure" of the pavement, and the failure condition was usually somewhat loosely defined to mean the need for rehabilitation or reconstruction. A major contribution of the AASHO Road Test was to quantify damage through, firstly, quantifying the subjective rating of pavement condition (made by a panel of people experienced in highway engineering and use) in the Present Serviceability Rating (PSR), and secondly, by correlating the PSR to physical measures of distress in what was defined as the Present Serviceability Index (PSI). The Serviceability Index was thus an explicit combination of various distress types, namely roughness, rut depth, cracking and patching (given in Appendix A, Equation A.8), and the terminal Serviceability Index (p_c) defined the lowest tolerable level of service "before resurfacing or reconstruction becomes necessary" (AASHTO 1981, p6). Damage was then defined by the fractional loss of serviceability between the new and terminal conditions.

The relative damaging power of different axle loadings was then evaluated for a specified level of damage by this definition, as elaborated first by Liddle (1962). As can be seen from Figure 9.1, the relative damaging power could vary with the level of damage considered because the ratios of the traffic intercepts depend on the relative curvatures of the serviceability trends.

Current needs however require a more refined, less aggregate approach to the definition of damage. It is recognized for example that different maintenance and rehabilitation alternatives, and thus different cost responsibilities, are triggered by different types of distress: the need for resealing is related to the amounts of cracking, ravelling or potholes, the need for smoothing overlay to the roughness, and the needs for rehabilitation or reconstruction are related to various combinations of all distress types. A single summary index such as PSI cannot distinguish between different combinations of distress to meet these various trigger criteria. Thus the definition of damage needs to be freed from...
Figure 9.1: Illustration of AASHTO definition of damage for formulation of relative load damage function

![Illustration of AASHTO definition of damage for formulation of relative load damage function](image)

Note: Relative load damage power for $P_1$, $P_2$, $P_3$ computed from $N_1$, $N_2$, $N_3$

Source: Author's schematic

the fixed combination of distress defined by the AASHO serviceability index formula to allow the terminal condition to be defined by different levels of each distress type. Further it is recognized from mechanistic principles that the relative influences of tire pressure, wheel load and wheel/axle configurations on distress vary according to the type of distress and the depth in the pavement from which the distress originates. Finally, in order to conduct an economic, rather than engineering, evaluation of standards of road condition and maintenance intervention, it is preferable to know the marginal damaging effects of individual load applications and of time (aging), without these necessarily being tied to a specific terminal condition (since the "terminal condition" becomes an economic, rather than an engineering, choice). The difference between the marginal and total damage approaches is evident from Figure 9.1, and Equation 9.1 would become:

$$f_n = \left(\frac{P}{P_s}\right)^n = \left(\frac{\Delta N_s}{\Delta N_p}\right)_g$$

(9.2)

for marginal damage at damage level $g$.

Based both on the findings from the present empirical research and also on findings reported from other theoretical and semi-empirical research, there appears to be a case for evaluating time and traffic effects on a case-by-case basis, and for classifying the load damaging effects by distress mode, since some modes such as disintegration seem independent of axle loading and others such as deformation seem highly dependent upon the loading levels. As noted earlier, the costs of repair tend to vary with the mode of distress. Thus the basic hypothesis which will be evaluated in this chapter is that the damaging effects of load and
time, and thus the damage costs, can be evaluated in four classes of distress mode namely:

1. **Superficial distress**, including ravelling, stone polishing, bleeding, spalling of cracks and possibly pothole development, which depend on traffic but are essentially independent of wheel loadings and may depend on tire pressures and sizes and speed;

2. **Shallow-seated distress**, including fatigue-cracking of thin surfacings, shoving and rutting within all bituminous surfacings, deformation of a (granular) base under thin surfacings and so on, which are influenced mainly by tire pressures and configurations and only slightly by wheel loadings (probably with a relative load damage power between nil and four);

3. **Structural deformation**, including fatigue-cracking of thick structural layers (bituminous or cementitious, of more than 80 - 100mm thickness), and deformation in deep base and foundation layers, which are generally understood to depend on the fourth, or greater, power of relative loading levels; and

4. **Non-traffic-associated distress**, including cold-temperature cracking, map-cracking due to ageing, linear cracking due to edge-dessication or subsidence, and so on, which occur independently of traffic (although traffic may later induce secondary effects such as crack-spalling).

### 9.2 LOADING CHARACTERISTICS OF MIXED TRAFFIC

Before proceeding to evaluate the evidence on the relative damaging power of axle loadings, it is useful first to consider the axle load spectra observed in mixed traffic, and to evaluate the importance of the damage relativities. In order to do this, we adopt the simple definition of relative load damage given in Equation 9.1 and consider the numerical effect of varying the relative load damaging power $n$ on the equivalent axle representation of mixed traffic. The Brazil data base provides a diverse range of axle load spectra observed on the study sections with two-lane road volumes ranging from 400 to 6,000 veh/day and the percentage of heavy or commercial vehicles ranging from 12 to 73 percent. The influence of the power $n$ on the equivalent axle loading computed from these diverse spectra is shown in summary form in Figure 9.2. The equivalent axle loadings are normalized with respect to the equivalent standard axle loadings having an $n$-value of 4, and the mean, 5th and 95th percentiles of 232 different load spectra are shown. The figure shows rather clearly that there was generally very little change in the numerical value of the total number of equivalent standard axles of the whole traffic stream over a range of $n$-values from 2 to 6, particularly for the mean load spectrum. Reference to the specific values given in Table 9.1 show that the differences were typically less than 20 percent and that a minimum tended to occur at an $n$-value of about 3 (in the range 2 to 4 in specific instances). Significantly large differences in the total number of equivalent axles appear only with $n$-values of less than 2, i.e., values which attribute relatively less damage to heavy vehicles. Thus, while the relative damage power $n$ has an enormous impact on the relative attribution of damage amongst vehicles, its impact on the total volume of equivalent loadings is fairly weak except at extreme values.
Figure 9.2: Normalized influence of relative load damage power $n$ on equivalent axle loading representation of mixed traffic for the range of axle load spectra observed in Brazil.

We can use the examples of load spectrum sensitivities shown in Figure 9.2, to define major types of spectrum by their influence curves, as shown in Figure 9.3. Specific examples of the load spectrum taken from the Brazil study are given for each type in Figure 9.4. Spectra of Type A tend toward the upper limits (95th percentile of Figure 9.2) at low $n$-values and typically tend toward the lower limits (5th percentile of Figure 9.2) at $n$-values above 4. Heavy loadings with high proportions of axle loadings above 80 kN are represented by spectra of Type C which tend toward the lower limit (5th percentile) at low $n$-values and higher limit (95th percentile) at high $n$-values.

Load spectrum Type B is similar to the "average" spectrum observed in the study. Two extreme types of spectrum, labelled AA and D, each typical of less than 2 percent of observations in Brazil, are also shown. In those countries in which single axle loading levels of above 80 kN are prevalent (particularly those having a dual-tired single axle load limit above 100 kN, see footnote to Table 9.1), the axle load spectra are likely to be of Types C and D when most heavy vehicles are laden.
### Table 9.1: Characteristics of the equivalent axle loading presentation of mixed traffic for the axle load spectra of heavy vehicles in the Brazil-UNDP study

<table>
<thead>
<tr>
<th>Relative load damage power ( n )</th>
<th>Equivalent axle loading</th>
<th>Normalized equivalent axle loading ( (Y_{E_n}; Y_{E_4}) )</th>
<th>Annual equivalent axle load flow (million axles/lane/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum ( P_{05} ) 1/</td>
<td>( P_{25} ) 1/</td>
<td>Mean ( P_{50} ) 1/</td>
</tr>
<tr>
<td>0</td>
<td>1.121</td>
<td>1.735</td>
<td>2.795</td>
</tr>
<tr>
<td>1</td>
<td>0.423</td>
<td>0.707</td>
<td>0.967</td>
</tr>
<tr>
<td>2</td>
<td>0.433</td>
<td>0.626</td>
<td>0.766</td>
</tr>
<tr>
<td>3</td>
<td>0.626</td>
<td>0.743</td>
<td>0.828</td>
</tr>
<tr>
<td>4</td>
<td>1.</td>
<td>1.</td>
<td>1.</td>
</tr>
<tr>
<td>5</td>
<td>0.405</td>
<td>1.044</td>
<td>1.136</td>
</tr>
<tr>
<td>6</td>
<td>0.175</td>
<td>1.153</td>
<td>1.354</td>
</tr>
</tbody>
</table>

- Annual equivalent axle load flow for damage power \( n \) \( (Y_{E_n}) \) divided by \( (Y_{E_4}) \).

**Note:** Prevailing axle load limits in Brazil were: front axle 5,000 kg, single rear axle 10,000 kg, tandem rear axle 17,000 kg, triple rear axle 25,500 kg.

**Source:** Author's analysis of paved road traffic data from Brazil-UNDP study.
These general trends and the hope that the influence lines could be characterized by one or two parameters, instead of discrete values of YE_n or manipulation of the many cell frequencies, prompted a pilot investigation to parameterize the load distributions. Two methods were tried, employing beta and gamma distributions of cell frequency data respectively, with some degree of success. A gamma distribution has also been applied by Saccomanno and Abd El Halim (1983). However, it was found that strongly bimodal distributions of loading, such as occur with fully laden and fully empty heavy vehicles, were difficult to parameterize satisfactorily, and the pilot study was not pursued further.

9.3 EVIDENCE FROM CONTROLLED LOADING EXPERIMENTS

9.3.1 AASHO Road Test

Quantification of the relationship between axle loading and pavement damage was one of the primary objectives of the AASHO Road Test conducted in Illinois between 1956 and 1961. Since the approximately fourth-power of load relationship that resulted has been widely applied subsequently both in pavement design methodologies and increasingly in road use taxation policies, some review is appropriate. The design of the experiment met the objective by trafficking 10 lanes on 5 test loops separately by ten different axle loads ranging from 9 kN (2,000 lbf) on single-tire single axles to 213 kN (48,000 lbf) on tandem axles, at
Figure 9.4: Examples of five major types of axle load spectra

(a) Type 'AA' Spectrum

<table>
<thead>
<tr>
<th>FREQUENCY (%)</th>
<th>FREQUENCY (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

SINGLE AXLE LOAD (kN)

- 12% heavy vehicles
- Axle loads > 80 kN
- Frequency 1.2%
- Mean 93 kN

Note: Section 164 Direction CS

(b) Type 'A' Spectrum

<table>
<thead>
<tr>
<th>FREQUENCY (%)</th>
<th>FREQUENCY (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

SINGLE AXLE LOAD (kN)

- 25.1% heavy vehicles
- Axle loads > 80 kN
- Frequency 3.1%
- Mean 97 kN

Note: Section 010 Direction SC

(c) Type 'B' Spectrum

<table>
<thead>
<tr>
<th>FREQUENCY (%)</th>
<th>FREQUENCY (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

SINGLE AXLE LOAD (kN)

- 24% heavy vehicles
- Axle loads > 80 kN
- Frequency 17.2%
- Mean 98 kN

Note: Section 163 Direction SC

(d) Type 'C' Spectrum

<table>
<thead>
<tr>
<th>FREQUENCY (%)</th>
<th>FREQUENCY (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

SINGLE AXLE LOAD (kN)

- 29.6% heavy vehicles
- Axle loads > 80 kN
- Frequency 34.2%
- Mean 109 kN

Note: Section 031 Direction SC

(e) Type 'D' Spectrum

<table>
<thead>
<tr>
<th>FREQUENCY (%)</th>
<th>FREQUENCY (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

SINGLE AXLE LOAD (kN)

- 35% heavy vehicles
- Axle loads > 80 kN
- Frequency 35.6%
- Mean 128 kN

Note: Section 159 Direction SC

Notes:

a. Only heavy vehicles (gross vehicle mass > 3,500 kg) are included in load distribution.
b. 15 kN load frequency does not include light vehicle classes.c. Tandem and triple axle loads are included as pro rata single axle loads.

Source: Axle load survey data in Working Document No. 12 of Brazil study (GEIPOT, 1982).
a rate of approximately one vehicle per minute over a period of 2 years (Nov. 1958
to Nov. 1960) reaching a total of 1,114,000 axle applications on each test
section. A sixth loop carried no traffic. The test was thus one of accelerated,
controlled, loading. Although such a loading rate is not considered "accelerated"
by today's standards for major primary roads, the time scale of deterioration from
new to "terminal" condition within two years, for many pavement sections, was
certainly accelerated.

The form of the mathematical model used to relate damage, axle load and
configuration, and number of transits, has an important bearing on the results.
For the relative load damaging effect, the damage \( g \) was expressed as a dimen-
sionless or normalized fraction of the change in serviceability index, as follows:

\[
g = \frac{P_i - P}{P_i - P_r} \tag{9.3}
\]

where \( g \) = the fraction of total damage, as at the current present service-
ability, \( p \); \( P_i \) = present serviceability index of pavement condition;
\( P_r \) = initial serviceability at time of construction; and
\( P_r \) = notional terminal serviceability index at which condition the
pavement is deemed to require rehabilitation, and at which \( g = 1 \).

Thus the damage \( g \) progresses from 0 to 1 over the life of the pavement, but the
meaning of \( g \) is dependent upon the initial construction quality (\( P_i \)) and the
rehabilitation intervention level (\( P_r \)). The damage function was estimated in
the form:

\[
g = \left[ \frac{N_{so}}{\rho} \right]^{\beta} \tag{9.4}
\]

where \( N_{so} \) = the number of 18,000 lbf (80 kN) equivalent single axle
loads (ESA);
\( \rho \) = a function of pavement and load variables that equals the number of
80 kN equivalent single axle loads when \( g = 1 \); and
\( \beta \) = a function of pavement and load variables which influences the
curvature of the damage function.

In the analysis presented by Liddle (1962), which was incorporated in the
subsequent AASHTO design code (AASHTO 1981), axle load equivalence factors were
defined for the particular condition \( g = 1 \), i.e., the terminal pavement condition
when \( \rho = N \) in Equation 9.4. The factors thus represent an average rather than
marginal effect, the differences depending on the curvature of the function. The
solution involved definition of a reference or standard load, the dual-tire single
axle load of 80 kN (18,000 lbf), and the factor \( F_{p,i,g} \) represents the number of
equivalent standard axles (\( N_{so} \)) that cause the same damage (\( g \)) as a number (\( N_p \))
of axle transits of a given load \( P \), as follows:

\[
F_{p,i,g} = \left[ \frac{P_i + L_i}{80 + L_s} \right]^{\beta} \left[ 10^{G/\beta}/10^{G/\beta L_i} \right] \tag{9.5}
\]
where \( \text{N}_{p,j,g} \) = number of axle transits of load \( P \) causing damage \( g \);
\( P_i \) = axle load \( P \) on axle type \( i \), in kN;
\( \beta_p \) = function's value for load \( P \) viz.,
\[ = 0.40 + [0.081 (P_i + L_1)^{3.23} / (SN + 1)^{5.19} L_1^{3.23}] \]

\( L_1 \) = axle configuration code (\( L = 1 \) for single axles and \( L = 2 \) for tandem axles when the load \( P \) is in thousand lbf units; or \( L = 4.44 \) and 8.89 respectively when \( P \) is in kN units as here);

\( G \) = log,\( g \)

Values for the factor \( F \) were generally tabulated (AASHTO 1981), but as the effective \( n \)-values varied only in the range of 3.8 to 4.5, with an average of 4.2, over the range of pavement strengths and terminal pavement conditions, it soon came to be known for convenience as a "fourth-power law", the "law" being mathematical rather than physical.

There has been general consensus that the magnitude of the relative damaging effect predicted by this relationship is correct (Yoder and Witzak 1975, and Lister 1981). However the form and details can be criticized in several ways. The scaling of the axle configuration parameter appears more arbitrary than rational. The form dictates that more early deterioration occurs under light axle loads than under heavy ones, and also that thick pavements are initially more vulnerable than thin ones (Lister 1981). Alternative model forms applied to the same data by Shook and Finn (1962), Painter (1962), and Kondner and Krizek (1966) have yielded large differences as shown in Table 9.2. The alternative studies all indicated greater damaging power for light axle loads than shown by Liddle's equation by factors ranging from 7 to 60, but much closer agreement for 133 kN axle loads, with one exception (Lister 1981).

There is also a fundamental error involved when applying the relationship to mixed traffic loading, because the application would be valid only if \( \beta_p \) were to be independent of load, which it is not (Scrivner and Duzan 1962). The point is amplified by Rauhut and others (1984) for two light, mixed traffic streams, one with more heavy vehicles than the other (load spectra of Types A and AA); the differences are large for a light pavement of structural number 2, the mixed traffic approach predicting approximately double the equivalencies given by Equation 9.5, but the differences are negligible for thick pavements of structural number 7. Addis and Whitmarsh (1981), applying an incremental analysis for mixed traffic effects to the AASHO data obtained the effective values of relative load damaging power shown in Table 9.3 for four different load spectra from light to very heavy loading (Types AA, A, B and C). The relative load damage power can be seen to vary least with wheel load and loading spectrum for thick pavements with structural number exceeding 5, but to vary more widely for thinner pavements, showing higher equivalency factors both for light wheel loads (effective powers less than 4), in agreement with Rauhut and others (1984), and also for heavy wheel loads (effective powers greater than 4, ranging up to 6.6). These and other
Table 9.2: Comparison of axle load equivalency results from analyses of AASHO Road Test data using alternative model forms

<table>
<thead>
<tr>
<th>Author</th>
<th>Repetitions of a 40 kN wheel load equivalent to one wheel load of</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>66.5 kN</td>
</tr>
<tr>
<td>Liddle</td>
<td>7.0</td>
</tr>
<tr>
<td>Shook and Finn</td>
<td>28.0</td>
</tr>
<tr>
<td>Painter</td>
<td>9.3</td>
</tr>
<tr>
<td>Kondner and Krizek</td>
<td>5.0</td>
</tr>
</tbody>
</table>


studies summarized by Yoder and Witzak (1975) indicate a large range of divergence at light wheel and axle loadings (less than 50 percent of the standard load) and a reasonable agreement for heavier wheel loadings, with a relative load damage power in the range of 2 to 6, averaging 4.

9.3.2 Simulated Trafficking Experiments

Experimental studies have also been conducted with the simulation of trafficking by applying controlled loading through a moving full-size wheel to full-size pavements. Some studies have used circular test tracks with specially-constructed pavements, and some are using a mobile loading frame that can apply loadings to in-service pavements. In each case regular repeated passages of the test wheel load are made on the same section of pavement, and acceleration of the damage is achieved by testing at relatively high loadings so as to achieve the failure criterion within a practical period of time (usually less than 4 months). This approach is therefore one step further from real mixed traffic conditions than the Road Test, but has the advantages of flexibility, control and lower resource requirements.

Rut depth progression and relative load damage effects in thick asphalt pavements were studied at the British TRRL circular test track (Lister 1981). Applying trafficking at a series of pavement temperature levels up to 50°C, with separate test pavements for various wheel loadings, a relative load damage power of 6 to 6.4 was observed (see also Equation 7.6). Other test track studies have indicated that tire contact pressures strongly influence the deformation of thick asphalt layers (e.g., Paterson 1972) so that pressure as well as load influences the relative damaging power.

Perhaps the most important empirical data from controlled load tests on full-size pavements since the AASHO Road Test has come from the Heavy Vehicle Simulator (HVS) testing program in South Africa. The Simulator, comprising a 23 m-long mobile loading frame, was capable of applying dual or single wheel loads of 20 to 100 kN in linear trafficking at a rate of 1200 transits per hour distributed over a test strip area 8 x 1.5 m, which was usually part of an in-service pave
Table 9.3: Effective values of relative load damage power derived from AASHO Road Test data by an incremental analysis for mixed traffic of four load spectra

<table>
<thead>
<tr>
<th>Structural number</th>
<th>Wheel-load spectrum 1/</th>
<th>20</th>
<th>40</th>
<th>50</th>
<th>65</th>
<th>80</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.9</td>
<td>very heavy</td>
<td>3.9</td>
<td>3.9</td>
<td>3.2</td>
<td>3.0</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>heavy</td>
<td>3.9</td>
<td>3.9</td>
<td>3.2</td>
<td>3.0</td>
<td>4.1</td>
</tr>
<tr>
<td>5.18</td>
<td>very heavy</td>
<td>3.8</td>
<td>3.8</td>
<td>3.3</td>
<td>3.5</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>heavy</td>
<td>3.8</td>
<td>3.8</td>
<td>3.3</td>
<td>3.5</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>medium</td>
<td>3.8</td>
<td>3.8</td>
<td>3.3</td>
<td>3.5</td>
<td>3.9</td>
</tr>
<tr>
<td>3.54</td>
<td>heavy</td>
<td>2.4</td>
<td>2.6</td>
<td>3.8</td>
<td>4.2</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>medium</td>
<td>2.8</td>
<td>3.1</td>
<td>4.6</td>
<td>4.7</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td>light</td>
<td>3.3</td>
<td>4.0</td>
<td>4.6</td>
<td>5.6</td>
<td>-</td>
</tr>
<tr>
<td>2.38</td>
<td>heavy</td>
<td>3.0</td>
<td>5.4</td>
<td>6.0</td>
<td>5.8</td>
<td>6.6</td>
</tr>
<tr>
<td></td>
<td>medium</td>
<td>3.3</td>
<td>5.7</td>
<td>6.0</td>
<td>5.8</td>
<td>6.6</td>
</tr>
<tr>
<td></td>
<td>light</td>
<td>4.2</td>
<td>6.0</td>
<td>6.0</td>
<td>5.8</td>
<td>-</td>
</tr>
</tbody>
</table>

1/ It is estimated that these classifications are best represented by axle load spectrum types as follows: Light = AA; Medium = A; Heavy = B; Very Heavy = C.


The relative load damage powers shown in Table 9.4 have been computed to differing terminal condition criteria, as noted in column 5, and for varying combinations of wheel load (col. 6), in the range of 40 to 100 kN. Rut depth was the visual damage criterion (roughness could not be measured meaningfully over such a short length), and cracking initiation was an alternative criterion in some tests. Rut depth progression was found to increase approximately linearly on a log-log scale and so, after the initial research phase when different loads were applied on separate strips, different loadings were applied in sequential stages on the same test strip, sometimes with the addition of water or rain to create saturated conditions, and relative damaging effects were computed by comparing the logarithmic rates of deformation in each stage (Maree 1982). Later analyses reverted to natural scales to characterize the loading phases.
Table 9.4: Summary of relative load damage powers for rut depth and cracking derived from Heavy Vehicle Simulator accelerated trafficking on in-service pavements

<table>
<thead>
<tr>
<th>Base/subbase type</th>
<th>Road category</th>
<th>Pavement state</th>
<th>Layer type and state</th>
<th>Criterion and method</th>
<th>Wheel load¹/2 compared (kN)</th>
<th>Relative load damage power ²/</th>
<th>Reference ³/</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed stone/</td>
<td>A Dry</td>
<td>-</td>
<td>Rate of deformation</td>
<td>40*; 55*; 65*; 75*</td>
<td>4.5</td>
<td>Van Vuren (3)</td>
<td></td>
</tr>
<tr>
<td>cemented</td>
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<td>-</td>
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<td>40*; 55*; 60*; 65*;</td>
<td>6.0; 6.7</td>
<td>Paterson (4)</td>
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<td>and 10 mm deformation</td>
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<tr>
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<td>-</td>
<td>Repeated to 20 mm</td>
<td>40; 60; 80; 100; 120</td>
<td>&lt;1</td>
<td>Maree (6)</td>
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<td></td>
<td>deformation</td>
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<tr>
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<td>C Wet</td>
<td>-</td>
<td>Repeated to 20 mm</td>
<td>40; 60; 80; 100; 120</td>
<td>&lt;1</td>
<td>Maree (6)</td>
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<td>-</td>
<td>Rate of deformation</td>
<td>70; 100; 120; 140</td>
<td>4.7</td>
<td>Maree (2)</td>
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<td>Base, dry</td>
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<td>Base, dry</td>
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<tr>
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<td>B Wet</td>
<td>Base, wet</td>
<td>Rate of deformation</td>
<td>40; 70; 100; 120</td>
<td>0</td>
<td>Maree (2)</td>
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<td>crusher run</td>
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<tr>
<td>Cemented/</td>
<td>A Dry</td>
<td>Base, uncracked</td>
<td>Rate of deformation</td>
<td>40*; 60*; 80*; 100*</td>
<td>6.5</td>
<td>Paterson (4)</td>
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<tr>
<td>Cemented/</td>
<td>B Cracked dry, uncracked</td>
<td>-</td>
<td>Rate of deformation</td>
<td>40; 70; 100; 120</td>
<td>1.0</td>
<td>Opperman (11)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Cemented/</td>
<td>C Cracked dry</td>
<td>-</td>
<td>Rate of deformation</td>
<td>40; 70; 100; 120</td>
<td>3.3</td>
<td>Opperman (12)</td>
<td></td>
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<td>gravel</td>
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</tr>
<tr>
<td>Cemented/</td>
<td>A Base and subbase cracked</td>
<td>-</td>
<td>Rate of deformation</td>
<td>40; 70; 100; 120</td>
<td>3.0</td>
<td>Opperman (12)</td>
<td></td>
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<td>cemented</td>
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<td></td>
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<tr>
<td>Recycled</td>
<td>A Base brittle, subbase</td>
<td>Base uncracked</td>
<td>Crack initiation;</td>
<td>40; 70; 100; 120</td>
<td>1.0</td>
<td>De Beer (8)</td>
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<td>asphalt/</td>
<td></td>
<td></td>
<td>mechanistic analysis</td>
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<tr>
<td>Asphalt/</td>
<td>A New structure</td>
<td>-</td>
<td>Rate of deformation</td>
<td>80; 100; 120; 140</td>
<td>1.4</td>
<td>Opperman (14)</td>
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</tbody>
</table>

¹/ Dual-wheel load, except * denotes single-wheel load.

²/ \( n = \log \left(\frac{N_L}{N_p}\right) \) where \( N_p \) = repetitions of wheel load \( P \) kN to the terminal condition defined in column 5. Where information is given on a pavement layer, the relative load damage power applies to the layer and not to the whole pavement.

³/ References in table are found in source document.

The early research showed that the relative load damage power for rut depth was between 6 and 6.7 for very strong semi-rigid inverted pavements (granular base, cemented subbase) (Paterson 1978) and generally of the order of 4.5 (ranging from 3.3 to 8.5) for granular pavements (Maree 1982); the higher values applied in general to the performance prior to cracking or to low values of rut depth such as 5 to 10 mm. The application of water changed the behavior of the granular-base pavements significantly, greatly increasing the rate of deformation (Maree and others 1982), as illustrated in Figure 7.1. The relative load damage power shown in the table for these cases is generally less than 1 or near zero, but this does not mean that the absolute damaging effect was negligible. It implies for example that the different loads caused similar rates of deformation, but at that high level of loading (from 40 to 100 kN dual wheel load, i.e., 80 to 200 kN axle load) and low shear strength due to the saturated conditions, the deformation rate was high in both cases; under lighter axle loadings, which induce stresses less than the saturated shear strength, some positive relative damaging effect would again be evident. The effect of saturation is thus to increase the absolute damaging effect enormously, while the relative damaging power of different wheel loads at high loadings reduces rapidly to zero. The research has also shown clearly the different sources of deformation, some from shear within the base and some from the subbase and subgrade, and also stress-dependent behavior. Maree (1982) and van Zyl and Freeme (1984) concluded from the data that typical average values of the relative load damage power for deformation of granular pavements ranged from 3 in the dry state to 2 in the wet state for crushed stone, and from 2 to 1 respectively for natural gravels; this is represented graphically in Figure 9.5(a). Those values are very low, and the conclusions seem clearly biased to the extremely heavy wheel loading regime applied during the tests, without taking explicit account of the influence that stress-dependent behavior would have on the findings at lower loading levels.

A preferable construct on Maree's results is shown in Figure 9.5(b) in which the number of repetitions ($N_f$) to a defined "critical" rut depth is depicted against the half-axle load $P$. The curves represent a range of effective shear strengths of the granular material in the pavement, and the slope of the curves on a logarithmic plot would be the effective relative load damage power, $n$. At low shear strength, as would result from saturation or very weak gravels, high loads all cause virtually immediate shear to the deformation limit with an effective $n$-value of near zero, while light loads, inducing lower stresses, would generate a more normal behavior with an $n$-value near 4. At the other extreme, for high quality crushed stone in a very stiff, low deflection pavement (such as the inverted designs), high loads and high induced stresses would be sustained by the high effective shear strength of the material and thus produce normal deformation behavior, while moderate or light loads would induce such low shear stresses that the material would be within its elastic range and sustain very little or no plastic deformation, a situation which would generate the high $n$-values of 6 to 8 that were observed. In point of fact, the rate of deformation even under high loadings was often so slow on unsaturated, strong pavements that the critical rut depth could not be reached within a reasonable period, so that $N_f$ could not be measured. In the extreme, these $n$-values conceivably may tend to infinity for very light loads or very strong pavements, meaning that vehicles would cause negligible deformation in the pavement, and for this situation an equivalency factor becomes meaningless. Thus, while an $n$-value of about 4 is probably quite meaningful for traffic loadings which are matched to pavement and material shear strengths, considerable distortions must occur at the extremes of heavy and light
Figure 9.5: Effect of saturation on relative load damaging power according to accelerated trafficking tests by Heavy Vehicle Simulator

(a) HVS Experimental Data

![Graph showing effect of saturation on relative load damaging power with data for high quality crushed stone and natural or partly crushed gravels.](image)

Source: After Maree (1982), Table 10.7

(b) Interpretation of Experimental Results in Relation to Ranges of Mixed Traffic and AASHO Road Test

![Graph showing load repetitions to critical rut depth vs. half-axle load with viable range for HVS, ALF and range of AASHO Road Test.](image)

Source: Author's schematic

Note: Strength of pavement—A. dry, strong; B. partially saturated, strong; C. saturated, strong; D. dry, weak.
loadings. This construct also appears to be more consistent with the extensive mechanistic analyses that accompanied the granular pavement studies (Maree 1982).

For cracking distress, the results show generally low relative load damage powers in the range from less than 1 up to 2.5 for granular pavements, but increasing with load above extreme wheel loadings of 80 kN; this is consistent with the strain-controlled behavior expected of thin-surfacing granular flexible pavements and confirms the hypothesis postulated earlier. For cracking in cemented pavements, brittle behavior is evident with high n-values of 4.5 to 13, which is also consistent with the hypothesis; following cracking initiation, however, the results indicate that the behavior of the cemented layers reverts towards that of granular materials as it breaks down.

The data from the simulated-trafficking research is clearly valuable and in many respects confirms the damage hypotheses. It is unfortunate however that the loadings could not extend further down into the normal working range in order to make the interpretation of the results more generally applicable. The situation highlights a problem of this kind of research: damage under light loadings on thick well-constructed pavements develops very slowly and requires prohibitively long testing periods, so it may be studied only through either extensive extrapolation or using very thin or weak pavements; on the other hand, acceleration of the trafficking by increasing the test loading to extremes renders some desired interpretations either invalid or infeasible. Similar equipment is now being used in Australia (Accelerated Loading Facility, ALF), and has begun to be used in the United States, so much more information will become available in the future.

9.4 THEORETICAL MECHANISTIC STUDIES

Theoretical studies of load equivalencies based on mechanistic principles of material and structural behavior fall in three categories with varying reliance on theoretical structural analysis, and experimental measurements of pavement response and material behavior.

Many studies have concentrated on extending the AASHTO load equivalences to multiple axle configurations and wheel configurations including wide tires (e.g., Southgate and others 1978, Deacon 1969, Treybig 1983, Terrel and Rimsritong 1976). Using linear elastic multilayer analytical models to predict strains in the pavement and applying laboratory-determined material behavior relationships to predict the numbers of load repetitions to a damage criterion, the equivalencies tend to vary depending on whether asphalt tensile strain or subgrade compressive strain was used as the criterion for equating damage.

Other studies have made experimental measurements of the pavement responses considered indicative of performance, including asphalt tensile strain, subgrade compressive strain and stresses, and deflections, instead of estimating the responses theoretically. Axle load equivalencies have been based on applying the measured influence of load, configuration and tire pressure on these values through laboratory material behavior models usually to a single failure criterion such as fatigue cracking, and general agreement with the AASHO function has been shown (e.g., Christison and Shields 1978).

Structural analyses which have been calibrated to observed pavement performance and which use multiple criteria for damage, i.e., deformation in all layers, fatigue in all bound layers and cracking propagation models, simulate the
real performance evaluation most closely. Adapting the equivalent single wheel load concept, Paterson (1979) showed that the load equivalency relationship varied markedly with pavement type and wheel load, as shown in Figure 9.6(a). The relative load damage power was found to depend upon the critical mode of distress, and yielded average n-values of 2.5 to 4 when deformation dominated, and of 0.6 to 1.3 when surfacing fatigue cracking dominated.

Tire pressures were shown to influence fatigue cracking in thin surfacings more than wheel loads by Freeme and Marais (1972). The relative damaging effect of tire pressure also seems to approximate a value of 4 as shown in Figure 9.6(b) (Paterson 1979), and several research studies are currently in progress on this topic (e.g., Roberts and Rosson 1985). High tire pressures in the range of 800 to 1200 kPa approach the unconfined strength of some granular base materials, including low quality stabilized bases, and the impact on surface treatments could be severe.

The most comprehensive mechanistic analysis was undertaken as part of the recent U.S.A. Federal Highway Administration Cost Allocation Study (Rauhut and others 1984). Using pavement study sections in four major climatic zones for "calibration", the factors influencing axle load equivalencies on flexible pavements were studied using the comprehensive viscoelastic pavement analytical model, VESYS III-B. The results, simplified to an average relative damage power n defined as earlier, are presented in Figure 9.7. In all four diagrams there is a clear distinction between the power values for surfacing distress (which was fatigue cracking) with an n-value of about 1.5, rut depth with an n-value of about 4.5, and serviceability index with an n-value of about 6. The values for surfacing distress and rut depth are strong confirmation for the hypothesis. The value for serviceability index however seems very high and this may reflect weaknesses in the underlying relationships in the VESYS model. The powers for cracking and rut depth are seen to be generally independent of axle load and subgrade stiffness (the influence of the subgrade is a new observation that was not feasible at the AASHO Road Test); both however increased as the thickness of asphalt surfacing increased over the range of 75 to 225 mm. The effects of climate shown in Figure 9.7(d), indicate that the rut depth was most sensitive to loadings in freezing climates, notably due to thaw effects, and in wet climates; in general however the VESYS model indicated a negligible effect of climate on the value of n, which is in contrast to the strong marginal effects of moisture noted in the HVS accelerated loading research.

Both the theoretical mechanistic studies and the accelerated loading studies in the previous section indicate quite clearly therefore that fatigue cracking and surfacing distress are sensitive only to the second-power, or less, of load on thin surfacings, and that this power increases to the order of 2.5 to 3 for thick asphalt surfacings of over 100 mm thickness. Subgrade strain and rut depth prediction models generally confirm a relative load damage power of about 4.

9.5 EMPIRICAL EVIDENCE OF LOAD DAMAGING EFFECTS

9.5.1 Identifying Effects under Service Conditions

Several shortcomings can be identified in the various methodologies used to relate axle loading with pavement damage, due chiefly to the ways in which the test conditions or the theoretical modelling differ from real road conditions. A pavement in service is subject to mixed traffic loading of light and heavy
Figure 9.6: Variation of equivalent standard axle load factor with wheel load and tire pressure on various pavements types, as derived by mechanistic analysis with full performance modelling and multiple failure criteria.

(a) Influence of Single Wheel Load

(b) Influence of Tire Contact Pressure

Note: a) Method - nonlinear elastic analysis with multiple failure criteria
b) Standard Axle Load. 80kN. 520kPa.

<table>
<thead>
<tr>
<th>Pavement</th>
<th>Failure Mode</th>
<th>Life [MESA]</th>
<th>Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cracking</td>
<td>17.0</td>
<td>Crushed stone</td>
</tr>
<tr>
<td>2</td>
<td>Deformation</td>
<td>0.5</td>
<td>base</td>
</tr>
<tr>
<td>3</td>
<td>Cracking</td>
<td>20.0</td>
<td>cemented subbase</td>
</tr>
<tr>
<td>4</td>
<td>Cracking</td>
<td>0.3</td>
<td>Bituminous base</td>
</tr>
<tr>
<td>5</td>
<td>Cracking</td>
<td>5.0</td>
<td>and surfacing</td>
</tr>
<tr>
<td>6</td>
<td>Deformation</td>
<td>0.2</td>
<td>Thin surfacing, granular base</td>
</tr>
</tbody>
</table>

Source: Paterson (1979): Figures 11 and 14, Table 4.
Figure 9.7: Sensitivity of relative load damage power to loading, pavement characteristics and climate: results of theoretical mechanistic analysis using the viscoelastic pavement model, VESYS III-B

(a) Effect of Axle Load

(b) Effect of Subgrade Stiffness

(c) Effect of Asphalt Thickness

(d) Effect of Climatic Zone

Source: Data from Rauhut and others (1984).
vehicles, laden and unladen, with tires of different sizes and contact pressures, arriving at varying time-intervals under a range of temperature and moisture conditions over the lifetime of the pavement, during which the material properties of the pavement, and the surfacing in particular, change. Real road conditions therefore comprise a shifting spectrum of pavement properties and a complex spectrum of loading, loading periods and rest periods, which, in the above methodologies, must be dealt with either by averaging or simulation. Inevitably questions arise about the validity of the shift factors and simulation methods that are involved in such a transformation.

It is important therefore to assess the validity of load-damage functions empirically from performance data. This has generally not been possible in the past, except in minor respects, because of the extensive data required for what must be a cross-sectional analysis of different combinations of pavement strength and traffic loading. The data base from the Brazil-UNDP road costs study however has provided this opportunity because it includes comprehensive data on the traffic, axle loading spectrum, pavement strength, age, and pavement condition history over the duration of the study period, for a wide range of service conditions. The Brazil data base thus provides a major independent source for the quantification of the relative damaging effects of different axle loadings, which moreover permits their evaluation under the real conditions of mixed traffic and natural environment, and on a variety of pavements which included thin-surfacing, and thick surfacing, construction. While the Brazil study clearly ranks in importance with the AASHO Road Test on this issue of load damaging effects, there are a few important distinctions between the two studies.

First, the evaluation of relative load damaging effects was not a major objective in the experimental design of the Brazil study as it had been in the AASHO Road Test. Indeed in any road network designed to uniform standards it is usually very difficult to find sample pavements either underdesigned or overdesigned with respect to traffic, as is required to determine the cross-sectional interaction between traffic loading and pavement strength; this of course is a natural result of engineering design practice. Nevertheless a good range of traffic loading within each pavement strength cell was achieved in the design of the Brazil study, and when this was supplemented by sometimes strong differences in the traffic loading between directions of travel on a given section, this range was further increased. This is shown in Table 9.5 which gives the cross-sectional distributions and ranges of traffic loadings (in terms of annual equivalent 80 kN standard axle loads) for various classes of pavement strength (in terms of the modified structural number) available in the study sample. In the table the traffic units are orders of magnitude of equivalent axles per lane per year, so it can be seen that there is generally a span of at least two, and up to five, orders of magnitude of traffic loading in each unit of pavement strength. These are considerable ranges, even though most of the sample is concentrated on the diagonal that represents design practice.

Second, the traffic loadings in the Brazil study come from mixed traffic of all vehicle classes and all degrees of loading, characterized by axle load spectra that differ for nearly every section-lane combination. Thus it is not possible from the data either to compare directly the performances of a given pavement structure under specific axle loads, such as say, 80 kN single axle loads and 120 kN single axle loads, or to evaluate, say, the specific damaging effect of tandem axle loads, as was possible from the separate lanes and loops in the AASHO Road Test.
Table 9.5: Cross-sectional distribution of pavement strength and annual traffic loading data in the Brazil-UNDP road costs study

<table>
<thead>
<tr>
<th>Pavement modified structural number, SNC</th>
<th>Annual traffic loading (ESA/lane/year)</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$10^2$</td>
<td>$10^3$</td>
</tr>
<tr>
<td>2</td>
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<td>6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Sample totals 2 7 30 87 122 92 34 4 378
Sample % 0.5 1.9 7.9 23.0 32.3 24.3 9.0 1.1 100

- No sections in the sample cell.

Note: The sample unit is a pavement subsection. Parameter values are midpoint values for each cell.
Source: Author's analysis of Brazil-UNDP study data.

Third, the pavement structures in the Brazil study vary considerably in age, materials, layer thicknesses, subgrade support and surfacing type, even within a given structural number category. By contrast, the AASHO Road Test, in the main factorial flexible pavement experiment that was used to develop the axle load damage relationships, comprised only new pavements of a single type (flexible with asphalt concrete surfacing), three standard materials (asphalt concrete surfacing, crushed stone base and gravel subbase), three thicknesses for each layer, and only one uniform weak subgrade. Thus, any imprecision in the structural modelling of the highly diverse range of pavement types (new, old, rehabilitated, semi-rigid), materials, thicknesses and subgrade support in the Brazil study, adds further to the difficulty of quantifying relative damaging effects.

Finally, all the pavements in the AASHO Road Test were essentially young and of the same age, and subject to a single climate with seasonal extremes of freeze, thaw and heat, whereas the Brazil pavements covered a range of ages with a much less variable and less harsh climate (subhumid, non freezing, summer rainfall). Thus the relative damaging effects, including interactive effects, of age, environment and traffic, are inherently different between the two studies.

These points reinforce the value and importance of evaluating load equivalency effects in the Brazil data base, but also the potential difficulties. Three approaches were adopted during the analysis as follows:

1. Evaluating alternative load equivalency effects by correlation comparing the incidence and level of each mode of distress between lanes and traffic and loadings within purportedly homogeneous sections;
2. Inferring the load equivalency effects by optimizing each empirical model of distress with respect to the traffic loading parameters; and

3. Characterizing the load spectrum of mixed traffic on each section by a small number of parameters so as to permit a direct statistical estimation of $n$ in the empirical models of distress.

The correlation approach yielded a strong basis for proceeding to the higher levels of analysis as shown below. The load parameterization approach however proved infeasible, as noted in Section 9.2, and so the second approach, the model optimization method, was further developed and formed the final basis for analysis.

9.5.2 Evaluating Effects by Correlation

The first angle from which to consider the relation between damage and axle loading is to eliminate all explanatory variables other than traffic loading by comparing the conditions of a pavement in adjacent lanes, in other words an analysis of within-section effects. With all structural variables being essentially constant (the sections being essentially homogeneous) one can test the hypotheses that age, number of vehicles (or axles), or number of equivalent axle loads have caused the observed damage. In compiling the data files it was assumed, based on good evidence in Brazil, that the average daily traffic flow on the two-lane roads was split equally in number of vehicles in the two directions, and thus only the loading, which was measured separately in each direction, varied across lanes within a given section. If the condition is worse, and the loading is greater, in one lane than in the other, then it can be deduced that loading is probably a dominant cause of the damage. If the condition is the same in both lanes and the loading is different, then one deduces that loading may not be a primary cause and that either time or number of vehicles is the dominant causative factor. When both the condition and the loading are the same in both lanes, none of the hypotheses can be tested. To embrace all these cases, the cross-sectional analysis described in the next section is necessary.

The simple correlations for the initiation of cracking and raveling, in lanes common to each section and for separate surface and pavement types, are shown in Figures 9.8 and 9.9, and the detailed correlation coefficients obtained using alternative parameters for comparison (time and equivalent axle loadings with relative load damage powers of 0, 2, 4, and 6) are given in Table 9.6. In the left-hand diagrams of each figure, the surfacing age at which distress appeared in one lane (direction CS) is compared with the age at which distress appeared in the other lane (direction SC). In the right-hand diagrams, the cumulative loadings (in ESA, using the common relative load damage power of 4) up to the times at which distress appeared in each lane are compared for the two lanes. Only the observed initiation events are presented, the prior and future events not being relevant here.

It is very clear from the figures that the highest correlation in each case is given by pavement age, or time, which emphasizes the rather dominant influence had by weathering on when surface distress first appeared in the study sample. For cracking initiation in semi-rigid cemented base pavements, and for raveling initiation in surface treatments, this tendency was very strong. It was less clear however for the cracking of flexible pavement surfacings, particularly
Figure 9.8: Relative load damaging effects in cracking initiation of asphalt concrete and surface treatment surfacings: empirical evidence from correlation within-sections

(a) Original Asphalt Concrete Surfacings

(b) Original Surface Treatment Surfacings

Source: Brazil-UNDP study data (World Bank 1985).
Figure 9.9: Relative load damaging effects in cracking initiation of cemented base pavements and in ravelling initiation of surface treatments: empirical evidence from correlations within-sections

(a) Cracking Initiation in Cemented-base Pavements

(b) Ravelling Initiation in Surface Treatment Surfacings

Source: Brazil-UNDP study data (World Bank 1985).
Table 9.6: Relative damaging effects on the initiation of cracking and ravelling: empirical evidence from within-section correlations of time, and of equivalent axle loads for various damaging powers, from the Brazil study

<table>
<thead>
<tr>
<th>Distress type and pavement type</th>
<th>Bivariate correlation coefficient across lanes for cited parameter</th>
<th>Equivalent axle loads for</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Age</td>
<td>All axles</td>
</tr>
<tr>
<td>Narrow cracking</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>0.96</td>
<td>0.95</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>0.81</td>
<td>0.72</td>
</tr>
<tr>
<td>Cemented base</td>
<td>0.97</td>
<td>0.96</td>
</tr>
<tr>
<td>Ravelling</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surface treatments</td>
<td>0.94</td>
<td>0.92</td>
</tr>
</tbody>
</table>

Note: n is the relative load damage power.
Source: Author's analysis of paved road data files from Brazil-UNDP study.

for surface treatments in which the correlations are essentially indistinguishable from one another. For the flexible pavement surfacings, therefore, traffic loading and weathering have an interactive influence on cracking that cannot be identified by simple bivariate correlations. For semi-rigid pavements, the poor correlation of loading with cracking initiation is counter-intuitive, and is probably due in part at least to the difficulty of distinguishing between load-associated and non-load-associated cracking at the early stages (after excepting shrinkage cracking, which is distinctive).

Rut depth and roughness progression, being progressive rather than discrete distress phenomena, need to be treated differently. The simplest test is to compare the ratio of the mean rut depth or roughness in the two lanes with the ratio of the traffic loadings. This is shown for the Brazil study sections in Figure 9.10 and Table 9.7. The correlations are not nearly as strong as the across-direction correlations for cracking initiation, but they are positive and significantly different from zero. Thus the data do provide empirical confirmation that increased loadings cause increased deformation. This was not the case incidentally when the incremental changes in rut depth and roughness observed during the study were tested; near-zero correlations resulted, as shown in the table, due primarily to the problems of compounded measurement errors and superficial effects (as discussed fully in Chapters 7 and 8). When the correlations were run on a subset of the data which excluded the observations most prone to measurement error, then positive correlations were achieved, as shown at the bottom of the table; the subset comprised only those observations having an average roughness increment greater than 0.3 m/km IRI and a minimum roughness increment greater than 0.17 m/km IRI. This emphasizes again the importance of high accuracy in the roughness data if there is to be any chance of determining significant effects of loading for mixed traffic during a short study period. The positive correlations for the across-lane ratios of mean rut depth and mean roughness indicated optimum
Figure 9.10: Relative load damaging effects in rutting and roughness: empirical evidence from correlations between the ratio of mean distress levels across lanes with the ratio of traffic loadings across lanes

(a) Mean Rut Depths in Each Direction

(b) Mean Roughness in Each Direction

Note: Symbols indicate pavement type, i.e., A = asphalt concrete; S = Surface treatment; C = thin surfacing on cemented base; O = asphalt overlay; R = reseal on surface treatment or asphalt concrete.

Source: Brazil-UNDP study data (World Bank 1985).
Table 9.7: Relative damaging effects in rutting and roughness: empirical evidence from correlations between the across-lane ratios of mean distress and the across-lane ratios of equivalent axle loadings (for various damage power values)

<table>
<thead>
<tr>
<th>Distress parameter expressed as dimensionless ratio across-lane within sections 1/</th>
<th>Bivariate correlation coefficient between distress ratio and traffic ratio for given relative load damage power, n 2/</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rut depth</td>
<td>n = 2</td>
</tr>
<tr>
<td>Mean rut depth (ratio)</td>
<td>0.37</td>
</tr>
<tr>
<td>Roughness</td>
<td></td>
</tr>
<tr>
<td>Mean roughness (ratio)</td>
<td>0.26</td>
</tr>
<tr>
<td>Increment of roughness (ratio)</td>
<td>0.00</td>
</tr>
<tr>
<td>Subset of data: AR &gt; 0.3 m/km IRI</td>
<td></td>
</tr>
<tr>
<td>Increment of roughness (ratio)</td>
<td>0.274</td>
</tr>
</tbody>
</table>

1/ Dimensionless ratios, e.g., RDM(CS): RDM(SC) where CS, SC identify the lane direction.
2/ Dimensionless ratio of incremental traffic observed during study "window", e.g., ANEn(CS): ANEn(SC).
Source: Author's analysis of Brazil-UNDP study data.

n-values in the order of 2, but in the high-discrimination subset of data, the optimum n-value for incremental roughness was in the range of 4 to 6.

The circumstantial evidence on relative load- and non-load-associated damage effects presented above is based on simple correlations and, as such, does not take account of interactive effects nor the correct forms of the traffic and age parameters; it also suffers particularly adverse effects from measurement errors. This is particularly true for roughness and rut depth progression. A definitive result thus needs to be based on an approach which includes all factors and their nonlinear effects, as considered next.

9.5.3 Inference by Optimizing Empirical Models of Distress

In order to take account of the nonlinear and interactive effects of multiple factors, the multivariate approach was based on optimizing the empirical models that had been developed for each distress mode. The primary assumption here was that the load equivalency formulation of the traffic term which resulted in the best statistical fit of each model to the data also represented the average load damaging effect for that particular mode and type of distress. The assumption implies for example that the correct equivalent traffic term is the one which best explains all the data, particularly the variations in distress development between those sections or lanes in which all other explanatory variables are constant (as considered above in the correlation study). A vital condition for the assumption to be valid is that the model being optimized is essentially correct, in the respect that all other explanatory variables and the algebraic
formulation of the model are a correct representation of the underlying causes of distress and their interactions, so that only the form of the traffic variable requires optimization.

The method of analysis involved the computation, for each subsection, of the average ESA per axle \( (EPA_n) \) and the equivalent axle loading flow \( (YE_n) \), in ESA per lane per year, for discrete values of the relative load damaging power \( n \), assuming the simplified form of the axle load equivalency factor given previously in Equation 9.2. For light vehicles (cars and utilities), no axle load data were available so an average load of 500 kg per axle was assumed.

For each distress type, the empirical models estimated from the Brazilian data, as presented in earlier chapters, were re-estimated using different traffic terms with \( n \)-values ranging from 0 to 6 (e.g., \( YE_0 \), \( YE_1 \), etc., in models using annual traffic flow). All other variables in the models were unchanged, although their coefficients were unconstrained. Various regression statistics were used as criteria to evaluate the damage power, namely goodness of model fit, normalized model prediction error, and the t-statistic of the traffic term.

Goodness of fit statistics, such as the coefficient of determination \( (r^2) \) for the deterministic models of rut depth and roughness, or the average log likelihood \( (AL) \) statistics in the case of the probabilistic models of cracking and raveling initiation, are not always the most discriminatory. They are dependent on the range of the dependent variable and may be biased by values at the extremes of the range. Furthermore the values of average log likelihood cannot be compared across distress types because they depend on the range and scaling of the dependent variable. The preferred statistic is the prediction error, presented here in normalized form as the coefficient of variation \( (CV) \) (standard error/mean of dependent variable, in percent) for deterministic models and the dispersion \( (SIQF, \text{see Section 5.3.3}) \) for probabilistic models. Third, for the distress initiation models, in which the traffic term was an unconstrained explanatory variable, the t-statistic of the traffic parameter estimate indicates how well-determined was the traffic effect. Finally, consistency of the result with prior theory and experience will be a governing criterion, which in this instance implies the theoretical mechanistic findings elaborated earlier.

Cracking

The results for cracking are given in Table 9.8. The overall optimum value of \( n \) is 2 for the initiation and progression of cracking in both flexible pavement types, although the results are not entirely consistent. For asphalt concrete surfacings, an \( n \)-value of 2 is optimum for most measures except that marginally better model fits for initiation are given by an \( n \)-value of 4. The differences between the results for alternative \( n \)-values are small, and thus not highly significant, but it is notable that this approach confirms stronger loading effects than did the within-section correlation approach. For surface treatments, \( n \) lies between 0 and 3 for initiation, and is clearly 2 for progression.

The trends are consistent with the effects of surfacing thickness as indicated by mechanistic theory. In thin surfacings, the tensile strains induced are relatively insensitive to wheel loading (though sensitive to tire pressures), and the fatigue behavior under the strain-controlled mode relates to the fourth-power or less of tensile strain, so that fatigue cracking relates to about the
Table 9.8: Relative load damaging power in cracking and ravelling: evaluation by optimization of empirical models on road data.

<table>
<thead>
<tr>
<th>Distress Type</th>
<th>Pavement Types</th>
<th>Goodness-of-fit statistics for given n-values</th>
<th>Traffic term</th>
<th>Optimum n value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Average log likelihood or $r^2$</td>
<td>SIQF or CV (%)</td>
<td>t-statistic</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0   2   4   6</td>
<td>0   2   4   6</td>
<td>0   2   4   6</td>
</tr>
<tr>
<td>Cracking Initiation 1/</td>
<td>Asphalt concrete</td>
<td>-1.308 -1.288 -1.279 -1.281 33.7 32.6 32.4 34.0</td>
<td>2.05 2.32 2.04 1.81</td>
<td>2, 4</td>
</tr>
<tr>
<td></td>
<td>Surface treatment</td>
<td>-1.277 -1.269 -1.269 - 25.9 26.6 27.7 - 3.00 2.81 2.65 - 0, 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cemented base</td>
<td>-1.051 -1.147 -1.175 -1.183 13.6 15.9 16.2 16.1 3.44 2.36 2.08 2.11 0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cracking Progression 2/</td>
<td>Asphalt concrete</td>
<td>0.477 0.510 0.509 0.492 41.5 28.7 31.6 38.1</td>
<td>- - - -</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Surface treatment</td>
<td>0.657 0.646 0.398 0.320 37.1 25.1 30.2 38.0</td>
<td>- - - -</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Cemented base</td>
<td>0.250 0.314 0.325 0.331 44.6 29.4 32.0 37.3</td>
<td>- - - -</td>
<td>2, 6</td>
</tr>
<tr>
<td>Ravelling Initiation 1/</td>
<td>Surface treatment</td>
<td>-1.212 -1.223 -1.224 -1.222 30.3 31.2 31.3 31.0 2.30 0.71 0.64 0.81 0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ravelling Progression 2/</td>
<td>No traffic effects</td>
<td>0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1/ Analysed using the probabilistic failure-time model method (Appendix B).
2/ Analyzed by linear least-squares regression.

Note: Underlining indicates the optimum value by trial-and-error.
Source: Author's analysis on data from Brazil-UNDP study.

square of the applied load (an n-value of 2). In thick asphalt surfacings, on the other hand, the tensile strains induced are almost linearly related to wheel loadings and the fatigue behavior under stress-controlled mode relates to about the fourth-power of tensile strain, so the resultant behavior corresponds to an n-value approaching 4. This tendency was evident in the theoretical study by Rauhut and others (1984), shown in Figure 9.7(c), and also in the HVS simulated trafficking studies referenced in Table 9.4. While the empirical method was unable to distinguish the effects of surfacing thickness on n, the general result of an n-value of 2 is clearly consistent with mechanistic theory.

For cracking initiation in semi-rigid pavements, with cemented base, the n-value of 0 is contrary to theory which indicates that a higher value of between 4 and 10 might be expected because of the brittle behavior of the cemented layer. All three statistics consistently indicate an n-value as low as zero, which suggests rather strongly that non-load-associated effects (cement shrinkage or aging) may be confusing the issue (as noted previously it is often difficult to discern whether initial irregular cracking is load- or non-load-associated). For cracking progression, higher n-values of 2 to 6 are evident. As the effective sample size was small (11 independent sections) and the ranges of traffic loading were narrow for the cemented base sections (a range factor in the order of 10 to 50 times), the initiation results are considered misleading. Based on the progression results and mechanistic theory, an n-value of 4 appears to be the most appropriate.

Ravelling

For the initiation of ravelling, all three statistics show a clear optimum for an n-value of zero. This implies that every axle transit of either light
or heavy vehicles causes the same amount of "damage", independent of axle load. Ravelling is thus a surface "wear" phenomenon, and falls in the same category as stone polishing and skid resistance. The ravelling or disintegration mechanism is strongly related to the horizontal and suction stresses generated by the contact of the moving tire on the exposed particles of the surfacing, and to binder adhesion and ductility. As those stresses are essentially independent of wheel load, the statistical result is in agreement with mechanistic theory. The data did not permit further refinement to determine whether tire pressure, tire size or light/heavy vehicle classification were significant, but these are likely to be very second-order effects if they are present. For ravelling progression, traffic flow did not enter the model explicitly but, as the time base is constant and because traffic causes the ravelling, the distress should again be allocated to all axle transits, independent of load.

Rut depth

For rut depth progression, the optimum n-value shown in Table 9.9 is 4. This is consistent with the mechanistic principles which relate permanent deformation to imposed stresses, particularly for the deformation within unbound granular materials in flexible, thin surfacing pavements. The mechanism is less clear for pavements with thick asphalt layers (which were sparsely represented in the Brazil study) because the pavement temperature, asphalt stiffness and tire contact pressure exert significant influences that may modify the effect of axle load; for those conditions the British TRRL experimental finding was for an n-value of about 6 and the FHWA Cost Allocation study theoretical finding was 4. Although the optimum appears to be rather weakly determined in the present analysis (values between 3 and 5 are equally significant), this again is due largely to the fact that the average rut depth observed was small compared with the measurement error.

Roughness

For roughness progression, the economically most important mode of distress, a number of approaches were used until a satisfactorily rigorous determination was obtained. Using the model optimization approach on the incremental model, the full data sample yielded an extremely weak discrimination between alternative n-values, as shown in the table, but when the high-discrimination subset was analysed (only those observations with more than 0.3 m/km IRI change over the 4-year period), a much stronger result was achieved, as shown under "subset" in the table, returning a best n-value of 4.

The statistically most rigorous determination was achieved after the development of the aggregate levels model of roughness progression, (Equation 8.20), since this had many fewer coefficients to be estimated and dealt with absolute rather than incremental values of traffic loading and roughness. The hypothesis that 4 was the preferred n-value was tested by permitting the model estimation to choose between 4 and an alternative n-value for the cumulative traffic loading term, NE\textsubscript{N}, as set out in the footnote to Table 9.9. For every value of n between 0 and 6, the hypothesis that n was different from 4 was overwhelming rejected; in each case the coefficient z, representing the fractional combination of the alternative NE\textsubscript{N} value which would improve the model fit, was less than one-sixth and never significantly different from zero.
Table 9.9 Relative load damage power in rutting and roughness progression under mixed traffic: evaluation by optimization of empirical models from road data

<table>
<thead>
<tr>
<th>Distress type and statistical parameter</th>
<th>Relative load damage power</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Rut depth: log-linear model</td>
<td></td>
</tr>
<tr>
<td>Coefficient of determination, (r^2)</td>
<td></td>
</tr>
<tr>
<td>Coefficient of variation, %</td>
<td></td>
</tr>
<tr>
<td>Roughness: incremental model</td>
<td></td>
</tr>
<tr>
<td>Full sample</td>
<td>0.545</td>
</tr>
<tr>
<td>Coefficient of variation, %</td>
<td>81.6</td>
</tr>
<tr>
<td>Subset (AR &gt; 0.4 m/km IRI)</td>
<td>0.527</td>
</tr>
<tr>
<td>Roughness: levels model (^1)</td>
<td></td>
</tr>
<tr>
<td>Preference for alternative (NE_n)</td>
<td></td>
</tr>
<tr>
<td>when compared to (NE_4):</td>
<td></td>
</tr>
<tr>
<td>Coefficient of alternative, (z)</td>
<td>0.075</td>
</tr>
<tr>
<td>Standard error of (z)</td>
<td>0.072</td>
</tr>
<tr>
<td>(t)-statistic of (z)</td>
<td>1.04</td>
</tr>
</tbody>
</table>

\(^{-}\) Not computed. Underscored value represents optimum fit.

\(^1\) The model estimated by non-linear regression was:

\[ R_{It} = [RI_0 + b (1+SNC)^c \left[ (1-z) NE_4 + z NE_n \right]] e^{mt} \]

where \(NE_n\) = cumulative ESA computed with damage power of \(n\) \((0 \leq n \leq 6)\); \(z\) = coefficient representing the fractional contribution of the alternative loading formulation \((NE_n)\) improving the model fit with respect to \(NE_4\) \((0 \leq z \leq 1)\); and all other variables are as defined in Chapter 8. Note that the coefficients \(b, c, m\) and \(z\) were all estimated without constraint.

**Source:** Author's analysis of Brazil-UNDP study data.

The empirical analysis therefore showed very strong evidence that the influence of axle loading on roughness progression, the most important form of damage, was best represented by a relative damage power of 4 when averaged over a wide range of pavement strengths and ages, and a wide range of axle load spectra types in mixed traffic. It thus strongly confirms that the "fourth-power damage law", deriving from the accelerated controlled loading conditions of the AASHO Road Test, is also the most applicable under the real conditions of mixed traffic and natural environment.
RELATIVE DAMAGING EFFECTS

General

Some general comments on this approach of empirical model optimization are appropriate. First, the relative load damaging effect determined by this method for a given distress mode is an average value across all pavements and all axle loadings in the sample, unlike the AASHO Road Test damage function which showed that the damage power varied slightly with the structural number and with the specific axle load. For the planner and policy maker, the averaging is a positive feature because it identifies the collective effects rather than the very disparate specific effects that have been revealed by the theoretical and accelerated-trafficking experimental research.

Second, much of the discriminatory power between different n-values was reduced when applied across all the observed axle load spectra of mixed traffic because many of those spectra were similar to the mean spectrum (type B), which is barely sensitive to the value of n within the range of 2 to 6, as shown in Figure 9.3. Fortunately this handicap was largely compensated by large differentials in loading levels, often in the opposing lanes of one section. The point does help to explain why this empirical approach has not been productive previously.

Third, the addition of light vehicle data into the analysis files, which was possible only late in the analysis phase, caused a significant shift in the influence of the n-value compared with the preliminary analyses. Thus for future empirical studies that might aim to evaluate load damaging effects, the inclusion of light vehicles in the data base by number and class is considered essential, though only a sample need be weighed.

Finally, for an empirical analysis to be feasible, the levels of distress must be sufficiently high in a large part of the sample for adequate discrimination between distress levels to be possible without the effects being swamped by measurement error. Thus a data base should preferably have broad cross-sectional ranges and combinations of traffic loading, axle loading spectrum type, pavement strength and distress level if an empirical evaluation of relative load damaging power is required. These are demanding requirements that are not easily met.

9.6 LOAD- AND NON-LOAD-ASSOCIATED DAMAGE ATPRIBUTION

Aside from the relative damaging effects of different axle loads and configurations, the amount of damage attributable to environmental effects is a major issue in pricing and taxation studies because the costs of such damage may be accounted as a social rather than a user cost. Environmental effects are perhaps most visible in freezing climates where a considerable acceleration of surfacing disintegration, potholing and rutting occurs during thaw periods. In arid climates and hot climates, the effects are usually less dramatic and take the forms of map cracking or ravelling of the surface due to rapid embrittlement of the bituminous binder, of shoving or rutting of thick asphalt layers due to high pavement temperatures and low binder viscosity, and so forth. In non-freezing wet climates, the rate of deformation after cracking is likely to be high. Other forms of environmental damage can also be identified, such as long-wavelength roughness due to moisture movements in expansive clays.

Mostly however the effects result from interactions between traffic and environmental factors, and the strict attribution of damage to one or the other
becomes a complex if not impossible task: for instance, the marginal damage due to a vehicle transit would vary from season to season or from hour to hour. There are also a number of engineering interventions which diminish the environmental damage, for example improving surface and subsurface drainage, eliminating or controlling moisture-susceptible materials within the frost-depth, and modifying the viscosity temperature sensitivity of binders. One of the most specific interventions perhaps is the seasonal control of axle and vehicle load limits during thaw periods in freezing climates, as is implemented in some northern countries. Such interventions counter the environmental impact on deterioration and cause the deterioration trends to tend towards a global mean. The associated costs of the engineering precautions are thus investment rather than marginal costs, and the impact on marginal costs may be considerably suppressed.

Many of these effects are highly regionalized and most of the few studies that have been made have been specific to one effect or one region. In this section discussion is restricted to the general non-catastrophic, time-related effects of aging and weathering in both nonfreezing and freezing climates and their relationship to load-associated damaging effects.

9.6.1 Attribution of Roughness Damage

The primary damage to be considered is roughness, since that occasions the major rehabilitation costs and also all the externality costs of the road user. We utilize the full empirical performance model to generate the long-term trends of roughness for typical pavements, and analyse the damage attribution through the major components in the incremental roughness model. Thus we write the model from Equation 8.13 in the form of the three component groups, namely structural deformation (D), surfacing distress (S), and age-environment (E) as follows:

\[
AR_{It} = AR_{Dt} + AR_{St} + AR_{Et}
\]

(9.6)

where

\[
AR_{Dt} = \text{incremental roughness occurring at pavement age, } t;
\]

\[
AR_{Dt} = 134 \cdot m t \cdot (1 + SNC - 0.000758 \cdot H \cdot CRX_t)^{-5.0} \cdot ANE_{At} + 0.114 \cdot ARDS_t
\]

\[
AR_{St} = \text{incremental roughness due to surfacing distress at age } t,
\]

\[
= 0.0066 \cdot ACRX_t + 0.42 \cdot AAPOT_t; \text{ and}
\]

\[
AR_{Et} = \text{incremental roughness due to age-environment-roughness interaction},
\]

\[
= n \cdot R_t \cdot At, \text{ with } n = 0.023 \text{ for the reference case.}
\]

This component view of damage is illustrated in the roughness trends shown in Figure 8.9, where it can be seen that the relative contributions of the components change over the performance cycle of the pavement. In absolute terms, the environmental component increases steadily at a rate proportional to the roughness levels so that the marginal (say annual) amount has doubled by the time the roughness has doubled. The deformation component begins to increase after cracking develops, and the amount is very dependent on the level of traffic loading relative to the pavement strength. The surface distress component begins only when cracking begins and increases rapidly after potholes develop.
There are a number of ways to illustrate these influences of loading, environment, pavement strength and maintenance standard (or, intervention level) on the attribution of damage. To do so it is convenient to express each damage component as a fraction of the total damage, which, for the cumulative damage since construction or rehabilitation is given by:

\[ f_{Dt} = \sum_{t=0}^{t} \Delta R_{Dt} / \sum_{t=0}^{t} ARI_t \]  

where \( f_{Dt} = \) fraction of total roughness increase \( \Delta R_{Dt} \) over the period from \( t = 0 \) to \( t \) which is attributed to deformation mechanisms, and where \( f_{St} \) for surface distress damage, and \( f_{Et} \) for environmentally-induced roughness are similarly defined in terms of \( ARI_{St} \) and \( ARI_{Et} \) respectively. Thus:

\[ f_{Dt} + f_{St} + f_{Et} = 1 \]  

and

\[ R_t = R_0 + (f_{Dt} + f_{St} + f_{Et}) (R_t - R_0) \]

and the total "damage" is the amount \( (R_t - R_0) \) in roughness units.

Influence of pavement design and maintenance standards

The influences of loading and pavement strength are shown in Figure 9.11 for what would be described in traditional engineering terms as overdesign, normal design and underdesign situations. In the figure this has been represented for a constant pavement strength (modified structural number 3) and three levels of loading differing by factors of five. The impact on the design life, shown by the arrows for alternative intervention criteria of cracking (CC) or roughness (CR), under pothole patching maintenance, is a range of design life before rehabilitation from 23 years to 9 years. The impact on the attribution of cumulative damage to traffic \( f_{Dt} + f_{St} \) at the end of the pavement "life" is to increase the fraction from 56 percent in the overdesign case, to 66 percent in the normal design case, and to 82 percent in the underdesign case. The three figures show clearly the increasing share of damage inflicted by the traffic loading, particularly through the deformation modes of distress, as the loading volume rises (or, by corollary as the pavement design standard drops).

This influence of loading and pavement design standard is generalized in Figure 9.12, which indicates the attribution of cumulative damage at the end of the pavement life for the same pavement, covering a range of traffic loading from 0.02 to 1 million ESA per lane per year. It is shown for two maintenance standards, namely rehabilitation at 3.5 m/km IRI for high maintenance standards and at 5.0 m/km IRI for low maintenance standards. At the lowest traffic volumes the attribution of damage to traffic is slightly less under high maintenance than under low maintenance (48 percent compared with 56 percent), but at the highest volume of 50 times more traffic (the underdesign/overloading case) the attribution is barely affected by the maintenance standard (about 90 percent in each case).

In each of these cases, the fractional attribution of damage to non-traffic-associated effects has decreased while that to traffic increased for higher traffic loadings. This has the interesting implication that the "non-use" share of both the damage and the associated maintenance cost is highest for high design standards. As network-level economic studies (e.g., Bhandari and others 1987) have shown that high design standards are optimal under unconstrained budgets, this further implies that about one half of the maintenance and rehabili-
Figure 9.11: Influence of traffic loading on attribution of roughness damage for constant pavement strength

(a) Overdesign or Light Loading: 0.02 million ESA/year

(b) Normal Design and Loading: 0.10 million ESA/year

(c) Underdesign or Overloading: 0.50 million ESA/year

Notes: Pavement characteristics - Modified Structural Number 3, Asphalt concrete surfacing

Code:
CC = critical cracking 30 percent
CR = critical roughness of 5 m/km IRI (after pothole patching)
E = roughness attributable to age and environmental effects
D = roughness attributable to deformation mechanisms
S = roughness attributable to surfacing distress

Source: Author's computation using Road Deterioration and Maintenance Model of HDM-III.
Figure 9.12: Influence of pavement design and maintenance standards on attribution of cumulative roughness damage

Notes: Pavement: Modified Structural Number 3, asphalt concrete surfacing.

Maintenance standards:
(a) Low. Patch all potholes, overlay at 5.0 m/km IRI roughness.
(b) High. Patch all potholes, overlay at 3.5 m/km IRI roughness.
Damage Component: D=deformation
S=surfacing distress
E=environment and age

Source: Author's computation using Road Deterioration and Maintenance Model of HDM-III.
tation costs for a high standard network should be considered as non-traffic costs to be allocated either as a purely social cost or as a general highway network cost, depending upon the taxation policy. By the same argument, much larger shares of the damage are attributable to traffic on networks of low design and maintenance standards.

It is well to note at this point that we have been discussing the fractional shares of total damage for fixed amounts of damage. The marginal amount of damage, or rate of damage, per ESA or per year does of course also decrease as the pavement design standards are increased. Thus increasing design standards decrease both the marginal amount, and the fractional share of that amount, of damage which is attributable to traffic.

Influence of environment

The cause of the non-use-attributed damage has been imputed to environmental factors, although it has been possible to quantify these effects only in a general way through the coefficient m used in the roughness-age term of the roughness prediction model. While it is understood that the environmental factors will also affect cracking, ravelling, potholing and rutting in individual ways that could influence the damage trends, the effect of environment on "roughness damage" is here considered only through the coefficient m in the roughness prediction model.

The influence of the environment on the damage attribution is seen in Figure 9.13 for a given pavement and traffic, which represents "normal design" (rehabilitation overlay after 15 years) for the model reference case (m = 0.023) of a subhumid, moderate temperature, climate. The trend is again shown for a fixed amount of damage (constant maintenance standards), as for Figure 9.12, so that the "life" before critical roughness is reached (shown in the upper diagram) is longer for low m-values (arid, warm climates) and shorter for high m-values (cold, moist climates). Under the arid, warm climatic conditions, the damage is almost entirely attributable to traffic with only 19 percent being attributable to the environment or non-use. (Note, however, that if we were considering only "surface distress damage" and resurfacing costs instead of roughness and rehabilitation costs, the portion attributable to the environment would probably be higher). At the other extreme, for the more aggressive cold, moist climate, less than 40 percent of damage is attributable to traffic and more than 60 percent is attributable to non-traffic or "environmental" effects.

In practice this apparent influence of environment would be moderated by engineering adaptation; adjustment to give comparable design lives across all environmental conditions would mean slightly thinner pavements in the arid climate, and rather thicker pavements in the aggressive cold climate, than the reference pavement for the given traffic loading. Maintenance standards might also vary.

Surface and structural distress distinctions

Finally, one might consider suballocating the traffic-attributable damage amongst vehicle classes on the basis of different relative load damaging powers for the surfacing distress and deformation components of the traffic-associated damage, following the findings of Section 9.5.
Thus for pavements with generally thin surfacings, the costs of maintenance and resurfacing which result from cracking, ravelling and other forms of surface distress, would be allocated using a relative damage power in the order of 0 to 2. The choice of 0 or 2 would depend on whether the prevalent distress modes were disintegration or cracking, and in most cases it is probably simplest to use an average power of 1, i.e., linearly proportional to axle load. Thus a two-axle 6 ton light truck would pay three times as much as a two-axle 2-ton utility in their shares of resurfacing costs. Rehabilitation and reconstruction costs on the other hand, in which the need for strengthening results mainly from structural distress, would be allocated using a relative load damaging power of 4. In their shares of these costs, the 6-ton truck would pay about 20 times as much as the 2-ton utility in our example.
For pavements with predominantly thick layers of bound materials, particularly bituminous materials in thicknesses greater than 100 mm, the appropriate relative load damage power is in the order of 3 to 5, and the traditional value of 4 would clearly be the sound choice.

9.6.2 Marginal Damage Attribution in a Network Sample

Another interesting view is to consider the average marginal attribution of damage for a network of roads, which typically will have a spectrum of conditions, loadings, ages, and standards at a point in time. This view is particularly pertinent when allocating costs with a view to full cost recovery of annual maintenance and rehabilitation expenditures.

Consider the roads included in the Brazil-UNDP Road Costs Study as an example, although note that in this instance the distribution of roads in the sample was not selected to be representative of the network as a whole. Applying the same approach to normalizing damage attribution as Equation 9.7, but defining the time increment as the 4-year study period, all the study sections were analysed to determine the fractions of the roughness change attributable to each cause in each individual case. The results are expressed by the frequency distributions shown in Figure 9.14.

From the figure, we see that the structural deformation mechanisms accounted for less than one-half of the roughness change in the majority (90 percent) of the study pavements; for 50 percent it accounted for less than two-tenths, and for only 3 percent did it account for more than three-quarters of the change in roughness. For about 10 percent of the study sections the deformation component was moderately substantial amounting to more than 0.1 m/km IRI per year, or 0.4 m/km IRI over the study period. These generally small amounts of damage occurring through deformation are not altogether surprising because they reflect the fact that the pavements had been designed with an engineering code to have adequate structural strength for the traffic loading they were to carry. This is corroborated by the fact that the mean rut depths were generally very small with about 90 percent less than 8 mm. The fact that structural deformation accounted for so little of the observed damage highlights the shortcomings of damage models that attribute all damage to traffic loading, and provides explanation for the various empirical studies that were unable to determine structurally-related effects. It also confirms the inference, drawn in the previous section, that both the fraction of roughness damage attributable to structural deformation, and the rate of roughness progression, will tend to diminish to very small amounts as the construction and structural design standards of a network are improved.

Surfacing distress accounted for widely varying fractions of the roughness change, according to the model. In 45 percent of the study pavements it accounted for less than one-tenth of the roughness change, and in 79 percent it amounted to small roughness changes of less than 0.1 m/km IRI per year. These of course were those pavements with either negligible or moderate incidence of either cracking or patching. Only on those pavements that developed extensive wide-cracking, patching or potholes, was the direct contribution of surfacing distress greater than these amounts, with a maximum contribution of 0.82 m/km IRI and a maximum amount of 3.0 m/km IRI over the 4-year period. The definition of this component in the roughness model is not exclusive because cracking also influences the deformation component through both diminishing the effective structural number and through accelerating the rutting process. Thus it is best not to distinguish
between the deformation and surfacing distress components too rigorously but rather to focus on total traffic-associated damage, which is the sum of the two and the complement of the non-traffic-associated damage.

Finally, the non-traffic-associated effects accounted for between one- and four-fifths of the roughness change in 86 percent of the study subsections, with an average value of 0.48 or slightly under one-half. This is a substantial fraction of the total damage for the bulk of the pavements studied although the magnitudes of roughness change involved were relatively small, 92 percent of the amounts being less than 0.1 m/km IRI per year. In the form that it was estimated, the model indicated that the time-related effect was a constant proportional increase of about 2.3 percent increase in roughness per year, while the validation studies for other climates have so far indicated that a value of from 3 to 7 percent per year would be appropriate for cold and freezing climates and a value of about 1 percent per year would be appropriate for semi-arid warm climates. Thus the marginal contribution of this time-related, traffic-independent component of roughness progression is likely to vary across major environments, in much the same way as indicated for the cumulative damage attribution in Figure 9.13.

9.6.3 Allocation Amongst Vehicle Classes

The preceding sections have indicated that the proportion of damage attributable to traffic is typically in the range of 0.6 to 0.8, with low traffic levels or high maintenance standards tending to give the lower values, but it may drop to as low as 0.2 for pavements in wet, freezing climates.
To consider the allocation of the attributable damage amongst vehicle classes, we return briefly to the axle load spectra typology developed in Section 9.2 for some sample traffic streams so as to evaluate the impact of the relative load damage power on this decision.

The sensitivity of the total number of equivalent axle loadings of mixed traffic to the relative load damage power is usually similar to that of the heavy vehicle classes. However, the sensitivity for an individual vehicle or class of vehicle may be very different from the average and it is that which affects the allocation of costs amongst users. This is illustrated in Table 9.10 which shows the sensitivity of the average equivalent axle loading per vehicle (separately for heavy and light vehicle classes) computed as a function of the relative load damage power $n$. The example is worked for three axle load spectrum types, A, B and C, using loading data from the Brazil study, and one example of a Type C spectrum from Tunisia as a cross-country comparison. The values appearing in column (6) for the Brazil axle load distribution examples come from the influence curves plotted in Figure 9.3. From the table it can be seen that the average ESA per heavy vehicle (column (2)) is almost directly proportional to the average ESA for all vehicles (columns (5), (6)) for $n$-values of 2 and greater. This is not surprising because the heavy vehicles dominate the loading effects (especially when they constitute over 20 percent of the traffic volume), but it also illustrates the useful point that the heavy vehicle load distribution can be used to classify the distribution type.

It is the intention here to examine damage and cost allocation amongst vehicle classes only in outline so as to demonstrate the importance of damage effects. A detailed study is given in Newbery and others (1988). A simple presentation of the percentage of damage allocatable to heavy vehicles for each load distribution type and under each damage power $n$-value is given in column (7), and the ratio of the heavy vehicles and light vehicle allocations is given in column (8). It is immediately apparent from column (7) that heavy vehicles would be responsible for over 90 percent of allocatable damage, regardless of whether the relative load damage power was 2 or 6, and only in the case of the Tunisian example, in which "light vehicles" include vehicles of up to 6,000 kg GVM and less than 20 percent of the volume are heavy vehicles, is the percentage lower than this (80 percent) for an $n$-value of 2. The primary issue of debate over the magnitude of the relative load damage power thus really impinges only on the allocation amongst classes of heavy vehicle of damage effects arising from those modes of distress that have a damage power of 2 or more, i.e., cracking and deformation. It is only in the allocation of surface characteristics damage, such as ravelling and stone polishing, which have a damage power of less than 2, that the costs of damage are allocatable over all vehicles more or less equally, and these costs are usually relatively minor, being the costs of resurfacing maintenance.

9.7 CONCLUSION

The relative damaging effects of different axle loadings are different for each of the major modes of distress. As different modes of distress may trigger maintenance under alternative maintenance policies and as the associated repair costs and repair effects differ in each case, it is appropriate to deal with the modes separately in an evaluation of damage attribution. The popular equivalent standard axle load concept derived from the AASHO Road Test, which defined damage in terms of a fractional loss of the serviceability index (which combined a roughness parameter, rut depth, cracking and patching in fixed propor-
Table 9.10: Examples of average number of equivalent axle loads per vehicle for major load distribution types and varying relative load damage power in Brazil and Tunisia

<table>
<thead>
<tr>
<th>Load damage power n</th>
<th>Average ESA/veh</th>
<th>Incidence</th>
<th>ESA/veh</th>
<th>Spectrum power ratio</th>
<th>Allocation to all heavy vehicles %</th>
<th>Ratio of damage power to all heavy vehicles</th>
<th>Light vehicles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>all heavy</td>
<td>all</td>
<td>all</td>
<td>All</td>
<td>ESA_n / ESA_n</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>vehicles</td>
<td></td>
<td></td>
<td>vehicles</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
<td>(5)</td>
<td>(6)</td>
<td>(7)</td>
</tr>
<tr>
<td>Brazil - Type A 2/</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>2.012</td>
<td>2.000</td>
<td>25.1</td>
<td>2.009</td>
<td>15.46</td>
<td>25.1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>0.722</td>
<td>0.020</td>
<td>25.1</td>
<td>0.196</td>
<td>1.51</td>
<td>92.4</td>
<td>36</td>
</tr>
<tr>
<td>4</td>
<td>0.516</td>
<td>0.0002</td>
<td>25.1</td>
<td>0.130</td>
<td>1.00</td>
<td>99.9</td>
<td>2580</td>
</tr>
<tr>
<td>6</td>
<td>0.609</td>
<td>0.0000</td>
<td>25.1</td>
<td>0.153</td>
<td>1.18</td>
<td>100.</td>
<td>99000</td>
</tr>
<tr>
<td>Brazil - Type B 2/</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>2.191</td>
<td>2.000</td>
<td>24.0</td>
<td>2.046</td>
<td>8.09</td>
<td>25.7</td>
<td>1.1</td>
</tr>
<tr>
<td>2</td>
<td>0.979</td>
<td>0.020</td>
<td>24.0</td>
<td>0.250</td>
<td>0.99</td>
<td>93.9</td>
<td>49</td>
</tr>
<tr>
<td>4</td>
<td>1.053</td>
<td>0.0002</td>
<td>24.0</td>
<td>0.253</td>
<td>1.00</td>
<td>99.9</td>
<td>5260</td>
</tr>
<tr>
<td>6</td>
<td>1.560</td>
<td>0.0000</td>
<td>24.0</td>
<td>0.374</td>
<td>1.48</td>
<td>100.</td>
<td>99000</td>
</tr>
<tr>
<td>Brazil - Type C 2/</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>2.422</td>
<td>2.000</td>
<td>29.6</td>
<td>2.125</td>
<td>1.73</td>
<td>33.8</td>
<td>1.2</td>
</tr>
<tr>
<td>2</td>
<td>2.485</td>
<td>0.020</td>
<td>29.6</td>
<td>0.750</td>
<td>0.61</td>
<td>98.1</td>
<td>124</td>
</tr>
<tr>
<td>4</td>
<td>4.144</td>
<td>0.0002</td>
<td>29.6</td>
<td>1.227</td>
<td>1.00</td>
<td>100.</td>
<td>20700</td>
</tr>
<tr>
<td>6</td>
<td>8.215</td>
<td>0.0000</td>
<td>29.6</td>
<td>2.433</td>
<td>1.98</td>
<td>100.</td>
<td>99000</td>
</tr>
<tr>
<td>Tunisia - (Type C) 3/</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.57</td>
<td>0.076</td>
<td>16.6</td>
<td>0.324</td>
<td>0.86</td>
<td>80.4</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>1.73</td>
<td>0.030</td>
<td>16.6</td>
<td>0.312</td>
<td>0.83</td>
<td>92.0</td>
<td>58</td>
</tr>
<tr>
<td>4</td>
<td>2.19</td>
<td>0.014</td>
<td>16.6</td>
<td>0.375</td>
<td>1.00</td>
<td>96.9</td>
<td>156</td>
</tr>
<tr>
<td>5</td>
<td>3.09</td>
<td>0.007</td>
<td>16.6</td>
<td>0.519</td>
<td>1.38</td>
<td>98.8</td>
<td>441</td>
</tr>
<tr>
<td>6</td>
<td>4.63</td>
<td>0.003</td>
<td>16.6</td>
<td>0.771</td>
<td>2.06</td>
<td>99.7</td>
<td>1540</td>
</tr>
</tbody>
</table>

1/ Light vehicles in Tunisia included those of payload capacity less than 3,500 kg, i.e., gross vehicle mass (GVM) of up to 6,000 kg. In Brazil, light vehicles were defined as those with GVM of less than 3,500 kg.
2/ Load distributions from Figure 9.4.
3/ Based on Table 1.14 in Part III of Newbery and others (1988).
Source: Based on data from Brazil-UNDP study and Newbery and others (1988).
tions), is inflexible for such an application and, without modification, is inappropriate for many types of surfacing distress.

The conclusions drawn are based in part on a review of alternative analyses of the AASHO load-damage relationship, of results from two major controlled-loading experiments, and of results from detailed mechanistic studies using theoretical structural analysis techniques and measured properties of material behavior. While these sources tend to generally confirm the magnitude of the original AASHO relationship of damage to approximately the fourth-power of axle load when the damage is due to structural deformation, the detailed studies point clearly to the large differences (damage power values ranging from 1 to 13) which can occur under certain conditions as a function of axle configuration, tire pressure and size, material and structural properties, drainage conditions, mixed traffic, etc., and to damaging effects for cracking that are typically of the order of the second-power of wheel load. As significant uncertainties exist in translating those experimental and theoretical findings to real conditions, the conclusions here are primarily based on a new major empirical analysis, reported here, of performance data from in-service roads under mixed traffic and normal aging and environmental conditions.

Three categories of load damaging effect have been identified comprising those modes of distress that are represented by relative load damage powers of 0, 2 and 4 respectively, as follows:

1. **Relative load damage power of 0.** Ravelling initiation and progression, and skid resistance (stone polishing in particular), fall into this category clearly and this is consistent with mechanistic principles. Damage in this category is essentially abrasive wear of the surfacing, and being independent of wheel loading would be attributed uniformly across vehicle axles. Refinement of this category to consider the possible effects of tire pressure, size and configuration has not been attempted, but, as the marginal costs of damage involved are very small, any such refinement would have negligible distributional impact on the total cost allocation across vehicles.

2. **Relative load damage power of 2.** The initiation and progression of crocodile cracking, i.e., load-associated cracking, can be placed in this category for thin to medium surfacings and flexible pavement types, with the exception of pavements with thick asphalt layers (100 mm thickness and more) and semirigid pavements. In reality there is considerable variation about the value of 2 for specific surfacings and conditions, due to the important influence of base support, weathering, and so forth, on the relative impacts that tire pressure and size, axle loading, and wheel configuration have on the mechanism of cracking. In particular, little is yet known of the relative load damage power for cracking alongside the wheelpath when this is caused by the wheel punching through the surfacing. If distinction is to be made for pavements with surfacings thicker than 100 mm, or bituminous-base pavements, a relative load damage power of 2.5 to 3 appears to be appropriate.

3. **Relative road damage power of 4.** Cracking in semirigid or thick bituminous pavements, and structural deformation such as roughness
and rutting in all pavement types, are characterized by a relative load damage power of 4. While some controlled loading experiments and some theoretical analyses have indicated that the power may vary widely from 1 to 6 (or occasionally more) under specific conditions, wheel configurations and loadings, the empirical evidence for a wide range of typical pavement and loading conditions strongly confirms that an average value of 4 is appropriate for most analytical purposes. For the same reasons, it appears generally inappropriate to include fine distinctions of pavement strength, maintenance standard, and wheel load levels in the computation of the equivalent standard axle load formula (as in Equation 9.5 for example), and the simple formula in Equation 9.1, utilizing an n-value of 4, is recommended for general use.

The implication of these findings for the allocation of costs is summarized in Table 9.11. Matching the maintenance activity to the triggering distress category, the table indicates the relative load damage power that is recommended for allocating the traffic-attributed share of maintenance costs amongst vehicle classes. In a further simplification, all maintenance and resurfacing costs for thin surface pavements could be allocated by a single n-value of 1 (instead of 0 and 2).

An important conclusion for future research strategy is that refinement of these relative load damage effects for particular axle, tire and pavement configurations is likely to be achieved only through a validated mechanistic analysis or extended controlled-load trafficking studies. Empirical analysis of data from roads in service under mixed traffic can quantify the relative load damaging effects only when two conditions are met, namely first that the measurement error

<table>
<thead>
<tr>
<th>Triggering distress category</th>
<th>Maintenance activities</th>
<th>Relative load damage power</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage, ravelling, polishing, bleeding, and potholing.</td>
<td>Routine maintenance, surface patching, preventive treatments, reseal (slurry or surface treatment).</td>
<td>0</td>
</tr>
<tr>
<td>Crocodile cracking and edge break in flexible pavements.</td>
<td>Reseal (surface treatment), thin overlay, recycling, deep patching.</td>
<td>2</td>
</tr>
<tr>
<td>Rutting and roughness; cracking in semirigid and thick bituminous pavements.</td>
<td>Rehabilitation (thick overlay, strengthening, recycling), reconstruction.</td>
<td>4</td>
</tr>
</tbody>
</table>

Note: When the maintenance activity remedies mixed distress categories, the relative damage power chosen should be an appropriate average.

Source: Author's recommendation.
Table 9.12: Guideline for attribution of roughness to non-traffic factors for various pavement design standards and climatic categories

<table>
<thead>
<tr>
<th>Climate</th>
<th>High-strength RL &gt; 20 yrs</th>
<th>Medium-strength RL = 15 yrs</th>
<th>Low-strength RL &lt; 10 yrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arid, warm</td>
<td>0.3 - 0.5</td>
<td>0.1 - 0.3</td>
<td>0.1 - 0.2</td>
</tr>
<tr>
<td>Subhumid, warm</td>
<td>0.4 - 0.6</td>
<td>0.2 - 0.4</td>
<td>0.1 - 0.2</td>
</tr>
<tr>
<td>Arid, freezing</td>
<td>0.5 - 0.8</td>
<td>0.4 - 0.5</td>
<td>0.2 - 0.4</td>
</tr>
<tr>
<td>Moist, freezing</td>
<td>0.6 - 0.9</td>
<td>0.5 - 0.8</td>
<td>0.4 - 0.6</td>
</tr>
</tbody>
</table>

1/ RL: Life before rehabilitation at roughness ranging from 3.5 to 5.0 m/km IRI. Source: Author's recommendation.

is small relative to the changes in rut depth or roughness being observed, and second that the axle load distributions of the mixed traffic should be of one of the extreme types (i.e. either A, light loading, or C, heavy loading) so as to permit some discrimination between alternative n-values of the damage power.

On the attribution of damage to non-traffic-associated effects, there is an important finding. The empirical analysis demonstrated that a regular annual increase in roughness was attributable to non-traffic-associated effects, which was 2.3 percent in the Brazil study and amounted to between two- and eight-tenths of the marginal roughness damage for 88 percent of the sections, with an average amount of one-half. These proportions are comparatively high because the study pavements were generally well designed and the resulting traffic-associated roughness damage was low. A generalized analysis for a variety of conditions indicated that two- to four-tenths of cumulative damage was attributable to non-traffic effects for normal pavement designs in moderate subhumid climates. Table 9.12 suggests guidelines on the figures to be used in cost allocation exercises for all conditions. There is preliminary evidence suggesting that the proportion rises to the range of four- to nine-tenths for pavements in wet-freezing climates, with intermediate levels for dry-freezing and wet-nonfreezing climates. In general, as pavements are strengthened relative to their traffic loading, and under high maintenance standards, the rate of roughness progression would be reduced and the proportion of damage not directly attributable to traffic would increase to the order of four- or five-tenths for nonfreezing climates.
CHAPTER 10

Summary and Conclusions

The main findings and conclusions of the study are summarized, highlighting particular contributions to knowledge, and evaluating the present status of predictive modeling for road deterioration and maintenance. The progress has been significant. The empirical models derived (and listed here for convenience), have been broadly validated and have wide application in road planning and management models, including and beyond the World Bank's Highway Design and Maintenance standards model (HDM-III) for which they were developed in the first instance. The more general distress models, of roughness in particular, will be of appreciable value in economic studies of costs and taxes in road transportation, and go well beyond previous models through their inclusion of quantified time, environment, and standards effects in addition to the traditional effects of traffic and structural strength. Lastly, it is believed that the models make significant contributions to the structural design of pavements and, along with their primary data base of the Brazil-UNDP-World Bank study, will serve as an important validation source and benchmark for future design methods and road research.

10.1 GENERAL

10.1.1 Requirements for Predictive Models

The application of formal management systems to the maintenance of road networks, and the application of economic criteria to the evaluation of appropriate standards, alternative maintenance policies, and to the pricing of road use, have created demands for reliable, well-quantified and validated means for predicting road deterioration. Various requirements for the predictive models to be relevant for these various applications can be summarized as follows.

First, mathematical models are needed for predicting the trend of road condition over time, and both the short- and long-term effects of maintenance on that trend, so that reasonable estimates can be made of both the likely timing and costs of future maintenance on a road, and of the resultant quality of service of the road with respect to the road user.

Second, the quality of service and the trend of condition must be quantified in terms that relate directly to the factors influencing an engineering decision to intervene with maintenance and to the factors giving rise to economic benefits. Extensive research, of which this study has been a part, has shown that roughness is the road surface characteristic which has the greatest influence on the economic benefits to be derived through maintenance, because of its influence on vehicle operating costs. Surface texture through its influence on surface friction (skid resistance), and pavement edge characteristics, also affect benefits through their influence on road safety and accident costs, but these are often small relative to the savings achievable in operating costs. Network-level evaluations have shown that the roughness standards dictated by economic criteria tend to be more stringent than the standards required to meet users' riding com-
fort requirements (except under severe budget constraints), so that the economic standards may be considered inclusive of other criteria.

Third, for their implementation within a management system to be feasible, predictive models must utilize only parameters which can be measured physically and obtained within the budgetary and human resources of highway agencies. These constraints of physical implementation differ at a national (or regional) network level and a project level, so that differing levels of sophistication in the models may be allowable depending upon the application.

Fourth, for decisions concerning the pricing of road use and the allocation of costs amongst users, predictive models must permit discrimination of the marginal effects of the various primary factors affecting the rate of deterioration, such as vehicle loading configurations, pavement strength, environment, and so forth. At the project level, some of the interactions between the many factors involved can be described by theoretical mechanistic models, but at the network level, where the many factors must be reduced to a small number of summary variables, the complex interactions of concurrent mixed traffic and environmental conditions on pavement behavior require models to be developed by empirical study. For surface disintegration and roughness, in particular, mechanistic modelling is restricted by numerous implicit assumptions, and empirical modelling is essential.

Fifth, to be valid, the predictions must have a well-quantified, empirical base and be demonstrably applicable for the region of interest, inclusive of its traffic, environment, materials and construction method characteristics. Models developed from entirely within one region typically lack a basis for extrapolation to conditions applying in other regions unless the empirical base embraces a wide range of conditions and factors. In the quest for universal models that include the features essential to deterioration prediction and that are readily adaptable to particular local conditions, an empirical data base therefore requires a sound experimental design, a wide range of conditions for the pertinent variables, and physical measures that can be applied universally.

Sixth, the reliability of predictions is dependent on three sources of variation, namely; the inherently stochastic behavior of materials under natural conditions, the inability of parameters in a model to fully represent all factors influencing pavement behavior, and the measurement errors arising from differences between the observed and actual behaviors of pavements. As the real variations of behavior within a nominally homogeneous length of pavement alone may be as great as a factor of three to ten, a concept of reliability or probability needs to be incorporated in pavement decision models, and quantification of the prediction reliability is thus an essential part of the modelling effort.

10.1.2 Study Objectives and Methodology

The primary purpose of the study was to develop predictive models of deterioration for both unpaved and paved roads to be applied in the economic evaluation of maintenance standards and expenditures at a network level, and in studies of pricing and taxation in the road transport sector, including the costs of road usage. The development however also took consideration of the more detailed needs for application at project level and in pavement management systems. The models were to be applicable or transferable to different regions, primarily in nonfreezing climates.
To meet these aims, the approach adopted for the modelling combined advanced empirical methods with mechanistic principles. The methodology was primarily empirical, developing parametric models by statistical regression of time-series data which had been collected in studies comprising a statistically-designed, factorial sample of in-service roads with differing structural and traffic characteristics. However, the form and parameters of the models, were based, where possible, on mechanistic theory and experimental knowledge of structural and material behavior so as to ensure that the relative marginal effects would be estimated appropriately. The forms are generally incremental, predicting the change in condition over an incremental time period as functions of the current condition, and structural, traffic and environmental factors. The empirical approach was necessary because the experimental and theoretical knowledge of pavement performance prior to the 1970s was not applicable to the full range of standards, practices and environments found in developing countries. Also the individual modelling of roughness, surface distress and maintenance effects, and the inclusion of non-traffic-related effects and unpaved roads, were essential for a policy tool.

The Brazil-UNDP road costs study of 1976 to 1982 was the primary data base utilized in this study, on account of the scope of its experimental design and the range of conditions studied. Other empirical studies, conducted in different countries and climates, were used to test the validity of the models developed from the primary data base and to determine the effects of environment and materials across regions. These included two factorial studies in Kenya and one in Ghana, network sample studies in Arizona, Kenya, Tunisia, and Texas, and special road studies in Colorado and Illinois, thus covering climates ranging from arid to moist, and hot nonfreezing to freezing. This validation and across-region phase of the study was important, though limited slightly by the availability of reliable time-series data, and by differences in measures of condition.

10.1.3 Road Roughness

The physical measure of roughness was perhaps the most important focus of this research because of its economic importance. Thus considerable attention was devoted to the interpretation of roughness measurements made in the empirical studies, and in the subsequent international experiment to correlate different road roughness measures (Sayers, Gillespie and Queiroz 1986). Prior to that experiment it was extremely difficult, and in some cases impossible, to compare the different roughness scales used in all the major empirical studies. A basis for conversion between scales (see Figure 2.15) has been established in this study using data from the international experiment, making it possible to compare studies across different countries and methodologies. The retrospective validity of applying these conversions to earlier studies is slightly uncertain, however, because the hardware used as references possibly varied with time, by small or large amounts in different cases, or else cannot now be reproduced accurately. Thus, the definition from that experiment of a profile-based, summary statistic of roughness, the International Roughness Index (IRI), compatible with all response-type and profilometry measuring systems and related to the economic impact of roughness on vehicles, provides a much-needed common standard for the comparison and validation of future deterioration studies worldwide. Other road profile statistics of individual wavebands, measurable only by profilometry equipment, are evolving for special applications, and potentially may provide more information on the causes and effects of roughness in the future, but are not suitable for planning models at this time.
Adoption of the IRI as a common reference, and sound measurement practices (Gillespie, Sayers and Paterson 1986), are thus urged for all agencies measuring roughness in order to facilitate technology transfer worldwide and the broadest possible basis for model development. Only with such exchange and transferability is it likely that the economically important influences of climate, construction techniques, and maintenance, which remain largely open research questions, will be able to be quantified. While many may wish to use a previous system in parallel for historical purposes, the need for a common denominator, the IRI, is urgent.

10.2 UNPAVED ROADS

The models developed predict the rate of roughness progression, the effect of maintenance blading on roughness, and the rate of surface material loss, as summarized in Table 10.1. The roughness progression and blading effect models may be used either independently or simultaneously. In the simultaneous application, the results predicted are the longterm average, low and high levels of roughness associated with a constant policy of blading frequency under time-scheduled, traffic-scheduled, or condition-responsive criteria. These "steady-state" results are summarized in part 3 of Table 10.1.

Alternative, exponential models for roughness progression, may be found in Equations 3.7 and 3.8, and for blading effects in Equation 3.11. Care needs to be taken with the roughness progression predicted by Equation 3.7 since it tends to overestimate the rate under a low frequency of blading.

10.2.1 Roughness and Effects of Maintenance Blading

The roughness on unpaved roads increases primarily as a function of cumulative traffic carried since the most recent maintenance blading, and the Brazil study has indicated that some further increase is attributable to time or non-traffic-associated factors. The most important mechanism causing roughness progression is thus the mechanical attrition of the surface material under traffic, which is usually worst under dry conditions. The presence of moderate rainfall, which maintains a sufficient level of cohesion in the material and promotes compaction, was found to suppress the rate of progression. Over long periods of observation without the application of blading maintenance, the roughness sometimes but not always reached very high levels, indicating that the maximum roughness to be expected was itself a function of road characteristics. This stands in contrast to previous studies which have shown (on often scant evidence) roughness progression accelerating to very high levels above 14 m/km IRI. Roughness above this level is invariably associated with numerous or major depressions and potholes, and material loss, and is aggravated by rainfall which creates standing water that is not shed as runoff.

Blading maintenance to control roughness is usually undertaken on a regular basis to redistribute the surface material and reduce roughness, so commencing a new roughness progression cycle. Recognizing this cyclic process, the model developed in this study predicts roughness progression and the effect of blading on roughness in separate relationships, which can be solved simultaneously to predict a longterm steady-state condition of cyclic roughness trends. The model indicates that the rate of progression is a function only of traffic, rainfall and time, and that the maximum roughness to be expected was a function of material properties and road geometry. The maximum increases with horizontal
Table 10.1: Predictive models for deterioration of unpaved roads

<table>
<thead>
<tr>
<th>Distress and model</th>
<th>Equations</th>
</tr>
</thead>
</table>

1. **Roughness Progression**

\[
RG(t_2) = RG_{\text{max}} - p \left[ RG_{\text{max}} - RG(t_1) \right] 
\]
\[
p = \exp \left[-0.001 \left(0.461 + 0.0174 \text{ ADL} + 0.0114 \text{ ADH} - 0.0287 \text{ ADT MMF} \right) (t_2 - t_1) \right]; 
\]
\[
RG_{\text{max}} = \max \left[21.4 - 32.4 \left(0.5 - \text{MGD}_j \right)^2 + 0.97 \text{ KCV} - 7.64 \text{ G MMF}; 12 \right].
\]

2. **Effective of Blading Maintenance on Roughness**

\[
RG_a = RG_{\text{min}} + q \left[ RG_b - RG_{\text{min}} \right] 
\]
\[
q = 0.553 + 0.230 \text{ MGD}; 
\]
\[
RG_{\text{min}} = \max \left[0.8; \min \left[8; 0.361 \text{ D95} \left(1 - 2.78 \text{ MG}' \right) \right] \right]; \text{ and} 
\]
\[
\text{MG}' = \min \left[\text{MG}; 0.36 \right].
\]

3. **Roughness Cycles at Steady-state under Constant Maintenance Policy**

\[
RG_{\text{avg}} = RG_{\text{max}} + \left( RG_{\text{max}} - RG_{\text{min}} \right) \frac{(1 - p)(1 - q)}{(1 - p q) \ln p} 
\]
\[
RG_{\text{H}} = \left[ RG_{\text{max}} \left(1 - p\right) + RG_{\text{min}} p \left(1 - q\right) \right] / \left(1 - p q\right); 
\]
\[
RG_{\text{L}} = \left[ RG_{\text{min}} \left(1 - q\right) + RG_{\text{max}} q \left(1 - p\right) \right] / \left(1 - p q\right); 
\]

Note: Valid for \(0 < p < 1; 0 \leq q < 1; \Delta t > 0\). For \(p, (t_2-t_1) = \Delta t\), the interval between bladings.

(a) Time-scheduled, for blading a constant time intervals: \(\Delta t = \text{scheduled time interval between bladings, in days}; \)

(b) Traffic-scheduled, for blading after every \(\text{VEHG} \times \text{ADT}; \)

(c) Condition-responsive, for blading when roughness reaches threshold \(RG_{\text{ha}}\):

\[
\Delta t = \ln \left[ \left( RG_{\text{max}} - RG_{\text{ha}} \right) / \left[ RG_{\text{max}} - (1 - q) \left( RG_{\text{min}} - q \cdot RG_{\text{ha}} \right) \right] \right] / c; 
\]

where \(c = -0.001 \left(0.461 + 0.0174 \text{ ADL} + 0.0114 \text{ ADH} - 0.0287 \text{ ADT MMF} \right). \)

4. **Surface Material Loss**

\[
GL = (30 + 180 \text{ MMP} + 72 \text{ MMP G}) h \text{ ADT} t 10^{-5} 
\]

Alternative model:

\[
MLA = 3.65 \left[3.46 + 2.46 \text{ MMP G + KT ADT} \right] 
\]
\[
KT = \max \left[0; (0.022 + 0.969 \text{ KCV} + 0.00342 \text{ MMP P075} - 0.0092 \text{ MMP FI} - 0.101 \text{ MMF}) \right]
\]

(Table continued next page.)
Table 10.1: (Continued)

Notes:  
\(RG(t_1)\) = roughness at time \(t_1\), in m/km IRI;  
\(RG(t_2)\) = roughness at time \(t_2\), in m/km IRI;  
\(RG_a\) = roughness after blading, in m/km IRI;  
\(RG_b\) = roughness before blading, in m/km IRI;  
\(RG_{avg}\) = longterm average roughness under constant maintenance policy;  
\(RG_H, RG_L\) = highest and lowest roughnesses in each cycle;  
\(\Delta t\) = time interval between bladings dependent on type of policy, in days;  
\(GL\) = surface material loss, in mm;  
\(MMP\) = mean monthly precipitation, in mm;  
\(h\) = proportion of heavy vehicles in traffic, fraction;  
\(t\) = time since surfacing was placed, in days.  
\(MLA\) = the predicted annual material loss, in mm/year; and  
\(KT\) = the traffic-induced material whip-off coefficient.  
Other explanatory parameters are defined in Table 3.10.

Source: As noted by Equation numbers.

curvature and decreases with gradient, primarily due to the effects of surface water erosion and horizontal vehicle stresses at curves. The effectiveness of blading was found to depend primarily on the roughness before blading, and secondarily on the maximum particle size and material gradation. The reduction in roughness was typically 22%, but operator skills had a significant impact on this and caused variations of about 34 percent in the roughness achieved after blading.

Comparison of the model and Brazilian data with those from other studies showed several major differences. First, in model form, the model here is mainly linear at low roughness levels and slightly concave, converging on a maximum in the range of 12 to 25 m/km IRI under 20,000 to 80,000 vehicle transits (2-way). The exponential model of GEIPOT (1982) and cubic model of TRRL (1975) show strongly convex functions with very high progression rates above 12 m/km IRI. In fact both patterns are apparent within the data, even within one section across different blading cycles, and the reasons for the variations have not yet been fully resolved. The convex behavior seems to be associated with newly-constructed or recompacted surfaces with cohesive gravels, and the linear progression to be associated with less cohesive materials or more arid conditions. Second, the average rates of progression range widely, from 0.03 (for coral gravels in Kenya) to 0.37 (for laterites in Brazil) m/km IRI per thousand vehicles, when comparing major material types across six different studies. The predominant range is from 0.08 to 0.20 and variation within this range has yet to be explained by physical parameters. The rates observed in Brazil were three to five times faster than those observed in Kenya; the biggest single difference between the studies being the blading frequency, which covered a wide range in Brazil but was very low or non-existent in the Kenyan study. Interestingly, the lower rates of progression, particularly those observed in Kenya, may be associated with a uniform low intensity rainfall distribution, as distinct from a highly seasonal rainfall with high maximum intensities.
10.2.2 Material Loss

The loss of surface material from the carriageway, due to whip-off by traffic and erosion by surface water, was found to be a function primarily of traffic volume, and secondarily of rainfall, horizontal curvature, vertical gradient, and the plasticity and fines content of the material. The average rate of loss observed in Brazil was 0.28 mm per 1,000 vehicles, or 20 mm per year per hundred vehicles per day (ADT).

The value observed in Brazil is close to the average of six studies in Africa reported by Jones (1984) which range from 12 to 27 mm per year per hundred vehicles per day (ADT) for volumes up to 200 veh/day, and similar also to the rates of 0.22 to 0.67 mm per thousand vehicles observed in the recent Kenya study (Jones 1984). A significant difference between the models however, is that the model based on Brazilian data includes a term for loss independent of traffic, yielding an intercept of 12.6 mm per year loss under nil traffic, and is a linear function of traffic volume, whereas most of the other models have zero intercept and are concave functions of traffic volume. Over the range of data observed it is unlikely that the difference in forms is significant. Combining the results of several studies produced the model in Table 10.1 which gives the general effects of rainfall, gradient and traffic. The Brazilian model is a good alternative.

Progress has yet to be made however in explaining the differences in the loss rates of various materials in terms of physical properties. Angularity, gradation and density are factors to be considered.

10.2.3 Surface Material Selection

Observations in the Brazil study showed that the longterm rate of roughness progression per thousand vehicles on earth surfaces was generally similar to that on gravel surfaces. As a result of this effect in the prediction model, it is probable that some economic analyses may indicate little economic advantage for the use of gravel materials over earth surfaces. However, other criteria need to be considered, including rutting, slipperiness and passability in wet weather, and erosion by surface water. Appropriate criteria adopted from various sources are listed in Table 10.2.

10.2.4 Environmental Effects

The effects of rainfall, and the differences in behavior of different materials (which are often geologically closely-related to environmental factors) are environmental effects which have been observed and, in the Brazil models, partially quantified. However, the major variations of behavior within a section over time warrant further research; these were observed but not effectively quantified in the studies, and may be attributable to rainfall patterns and changes in drainage characteristics. Rainfall intensity and its distribution throughout the year appear to be factors influencing both gravel loss and roughness progression. The range of rainfalls encountered in the studies have tended to be in arid to subhumid climates with a maximum of 1,750 mm/year in Brazil that was highly seasonal. The effects of uniformly distributed rainfall, higher rainfalls up to 7,000 mm/year, and results for weathered materials with high fines and plasticity and for coarse gravels require research.
Table 10.2: Design criteria for surface material of unpaved roads

<table>
<thead>
<tr>
<th>Surface characteristic and design criteria</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Control of Passability</td>
<td></td>
</tr>
<tr>
<td>SFCBR ( \geq 8.25 + 3.75 \log_{10} (ADT) ) (3.2)</td>
<td></td>
</tr>
<tr>
<td>where SFCBR = the soaked California Bearing Ratio of surfacing material at standard Proctor laboratory compaction ((600 \text{ kJ/m}^3)), in percent, which is the minimum for ensuring passability;</td>
<td></td>
</tr>
<tr>
<td>ADT = average daily traffic in both directions, in veh/day.</td>
<td></td>
</tr>
<tr>
<td>2. Control of Looseness</td>
<td></td>
</tr>
<tr>
<td>P075 ( \geq 14 ) (3.1)</td>
<td></td>
</tr>
<tr>
<td>where P075 = percentage of surfacing material finer than 0.075 mm.</td>
<td></td>
</tr>
<tr>
<td>3. Control of Rutting</td>
<td></td>
</tr>
<tr>
<td>( \log_{10} \text{HG} = 1.40 + 12.3 \text{C1} -0.466 \text{C2} -0.142 \text{NE} 0.124 \text{RD}_{c} -0.5 ) (3.3)</td>
<td></td>
</tr>
<tr>
<td>where HG = the thickness of gravel surfacing, in mm;</td>
<td></td>
</tr>
<tr>
<td>C1 = soaked CBR of surfacing material, in percent;</td>
<td></td>
</tr>
<tr>
<td>C2 = soaked CBR of roadbed soil, in percent;</td>
<td></td>
</tr>
<tr>
<td>NE = design number of cumulative equivalent 80 kN single axle loads; and</td>
<td></td>
</tr>
<tr>
<td>RD_{c} = maximum allowable mean rut depth, mm.</td>
<td></td>
</tr>
</tbody>
</table>

Source: As noted, based on Visser (1981), and Barber, Odom and Patrick (1978).

Drainage characteristics and their interaction with road geometry have been poorly represented in the models through not being quantified in the studies. Level tangent sections may either perform poorly, under bathtub drainage conditions which greatly accelerate the development of potholes and result in high maximum roughness levels, or alternatively, with adequate crown and side drainage, may perform excellently and show very low rates of roughness progression.

10.2.5 Research and Measurement Issues

In addition to the environmental issues just enumerated, the following suggestions for future research are made as a result of this study.

The most important remaining need is to extend the research to further climates and material types and, in particular, to quantify the effects of cohesion and density of the surfacing materials in place. The climates need to
include those with high rainfall (more than 2,500 mm per year) or nonseasonal rainfall. Material types need to include coarse gravels, alluvial gravels, and many soil types on earth roads, with some measure of particle shape (angularity). With respect to apparent cohesion and density effects, the Brazilian study showed large variations in the rate of roughness progression across successive blading cycles, even for a given road section, that could not be explained, and the Bolivian study showed that reshaping and recompaction suppressed the rate of roughness progression considerably for the initial cycle before subsequent blading cycles. Large differences also remain between the different country studies that have not yet been fully explained. Explanation of these variations and differences has potentially significant economic implications about the effectiveness of maintenance methods.

Surface drainage conditions are expected to influence the deterioration rates significantly, but these conditions were not quantified in the studies to date. Surface drainage conditions need to be defined and monitored in conjunction with the condition survey, for example by camber or crown, watershed or windrows on road shoulders, bathtub or bird bath depressions, etc. In addition to the vertical and horizontal geometry of the section, should be added information about the approach geometry and topography to indicate whether the section receives runoff in addition to direct precipitation.

Interpretation of roughness trends, and particularly of the trends or limits at high roughness levels, would be improved by the gathering of elementary condition survey information. This should include the number and size of potholes or depressions in the measured wheelpath, the presence of standing water, the wetness of the surface, the presence and direction of scour channels on the surface, exposure of boulder-size particles in the wheelpaths, etc.

The measurements of roughness and gravel loss tend to be made with considerable error, in the order of 30 to 40 percent. Very careful attention therefore needs to be made to the measurement method; in the case of roughness measurement the guidelines of Sayers, Gillespie and Paterson (1986) should be followed.

10.3 PAVED ROAD DETERIORATION

10.3.1 Definition of Deterioration and Roughness

Paved road deterioration has been defined here by the trends of separate distress types in physical measures, namely:

1. Roughness, in terms of a surface profile statistic relevant to the response of moving vehicles (m/km IRI, and the parallel measures of counts/km QI_m and mm/km Bump Integrator trailer used in particular empirical studies);

2. Cracking, in terms of (a) extent, as the normalized area of the pavement surfacing (%), (b) severity by class of crack width, (c) type by visible pattern of cracks, and (d) intensity, by length of crack per unit area (m/m^2) (available only in TRRL studies);

3. Ravelling, in terms of extent (as for cracking);
4. Potholing, in terms of (a) extent (as for cracking), or (b) volume of open potholes per lane-km \( (m^3/\text{lane-km}) \) where available;

5. Rutting, in terms of the mean and standard deviation of rut depth in both wheelpaths within a traffic lane (mm); and

6. Surface friction, in terms of a skid resistance coefficient (units varying with test method).

Cracking, ravelling and potholing are characterized by separate phases of initiation and areal progression. Roughness and rut depth are characterized by continuous progression. Surface friction is represented by progression to a critical level.

Separate measures such as these are preferable in economic evaluations to a composite measure, such as serviceability index, because different types of maintenance intervention are triggered by different types of distress, and the consequences of any one type of distress differ across types of pavement construction and across climatic and traffic conditions. Physical measures of distress are preferred to damage functions, which are normalized to an implicit maintenance intervention criterion (such as a serviceability index of 2.5), because this permits a direct economic comparison of alternative maintenance intervention levels and strategies.

10.3.2 Model Forms and Statistical Methodology

The incremental, or in its purest sense derivative, form of distress function is the most versatile form for the variety of applications of road deterioration predictions. In essence the incremental model form permits prediction of future deterioration as a function of time, given only the current surface condition and the imposed traffic, structural and environmental conditions, that is:

\[
A \quad (\text{future deterioration} = f \left( \text{current condition, traffic volume and over incremental loading, pavement strength, environment, time} \right) \text{ maintenance})
\]

The generally slow rate of deterioration of paved roads however means that the changes of condition observed in empirical deterioration studies are usually small, and very sensitive to measurement error. Furthermore, many effects are statistically collinear, that is increasing simultaneously, such as time and cumulative traffic, traffic volume and age, pavement strength and traffic loading, etc., making it difficult to distinguish the true causes of deterioration. To cope with these factors, the statistical estimation methods applied to the time-series, cross-sectional data in this study included:

1. Linear regression based on the ordinary least squares of residuals estimation procedure;

2. Linear regression on the natural logarithmic transform of parameters, with correction for the logarithmic mean;

3. Linear regression on logistic transformation of parameters (for sigmoidal functions);
4. Nonlinear regression procedures based on the ordinary least squares of residuals;

5. Probabilistic failure-time modelling for the initiation of surfacing distress using maximum likelihood estimation (with a variant of the Newton Raphson procedure for maximization) and a Weibull distribution of failure times;

6. Reduction of the data from individual road subsections to the aggregated incremental distress over the duration of the study period, in order to diminish the variance due to measurement errors and enhance the statistical discrimination of the desired parameter effects; and

7. The use of time as the base for predicting incremental distress, with traffic volume or loading rate, and pavement or surfacing age, acting as continuous explanatory variables.

The use of the probabilistic failure-time models for the predictions of distress initiation was a major advance, which permitted the estimation of concurrent time and traffic effects, the distinction between pavement variability and model error, and the use of censored data (that is data from those sections on which the "failure" event was not actually observed during the study period). Nonlinear regression proved to be essential in the modelling of roughness progression, which involved a combination of both linear and nonlinear terms.

10.3.3 Distress Interaction

Pavement deterioration was modelled to permit interaction between different distress types and maintenance. For example, the occurrence of wide cracking was made consequential on the occurrence of narrow cracking, cracking and rainfall were found to influence rut depth progression, and cracking, potholing, patching and rut depth were found to influence roughness progression.

The prediction models for individual types of distress, as developed in this study, are summarized and discussed in the following sections.

10.3.4 Prediction of Cracking

The development of cracking was represented by two separate phases, namely the time before initiation of cracking (or the age of the surfacing at "first failure") and the rate of progression of the area of cracking, and by two severity levels, namely, all cracking (narrow and wide cracks, that is more than 1 mm wide) and wide cracking (that is cracks wider than 3 mm, or spalled). The models apply to crocodile cracking only, which is the traffic-associated cracking having the most important economic consequences in nonfreezing climates; other types of cracking such as linear cracking were not represented in the data base and occur primarily in freezing climates. Predictions are separated also by major categories of surfacing and pavement type.

The predictive models are summarized in Table 10.3 and 10.4 for the initiation of all and wide cracking respectively, and in Table 10.5 for the progression of cracking.
The prediction of cracking initiation has a probabilistic form, which is to say that there is a range of ages over which initiation is expected to occur. The model gives both the expected time of initiation (i.e., with about 50 percent probability that failure has not occurred earlier), and the distribution of failure times about that value. The latter is a Weibull distribution, selected because its shape is flexible and can be estimated from the data, and because it is highly appropriate to the fatigue mechanism causing cracking (representing an increasing likelihood of failure over time if it has not already occurred). The estimated dispersion of failure times is shown in the tables by the semi-interquartile factor (SIQF), which is half the spread between the times for 25 and 75 percent probabilities of failure, as a fraction of the expected time. More detailed statistics of the dispersion may be derived from the $\beta$ parameter given in the source tables, Equation 5.14, and Figure 5.11.

The surfacing age at which cracking initiates was found to be strongly influenced by aging, traffic loading and pavement stiffness for all original surfacings, that is asphalt concrete or surface treatment (chip seal), on either granular or cemented bases. The form for each case was similar although the parameter estimates differed. The 95th percentile confidence intervals for the prediction ranged from 1.1 years for asphalt concrete surfacings to 1.6 to 1.7 years for surface treatment/granular base and cemented base pavements. The distribution of lives due to stochastic variability indicated that about half of the pavements were likely to have lives either less than 70 percent or more than 130 percent of the expected life.

The effect of aging was so strong in each case that cracking initiation generally occurred within 6 to 13 years even at low traffic volumes. In the Brazilian data base, surface treatments had longer life expectancy than asphalt concrete at similar traffic and deflection levels up to 0.3 million ESA/lane/year, due apparently to the fairly high stiffness of the asphalt mixes, oxidation, and the thicknesses being near to the critical 60 to 80 mm range. This is consistent with the longer fatigue life and greater resistance to oxidation given by the thick binder film and low stiffness of surface treatments. Oxidation increases the viscosity of bituminous binders over time by chemical reaction to a critical level at which fracture can be caused either by traffic-induced strain or by daily thermally-induced strains. As oxidation effects diminish rapidly with depth below the exposed surface in low permeability materials, their influence on cracking initiation is likely to be stronger on the thin surfacings (less than 100 mm thick) included in the Brazil study data base than on thicker asphalt layers. The oxidation rate depends also on temperature and the chemical resistance of the bitumen. Thus the aging effects estimated in the models will not apply for all applications, and this point is addressed further under transferability.

Increasing traffic loading rate, and decreasing pavement strength both reduced the time to cracking initiation significantly. In the models, these effects are represented interactively, although the precise form of the interaction was somewhat indistinct in the data due to scatter. The pavement strength effect was well represented by either the modified structural number or Benkelman beam deflection for granular-base pavements, and by surface deflection and the cemented base modulus for cemented base pavements.

In asphalt concrete surfacings, the observed behavior compared well with a classical mechanistic fatigue model based on tensile strain, but only when the strain was the maximum occurring at either the underside or exposed surface of the
Table 10.3: Expected time or traffic to initiation of all cracking for original and maintenance surfacings

<table>
<thead>
<tr>
<th>Expected time or traffic to initiation of all cracking</th>
<th>SIQF</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Asphalt Concrete Original</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$TY_{cr2} = 4.21 \exp \left(0.139 \text{ SNC} - 17.1 \text{ YE}_4 / \text{ SNC}^2\right)$</td>
<td>0.320</td>
<td>5.21</td>
</tr>
<tr>
<td>$TY_{cr2} = 8.38 \exp \left(1.21 \text{ BNO} - 18.6 \text{ YE}_4 / \text{ SNC}^2\right)$</td>
<td>0.388</td>
<td>5.23</td>
</tr>
<tr>
<td>$TE_{cr2} = 0.0342 \text{ EHM}^{-2.86} e^{-0.198 \text{ EY}}$</td>
<td>0.545</td>
<td>5.24</td>
</tr>
<tr>
<td><strong>Surface Treatment Original</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$TY_{cr2} = 13.2 \exp \left[-20.7 (1 + \text{ CQ}) \text{ YE}_4 / \text{ SNC}^2\right]$</td>
<td>0.297</td>
<td>5.28a</td>
</tr>
<tr>
<td><strong>Asphalt or Surface Treatment on Cemented Base</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$TY_{cr2} = 1.11 \exp \left(0.035 \text{ H}_b + 0.371 \ln \text{ CMOD} - 0.418 \ln \text{ DEF} - 2.87 \text{ YE}_4 \text{ DEF}\right)$</td>
<td>0.160</td>
<td>5.33</td>
</tr>
<tr>
<td><strong>Asphalt Overlays</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$TY_{cr2} = 10.8 \exp \left[-1.21 \text{ DEF} - 1.02 \text{ YE}_4 \text{ DEF}\right]$</td>
<td>0.393</td>
<td>5.34</td>
</tr>
<tr>
<td><strong>Reseals on Uncracked Surface</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$TY_{cr2} = 13.2 \exp \left[-20.7 (1 + \text{ CQ}) \text{ YE}_4 / \text{ SNC}^2\right]$</td>
<td>0.297</td>
<td>5.28a</td>
</tr>
<tr>
<td><strong>Reseals on Cracked Surface</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$TY_{cr2} = \begin{cases} 0.4 \text{ H}_b \text{ (on surface treatment)} \ 0.2 \text{ H}_b \text{ (on asphalt concrete)} \end{cases}$</td>
<td>-</td>
<td>section 5.4.5</td>
</tr>
<tr>
<td><strong>Slurry Seals on Cracked Surface</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$TY_{cr2} = \begin{cases} 1.4 \text{ (on surface treatment)} \ 0.8 \text{ (on asphalt concrete)} \end{cases}$</td>
<td>0.50</td>
<td>section 5.4.5</td>
</tr>
</tbody>
</table>

**Notes:**
- $TY_{cr2}$ = expected (mean) age of surfacing at initiation of narrow cracking, years;
- $TE_{cr2}$ = expected (mean) cumulative traffic at initiation of narrow cracking, million ESA;
- SNC = modified structural number;
- DEF = Benkelman beam deflection under 80 kN single axle load, mm;
- YE = annual traffic loading, million ESA/lane/year;
- $H_b$ = thickness of bituminous layers, mm;
- BNO = excess of binder content with respect to optimum, fraction;
- EHM = maximum tensile strain in surfacing, $10^{-3}$; and
- CMOD = resilient modulus of cemented base, GPa;
- EY = $1 / (\text{EHM} \cdot 1000 \text{ YE}_4)$, provided that EY $\leq 6$.
- SIQF = semi-interquartile probability factor (half the spread between the times for 25 and 75 percent probabilities of failure, as a fraction of the expected time).

**Source:** Tables 5.5, 5.6, and 5.7, and Equations as noted.
Table 10.4: Expected time of initiation of wide cracking as a function of the initiation-time of all cracking

<table>
<thead>
<tr>
<th>Pavement type</th>
<th>Time to wide cracking (yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt concrete</td>
<td>2.46 + 0.93 $TY_{cr2}$</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>2.66 + 0.88 $TY_{cr2}$</td>
</tr>
<tr>
<td></td>
<td>1.16 $TY_{cr2}$</td>
</tr>
<tr>
<td>Surfacing on cemented base</td>
<td>1.46 + 0.98 $TY_{cr2}$</td>
</tr>
<tr>
<td>Slurry seals</td>
<td>0.70 + 1.65 $TY_{cr2}$</td>
</tr>
<tr>
<td>Reseals (ST)</td>
<td>1.85 + 1.00 $TY_{cr2}$</td>
</tr>
<tr>
<td>Asphalt overlays</td>
<td>2.04 + 0.98 $TY_{cr2}$</td>
</tr>
<tr>
<td>Open-graded cold mix asphalt</td>
<td>0.26 + 1.44 $TY_{cr2}$</td>
</tr>
</tbody>
</table>

Note: Time origin is construction of the most recent surfacing.
Source: Table 5.8.

Table 10.5: Models for predicting cracking progression as functions of incremental time or traffic

<table>
<thead>
<tr>
<th>Cracking class and surfacing</th>
<th>Time-base$^1$/</th>
<th>Traffic-base$^2$/</th>
<th>Average linear rate$^3$/</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$a$</td>
<td>$b$</td>
<td>$a$</td>
</tr>
<tr>
<td>All cracking</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>1.84</td>
<td>0.45</td>
<td>450 SNC $^{-2.27}$</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>1.76</td>
<td>0.32</td>
<td>1,760 SNC $^{2.23}$</td>
</tr>
<tr>
<td>On cemented base</td>
<td>2.13</td>
<td>0.36</td>
<td>0.005 DEF $^{0.64}$ CMOD $^{0.90}$</td>
</tr>
<tr>
<td>Asphalt overlays</td>
<td>1.07</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>Reseals and slurry seals</td>
<td>2.41</td>
<td>0.34</td>
<td></td>
</tr>
<tr>
<td>Wide cracking</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt concrete</td>
<td>2.94</td>
<td>0.56</td>
<td>718 SNC $^{2.52}$</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>2.50</td>
<td>0.25</td>
<td>160 DEF $^{1.48}$</td>
</tr>
<tr>
<td>On cemented base</td>
<td>3.67</td>
<td>0.38</td>
<td>0.061 CMOD $^{0.56}$</td>
</tr>
<tr>
<td>Asphalt overlays</td>
<td>2.58</td>
<td>0.45</td>
<td></td>
</tr>
<tr>
<td>Reseals</td>
<td>3.4</td>
<td>0.35</td>
<td></td>
</tr>
</tbody>
</table>

$^1/ A_{cri}$ = a SA_{cri} $^{1-b}$ $At$ (exact form in Equation 5.41).

$^2/ A_{cri}$ = a SA_{cri} $^{1-b}$ $ANE_4$ (exact form in Equation 5.41).

$^3/ A_{cri}$ = a $At$, in percent per year.

Notes: $A_{cri}$ = increment in area of cracking $A_{cri}$, in percent of total area;

$SA_{cri}$ = minimum ($A_{cri}$, 100 - $A_{cri}$);

$A_{cri}$ = area of cracking of class i and greater, in percent of total area;

$At$ = incremental time, years.

$ANE_4$ = incremental traffic, million ESA.

SNC, DEF, CMOD are defined in Table 10.3.

Source: Tables 5.9, 5.10 and 5.11.
surfacing layer, and when a term representing aging effects was included. The behavior correlated poorly with the strain at the underside of the layer, contrary to some applications of mechanistic theory. Both these findings however are applicable primarily to thin surfacings of less than 100 mm thickness and are unlikely to be directly applicable to thicker layers. There was strong evidence that a surfacing thickness of 50 to 70 mm on a granular base gave a minimum life expectancy, the life increasing for thicknesses both thinner and thicker than that range, and this also agrees with mechanistic modelling. For pavements with surfacing thickness in this range, stiff pavement support and high base quality are crucial to achieving reasonable life expectancy.

Wide cracking initiation was observed to occur typically 1.5 to 2.5 years after narrow cracking, and tended to develop slightly earlier in original surface treatment surfacings and cemented base pavements than in original asphalt concrete surfacings.

The rate of cracking progression was found to be fairly strongly nonlinear, varying mainly with the area cracked in an S-shaped or sigmoidal form. Average progression rates varied slightly across surfacing types, ranging from seven percent per year in asphalt concrete original and overlay surfacings to 19 percent per year in reseals. The progression rates of wide cracking were generally 20 to 40 percent faster than those of all cracking. Some dependence of the progression rates on pavement strength and traffic loading was found, but this was not statistically very much superior to simple stochastic models based on time elapsed. Thus both kinds of models are provided in Table 10.5.

10.3.5 Prediction of Ravelling

The predictive models for the initiation and progression of ravelling and potholing are summarized in Table 10.6. The probabilistic failure-time approach was again applied to modelling the initiation of ravelling, and predictions were estimated for three types of bituminous surface treatment, namely, double chip seal (surface treatment), slurry seal and open-graded cold mix. No data were available on the ravelling of asphalt concrete surfacings, but this is a rare occurrence except under inadequate material specifications.

The initiation of ravelling, which is the loss of material from the surface by disintegration, was found to be most strongly influenced by age and construction quality, and secondarily by traffic volume independent of loading, with different life expectations for each treatment type. The progression of ravelled area was found to be a stochastic phenomenon not related directly to traffic, a strongly sigmoidal function of the area ravelled, and not significantly different for the various treatment types.

The life expectancy of chip seal double surface treatments ranged from 10 to 12.5 years for traffic volumes of 2,500 veh/lane/day or less, for a maintenance intervention criterion of 50 percent area ravelled (which occurs about 2 years after initiation). Considerable dispersion was observed about this mean life expectancy, with the quartile lives being about 30 percent below and above the expected life respectively, that is, one quarter survive less than 7 to 8.5 years and one quarter survive longer than 13 to 16.5 years. The maximum lives observed in the Brazil study were 16.5 and 18 years. For higher traffic flows, up to 6,000 veh/lane/day, ravelling is predicted to occur only 2 to 2.5 years earlier than the lower values cited above.
Table 10.6: Predictive models for ravelling and potholing

<table>
<thead>
<tr>
<th>Distress and model</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Initiation of Ravelling</td>
<td>$\frac{TY_{\text{rv}}}{K_p s} = \exp(-0.655 CQ - 0.156 \text{YAX})$</td>
</tr>
<tr>
<td></td>
<td>$a_s = 10.5$ (surface treatment); 14.1 (slurry seal); 8.0 (open-graded cold-mix)</td>
</tr>
<tr>
<td>2. Progression of Ravelling</td>
<td>$\Delta A_{\text{rv}} = \left{ \begin{array}{ll} 0.24 \Delta t; &amp; \text{or} \ 4.42 S_{A_{\text{rv}}} 0.65 \Delta t &amp; \end{array} \right.$</td>
</tr>
<tr>
<td>3. Initiation of Potholing</td>
<td>$\min {2 + 0.04 H_s - 0.5 \text{YAX}; 2}$ when base is granular or bituminous</td>
</tr>
<tr>
<td></td>
<td>$\frac{TY_{\text{pot}}}{H_s} = \left{ \begin{array}{ll} \max {6 - \text{YAX}; 2} &amp; \text{when base is cemented} \ \max {2 + 0.04 H_s; 0.3} &amp; \text{for granular base; and} \ 0.6 &amp; \text{for cemented base; and} \ 0.3 &amp; \text{for bituminous base} \end{array} \right.$</td>
</tr>
<tr>
<td>4. Progression of Potholing</td>
<td>$\Delta A_{\text{pot}} = \min {\Delta A_{\text{pcr}} + \Delta A_{\text{prv}} + \Delta A_{\text{ppe}}; 10}$</td>
</tr>
<tr>
<td></td>
<td>$\Delta A_{\text{pcr}} = \min {2 A_{\text{crw}} U; 6}$</td>
</tr>
<tr>
<td></td>
<td>$\Delta A_{\text{prv}} = \min {0.4 A_{\text{rv}} U; 6}$</td>
</tr>
<tr>
<td></td>
<td>$U = (1 - CQ) (\text{YAX} / \text{SNC}) / 2.7 H_s$</td>
</tr>
<tr>
<td></td>
<td>$\Delta A_{\text{ppe}} = \min {\Delta A_{\text{pot}} \left{KB \text{YAX} (\text{MMP} + 0.1)\right}; 10}$</td>
</tr>
</tbody>
</table>

Notes: $TY_{\text{rv}}$ = age of surfacing at initiation of ravelling, years; $\text{YAX}$ = annual volume of traffic, million axles/lane/year; $\Delta A_{\text{rv}}$ = increment of ravelling area, percent; $S_{A_{\text{rv}}}$ = minimum $(A_{\text{rv}}, 100 - A_{\text{rv}})$ where $A_{\text{rv}}$ = area of ravelling, percent; $TY_{\text{pot}}$ = time between cracking initiation and potholing initiation, years; $\Delta A_{\text{pcr}}$ = increment of potholing area due to cracking, percent; $\Delta A_{\text{prv}}$ = increment of potholing area due to ravelling, percent; $\Delta A_{\text{ppe}}$ = increment of potholing area due to enlargement, percent; $H_s$ = thickness of bituminous surfacing layers, mm; Other parameters are defined in Tables 10.3 to 10.5.

Source: Equations as noted.
In practice, the distress manifested by surface treatments at these higher traffic volume levels is more likely to be polishing, stone embedment or bleeding, rather than ravelling. These conditions were not measured systematically in the study, so modelling was not possible. However, the lives predicted by the ravelling model appear to be highly representative of the rescaling intervals observed in a number of other countries where chip seal double surface treatments are common construction, even though the modes of distress differ. Comparison with quantitative data from Tunisia, and qualitative data from Australia, New Zealand and South Africa appear to confirm the life predictions summarized as follows:

<table>
<thead>
<tr>
<th>Traffic volume veh/day (ADT)</th>
<th>Expected reseal interval years</th>
<th>90 percent within range of (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>12</td>
<td>4-21</td>
</tr>
<tr>
<td>5,000</td>
<td>10</td>
<td>3.5-17</td>
</tr>
<tr>
<td>10,000</td>
<td>8</td>
<td>3-14</td>
</tr>
</tbody>
</table>

Poor construction quality was observed to reduce the life expectancy considerably, by the order of one-half, so that the risk of early failures occurring within 4 years of construction becomes very high in that instance. Significant benefits therefore can be gained from improving the construction technology of surface treatments in regions where poor quality has led to short life expectancies.

10.3.6 Prediction of Potholing and Its Effects on Roughness

Potholing, the most severe form of surfacing distress, is an essential component in deterioration modelling because it provides the economic penalty of deferred maintenance. Its modelling however is difficult, and in this study use was made of data from four countries to model the initiation and progression of potholing as a consequence of wide cracking, ravelling and existing potholes (see Table 10.6). Both initiation and progression are hastened by increasing traffic flow or decreasing surfacing thickness, the rates being based on judgment in addition to the data.

As the development of potholing is highly dependent on many factors peculiar to a material, climate and construction quality, the models presented are a generalization of major effects, and considerable dispersion can be expected about the predictions. Nevertheless, they represent a logical ordering of the variables most likely to cause potholing and so, with possible adaptation to local circumstances, provide estimates that are appropriate to economic predictions.

An important contribution of the study was to quantify the effect of potholes and depressions on roughness. As this cannot be measured physically with roughness roadmeters because of the risk of mechanical damage, a computer simulation study was undertaken using road profile data and the vehicle response simulation incorporated in the International Roughness Index. Over a range of different pothole sizes and shapes, and on roads covering a wide range of roughness, a unique relationship was obtained showing that the increase in roughness sensed by
vehicles was linearly proportional to the volume of the pothole cavities encountered in the wheelpaths. After adjustment to take account of avoided potholes, the simulation model showed that roughness was increased by 0.75 m/km IRI per cubic meter of potholes per lane-kilometer. Field data from a Caribbean study showed an effect that was only one-fifth of this amount, however, due probably to the wide shallow shape of the potholes in that study which would lessen the impact on the sensed roughness. Currently, the lesser effect (0.16 m/km IRI per m$^3$ per lane-km) is recommended.

This effect also applies in principle to the impact of uneven patching, and protrusions above the pavement surface, on road roughness.

10.3.7 Rutting

In modern pavement construction, rutting due to densification and deformation in the lower pavement layers under traffic loading is usually minor because it is taken into account in structural design methods, but can become significant when the pavement is weakened by water ingress. Rutting may also develop by plastic flow in bituminous surface layers if the bituminous materials are soft under high temperatures or tire pressures. The variation of rut depth is an important condition parameter because it has a direct physical relationship to roughness.

The standard deviation of rut depth (across both wheelpaths in a lane), was found to depend strongly on the mean rut depth. Both the mean and the standard deviation were found to be nonlinear functions of cumulative equivalent standard axle loadings, modified structural number, the average relative compaction, and cracking, as summarized in Table 10.7. In the study data, the rate of rut depth progression generally decreased over time, indicating that the mechanism of rutting was primarily densification and deformation. This is represented in the models by the power terms (ERM and ERS) applied to the cumulative traffic loading which are generally in the order of 0.1 so that the rate of progression is strongly nonlinear. The rate increases as cracking occurs, particularly in combination with rainfall, but is not as sensitive to these conditions in the model as has been observed in other experimental research. The clear relationship of rut depth to relative compaction indicates an economic advantage for enforcing construction compaction specifications as much as possible.

Validation of the models against independent data showed that the predictions were good for pavements that were not highly susceptible to water ingress and that had adequate stiffness in the bituminous layers to avoid plastic flow. For the exceptions, other studies indicate that the power terms (ERM and ERS in Table 10.7) would be higher than predicted, tending towards a value of unity in the worst cases. Appropriate parameters and estimates for these conditions are not available from this study, but the effects can be accommodated by calibration of the model to the conditions.

10.3.8 Roughness

As roughness is the type of distress by which pavement performance is ultimately evaluated, its modelling was a most important aspect of the study. Previous predictive models, notably the serviceability index functions developed from the AASHO Road Test and the roughness function developed from the Kenya road costs study, represented roughness progression as an entirely structural
Table 10.7: Predictive models for rutting and roughness progression

<table>
<thead>
<tr>
<th>Distress and model</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. Mean Rut Depth</strong></td>
<td></td>
</tr>
<tr>
<td>( R_{DM} = t^{0.166} \text{ SNC}^{-0.502} \text{ COMP}^{-2.30} \text{ NE}_4 \text{ ERM} )</td>
<td>7.11</td>
</tr>
<tr>
<td>( \text{ ERM} = 0.0902 + 0.0384 \text{ DEF} - 0.009 \text{ RH} + 0.00158 \text{ MMP A}_{crx} )</td>
<td></td>
</tr>
<tr>
<td><strong>2. Rut Depth Standard Deviation</strong></td>
<td></td>
</tr>
<tr>
<td>( R_{DS} = 2.06 R_{DM}^{0.532} \text{ SNC}^{-0.422} \text{ COMP}^{-1.66} \text{ NE}_4 \text{ ERS} )</td>
<td>7.12</td>
</tr>
<tr>
<td>( \text{ ERS} = -0.009 \text{ RH} + 0.00116 \text{ MMP A}_{crx} )</td>
<td></td>
</tr>
<tr>
<td><strong>3. Roughness Increment</strong></td>
<td></td>
</tr>
<tr>
<td>( \Delta \text{RI} = 134 e^{mt} \text{ SNCK}^{-5.0} \text{ ANE}<em>4 + 0.114 \Delta \text{ARDS} + 0.0066 \Delta \text{A}</em>{crx} )</td>
<td>8.13</td>
</tr>
<tr>
<td>[ + 0.003 H_p \Delta \text{A}<em>{pat} + 0.16 \Delta V</em>{pot} + m \Delta \text{RI} \text{At} ]</td>
<td></td>
</tr>
<tr>
<td>( \text{ SNCK} = 1 + \text{ SNC} - 0.000758 \text{ H}<em>c \text{ A}</em>{crx} )</td>
<td></td>
</tr>
<tr>
<td><strong>4. Roughness</strong></td>
<td></td>
</tr>
<tr>
<td>( \text{RI}(t) = [\text{RI}_0 + 725 (1 + \text{SNC})^{-5.0} \text{ NE}_4(t)] e^{0.0153t} )</td>
<td>8.20</td>
</tr>
</tbody>
</table>

**Notes:**
- \( R_{DM} \) = mean rut depth in both wheelpaths, mm;
- \( R_{DS} \) = standard deviation of rut depth in both wheelpaths, mm;
- \( t \) = age of pavement since rehabilitation or construction, year;
- \( \text{ SNC} \) = modified structural number of pavement strength;
- \( \text{ COMP} \) = compaction index of flexible pavements (see Chapter 7), fraction;
- \( \text{ DEF} \) = pavement surface deflection by Benkelman beam, mm;
- \( \text{ RH} \) = rehabilitation state (= 1 if pavement overlay = 0 otherwise);
- \( \text{ MMP} \) = mean monthly precipitation, mm per month;
- \( \text{ A}_{crx} \) = area of indexed cracking (Equations 5.3, 5.4), percent;
- \( \text{ RI}(t) \) = roughness at age \( t \), m/km IRI;
- \( m \) = environmental coefficient (nominally 0.023, see Table 8.7);
- \( \text{ NE}_4(t) \) = cumulative traffic loading at time \( t \), million ESA (with load damage power 4);
- \( H_p \) = average deviation of patch from original surface profile, mm;
- \( H_c \) = total thickness of cracked layers of bound materials mm;
- \( \Delta \text{RI}, \Delta \text{ARDS}, \Delta \text{A}_{crx}, \Delta \text{ANE}_4, \Delta t \) = incremental values of \( \text{RI}(t), \text{ARDS}, \text{A}_{crx}, \text{NE}_4, \text{t} \), respectively;
- \( \Delta \text{A}_{pat} \) = incremental area of patching, percent; and
- \( \Delta V_{pot} \) = incremental total volume of potholing, in m\(^3\)/lane/km.

**Source:** As noted by equations.
phenomenon depending on pavement strength (the structural number, modified for subgrade support) and cumulative traffic loading (expressed in the summary statistic of equivalent 80 kN standard axle loads). Other studies, notably in Arizona, Australia and Canada, have been unable to detect traffic or structural effects and have related roughness progression only to time and environment.

**Incremental model**

The incremental roughness model developed in this study includes both structural and time-related environmental mechanisms, as well as the effects of surface distress such as cracking, patching and potholing, as summarized in Table 10.7. The incremental roughness is predicted as a function of incremental traffic loading, time and surface distress, given the current surface condition, pavement strength and environmental properties. The standard error of prediction was 0.12 m/km IRI for a one-year increment of which about one-half was found to be model error and one-half to be measurement error in the data.

The roughness arising through structural deformation is derived from two sources. In the first, the incremental roughness is related to traffic loading, the net pavement strength (after due allowance is made for weakening due to cracking), environment and age. This function is similar in some respects to those derived from the AASHO and Kenya studies, but differs in its inclusion of cracking, environmental and age effects, and in the marginal effects of pavement strength. The latter are similar to those in the Kenya model (Parsley and Robinson 1982) but less than those in the AASHTO model (AASHTO 1981), which appear to be higher due to the thaw effects of a freezing environment.

In the second deformation source, there is a clear relationship between incremental roughness and the incremental variation of rut depth, which amounted to 0.11 m/km IRI per mm of rut depth standard deviation. The relationship is inherently sensible because both are measures of the surface profile, but this statistical estimation of the effect is important because it provides a link with the mechanistic predictions of behavior, through rutting. Its inclusion in parallel to the first deformation function allows for the variation in pavement properties and behavior, and compensates for situations in which rutting comes not from densification but from plastic flow. One practical implication is the economic benefit deriving from uniformity and quality control in construction.

Cracking and patching were both found to increase roughness. Their inclusion in separate terms means that changes in the quality of patching could be reflected by a change in the patching coefficient. The patching present in the data base was skin patching with an average protrusion height of about 3 mm and the more general effect is directly proportional to the average (rectified) protrusion height or depth of the patching. The effect of potholing was included directly from the calibrated simulation study mentioned above, as pothole effects were either excluded in the data base or separated in the estimation. The effect is large and powerful when the volume of potholes reaches significant levels.

An appreciable amount of roughness change was found to be essentially independent of pavement strength, traffic or surface distress, but related instead to time and the environment. This was attributed to the effects of cyclic temperature and moisture changes in the pavement and roadbed which occur daily and seasonally, and is thus likely to be a function of several environmental factors including the climate (macroenvironment) and drainage (microenvironment). Within
the Brazilian data base, which covered a rather narrow range of moderate rainfall and humid climate, no explanatory parameter could be found and the average amounted to a 2.3 percent increase in roughness annually. The environmental effects were quantified by application of the model to seven independent data bases with a wide range climates, including dry nonfreezing (Kenya, Tunisia and Arizona), dry freezing (Arizona, Colorado) and wet freezing (Illinois). The results followed a clear pattern, with the annual (independent) increase in roughness relating to climate as follows (see Table 8.7 for detail):

- Arid and semi-arid nonfreezing - one percent or less;
- Subhumid to humid nonfreezing - two to three percent;
- Arid to semi-arid freezing - three to five percent; and
- Wet freeze-thaw to freezing - five to ten percent, or more.

The coefficient $m$ expresses this as a fraction. Any specific effects of the microenvironment (pavement drainage, etc.) were not quantifiable.

Generally strong validation of the model has been demonstrated on eight independent data sets from countries other than Brazil. The prediction errors, after inclusion of the environmental effects above, were in the same order as the original model estimation (about 0.5 m/km IRI on a four-year increment, or 50 percent of the increment), the biases were negligible (1 percent, with one case of -17 percent) and the correlations were generally good (in the range of 0.5 to 0.95). Much of the error and the scatter in the correlations were attributable to measurement errors similar to those experienced in the Brazilian study, because there were generally no residual correlations with any of the five main components of the model. The worst scatter was found on the Texan data and was attributable to problems with data interpretation. Moderate residual correlations with roughness and the surface distress components on the AASHO road test data indicate that the model may underpredict the roughness increments in wet freezing climates by about thirty percent. Alternative predictive models taken from published sources produced considerably worse prediction errors in the order of 30 to 300 percent higher than those of the study model, when compared on three of the independent data sets. Thus the validity of the model appears to be exceptionally strong and sufficient for it to be applied with reasonable confidence to a wide variety of conditions and countries. This validity is qualified in respect of freezing climates, and for the determination of the environmental coefficient $m$.

Aggregate model

Since implementation of the incremental model is complicated by the inclusion of surface defect terms that need separate prediction, an alternative model which aggregates these effects was also developed. As shown in Table 10.7, the model predicts roughness as a function of only pavement strength, traffic loading, time, and initial roughness and may be used in absolute, incremental or derivative forms. It is valid for low levels of surface distress (less than ten percent) and requires adjustment for higher levels. Most of the foregoing discussion on the incremental model applies also to this model.

10.3.9 Combined Modes of Distress

The combination of the relationships in Tables 10.3 to 10.7 represent the predictive model of deterioration for paved roads. These relationships have
been incorporated in HDM-III, the third version of the Highway Design and Maintenance Standards model (Watanatada and others 1987a), in which the trend of pavement condition is computed year by year over the analysis period. They can also be incorporated in a separate computational package which computes only road deterioration and maintenance effects, such as may be developed for pavement analysis and pavement management.

One example of the accuracy of the combination of relationships in the model is shown in Figure 10.1, applied to one of the sections in the Brazil-UNDP-World Bank Road Costs Study. The initiation of cracking after 12.2 years was accurately predicted, as seen in (a), and the progression of cracking to cover the full area in 4.7 years was moderately well-predicted with the exception that the initial progression rate observed was unusually high. The prediction of roughness progression, shown in (b), utilized predictions of cracking, raveling, potholing and rutting, not observed values. The change of roughness over the observation period was well-predicted, as was the increase in the rate of progression which followed cracking, although a slight time-lag is evident due to the difference between the observed and predicted rates of cracking progression.

In addition to applications through HDM-III in several countries during the past three years, specific calibration of the relationships from network data has been undertaken in a number of cases, including Tunisia, Niger, and Saskatchewan, Canada. For Tunisia, a semi-arid climate, the predictions of cracking initiation age needed to be increased by only 5 percent for surface treatments (Table 5.13), and the environmental coefficient \( m \) had to be reduced to 0.011 (see Section 8.6.4, and Table 8.6, page 314. These adjustments were consistent with the effects of Tunisia's semi-arid, mediterranean climate. In Niger the predictions of cracking initiation age had to be reduced by 24 percent.

### 10.4 RELATIVE DAMAGING EFFECTS

Quantification of the relative damaging effects of different axle loadings in mixed traffic and of non-traffic-associated effects are crucial to the allocation of road costs in the pricing of road use, and to the determination of economically optimum limits on vehicle and axle loading, tire pressures and axle configurations. To date, the relative effect of different axle loadings has been based primarily on the results of one major controlled-load factorial experiment, the AASHO Road Test, which showed the relative damage to be related approximately to the fourth-power of axle load. Subsequent studies, both analytical and with accelerated controlled loading, have shown the effects of loading to vary considerably from this value under various circumstances, particularly in relation to the criteria chosen to define damage, but there has been general consensus that the fourth-power "law" is an adequate representation of damaging effects. However, the validity of this relationship under real road conditions has not previously been demonstrated.

The current study included an empirical evaluation of load-damaging effects on in-service road sections under real mixed traffic and aging conditions. Using the Brazil data base of widely varying axle load distributions, traffic volumes and pavement strengths, the effect of axle loading on each individual type of distress was tested statistically by varying the form of the traffic term in the estimation of the prediction models. The results indicated the relative damaging effect attributable to load on the progression of each mode of distress (independent of the maintenance intervention criterion which defined damage in the earlier AASHO-based functions).
Figure 10.1: Comparison of observed and predicted deterioration from combined predictive models for paved roads

(a) Area of All Cracking

(b) Roughness Progression

Note: Section 123 SEM CS, Brazil-UNDP study; SNC=3.6; Benkelman Deflection=1.0 mm; Traffic=560 veh/day, 30,000 ESA/lane/year.

Source: Analysis by submodel of HDM-III (Watanatada and others 1987a).
The damaging effects were found to fall in three categories, defined by
the major modes of distress. Axle loading had negligible influence on surfacing
disintegration modes, represented by ravelling but apparently also applying to
surface friction by association; potholing progression was also placed in this
category although there was no direct experimental evidence on this. In the cracking
mode of distress (excepting thermal and shrinkage-induced cracking which
are non-traffic-related), the relative damaging effect was best represented by the
second-power of axle load; this intermediate effect of loading on damage is
consistent with the fatigue mechanism in thin bituminous surfacings and there is
theoretical evidence that the power increases somewhat to at least three in thick
surfacings (over 100 mm thick). In deformation modes of distress, that is rutting
and roughness, the relative damage was related to the fourth-power of axle
load.

These empirical findings, determined with the aid of distress models
consistent with mechanistic principles of pavement behavior, are in general agree-
ment with a recent American theoretical mechanistic analysis (Rauhut and others
1984) with the exception that the theoretical study found slightly higher relative
load powers in the order of four to six for deformation-related damage. Given the
representation of the real effects of concurrent aging and mixed traffic in the
empirical study, the allocation of damage by loading power values of zero, two and
four for disintegration, cracking and deformation modes of distress respectively
is the strongest conclusion possible from current evidence. The relative costs
attributable to each of these modes will vary with pavement construction and traf-
fic through the relative rates of deterioration by each mode. Given the levels of
variation in damaging effect under particular circumstances it appears unrealistic
to define damage powers with more precision than given by the above, at least for
economic purposes at a network level.

A classification of axle load spectra, which is useful for identifying
those types of spectra for which the computation of equivalent axle loadings
(ESAn) is sensitive to the relative load damage power n, is summarized in Table
10.8.

Table 10.8: A classification of axle load spectra with respect to sensitivity of
the damaging power assessment

<table>
<thead>
<tr>
<th>Type of axle load spectrum</th>
<th>Characteristics of loads exceeding 80 kN</th>
<th>Typical ESAn ratios</th>
<th>Sensitivity of ESAn to n</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>% of HV</td>
<td>Mean load (kN)</td>
<td>ESA0</td>
</tr>
<tr>
<td>AA</td>
<td>&lt;5</td>
<td>&lt;95</td>
<td>&gt;50</td>
</tr>
<tr>
<td>A</td>
<td>2-15</td>
<td>&lt;100</td>
<td>20</td>
</tr>
<tr>
<td>B</td>
<td>3-20</td>
<td>&lt;105</td>
<td>8</td>
</tr>
<tr>
<td>C</td>
<td>5-35</td>
<td>105-120</td>
<td>2</td>
</tr>
<tr>
<td>D</td>
<td>&gt;10</td>
<td>&gt;120</td>
<td>1</td>
</tr>
</tbody>
</table>

Note: ESAn = equivalent single 80 kN axle loads per axle using a relative load
damage power of n. HV = heavy vehicles.

Source: Figures 9.3 and 9.4.
10.5 TRANSFERABILITY AND APPLICATION OF FINDINGS

The deterioration prediction models developed in this study have been statistically-estimated from data collected on in-service roads under experimentally-designed study conditions, and structured on mechanistic principles of pavement behavior. The external validation exercises have established the applicability of the models in a wide range of conditions, in most cases, and the model forms are thus considered to be suitable for application in regions other than the original study base in central Brazil. It is inevitable however that some adjustment will need to be made when applying the models elsewhere, because of differences in some influential factors that were not varied within the original study framework. In broad terms these factors include climate, certain materials and types of pavement construction, and construction techniques.

It is anticipated that most adjustments can be made by simple multiplicative factors, increasing or reducing the predicted rates of deterioration to levels observed in the region of application, and provision for such factors has been incorporated in the HDM-III model. The gathering of local data, to quantify such an adjustment rationally, requires carefully controlled methods. In many instances, the collection of time-series data over a period of five years will be infeasible, and an alternative is the use of data from a one-time survey and back-analysis, as undertaken for Tunisia on paved roads and Niger for unpaved roads, for example. In other instances, a suitable data base will evolve from regular monitoring of a network in regions where a pavement management system has been implemented. Where the condition parameters being measured have been selected and controlled to be compatible with the predictive model framework described here, such instances will provide an extremely rich basis for validating and improving the prediction of road deterioration in the future.
APPENDIX A

Summary of Distress Prediction Models from Selected Major Studies

A summary of the flexible pavement distress prediction models developed from five major pavement performance studies in the United States of America, Kenya and Brazil is presented. The selection is not exhaustive, but comprises studies based on large data bases which produced comprehensive sets of distress models and which are appropriate for cross-validation purposes. The five sets of models are as follows:

1. AASHTO Models: The model predicting pavement performance developed from the 1958-1960 AASHO Road Test in Illinois, U.S.A., and incorporated in the interim design guide of the American Association of State Highway and Transportation Officials (AASHTO) (AASHTO 1981), comprises one damage function for serviceability (only the flexible pavement model is considered here).

2. RTIM2 Models: The models used in the Road Transport Investment Model (RTIM2), of the British Transport and Road Research Laboratory (TRRL) Overseas Unit (Parsley and Robinson 1982), are based on the Kenya road costs study (Hodges, Rolt and Jones 1975), and comprise functions for roughness progression and cracking.

3. GEIPOT, Brazil Models: The models for paved road deterioration developed by Queiroz (1981) for the 1975 to 1981 Brazil-UNDP road costs study conducted by the Brazilian Transportation Planning Agency (GEIPOT) (GEIPOT 1982), comprise functions for roughness progression, cracking initiation and cracking progression.

4. Arizona DOT Models: The models, developed for a pavement management system by the Arizona Department of Transportation (ADOT) and Woodward-Clyde Associates (Way and Eisenberg 1980), were derived from two data bases sampling the Arizonan road network, and comprise functions for roughness progression, and cracking initiation and progression; and

5. Texas FPS Models: Developed for the Flexible Pavement Design System (FPS) at the Texas A&M University (Lytton, Michalak and Scullion, 1982) for the Federal Highway Administration (FHWA) and Texas State Department of Highways and Public Transportation (TSDHPT), the models were derived from samples of the Texan road network, and comprise functions for serviceability (roughness) and cracking progression.

A.1 AASHTO PERFORMANCE MODEL

The following, based on the AASHTO Interim Design Guide (Appendix C in AASHTO 1981) summarizes the model predicting the loss of serviceability, which is
closely related to roughness over a range of 0 to 8 m/km IRI, in terms of a dimensionless damage function, $G_t$. The damage function expresses the fractional loss of serviceability between "new" (original construction) and "terminal" (a selected criterion for rehabilitation intervention) conditions.

A.1.1 Serviceability Damage Function

The general damage function adopted at the AASHO Road Test (Highway Research Board, 1962) expressed deterioration as the fractional loss of serviceability relative to defined limits of serviceability for new and terminal conditions, and related damage to traffic loading as follows:

$$ g_t = \frac{N_t}{P_t} $$

where $g_t$ = damage at time $t$, ranging from 0 at new condition to 1 at terminal condition, where

$g_t = \frac{(P_0 - P_t)}{(P_0 - P_r)}$;

$P_0$ = initial serviceability;

$P_t$ = serviceability at time $t$;

$P_r$ = terminal serviceability, at which rehabilitation is required;

$N_t$ = number of axle load applications at end of time $t$;

$\beta$ = a function of design and load variables that influence the shape of the $p$-versus-$N$ serviceability curve; and

$\rho$ = a function of design and load variables that denotes the expected number of axle load applications to a serviceability index of 1.5.

For convenience, Equation A.1a is frequently transformed as follows:

$$ G_t = \beta (\log_{10} N_t - \log_{10} P_t) $$

where $G_t = \log_{10} g_t$.

At the AASHO Road Test, the terms $\beta$ and $\rho$ in Equation A.1 were related to the load and pavement variables for flexible pavements as follows:

$$ \beta = 0.081 (L_1 + L_2)^{3.23} $$

$$ (SN + 1)^{5.19} L_2^{3.23} $$

and

$$ \log_{10} \rho = 5.93 + 9.36 \log_{10} (SN + 1) - 4.79 \log_{10} (L_1 + L_2) $$

$$ + 4.33 \log_{10} L_2 $$

where $L_1$ = load on one single axle or on one tandem-axle set, kips;

$L_2$ = axle code (1 for single axle and 2 for tandem axle); and

$SN$ = structural number (see Section 4.3.4 for definition).

Since the equations for $\beta$ and $\rho$ both contain the terms $L_1$, $L_2$, and $SN$, the solution of Equation A.1 for $SN$ is an involved iterative process. The solution is simplified if all load factors are expressed in terms of a common denominator. The common denominator used in the guide is an 18,000 lb single-axle load and for these conditions, $L_1 = 18$ kips, $L_2 = 1$ in Equations A.2 and A.3. The "serviceability progression" function then becomes:
\[
\log N_{t_{18}} = 9.36 \log_{10} (SN + 1) - 0.20 + \frac{G_t}{0.40 + 1094 (SN + 1)^{-5.19}}
\text{(A.4)}
\]

where \( G_t = \log_{10} \left( \frac{4.2 - p_t}{4.2 - 1.5} \right) \)

\( N_{t_{18}} \) = number of 18 kip (80 kN) single-axle load applications to time \( t \).

In order to apply this function to pavements with soil and climatic conditions different from those of the AASHO Road Test site in Illinois, a Regional Factor (R) and Soil Support Value (\( S_I \)) are used to define functions modifying Equation A.4 as follows:

\[
\log N_{t_{18}} = 9.36 \log (SN + 1) - 0.20 + \frac{G_t}{0.40 + 1094 (SN + 1)^{-5.19}}
+ \log R + 0.372 (S_I - 3.0)
\text{(A.5)}
\]

Methods for quantifying R and \( S_I \) vary according to different agencies (see AASHTO 1981).

### A.1.2 Load Equivalence Factors

Mixed traffic with a range of different axle loadings and configurations are converted to 18,000 lbf equivalent single axle loads by the following equations (where the ratio \((N_t / N_{t_{18}})\) is often termed an "equivalence factor") (see also Liddle 1962):

For single axles \((L = 1)\),

\[
\log_{10} \left( \frac{N_t}{N_{t_{18}}} \right) = 4.79 \log_{10} (18 + 1) - 4.79 \log_{10} (L + 1)
+ G_t/\beta_x - G_t/\beta_{18}
\text{(A.6)}
\]

and, for tandem axles, \((L = 2)\):

\[
\log_{10} \left( \frac{N_t}{N_{t_{18}}} \right) = 4.79 \log_{10} (18 + 1) - 4.79 \log_{10} (L + 2)
+ 4.33 \log 2 + G_t/\beta_x - G_t/\beta_{18}
\text{(A.7)}
\]

### A.1.3 Serviceability Index

Serviceability ratings made by a panel of engineers on a scale of 0 (very poor) to 5 (excellent), when correlated to physical measures of pavement condition, yielded the following definition of "serviceability index":

\[
p = 5.03 - 1.91 \log (1 + SV) - 0.01 \sqrt{(C + P)} - 1.38 \text{ RD}^2
\text{(A.8)}
\]

where
- \( p \) = the present serviceability index;
- \( SV \) = the mean of the slope variance in the two wheelpaths \( \times 10^6 \);
C + P = a measure of cracking and patching in the pavement surface (ft² per 1,000 ft²); and
RD = average rut depth in the wheelpaths (inches).

A.2 RTIM2 MODEL

A.2.1 Roughness Progression

The model derived from the TRRL road costs study in Kenya (Hodges, Rolt and Jones 1975; Parsley and Robinson 1982) has a simple incremental form, with the change of roughness being a function of pavement strength and traffic loading as follows:

\[ R_t = R_0 + m N E_t \]  \hspace{1cm} (A.9)

where

- \( R_t \) = predicted roughness at time t (mm/km Bump Integrator trailer);
- \( R_0 \) = initial roughness at time t = 0, constant for given range of modified structural number, SNC;
- \( N E_t \) = cumulative traffic at time t, in millions of equivalent 80 kN standard axle loads (million ESA); and
- \( m \) = constant for a given range of pavement structural number.

In the original form of the model (Hodges and others 1975), the values of the parameters were fixed for ranges of SNC, as follows:

Given \( 2.75 < SNC < 3.25; R_0 = 2,500; m = 483; \)
\( 3.25 < SNC < 3.75; R_0 = 2,700; m = 159. \)

Subsequently, in the RTIM2 model (Parsley and Robinson 1982), \( m \) was modelled as a continuous function through the discrete ranges above, as follows:

\[ m = \frac{1250}{\text{antilog}_{10}(a - b - 1.3841)}; \]  \hspace{1cm} (A.10)

where

\[ a = [(0.20209 + 23.1318 c^2)^{0.5} - 4.8096 c]^{0.33} \]
\[ b = [(0.20209 + 23.1318 c^2)^{0.5} + 4.8096 c]^{0.33} \]
\[ c = 2.1989 - SNC \]

The range over which the model was estimated was limited to \( 2.75 < SNC < 3.75. \) The predictions of \( m \) in the model above, which has a cubic form, change rapidly for values of SNC less than 3. To compensate for this recognized limitation, the author modified the function for this study to improve the extrapolation to values of SNC less than 3.0, as follows:

\[ m' = m_a (3/SNC)^4 \]  \hspace{1cm} (A.11)

where

- \( m' \) = modified value of \( m \) valid for \( 1.5 < SNC < 3; \)
- \( m_a \) = value of \( m \) for SNC = 3.

The exponent of 4 and inverse proportionality form in the above relationship were adopted from the structural component of the Brazil model, and compares well with the form in the AASHTO Interim Design Guide (see Equation A.4) where:

\[ m = k/(1 + SN)^{5.2} \]

approximately, and SN is the (unmodified) structural number.
The Kenya prediction model can also be written in an incremental form, which is linear as follows:

\[
\Delta R_t = m' \Delta N E_t
\]

(A.12)

where

\(\Delta R_t\) = increment of roughness from any time to time \(t\);
\(\Delta N E_t\) = increment of cumulative ESA (in millions) between times \(t_0\) and \(t\); and
\(m'\) = modified value of \(m\).

A.2.2 Cracking

The Kenya model combines cracking initiation and progression in one relationship expressed in terms of cracking plus patching, as follows:

For \(SNC < 4.0\), \(C + P \geq 0\):

\[
(C + P)_{i} = 21600 \, N E_s \, SNC^{\frac{SNC}{SNC}}
\]

(A.13)

This is a linear model which in incremental form for cracking progression becomes:

\[
\Delta(C + P)_{i} = 21600 \, SNC^{\frac{SNC}{SNC}} \, \Delta N E_s
\]

(A.14)

By reduction, the occurrence of cracking initiation is expressed in terms of the cumulative number of ESAs applied, as follows:

\[
NCA = \max \{((4 / SNC) - 1 \, [SNC^{\frac{1 + SNC}{SNC}}]) / 72; 0\}
\]

(A.15)

where \((C + P)\) = sum of areas of cracking and patching (\(m^2/\text{km/lane}\))

(note: \(CR_{i} = (C + P)/3500\));

\(SNC\) = modified structural number;

\(NE_s\) = cumulative traffic loadings since latest resurfacing (million ESA); and

\(NCA\) = cumulative ESAs applied during the period before cracking initiation (million ESA).

A.2.3 Rutting

A model predicting rut depth was not estimated from the study.

A.3 QUEIROZ-GEIPOT MODELS

The prediction relationships developed by Queiroz (1981), presented in Volume 7 of the final report of the Brazil-UNDP study (GEIPOT 1982), were based on data collected over the period 1977 to 1980.

A.3.1 Roughness Progression

A number of relationships were estimated for different groups of explanatory variables, including the following (for 73 observations):

\[
QI_m = 12.63 - 5.16 \, RH + 3.31 \, ST + 0.393 \, AGE + 8.66 \, (LN/SNC)
+ 7.17 \times 10^{-9} \, (B \, LN)^2
\]

\((r^2 = 0.53; \text{S.E.} = 10.2 \text{ counts/km } QI_m)\)
MODELS FROM MAJOR STUDIES

\[ Q_{Im} = 21.8 - 7.52 \text{RH} + 5.16 \text{ST} + 0.515 \text{AGE} + 7.22 \times 10^{-8} (B \ln)^2 \]
\[ (r^2 = 0.48; \text{S.E.} = 10.6 \text{counts/km } Q_{Im}) \quad (A.17) \]

\[ LQI = 1.391 - 0.1315 \text{RH} + 0.0414 \text{P} + 0.00751 \text{AGE} + 0.0248 D \ln \]
\[ (r^2 = 0.32; \text{S.E.} = 0.13) \quad (A.18) \]

\[ LQI = 1.487 - 0.1383 \text{RH} + 0.00795 \text{AGE} + 0.0224 (\ln / \text{SNC})^2 \]
\[ (r^2 = 0.26; \text{S.E.} = 0.14) \quad (A.19) \]

where

- \( Q_{Im} \) = roughness, in counts/km \( Q_{Im} \) (see Section 2.3);
- \( LQI \) = logarithm to the base 10 of roughness, in counts/km \( Q_{Im} \);
- \( \text{SNC} \) = modified structural number of pavement;
- \( \ln \) = logarithm to the base 10 of the number of 80 kN cumulative equivalent axles;
- \( B \) = Benkelman beam mean deflection (0.01 mm);
- \( D \) = Dynaflect maximum deflection, surface curvature index, and base curvature index (0.001 in);
- \( \text{AGE} \) = surface age since construction or overlay (years);
- \( \text{P} \) = percent area of the pavement which received repairs in the form of deep patches (%);
- \( \text{ST} \) = surface type dummy variable, where \( \text{ST} = 0 \) asphalt concrete, and \( \text{ST} = 1 \) double surface treatment; and
- \( \text{RH} \) = state of rehabilitation dummy variable, where \( \text{RH} = 0 \) for original construction, and \( \text{RH} = 1 \) overlaid pavement.

A.3.2 Cracking

Relationships predict the number of equivalent 80 kN single axles to initiation of class 2 cracking (1 mm wide cracks), and the progression of cracking in terms of percentage of area cracked, as follows:

**Initiation of cracking**

\[ \log_{10} N_c = 1.205 + 5.96 \log_{10} \text{SNC} \]
\[ (\text{Sample} = 19 \text{ observations}; \ r^2 = 0.52; \text{S.E.} = 0.44) \quad (A.20) \]

where \( N_c \) = the number of ESAs to first crack; and \( \text{SNC} \) = modified structural number.

**Progression of cracking**

\[ \text{CR} = -18.53 + 0.0456 B \ln + 0.00501 B \text{AGE} \ln \]
\[ (\text{Sample} = 76 \text{ observations}; \ r^2 = 0.64; \text{S.E.} = 12.6); \quad (A.21) \]

\[ \text{CR} = -14.10 + 2.84 D \ln + 0.395 D \text{AGE} \ln \]
\[ (\text{Sample} = 76 \text{ observations}, \ r^2 = 0.44; \text{S.E.} = 15.8); \quad (A.22) \]

\[ \text{CR} = -57.7 + 53.5 \ln / \text{SNC} + 0.313 \text{AGE} \ln \]
\[ (\text{Sample} = 76 \text{ observations}; \ r^2 = 0.35; \text{S.E.} = 17.1); \quad (A.23) \]

where \( \text{CR} \) = amount of cracking, in percentage of area (equivalent to \( \text{CR}_a \), see Section 5.1.2);
MODELS FROM MAJOR STUDIES

B = mean surface deflection by Benkelman Beam (0.01 mm);
LN = logarithm to the base 10 of the number of cumulative equivalent axles (ESA);
AGE = pavement age since construction or overlay (years);
D = mean surface deflection by Dynaflect (0.001 in); and
SNC = modified structural number.

A.3.3 Rutting

No relationship was estimated for rut depth progression.

A.4 ARIZONA DOT MODELS

A.4.1 Roughness Progression

The model derived from Arizona data base "A" (Way and Eisenberg 1980), which comprised data over an observation period of only 1 to 2 years, is Markovian and annually recursive, predicting the roughness change in one year as a function of the previous roughness and an environmental constant, as follows:

\[ \Delta R_n = 0.138 R + 2.65 R G^2 - 0.047 R G - 0.125 \]  

(A.24a)

where \( \Delta R_n \) = predicted change in ride during next year (inch/mile);
R = ride at beginning of year as measured by ADOT Mays-Ride-Meter (inch/mile);
RG = regional factor specific to Arizona, as derived empirically by ADOT and computed by
RG = \( 0.1(ELEVFT + MAPIY + ZONE) \);
ELEVFT = elevation of road section (thousand feet), (Note: the range in Arizona was 500 to 9,000 ft);
MAPIY = mean annual precipitation (inch/year), (Note: the range in Arizona was 4 to 25 inch/year);
ZONE = selected integer representing other climatic effects such as temperature, number of freeze-thaw cycles, etc. (Note: the range in Arizona was from 1 (dry, no freezing) to 9 (freezing and frost-susceptible), see Figure 2 in Way and Eisenberg (1980).)

Arising from a verification study, utilizing Arizona data base "B" which had a 5 to 6 year observation period (Way and Eisenberg 1980, p. 11 ff.), the original model above was adjusted by a factor of 0.48. Thus the resulting prediction of annual roughness increment, \( \Delta R \), was as follows:

\[ \Delta R = 0.48 \Delta R_n \]  

(A.24b)

For analytical purposes, we wish to adapt the above model form of annual roughness increment to the roughness increment over any length of time. (This is the technique adopted in the Brazil analysis to minimize the influence of errors in the roughness measurement). We substitute

\[ \Delta R_i = (A R_{i-1} + B) k \]  

(A.25)

where \( A = 0.138 - 0.047 RG \)
B = \( 2.65 R G^2 - 0.125 \)
R_{i-1} = \text{roughness at end of year } i-1 \text{ (inch/mile)}

\Delta R_i = \text{increment of roughness during year } i \text{ (inch/mile)}.

Now the increment in the \( i \)th year \( \Delta R_i \) with respect to the roughness \( R_0 \) at the beginning of the initial year is:

\[
\Delta R_i = k (A R_0 + B) (A k + 1)^{i-1}
\]

(A.26)

Then the cumulative increase of roughness \( \Delta R_N \) over the period of \( N \) years from the initial year can be written as:

\[
\Delta R_N = \sum_{i=1}^{N} \Delta R_i
\]

\[
= \left[ \frac{(A R_0 + B)}{A} \right] \left[ (1 + A k)^N - 1 \right]
\]

(A.27)

It is also true in the general case of prediction between times \( t_1 \) and \( t_2 \), where

\[
\Delta R_{At} = \left[ \frac{(A R_1 + B)}{A} \right] \left[ (1 + A k)^{At} - 1 \right]
\]

(A.28)

where \( At = t_2 - t_1 \);

\( R_1 \) = roughness at time \( t_1 \); and

\( \Delta R_{At} \) = cumulative increase of roughness between \( t_1 \) and \( t_2 \).

(Note: The coefficients \( A \), \( B \) and \( k \) as given above, only apply when \( R \) is in ADOT inch/mile units).

**A.4.2 Cracking**

The Arizona PMS has a model for cracking progression and tabulated values for cracking initiation. The progression model is Markovian and annually recursive, and predicts the increment of cracking index as a function of the previous rate of cracking, the current cracking index, the regional factor and, in the case of overlays, the overlay thickness, as follows:

**Original construction**

\[
\Delta CR_i = \Delta CR_{i-1} \left( 1 + 0.031 CR_i \right) - 0.0059 CR_i^2 + 0.05 CR_i RG
\]

\[+ 0.01 RG^2 + 0.186 \]

(A.29)

**Overlays**

\[
\Delta CR_i = 0.52 \Delta CR_{i-1} + 0.068 \Delta CR_{i-1}^2 + 0.069 CR_i
\]

\[ - 0.003 CR_i^2 - 0.0034 HO^2 + 0.51 \]

(A.30)

where \( \Delta CR_i \) = increment of cracking index in \( i \)th year (percent);

\( \Delta CR_{i-1} \) = increment of cracking index in previous, \((i-1)\)th year (percent);

\( CR_i \) = cracking index at beginning of \( i \)th year (percent);

\( RG \) = ADOT regional factor (as defined for Equation A.24); and

\( HO \) = thickness of asphalt overlay (inch).
In the verification exercise, it was found that the predicted mean values required no correction but that the estimates became very poor for predictions beyond 20 years.

The age at which cracking initiation occurred appeared to be a function of the year in which the surfacing was constructed, but this was probably a "survivor-effect" that was observed. In practice, ADOT seem to use an age which is a fixed function of the regional factor, ranging from 8 to 10 years for a mountainous region (RG = 4.5 to 3.0), up to 16 to 22 years for a desert region (RG = 1.0 to 0.5), as shown in Table A.1.

A.4.3 Rutting

A model predicting rut depth was not estimated in the study.

Table A.1: Age of surfacing when cracking index is 10 percent for Arizona PMS

<table>
<thead>
<tr>
<th>Region</th>
<th>Regional factor</th>
<th>Predicted age (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Desert</td>
<td>0.5</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>13</td>
</tr>
<tr>
<td>Transition</td>
<td>2.0</td>
<td>12</td>
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<tr>
<td></td>
<td>2.5</td>
<td>11</td>
</tr>
<tr>
<td>Mountainous</td>
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<td>10</td>
</tr>
<tr>
<td></td>
<td>3.5</td>
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<td>4.0</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>4.5</td>
<td>8</td>
</tr>
</tbody>
</table>


A.5. TEXAS FLEXIBLE PAVEMENT DESIGN SYSTEM

The Flexible Pavement Design System (FPS) used in Texas incorporates pavement distress prediction models which were derived empirically from data collected on 337 road sections in Texas, each in one lane and 2 miles long. The current prediction models were developed at Texas A&M University by Lytton, Michalak and Scullion (1982) and are under further revision at present.

The basic model form is a modification of the AASHO Road Test damage model which introduces a variable sigmoidal shape to the trend of condition and thus permits the condition to approach an asymptotic value. A general pavement damage function, g', has the form:

\[ g' = \exp\left[-(\rho/N)^\beta\right] \]  
(A.31)

where \( \rho \) = a magnitude parameter to be estimated as a function of pavement and traffic variables;

\( \beta \) = a shape parameter of the condition trend curve to be estimated as a function of pavement and traffic variables (see diagram);

\( N \) = number of 80 kN equivalent single axle loads (ESA); and
An advantage of the form is the variety of trend shapes which is afforded by the parameter $\beta$ as shown in Figure A.1. However, the appropriateness of a sigmoidally-shaped function for roughness distress models is questionable (see Chapter 8). The parameter $\beta$ can be estimated from the ratio $(\Delta \log g'/\Delta \log N)$ as shown in the figure. Garcia-Diaz and Riggens (1984) describe a statistical procedure for estimating $\rho$, $\beta$ and the asymptotic distress value ($p_f$ in the case of serviceability).

A.5.1 Roughness Progression

Roughness is defined by serviceability loss through the damage function:

\[ g' = \frac{p_i - p}{p_f - p} \]  

(A.32)

where $p = \text{current serviceability (PSI)}$; $p_f = \text{asymptotic value of ultimate serviceability (PSI)}$.

Figure A.1: Characteristics of general damage function used in the Texas Flexible Pavement Design System (FPS)

(a) Influence of $\beta$ on Shape of Damage Function $g'$

(b) Basic Ranges of $\beta$ Parameter

Source: After Lytton, Michalak and Scullion (1982).
MODELS FROM MAJOR STUDIES

For all pavements, $\beta = 1$ by assumption. The other factors, $p$ from Equation A.31, and $p_f$, the ultimate serviceability (Equation A.32) have been estimated as follows:

For surface treatments:

\[ p = [0.29 - 0.0076 \text{TI} + 0.015 \text{H}_2 + 0.0004 \text{FTC} - 0.035 \text{DMD}] \times 10^6 \]  \hspace{1cm} (A.33)  
\[ p_f = 0.84 \]

For asphalt concrete on bituminous base:

\[ p = [0.34 + 0.0075 \text{H}' - 0.032 \text{PI}] \times 10^6 \]  \hspace{1cm} (A.34)  
\[ p_f = 0.055 (\text{H}')^{-1} \text{PI}^{0.867} \]

where  
- TI = Thornwaite Index (1948) + 50;  
- H2 = thickness of base layer (inch);  
- FTC = number of annual freeze-thaw cycles;  
- DMD = Dynaflect maximum deflection (0.001 inch);  
- PI = plasticity index of subgrade soil (percent); and  
- H' = transformed total thickness of pavement above rigid base (see Lytton, and others 1982) (inch).

A.5.2 Cracking

Surfacing distress was quantified by TTI as a decimal score ($s$) that was an approximately linear function of area and severity (class of crack) up to values of 0.5 for an area greater than 30 percent or severity of 4, which are summed to give a maximum value of 1.0 [In terms of the Brazil variables, 40 percent area at Class 4 cracking gives $s = 1.0$; 20 percent of area at Class cracking, $s = 0.4$].

For crocodile cracking in surface treatments:

\[ p = [-0.97 + 0.039 T + 0.0034 \text{T} + 0.018 \text{H}_2 - 0.0046 \text{LL} + 0.0056 \text{PI} + 0.0066 \text{FTC}] \times 10^6 \]  \hspace{1cm} (A.35)  
\[ \beta = 0.39 \text{PI}^{-0.63} \text{DMD}^{0.54} T^{1.02} \]

where  
- T = mean average monthly air temperature less 50° F; (^oF)  
- LL = liquid limit of subgrade soil (percent)

For transverse and longitudinal cracking, the variable N in the general model was defined as the number of months since the previous major maintenance or construction, and four models were cited (Lytton and others 1982).
APPENDIX B
Probabilistic Failure-Time Models for Predicting Surfacing Distress: Concepts and Theory

The initiation of surfacing distress, such as cracking, or ravelling, marks a significant stage in the deterioration of a pavement. From this point, the rate of deterioration usually accelerates at a rate that varies with traffic, pavement and climatic conditions. The timing of maintenance to control the deterioration is thus largely dependent upon the time of distress initiation. The fact that distress does not occur instantaneously over the entire length of roads under like conditions is also important because the needs for maintenance expenditure are thereby spread over time.

Described here is a statistical procedure for estimating probabilistic models of distress from field data, giving the capability of predicting failure times and the probabilities of distress appearing. The method, based on failure-time theory, incorporates the variability of pavement behavior, and represents the concurrent effects of traffic-related fatigue and time-related aging, which can vary considerably from region to region. The method was developed because the variability evident in real pavement data, and the fact that the time of appearance of distress usually could not be observed on all sections within a finite study period, were hindering the analysis of cracking and ravelling data from the Brazil-UNDP road costs study.

B.1 CONCEPTS OF FAILURE TIME AND VARIABILITY

The initiation of distress such as cracking or ravelling is a discrete but highly variable event. That is, a given type of distress will appear at different times at various locations along a nominally homogeneous road. We term the first of these appearances the initiation of that type of distress. Another pavement of nominally identical properties and traffic will have initiation at yet a different time $T_i$, where $i$ indexes the pavement section.

The time, $T_i$, or age of the surfacing at "failure" (here defined as the appearance of distress), thus varies in the real world, even when given nominally identical conditions. This can be represented by a probability-density function $f(t)$, as shown by a hypothetical example for cracking in Figure B.1(a). In the function drawn, the first crack is unlikely to appear within "A" years of surfacing construction and nearly certain to appear before the surfacing is "B" years old. On about one-half of all identical pavements, the first crack is likely to have appeared within "C" years. The probability or "chance" that the pavement will not have cracked by a certain age is represented by the survivor function $F(t)$ in Figure B.1(b). The location of the functions along the time axis

Note: This appendix in collaboration with Andrew D. Chesher, currently Professor of Econometrics at the University of Bristol, UK, who undertook the theoretical formulation, application and programming of the model presented here.
Figure B.1: Variability of failure times represented by probability density and survivor functions of the time to first appearance of distress

(a) Probability Density Function

(b) Survivor Function

Source: Author.
and their shapes can be expected to depend on the properties of the pavement and the intensity of traffic and loading stresses to which it is subjected.

In some instances, when modelling fatigue cracking, we may wish to use cumulative traffic (for example the cumulative equivalent standard axles) in place of chronological time, but the modelling principles are similar for both cases. In general, time is the most convenient unit for planning models and is used in the argument that follows.

In addition to the considerable variability in failure times, there is the difficulty of unobserved failure events in a typical set of pavement condition data, because data collection surveys are typically of limited duration. Amongst a uniform cross-section of pavements with a range of different ages, strengths and traffic loadings, some of the pavements will have been already cracked on the first survey date, some will begin to crack during the survey period, and on others cracking will begin only after the end of the survey, as shown in Figure B.2. If only the cracking initiation events observed during the survey were included in a statistical analysis, important information about the stochastic and mechanistic properties of the phenomenon coming from the "before" and "after" events may be excluded, and thus cause a bias in the model. These latter events, known as "censored data," can be of vital importance particularly in representing long-life pavements in an analysis.

Both features, namely stochastic variations and censored data, were addressed by developing an estimation procedure based on the principles of

Figure B.2: Unobserved or "censored" data of distress initiation and progression: example of three pavement sections with prior, observed and future failure events respectively

Source: Author.
failure-time analysis, originally developed to study the reliability of industrial components. The procedure uses the statistical method of maximum likelihood estimation to exploit both censored and uncensored data, as described in the following section, and a flexible form of distribution which enables the variability of failure-times to be determined by the data, as outlined in the succeeding section.

B.2 CENSORED DATA: MAXIMUM LIKELIHOOD ESTIMATION

We define \( T \) as the time from construction of a section of surfacing to failure, where \( T \) is a random variable indexed by \( i \), a section identifier to indicate that the distribution of \( T \) depends on section characteristics. The term failure is used to describe the first appearance of the mode of pavement distress which is of interest, for example narrow cracking, wide cracking or ravelling.

We regard \( T \) as a continuous non-negative random variable and denote its probability density function by \( f(t) \), its distribution function by \( F(t) \), and its survivor function by \( F(t) \), which is the probability of the failure time \( T \) occurring after the point in time \( t \), as shown in Figure B.1(b) for example, where \( F(t) = P(T > t) = 1 - F(t) \).

Suppose a road section is selected at random, initially observed \( S_0 \) years after surfacing, and finally observed \( S_1 \) years after surfacing (where \( S_1 > S_0 \)). One and only one of the following events may be observed:

1. \( T < S_0 \), i.e., failure occurred prior to observing the road. In this case define \( D_1 = 1 \), otherwise define \( D_1 = 0 \).

2. \( S_0 \leq T < S_1 \), i.e., failure occurred while the road was observed. In this case define \( D_2 = 1 \), otherwise define \( D_2 = 0 \). Let \( z \) be the observed value of \( T \).

3. \( S_1 < T \), i.e., failure would occur in the future after the road was last observed. In this case define \( D_3 = 1 \), otherwise define \( D_3 = 0 \).

Let \( T = \) \( S_0 \) if \( D_1 = 1 \),
\( = z \) if \( D_2 = 1 \), and
\( = S_1 \) if \( D_3 = 1 \).

Selecting a road at random we obtain values for \( D_1, D_2, D_3 \) and \( t \).

In order to exploit data on all sections it is necessary to develop a maximum likelihood estimator. Accordingly, we consider the joint probability - probability density function of the discrete \( D_1, D_2 \) and \( D_3 \) and the continuous \( t \).

Standard probability theory gives \( D_1, D_2 \) and \( D_3 \) as multinomially distributed with \( P(D_1 = 1) = F(S_0) \), \( P(D_2 = 1) = F(S_1) - F(S_0) \), and \( P(D_3 = 1) = F(S_1) \). Conditional on \( D_1 = 1 \) or \( D_3 = 1 \), \( t \) is either \( S_0 \) or \( S_1 \) with probability 1 in each case. Conditional on \( D_2 = 1 \), \( t \) has the truncated probability density function \( (f(t) / [F(S_1) - F(S_0)]) \). Multiplying marginal and conditional probabilities gives:

\[
P(D_1 \cap D_2 \cap D_3 \cap t) = F(S_0)^{D_1} f(t)^{D_2} \frac{f(S_1)}{F(S_1)}^{D_3} = F(t)^{D_1} f(t)^{D_2} f(t)^{D_3} \frac{F(t)}{F(t)} \quad (B.2)
\]
Now write $f(t)$ as a conditional failure time - probability density function, depending on section characteristics $x$ and parameters $\theta$. Then:

$$ P(D_1 \cap D_2 \cap D_3 \cap t | x, \theta) = F(t | x, \theta)^{D_1} f(t | x, \theta)^{D_2} F(T | x, \theta)^{D_3} \quad (B.3) $$

Estimation of $\theta$ can be achieved by calculating the maximum likelihood estimator. Index $t$ and $x$ by $i$ which distinguishes sections. Then the probability - probability density function of the observed $D_i$'s, $t_i$'s, $D$ and $t$, given the $x$'s, $x$, and $\theta$ is:

$$ P(D \cap t | x, \theta) = \prod_{i=1}^{n} \left[ F(t_i | x_i, \theta)^{D_i} f(t_i | x_i, \theta)^{D_i} \right] F(t_i | x_i, \theta)^{D_i} \quad (B.4) $$

Taking logs we obtain Equation B.5, the log-likelihood function. The maximum likelihood estimator $\hat{\theta}$ is that value of $\theta$ which maximizes Equation B.5. $\theta$ must be obtained using numerical methods. Under fairly general conditions, $\theta$ is consistent and efficient. The variance covariance matrix of $\theta$ is estimated by minus the inverse of the Hessian of Equation B.5 at $\theta = \theta$. See Rao (1973) or Theil (1971) for further details of the properties of $\theta$.

$$ L(\theta | D, t, x) = \sum_{i=1}^{n} \left[ D_i \log F(t_i | x_i, \theta) + D_i \log f(t_i | x_i, \theta) + D_i \log F(t_i | x_i, \theta) \right]. \quad (B.5) $$

The log likelihood function Equation B.5 is maximized by some variant of the Newton Raphson procedure. A program was developed which maximizes Equation B.5 either by Newton Raphson as modified by Berndt-Hall-Hall & Hausman (1974) or by steepest ascent.

The Berndt, Hall, Hall & Hausman (BHHH) modification of the Newton Raphson procedure makes use of the identity:

$$ E \left[ \frac{-1}{\delta \theta} \frac{\partial^2 L}{\partial \theta \partial \theta'} \right] = E \left[ \left. \frac{-1}{\delta \theta} \frac{\partial L}{\partial \theta} \frac{\partial L}{\partial \theta'} \right| \right] \quad (B.6) $$

so that the matrix of second derivatives

$$ E \left[ \frac{\partial^2 L}{\partial \theta \partial \theta'} \right] $$

is replaced by the approximation

$$ -n^{-1} \sum_{i=1}^{n} \left[ \frac{\partial L_i}{\partial \theta} \frac{\partial L_i}{\partial \theta'} \right], $$

where $\frac{\partial L_i}{\partial \theta}$ is the $i$th term in the summation $\frac{\partial L}{\partial \theta} = \sum_{i=1}^{n} \frac{\partial L_i}{\partial \theta}$. 
The program uses analytic expressions for first derivatives of the log likelihood function. At the termination of the optimization, the Hessian associated with the log likelihood function is calculated by differencing the analytic first derivative vector. Eigenvalues are calculated to check on the definiteness of the Hessian and the Hessian is then inverted to obtain estimates of asymptotic variances and covariances of estimated coefficients. Various predictions are provided for each observation so that the equivalent of residuals can be examined. The program will calculate asymptotic confidence intervals around predictions and provide various graph plots if required.

Asymptotic confidence intervals for expected failure times are provided by exploiting the local large sample linearity of the expression for the expected failure time, given pavement characteristics, and by computing the asymptotic variance of the resulting linear approximation. 1.96 times the square root of this variance is then added to, and subtracted from, the predicted expected failure time to give the required interval.

Starting values for the parameters are provided automatically but there is provision for using a manual start if required. The program is written in SAS's MATRIX procedure (1979).

B.3 MEAN FAIURE TIME AND VARIABILITY

The underlying variation of failure times was assumed in the failure-time model to follow a Weibull distribution (for which, general statistical results may be found in Chapter 20, Volume 1 of Johnson and Kotz, 1970). A log normal distribution is also available in the program, but the Weibull distribution was considered the most representative of the joint mechanisms of fatigue and aging, for the following reasons.

The time to failure of a section is the first failure to occur amongst all individual elements of the surfacing, where each fails at a time following some probability law, as illustrated in Figure B.3(a). The Weibull distribution, being a Type 3 extreme value distribution, is suitable for determining the minimum (or limiting distribution) of a series of minima (that is the failure times).

The mechanisms of fatigue under traffic and oxidation, which reduces the available fatigue life, work concurrently, and thus the probability of cracking occurring in the surfacing is expected to increase as the pavement ages. For example, the chances that a pavement will crack in its fifteenth year, if it has not already cracked by that time, are considered greater than the chances of its cracking in, say, the previous year. This can be described as an increasing "hazard" of cracking. The hazard function \( h(t) \), a concept used in reliability theory, is proportional to the probability that failure will occur in a short time interval at time \( t \) given that it has not occurred previously. It is defined by:

\[
h(t) = \frac{f(t)}{1 - F(t)} = \frac{f(t)}{F(t)}
\]

where \( f(t) \) = probability density function associated with \( T \); \( F(t) \) = probability distribution function, \( = P(T \leq t) \); and \( F(t) = P(T \geq t) \).
Figure B.3: Two hypotheses on the probability of the appearance of cracking which indicate that a Weibull distribution is representative of pavement failure times

(a) Cracking initiation in a nominally homogeneous section represented by elements over which pavement properties are randomly distributed. Failure occurs at the minimum of the failure times of all elements.

\[ T = \min \{ T_j \mid j = 1 \text{ to } J \} \]

(b) Fatigue is a hazard which increases monotonically with time or with cumulative number of axle transits (we expect \( \beta > 0 \)).

Source: Author.
To model fatigue therefore, the hazard is expected to be an increasing function of time as illustrated in Figure B.3(b). When the hazard is specified by

\[ h(t) = \alpha^{-\beta} t^{\beta-1} \quad \alpha, t > 0 \text{ and } \beta > 1, \]

then the probability distribution function is defined by:

\[ F(t) = 1 - \exp \left\{ - \frac{1}{\beta} \alpha^{-\beta} t^{\beta} \right\} \quad (B.9a) \]

and the probability density function, by:

\[ f(t) = -\beta t^{\beta-1} \exp \left\{ - \frac{1}{\beta} \alpha^{-\beta} t^{\beta} \right\} \quad (B.9b) \]

where \( \alpha \) = function of characteristics causing failure;
\( \beta \) = curvature of hazard function; and
\( t \) = time (or cumulative traffic).

This is the distribution function associated with a Weibull distribution describing a positively-skewed distribution over the non-negative real axis. The expected, or mean, time to failure \( E(T) \) is given by:

\[ E(T) = \frac{\alpha}{B(\beta)} \quad (B.10a) \]

where \( B(\beta) \) is a constant function of \( \beta \) given by:

\[ B(\beta) = \beta^{(1-\beta)/\beta} \Gamma(1/\beta) \quad (B.10b) \]

and selected values of \( B(\beta) \) are given in Table B.1.

The selection of the type of distribution is important because results are sensitive to it when many of the data have censored values. The Weibull distribution seems appropriate for representing the variability of failures for the two conceptual reasons just outlined. It also is flexible as it describes a family of skewed curves for different values of \( \beta \) and \( \alpha \) that seem realistic for pavement data, as shown in Figure B.4. The \( \beta \) parameter determines the shape of the distribution, which becomes narrower as \( \beta \) increases. The \( \alpha \) parameter is a scaling function which locates the distribution along the time axis. As both the \( \beta \) and \( \alpha \) parameter values are estimated from the observed data, and neither is fixed, the Weibull model is particularly adaptable to actual circumstances.

The objective of the estimation is not only to determine the average time to failure and its distribution, but also to estimate how the expected time to failure depends on pavement and traffic characteristics. These are represented in the \( \alpha \) parameter of the model which must be non-negative. Although other forms could be used, \( \alpha \) here was defined by the vector:

\[ \alpha = \exp \left\{ [X' \ y] \right\} \]

\[ = \exp \left\{ (\gamma_0 + \gamma_1 x_1 + \gamma_2 x_2 + \gamma_3 x_3 \ldots) \right\} \quad (B.11) \]

where \( y = \) vector of coefficients \( \gamma_i \); and
\( x = \) vector of parameters (pavement strength, traffic flow, etc.).
Figure B.4: Family of probability density functions represented by Weibull distributions of time to failure

Source: Equations 8.9a, 8.10a.

Table B.1: Table of constants representing Weibull distribution of failure times: function E(β), and factors K(β) and F(β) of expected failure time K(T), for various probabilities and values of β

| BETA (β) | 0.50 | 0.55 | 0.60 | 0.65 | 0.70 | 0.75 | 0.80 | 0.85 | 0.90 | 0.95 | 1.00 | 1.05 | 1.10 | 1.15 | 1.20 | 1.25 | 1.30 | 1.35 | 1.40 | 1.45 | 1.50 | 1.55 | 1.60 | 1.65 | 1.70 | 1.75 | 1.80 | 1.85 | 1.90 | 1.95 | 2.00 | 2.05 | 2.10 | 2.15 | 2.20 | 2.25 | 2.30 | 2.35 | 2.40 | 2.45 | 2.50 |
|----------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|
| B        | 0.505| 0.510| 0.514| 0.517| 0.519| 0.520| 0.521| 0.522| 0.522| 0.521| 0.520| 0.518| 0.515| 0.512| 0.508| 0.504| 0.500| 0.496| 0.492| 0.488| 0.484| 0.480| 0.476| 0.472| 0.468| 0.464| 0.460| 0.456| 0.452| 0.448| 0.444| 0.440| 0.436| 0.432| 0.428| 0.424| 0.420|
| K10      | 0.005| 0.010| 0.014| 0.017| 0.019| 0.021| 0.022| 0.022| 0.022| 0.022| 0.021| 0.020| 0.018| 0.016| 0.014| 0.012| 0.010| 0.008| 0.006| 0.004| 0.002| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000|
| K25      | 0.005| 0.010| 0.014| 0.017| 0.019| 0.021| 0.022| 0.022| 0.022| 0.022| 0.021| 0.020| 0.018| 0.016| 0.014| 0.012| 0.010| 0.008| 0.006| 0.004| 0.002| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000|
| K50      | 0.005| 0.010| 0.014| 0.017| 0.019| 0.021| 0.022| 0.022| 0.022| 0.022| 0.021| 0.020| 0.018| 0.016| 0.014| 0.012| 0.010| 0.008| 0.006| 0.004| 0.002| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000|
| K90      | 0.005| 0.010| 0.014| 0.017| 0.019| 0.021| 0.022| 0.022| 0.022| 0.022| 0.021| 0.020| 0.018| 0.016| 0.014| 0.012| 0.010| 0.008| 0.006| 0.004| 0.002| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000|
| K95      | 0.005| 0.010| 0.014| 0.017| 0.019| 0.021| 0.022| 0.022| 0.022| 0.022| 0.021| 0.020| 0.018| 0.016| 0.014| 0.012| 0.010| 0.008| 0.006| 0.004| 0.002| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000|
| F5       | 0.005| 0.010| 0.014| 0.017| 0.019| 0.021| 0.022| 0.022| 0.022| 0.022| 0.021| 0.020| 0.018| 0.016| 0.014| 0.012| 0.010| 0.008| 0.006| 0.004| 0.002| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000|
| FM       | 0.005| 0.010| 0.014| 0.017| 0.019| 0.021| 0.022| 0.022| 0.022| 0.022| 0.021| 0.020| 0.018| 0.016| 0.014| 0.012| 0.010| 0.008| 0.006| 0.004| 0.002| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000|
| FL       | 0.005| 0.010| 0.014| 0.017| 0.019| 0.021| 0.022| 0.022| 0.022| 0.022| 0.021| 0.020| 0.018| 0.016| 0.014| 0.012| 0.010| 0.008| 0.006| 0.004| 0.002| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000| 0.000|

(Table continues next page.)
Table B1: Contd.

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<td>1.5069</td>
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<td>1.0369</td>
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</table>

Source: Equations B.10(b), B.13(b), B.15.
Given various pavement, traffic and environmental parameters $x_i$, the model estimates the coefficients $y_j$, and the shape parameter $\beta$, which is assumed constant for all pavements within the data set.

The mean time to failure, $E(T)$, from Equation B.10, can be used directly in road deterioration predictions in the same way as the mean time from a deterministic model. In practice then, the prediction of the expected time to failure took the following simple form:

$$E(T) = B(\beta) \exp [y_0 + y_1 x_1 + y_2 x_2 \ldots]$$  \hspace{1cm} (B.12)

The probability with which failure might occur at some time other than the expected time derives from the value of $\beta$, shown for example in the family of probability functions generated by different values of $\beta$ in Figure B.4. At low values of $\beta$, say below 2.5, the distributions tend to be widely-dispersed and highly skewed with more than half of the observations having values less than the mean but with a number of very late failures. At high values of $\beta$, the distributions tend to be more concentrated and centered about the mean.

In general, the failure time, $T_p$, associated with any probability, $p = P[T < T_p]$, can be related directly to the mean $E(T)$ by:

$$T_p = K(p) \cdot E(T)$$  \hspace{1cm} (B.13a)

where

$$K(p) = [-\beta \ln(1-p)]^{1/\beta} / B(\beta).$$  \hspace{1cm} (B.13b)

$K(p)$ is a constant function of $\beta$, and values for selected values of probability $p$ are given in Table B.1. The median time to failure, $M(T)$, and lower quartile time, $T_{0.25}$, are thus given by:

$$M(T) = T_{0.5} = K(0.5) E(T)$$  \hspace{1cm} (B.14a)

$$T_{0.25} = K(0.25) E(T)$$  \hspace{1cm} (B.14b)

Thus, when $\beta = 2.5$, $M(T) = 0.973 E(T)$ and $T_{0.25} = 0.685 E(T)$; or in other words, 75 percent of the pavements are expected to survive at least 0.685 $E(T)$ years before distress appears.

For practical purposes the width of dispersion is most simply described by the spread of time between two specific probabilities, usually the quartiles. For convenience we define the semi-interquartile factor (SIQF) to represent half the age range between the 25 percent and 75 percent failure probabilities, that is the range either side of the median within which about 25 percent of the pavements may be expected to fail, i.e., $(1 \pm \text{SIQF}) E(T)$, where

$$\text{SIQF} = [K(0.75) - K(0.25)]/2.$$  \hspace{1cm} (B.15)

For example, when $\beta = 2.5$, $\text{SIQF} = 0.300$.

**B.4 APPLICATION IN THE HDM MODEL**

When evaluating pavement maintenance strategies, we need to take into account that failures are spread randomly across time so that not all the length of roads in a network under similar conditions fail simultaneously. Thus mainte-
nance expenditures for a given category of pavement and traffic tend in practice to be distributed over time also. In the previous structure of the HDM model (version II), road deterioration predictions were deterministic which led to the lumping of expenditures when long lengths of road in a network would apparently reach critical condition and require maintenance all at one time. When the introduction of distributed deterioration predictions was considered during the revision of the model for version III, the necessary restructuring of the model was considered prohibitive. For such a purpose, a dynamic decision model using a Markovian process, as proposed for example by Kulkarni (1984), seems appropriate.

Therefore, in order to incorporate some effects of stochastic variations of performance into the HDM model (version III), a simplified approach was adopted. Distributed times for the initiation of surfacing distress were incorporated by subdividing each homogeneous section into three subsections of equal length, representing segments of pavement with:

1. Short life or early failure times, for which the probability of failure having occurred is less than or equal to one-third;

2. Average life or median failure times, in which the probability of failure having occurred is more than one-third and not greater than two-thirds; and

3. Long life or late failure times, in which the probability of failure having occurred earlier is greater than two-thirds.

This is illustrated in Figure B.5. In the figure, (a) shows the road divided in equal segments of short, average and long lives; this is of course a conceptual idealization because the segments of roads in a given category that have short lives would not be contiguous but be scattered along the "link" or over different roads within that category. The probability distribution function is shown in (b) of the figure. The identity between the "length" of road in each failure category and the fraction of the probability distribution rests on the assumption of either a large number of roads of similar characteristics, or a long length of road, within the pavement/traffic category.

The model then considers the three subsections separately, so we wish to know the expected time to failure of each third, as shown in Figure B.5(b). This requires determining the means of truncated portions of the Weibull distributions, calculated as follows. We require \( E(T | a < T < b) \), that is the expected life of roads that fail after \( a \) years and before \( b \) years (in particular for the early third \( a = 0 \), and for the late third \( b = \infty \)). In general we have:

\[
E(T | a < T < b) = \int_a^b \frac{t f(t) \, dt}{F[a < T < b]} = I(a, b)
\]  

where in this case, \( P[a < T < b] = 1/3 \); and \( a \) and \( b \) are given by

- short-life: \((a, b) = (0, \infty)\)
- mid-life: \((a, b) = (t_{0.33}, \infty)\)
- long-life: \((a, b) = (\infty, \infty)\).
Figure B.5: Approximation of the variation in failure times as adopted for the HDM-III model: subdivision of nominally homogeneous pavement into three equal lengths with short, medium and long lives.

(a) Segmented Distribution of Failure Times along Length of Road

Nominally Homogeneous Road: Equal Thirds of Length, L

Short Life | Medium Life | Long Life

\[ P(T < t) \leq 1/3 \]

\[ P(T < t) > 2/3 \]

(b) One-Third Portions of Probability Distribution

Cumulative Probability, \( F(t) \)

\[ F(T_{0.33}) \]

\[ F(T_{0.67}) \]

Source: Author.
Now for the Weibull distribution (from Equation B.9b):

\[
I(a, b) = 3 \int_a^b t^\beta \exp\left(-\frac{a}{t^\beta}\right) dt
\]

\[
= 3 \left(\frac{1}{\beta}\right)^{1/\beta} \int_{u a}^{u b} \frac{1}{\beta} (1 + 1) - 1 \exp(-w) dw,
\]

(B.17)

after the transformations

\[
w = \frac{a^\beta}{\beta} t^\beta \quad \text{and} \quad u = \frac{a^\beta}{\beta}.
\]

This has the form of an incomplete Gamma function which was solved by a Taylor Series expansion, viz.:

\[
I(a, b) = 3 \sum_{n=0}^{\infty} \frac{(-1)^n}{\Gamma(1 + n + 1/\beta)} \left[-n(1 - p)\right]^{1 + n + 1/\beta}
\]

(B.18)

This reduces again to a direct proportion of the expected time to failure of the whole population through the identity

\[
u^{-1/\beta} = a^\beta u^{-1/\beta} = \frac{\beta^1/\beta}{B(\beta)} E(t).
\]

Thus the expected failure time, for example, for the short-life segments, \(E(t_s)\) is given by:

\[
E(t_s) = F_s E(t), \quad (B.19)
\]

where \(p = 0.33\)

\[
F_s = 3 \frac{\beta^{1/\beta}}{B(\beta)} \sum_{n=0}^{\infty} \frac{(-1)^n (0.4055)^{1+n+1/\beta}}{n!}
\]

(B.19)

The factor \(F_s\) converges to within 10\(^{-5}\) in \(n = 5\) terms. The factor \(F_\ell\) for the mean life of the long-life segments, \(E(t_\ell)\), is determined likewise, the probability constant \((-n(1-p))\) being 1.0986 instead of 0.4055 and convergence requiring 7 terms in the expansion. The factor \(F_m\) for the mean life of the mid-life segments, \(E(t_m)\), is given by:

\[
F_m = 3 - F_s - F_\ell
\]

(B.20)

Values for all three factors are given as a function of \(\beta\) in Table B.1. A plot of the values in Figure B.6 illustrates how the width of dispersion diminishes as the value of \(\beta\) increases.

B.5 GOODNESS OF FIT

The goodness of fit of probabilistic models requires special interpretation because two components are present: the probability of failure occurring by the estimated time, and the error of estimate.
Figure B.6: Trend of factors describing the expected life of truncated thirds of a Weibull distribution, with value of shape parameter, $\beta$

Note: Expected time to failure of one-third segments:
- Short-Life: $E(T_s) = FS \cdot E(T)$
- Medium-Life: $E(T_m) = FM \cdot E(T)$
- Long-Life: $E(T_l) = FL \cdot E(T)$

where $E(T)$ is the expected time to failure of the whole length.


First, we note that whereas the traditional measures of goodness of fit indicate what proportion of the variation in the dependent variable can be accounted for by variation in the independent variables, in failure time models there is an upper bound on this proportion. In principle, the exact time to failure is not predictable because the failure time is a random variable distributed over the positive axis, and thus $r^2$ and similar measures can never approach 1.0. The model explains the variation of failure times that can be attributed to pavement and traffic characteristics, but further than that the influence of chance ensures that 100 identical pavements will crack at 100 different times. No model explains the latter variation, but the Weibull model predicts that it exists and estimates the shape of its distribution through the $\beta$ parameter. This is typified by the semi-interquartile factor (SIQF) defined in Equation B.15.

Second, the value of log likelihood (LL) which is maximized in the estimation is not a dimensionless proportion like $r^2$; its value varies with the magnitude of the dependent variable and with the number of observations, so that it is meaningless to compare LL values across models or groups of models (except where identical sets of dependent variable are involved). When the dependent variable is of a fixed dimension, it is useful to define an average log likelihood (ALL) (LL divided by the number of observations) as a normalized measure across models having different number of observations within the same data set.
Third, since the model used here is not a linear regression model, there is no obvious equivalent of the conventionally reported "standard error". In order to assess the predictive power of the model, confidence intervals (with asymptotic validity) around predicted expected failure times were calculated as described earlier. As the size of the interval depends in part on the error of estimate of the parameter coefficient, it varies also with the values of the pavement parameters. Thus the intervals do not form a locus of the expected time to failure but instead form loci of the explanatory parameters. As a practical measure, we compute an "average confidence interval," being the arithmetic average of the estimated 95th percentile confidence intervals of all observations, expressed in units of the dependent variable, for example ± 1.2 years. While this is not a precise statistic, it provides a meaningful estimate of the intervals to be expected given the observed ranges of explanatory variables.

Finally, assessing the adequacy of models for censored data remains an open research question, recently addressed by Chesher and Lancaster (1985), for example. Future applications of the procedure should consider these developments. In this instance the three practical measures noted above have been used, i.e.,

1. Goodness of fit: average log likelihood value (for internal best fit for given data set and dependent variable);

2. Predictive power: average 95th percentile confidence intervals (average for given data set); and

3. Stochastic variation: Semi-interquartile probability factor, (SIQF); for example, a value of SIQF = 0.36 for β = 2.0 indicates a fairly wide dispersion, and a value of 0.09 for β = 10 indicates a very narrow dispersion (meaning that like roads will fail at very similar times).
APPENDIX C

Formulation of Models Predicting
Initiating of Cracking

The following tables present a selection of the models estimated by the failure-time theory outlined in Appendix B, and leading to the final selections presented and discussed in Chapter 5. Only models in which all parameters were significant (asymptotic t-statistics of 2 or more) are included, except in a few instances when it was considered useful to indicate how certain apparently important parameters proved to be not significant in certain formulations. The parameter definitions are as follows:

**Parameters**

- **TYCR2** - surfacing age at initiation of CR2 cracking, years;
- **TYCR3** - surfacing age at initiation of CR3 cracking, years;
- **TYCR4** - surfacing age at initiation of CR4 cracking, years;
- **TE2** - cumulative ESA2 at initiation of CR2 cracking, million;
- **TE4** - cumulative ESA4 at initiation of CR4 cracking, million;
- **YE4** - ESA4 (computed with damage power 4), millions/lane/year;
- **YE2** - ESA2 (computed with damage power 2), millions/lane/year;
- **YHX** - number of heavy vehicle axles, millions/lane/year;
- **YAX** - number of all axles, millions/lane/year;
- **H** - thickness of surfacing, mm;
- **SN** - structural number of pavement (excluding subgrade);
- **SNC** - modified structural number of pavement (in situ);
- **DEF** - Benkelman beam surface deflection 80 kN axle load, mm;
- **DMD** - Dynaflect maximum deflection, 0.1 mm;
- **DSCI** - Dynaflect surface curvature index, .01 mm;
- **EH1** - maximum horizontal tensile strain at base interface, 10^-3;
- **EHM** - maximum tensile strain in surfacing, 10^-3;
- **BNO** - deviation of binder content from optimum, as fraction of optimum;
- **RMOD** - resilient modulus of asphaltic materials at 30°C, in MPa;
- **CUM** - sum of all patched areas;
- **CQ** - = 1 if surface has original construction defects, = 0 otherwise;
- **CM** - = 1 if surface is opengraded cold mix, = 0 otherwise;
- **SLU** - = 1 if surface is slurry seal, = 0 otherwise;
- **CBR2** - in situ CBR of basecourse layer, percent;
- **COMF** - average compaction;
- **PLS** - plastic index of soil;
- **GI** - group index of soil;
- **CHOD** - resilient modulus of cemented base, GPa;
- **SIQF** - semi-interquartile factor, defining the dispersion of observations about the mean, see Equation B.15, p. 421.
- **ACI** - average confidence interval, see Section B.5.
- **B(β)** - constant, see Equation B.10.
Table C.1: Examples of analytical development of prediction models for the initiation of cracking: asphalt concrete pavements

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<th>Dependent variable</th>
<th>B(g)</th>
<th>Model parameter estimates (t-statistics in parentheses)</th>
<th>Model statistics</th>
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<td>exp(2.037 - 0.072 H + 0.00064 H^2 + 0.527 SN - 2.48 YE4)</td>
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**Note:** Parameter names are defined on p. 427. SIQF = semi-interquartile factor. ALL = average log likelihood. ACI = average confidence interval of expected life. B, B(g), SIQF, ALL and ACI are defined in Appendix B.

**Source:** Estimation of failure-time models (Equation 5.13) on Brazil-UNDP study data.
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<td>[ \begin{align*} (13.1) &amp; \quad (5.2) \ (10.2) &amp; \quad (2.6) \ (1.3) &amp; \quad (1.4) \end{align*} \right]</td>
<td>[ \begin{align*} 4.65 &amp; \quad 36 \ -1.580 &amp; \quad 0.168 \end{align*} \right]</td>
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<td>1.284 exp(7.470 - 0.633 Q - 0.30 H + 0.004 H² + 0.5 Y64 - 0.026 SN + 0.005 QM)</td>
<td>[ \begin{align*} (3.7) &amp; \quad (3.5) \ (2.9) &amp; \quad (3.0) \ (0.5) &amp; \quad (0.2) \ (3.3) &amp; \quad (6.0) \end{align*} \right]</td>
<td>[ \begin{align*} 3.77 &amp; \quad 78 \ -1.533 &amp; \quad 0.205 \end{align*} \right]</td>
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<td>1.226 exp(-0.242 + 1.99 Y6K)</td>
<td>[ \begin{align*} (0.3) &amp; \quad (2.4) \ (10.4) &amp; \quad (8.4) \end{align*} \right]</td>
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<td>[ \begin{align*} (9.0) &amp; \quad (1.6) \ (5.8) &amp; \quad (0.02) \ (2.8) &amp; \quad (8.4) \end{align*} \right]</td>
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<td>[ \begin{align*} (11.3) &amp; \quad (1.8) \ (4.6) &amp; \quad (2.1) \ (1.4) &amp; \quad (3.2) \end{align*} \right]</td>
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<td>[ \begin{align*} 5.17 &amp; \quad 40* \ -1.295 &amp; \quad 0.152 \end{align*} \right]</td>
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<td>1.288 exp(2.20 - 1.50 C - 0.27 SLU - 0.72 Q + 0.006 lnY64 - 0.43 lnDEF)</td>
<td>*(Q = 0)</td>
<td>[ \begin{align*} 3.20 &amp; \quad 142 \ -1.251 &amp; \quad 0.240 \end{align*} \right]</td>
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<td>[ \begin{align*} 2.78 &amp; \quad 102* \ -1.251 &amp; \quad 0.272 \end{align*} \right]</td>
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<tr>
<td></td>
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<td>(3.0) (2.9) (5.5) (0.5)</td>
<td></td>
<td>(7.9)</td>
</tr>
<tr>
<td>12 TYCR2</td>
<td>1.274 ( \exp(2.34 - 1.82 \text{ YE4}) )</td>
<td>2.34</td>
<td>102</td>
<td>-1.209</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(18.7) (1.6)</td>
<td></td>
<td>(7.9)</td>
</tr>
<tr>
<td>12a TYCR2</td>
<td>1.287 ( \exp(2.32 - 0.13 \text{ YE4} - 0.59 \text{ Q}) )</td>
<td>2.88</td>
<td>102</td>
<td>-1.209</td>
</tr>
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<td></td>
<td>(22.2) (0.1) (5.7)</td>
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<td>(7.6)</td>
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<tr>
<td>12b TYCR2</td>
<td>1.279 ( \exp(1.20 - 1.86 \text{ YE4} + 0.25 \text{ SNC} \text{ YE2}) )</td>
<td>2.47</td>
<td>102</td>
<td>-1.293</td>
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<td>(1.0) (1.9) (1.0)</td>
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<td>(6.4)</td>
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<td>13 TYCR2</td>
<td>1.274 ( \exp(2.200) )</td>
<td>2.32</td>
<td>102</td>
<td>-1.312</td>
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<td></td>
<td>(37)</td>
<td></td>
<td>(8.3)</td>
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<tr>
<td>14 TYCR2</td>
<td>1.272 ( \exp(2.39 - 0.285 \text{ DEF}) )</td>
<td>2.29</td>
<td>102</td>
<td>-1.305</td>
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<td></td>
<td></td>
<td>(11.3) (0.9)</td>
<td></td>
<td>(7.3)</td>
</tr>
<tr>
<td>15 TYCR2</td>
<td>1.287 ( \exp(2.31 - 0.592 \text{ Q}) )</td>
<td>2.88</td>
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<td>-1.209</td>
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<td></td>
<td></td>
<td>(39) (5.8)</td>
<td></td>
<td>(7.6)</td>
</tr>
<tr>
<td>16 TYCR2</td>
<td>1.287 ( \exp(2.33 - 0.57 \text{ Q} + 4.94 \text{ YE4/SNC}^2) )</td>
<td>2.88</td>
<td>102</td>
<td>-1.208</td>
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<tr>
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<td>(24) (5.2) (0.3)</td>
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<td>(7.6)</td>
</tr>
<tr>
<td>17 TYCR2</td>
<td>1.281 ( \exp(2.33 - 20.7 (1 + \text{ Q}) \text{ YE4/SNC}^2) )</td>
<td>2.53</td>
<td>102</td>
<td>-1.269</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(25.4) (2.7)</td>
<td></td>
<td>(6.9)</td>
</tr>
<tr>
<td>17a TYCR2</td>
<td>1.281 ( \exp(2.33 - 24.3 (1 + \text{ Q}) \text{ YE2/SNC}^2) )</td>
<td>2.54</td>
<td>102</td>
<td>-1.269</td>
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<tr>
<td></td>
<td></td>
<td>(29.3) (2.8)</td>
<td></td>
<td>(6.7)</td>
</tr>
<tr>
<td>17b TYCR2</td>
<td>1.283 ( \exp(2.44 - 11.4 (1 + \text{ Q}) \text{ YE4} - 5/\text{ SNC}^2) )</td>
<td>2.62</td>
<td>102</td>
<td>-1.244</td>
</tr>
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<td></td>
<td></td>
<td>(25.4) (3.8)</td>
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<td>(7.2)</td>
</tr>
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(Table continues on next page)
Table C.2: continued

<table>
<thead>
<tr>
<th>Traffic Loadings to Initiation</th>
<th>Parameter</th>
<th>Value</th>
<th>Standard Error</th>
<th>Confidence Interval</th>
<th>Expected Life</th>
<th>Average Log Likelihood</th>
<th>Average Confidence Interval of Expected Life</th>
<th>Source</th>
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<tr>
<td>17c TYCR2</td>
<td>1.283 exp(2.24 - 31.0 (1 + OQ) YE4/SNC² + 2.0 YE4)</td>
<td>2.60</td>
<td>102</td>
<td>-1.262</td>
<td>0.290</td>
<td>-</td>
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<tr>
<td>17d TYCR2</td>
<td>1.279 exp(2.36 - 3.19 (1 + OQ) YE2 DEF)</td>
<td>2.46</td>
<td>102</td>
<td>-1.269</td>
<td>0.304</td>
<td>1.73</td>
<td></td>
<td></td>
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<tr>
<td>17e TYCR2</td>
<td>1.277 exp(2.37 - 2.70 (1 + OQ) YE4 DEF)</td>
<td>2.42</td>
<td>102</td>
<td>-1.274</td>
<td>0.309</td>
<td>1.81</td>
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<tr>
<td>17f TYCR2</td>
<td>1.268 exp(2.75 - 4.77 (1 + OQ) YE2 DMD)</td>
<td>2.23</td>
<td>46</td>
<td>-1.618</td>
<td>0.327</td>
<td>4.33</td>
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<tr>
<td>17g TYCR2</td>
<td>1.262 exp(2.68 - 3.19 (1 + OQ) YE4 DMD)</td>
<td>2.12</td>
<td>46</td>
<td>-1.647</td>
<td>0.345</td>
<td>4.49</td>
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<td>18</td>
<td>1.160 exp(2.55 - 0.115 DEF)</td>
<td>14.6</td>
<td>102</td>
<td>-0.804</td>
<td>0.055</td>
<td>-</td>
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</tr>
<tr>
<td>18a</td>
<td>1.107 exp(2.80 - 0.089 DEF - 0.11 OQ + 0.070 YE4)</td>
<td>27.1</td>
<td>102</td>
<td>-0.589</td>
<td>0.029</td>
<td>-</td>
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<td></td>
</tr>
<tr>
<td>18b</td>
<td>1.110 exp(2.55 + 0.038 SND - 0.08 YE4 + 0.061 YE4)</td>
<td>26.2</td>
<td>102</td>
<td>-0.625</td>
<td>0.030</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>1.106 exp(2.61 - 0.091 DEF - 0.12 OQ + 0.074 YE2)</td>
<td>27.4</td>
<td>102</td>
<td>-0.589</td>
<td>0.029</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>1.077 exp(0.446 + 1.06 DEF)</td>
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<td>102</td>
<td>-0.578</td>
<td>0.519</td>
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<tr>
<td>21</td>
<td>1.109 exp(-5.03 + 3.97 DEF - 0.39 OQ)</td>
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<td>102</td>
<td>-0.571</td>
<td>0.501</td>
<td>0.76</td>
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</tbody>
</table>

Note: Parameter names are defined on p. 427. SIQF = semi-interquartile factor. ALL = average log likelihood. ACI = average confidence interval of expected life. G, B(g), SIQF, ALL and ACI are defined in Appendix B.

Source: Estimation of failure-time models (Equation 5.13) on Brazil-UNDP study data.
Table C.3: Examples of analytical development of prediction model for the initiation of cracking: cemented base pavements

<table>
<thead>
<tr>
<th>Ref. No.</th>
<th>Model parameter estimates (t-statistics in parentheses)</th>
<th>B(β)</th>
<th>Model statistics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>β</td>
<td>obs.</td>
<td>ALL</td>
</tr>
<tr>
<td>1 TYCR2</td>
<td>1.275 exp(1.96 - 4.20 YE4 - 0.03 SN + 0.018 H)</td>
<td>(5.4) (5.7) (0.3) (2.6)</td>
<td>4.50</td>
</tr>
<tr>
<td>2 TYCR2</td>
<td>1.267 exp(1.82 - 2.20 YE4 - 1.15 DEF + 0.026 H + 0.0017 CMOD)</td>
<td>(6.6) (6.7) (4.7) (4.8) (1.7)</td>
<td>5.01</td>
</tr>
<tr>
<td>3 TYCR3</td>
<td>1.240 exp(2.80 - 3.16 YE4 - 3.98 DEF + 0.036 H + 0.003 CMOD)</td>
<td>(9.2) (4.6) (4.0) (4.9) (2.5)</td>
<td>6.67</td>
</tr>
<tr>
<td>4 TYCR4</td>
<td>1.151 exp(2.37 - 1.84 YE4 - 1.34 DEF + 0.030 H)</td>
<td>(17.3) (2.9) (4.5) (6.8)</td>
<td>16.0</td>
</tr>
<tr>
<td>5 TYCR2</td>
<td>1.243 exp(2.01 - 2.50 YE4 - 3.33 DEF + 0.034 H + 0.0053 CMOD)</td>
<td>(8.2) (4.2) (3.9) (5.4) (3.8)</td>
<td>6.5</td>
</tr>
<tr>
<td>6 TYCR2</td>
<td>1.262 exp(-3.91 - 0.21 YE4 - 0.47 DEF + 0.024 H + 0.44 CMOD)</td>
<td>(5.2) (2.4) (2.3) (2.7) (4.5)</td>
<td>5.30</td>
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<tr>
<td>7 TYCR2</td>
<td>1.275 exp(4.28 - 1.04 YE4 + 0.41 DEF + 0.027 H + 0.56 CMOD)</td>
<td>(2.5) (1.0) (1.4) (3.2) (4.2)</td>
<td>4.48</td>
</tr>
<tr>
<td>8 TYCR2</td>
<td>1.265 exp(-2.64 - 1.24 YE2 - 0.55 DEF + 0.030 H + 0.388 CMOD)</td>
<td>(2.1) (2.3) (2.8) (3.9) (3.3)</td>
<td>5.11</td>
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<tr>
<td>9 TYCR2</td>
<td>1.278 exp(0.57 - 7.39 YE4 DEF2 + 0.032 H + 0.264 CMOD)</td>
<td>(1.5) (3.8) (5.6) (1.9)</td>
<td>4.32</td>
</tr>
<tr>
<td>10 TYCR2</td>
<td>1.269 exp(-0.13 - 2.87 YE4 DEF - 0.418 DEF + 0.035 H + 0.371 CMOD)</td>
<td>(0.2) (2.1) (2.1) (5.0) (2.3)</td>
<td>4.85</td>
</tr>
<tr>
<td>10a TYCR2</td>
<td>1.268 exp(0.07 - 4.87 YE2 DEF - 0.374 DEF + 0.033 H + 0.333 CMOD)</td>
<td>(0.1) (2.4) (1.9) (4.8) (2.3)</td>
<td>4.94</td>
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<tr>
<td>10b TYCR2</td>
<td>1.254 exp(0.07 - 1.70 YAX DEF - 0.320 DEF + 0.031 H + 0.422 CMOD)</td>
<td>(0.2) (3.4) (1.9) (5.1) (5.1)</td>
<td>5.78</td>
</tr>
</tbody>
</table>

Note: Parameter names are defined on p. 427. SIQF = semi-interquartile factor. ALL = average log likelihood. ACI = average confidence interval of expected life. B(β), SIQF, ALL and ACI are defined in Appendix B.

Source: Estimation of failure-time models (Equation 5.13) on Brazil-UNDP study data.
APPENDIX D

Roughness Progression on Paved Roads: Analytical Summary for Development of Aggregated Levels Models

A selection of various model forms analyzed during the development of an aggregated model for roughness progression on paved roads is summarized below. The form was intended to aggregate the effects of surface distress under the primary explanatory parameters of

1. initial (or existing) roughness;
2. pavement strength;
3. traffic loading; and
4. pavement age.

The models were estimated in various forms, e.g.,

1. roughness levels, $R_t$, at time $t$;
2. progression slope by finite differences, i.e., incremental roughness, $\Delta R = R_t - R_t'$, in time and traffic increments;
3. progression slope by derivative of roughness levels form, i.e.,
   \[ \frac{dR_t}{dt} \quad \text{or} \quad \frac{dR_t}{dX} \quad \text{where} \ t \ \text{denotes time and} \ X \ \text{denotes traffic.} \]

The finite difference and derivative forms analyzed for many of the "levels" model forms listed here have been omitted for the sake of brevity, and may be readily derived.

The data analysed were the same as analysed for the component incremental roughness model, as summarized in Table 8.1 on p. 285. Parameters are here defined by general category, with units occasionally varied according to the specific specification being estimated, as follows:

- $R_t$ - roughness at time $t$;
- $t$ - time since most recent, construction or rehabilitation of pavement;
- $X$ - cumulative traffic during time $t$;
- $X'$ - rate of trafficking, $dX/dt$;
- $S$ - pavement strength parameters (e.g., SNC, DEF);
- $SD_i$ - surface distress of type $i$

$s_j$, $m$, $\alpha$, $\beta$, $y$, $\delta$ are coefficients to be estimated.

D.1 SERIES I

1.1 $R_t = R_0 + a S^y X$

1.2 $R_t = R_0 + a S^y X^\alpha$

1.3 $R_t = R_0 + a S^y X^{A(\alpha)}$
   where $A(\alpha) = \alpha_0 + \alpha_1 \ln S$

1.4 $R_t = R_0 + a S^y A(\alpha)^{-1} X^{A(\alpha)} X^\delta$
   \[ dR = a S^y X^\delta X^{A(\alpha)-1} dX \quad \text{(for} \ X = \text{constant)} \]
1.5 \[ R_t = R_o + a_o e^{a_1 t} S^Y X^A(a) \]

\[ \ln (dR/dX) = \ln a_o + A(a) \ln X + \gamma \ln S + a_1 t \]

\[ dR_t = A(a) dX/X + a_1 t \]

\[ \ln (R_t) = \ln a_o + A(a) \ln X + a_2 \ln S + a_4 \ln (x dt) \]

1.6 \[ \ln(R_t/R_o) = a_o + A(a) \ln X_t + a_2 \ln S + a_4 t \]

where \( R_t = [R_t - R_o] \)

1.7 \[ (dR_t/dt)/R_t = \alpha + A(a) \dot{X}/X \]

1.8 \[ R_t = R A(a)^{-1} S^Y X^A(a) e^{B(t)} \]

where \( B(t) = a_3 t + a_4 t^2 \).

1.9 \[ \frac{R_t}{R_o} = [a_o A(a)^{-1} S^Y X^A(a) + \sum a_i S^i] e^{a_4 t} \]

Comment: Difficulty with all these model estimates was that \( 0 < A(a) < 1 \) which gives a concave \( \{R, X\} \) curve, contrary to time-series trends.

D.2 SERIES II

2.1 \[ R_t = R_o [A(a^{-1} X^A(a) + a S^Y)^\beta] \]

\[ \ln \left[ \frac{dR}{dX} \cdot \frac{1}{R_o} \right] = \ln \beta + \frac{\beta - 1}{\beta} \ln (\frac{R_t}{R_o}) + (\alpha - 1) \ln X \]

2.2 \[ R_t = a_o R_o S^Y A(a)^{-1} X^A(a) \]

2.3 \[ R_t = R_o e^{a_4 t} [1 + a_o S^Y (1000 + X)^A(a) A(a)^{-1}] \]

2.4 \[ R_t = e^{a_4 t} [R_o^{1-\delta} + a (1-\delta) S^Y X]^1/(1-\delta) \]

2.5 \[ R_t = \left[R_o^{1-\delta} + (\frac{1-\delta}{\delta}) (S - a_c H_c CRX)^Y X^\beta\right]^1/(1-\delta) \]

Comment: Comparison of derivative and finite difference results revealed significant differences in some cases because many increments in the data were large. Thus the finite difference form was preferable for incremental roughness.
D.3 SERIES III

Specifications allowing surface distress to interact with cumulative traffic, when distress present.

3.1 \( R_t = \left[ R_0^{1-\delta} + (1-\delta) S^Y \left[ A(\alpha)^{-1} X^A(\alpha) + \beta^{-1} \text{MSD} (X-X_c)^\beta \right] \right]^{1/(1-\delta)} \)

and variants,

where \( \text{MSD} = \sum a_i SD_i / 2 \), i.e., average distress since initiation, and \( X_c = \) cumulative traffic at time of distress initiation.

3.2 \( R_t = \left[ e^{m t} \left[ R_0^{1-\delta} + (1-\delta) S^Y X (1 + SD) \right] \right]^{1/(1-\delta)} \)

where \( SD = \sum a_i SD_i \).

Note: Relationship is unstable as \( \delta \rightarrow 1 \).

3.3 \( R_t = \exp\left[ a_o R_o + a_1 S^Y (1 + SD) X + m t \right] \)

\( \Delta R_t = a S^Y \Delta X + \Delta SD + m \Delta t \).

3.4 \( R_t = (R_o + a SD_t) e^{m t} \)

\( R_t = R_o e^{m t} + a SD_t \)

3.5 AASHO-type model modified by age and distress:

\( R_t = R_o e^{m t} \left[ 1 + \exp\left( a S^Y / X_t^\beta \right) \right] + SD_t \)

3.6 \( R_t = R_o e^{m t} \left[ 1 + a S^Y X_t^\beta \right] + SD_t \)

3.7 \( R_t = R_o + \exp\left[ a_o + a_1 S^Y + a_2 S^s(S) \right] X_t \)

D.4 SERIES IV

AASHO and Texas TTI forms, modified for aging.

4.1 \( R_t = R_o + \exp\left\{ -[s(S)/X]^\beta + m t \right\} \),

where \( s(S) = \) general function of strength, \( S \).

4.2 \( R_t = R_o \exp\left\{ -[s(S)/X]^\beta + m t \right\} \)

4.3 \( R_t = R_o s(S) N^\beta \)
4.4 \[ R_t = R_o + s(S) N^\beta \]

4.5 \[ R_t = (R_o + a_o S^\gamma N^\beta) e^{m t} \]

Comment: All models of the general forms 4.1 or 4.2, based on the AASHTO and Texas TTI model forms (with or without the addition of aging effects by \(m t\)), tended to give values of \(\beta\) less than 1, i.e., a concave trend, contrary to the convex time-series trends evident on individual sections.

Any power coefficient applied to roughness in the explanatory function (as in 2.4 to 3.2) tended to make the derivative form highly unstable since the power coefficient was typically close to a value of 1.

Most models with suitable fit in the levels form had poor fit when applied to slope predictions.

All aspects of model estimations improved when surface distress parameters were included.

Model form 4.5 was the final selection for the aggregated levels model. The curvature problem, mentioned above, was corrected by achieving a balanced weighting of time and traffic increments through interpolating data for sections having large traffic or roughness increments. Coherent fits for both "levels" and "slope" forms of the model were achieved by placing minor constraints on \(y\) once its optimum had been estimated statistically. The most stable form was achieved with \(\beta = 1\), which was subsequently constrained to 1.
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