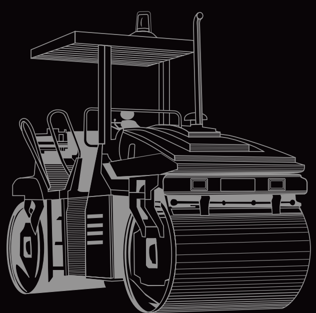
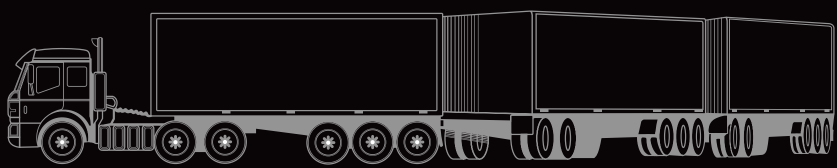
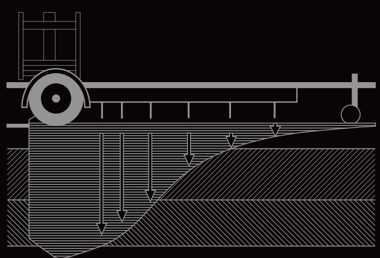
PAVEMENT TECHNOLOGY SERIES

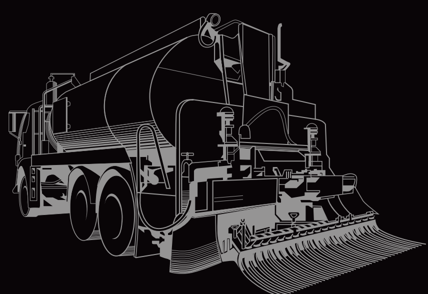
Pavement Design

A Guide to the Structural Design of Road Pavements









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Pavement Design –
A Guide to the Structural Design of Road Pavements
AP-G17/04

ERRATA SHEET
May 2006

Section	Page	Amended text
6.5.3.3	6.20	Right-hand side of Equation 6.6 should read: **' = exp (-0.08[WMAPT – T])'**
Appendix 7.6	A7.6-1	The Percentage of Heavy Vehicles should be 4% rather than 8%, this changes equation A.7.6.3 to: **'N_{DT} = 365 x (5350 x 0.5) x 4/100 x 1.0 x 29.8 x N_{HVAG}'**
8.2.3	8.5	In dot point 3, the equation for f should be **'$f = E_v/(1 + v_v)$'**
8.2.4	8.6	Second last paragraph should read: **'Note that eqn 8.5 is only applicable if N_{1stA} exceeds N_c and eqn 8.6 is only applicable if N_{1stS} exceeds N_c.'**
8.4	8.16	EC07 – Example Design Chart 7. The title of the x axis should read **'Thickness of Cemented Material (mm)'**.
9.5.4.1	9.13	The reference within ε_t is **'(Roads and Traffic Authority, 1991)'** not '(Standards Australia, 1991)'.
6.2.3.2 Table 6.4(b)	6.8	Where the cover material modulus is **2000 MPa** and its thickness is 200 mm, the top granular modulus should read **210 MPa** not 270 MPa

Pavement Design

A Guide to the Structural Design of Road Pavements

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PAVEMENT TECHNOLOGY SERIES

Pavement Design

A Guide to the Structural Design of Road Pavements

Sydney 2004

AUSTROADS

AUSTROADS PROFILE

Austroads is the association of Australian and New Zealand road transport and traffic authorities whose purpose is to contribute to the achievement of improved Australian and New Zealand transport related outcomes by:

- developing and promoting best practice for the safe and effective management and use of the road system
- providing professional support and advice to member organisations and national and international bodies
- acting as a common vehicle for national and international action
- fulfilling the role of the Australian Transport Council's Road Modal Group
- undertaking performance assessment and development of Australian and New Zealand standards
- developing and managing the National Strategic Research Program for roads and their use.

Within this ambit, Austroads aims to provide strategic direction for the integrated development, management and operation of the Australian and New Zealand road system – through the promotion of national uniformity and harmony, elimination of unnecessary duplication, and the identification and application of world best practice.

Austroads membership

Austroads membership comprises the six State and two Territory road transport and traffic authorities and the Commonwealth Department of Transport and Regional Services in Australia, the Australian Local Government Association and Transit New Zealand. It is governed by a council consisting of the chief executive officer (or an alternative senior executive officer) of each of its eleven member organisations:

- Roads and Traffic Authority New South Wales
- Roads Corporation Victoria
- Department of Main Roads Queensland
- Main Roads Western Australia
- Department of Transport and Urban Planning South Australia
- Department of Infrastructure, Energy and Resources Tasmania
- Department of Infrastructure, Planning and Environment Northern Territory
- Department of Urban Services Australian Capital Territory
- Commonwealth Department of Transport and Regional Services
- Australian Local Government Association
- Transit New Zealand

The success of Austroads is derived from the synergies of interest and participation of member organisations and others in the road industry.

DRAFTING COMMITTEE

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Mr A. Leask, Department of Main Roads, NSW
Drafting Committee, Convenor

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FOREWORD

Austroads works towards sharing of national best practice in design, construction and user aspects of roads and, with this purpose in view, arranges for the preparation and publication of specifications, manuals and guides dealing with standards and general procedures.

This Guide is intended to assist those required to plan and design new pavements. It was originally produced in 1987 as a result of a review of the NAASRA "Interim Guide to Pavement Thickness Design, (1979). In 1992, the Guide was revised to include an updated procedure for the design of rigid pavements and also relevant portions of Chapters 6 (Pavement Materials) and 7 (Design Traffic).

This revision consists of new procedures for the design of pavements to a desired reliability of outlasting the design traffic, improvements to materials characterisation, design traffic calculations and thickness design procedures, together with an editorial review of the entire Guide. The design overlays section has now been removed from this Guide and included in the Austroads *Guide to the Design of Rehabilitation Treatments for Road Pavements*.

The methods described have been generally developed from the approaches followed by the member Authorities. However, as a Guide which encompasses the wide range of materials and conditions found in Australia and New Zealand, some parts are broadly based.

The Guide covers the assessment of input parameters needed for design, design methods for flexible and rigid pavements and gives guidance to the economic comparisons of alternative pavement designs.

Pavement composition is related to availability of materials and knowledge of their performance in any particular locality. It is necessary that users of this Guide apply such experience as a basis for interpreting its requirements. The selection of pavement materials is not detailed in this document, but is dealt with in a series of documents published by NAASRA and Austroads.

Amendments/updates

To check for any available amendments to this Guide, users are requested to visit the pavements section of www.austroads.com.au

> At the time of writing, Austroads was in the process of creating a Series of Pavement Technology publications, to be known as the Austroads Pavement Technology Series. The Series is to comprise a number of Parts, one of which will deal exclusively with Pavement Design. As such it will encompass most of the material in this Guide as well as additional items deemed necessary for inclusion in a Part on Pavement Design, such as designing for light traffic.
>
> In the interests of minimising delays in making this document available to the Pavement Design industry, Austroads has decided to release it in its current form, to ensure that the most up to date information is available. Eventually, the information in this document will be available in Part 3 of the Austroads Pavement Technology Series.

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Chapter 5: Subgrade Evaluation

Chapter 6: Pavement Materials

Chapter 7: Design Traffic

Chapter 8: Design of New Flexible Pavements

Chapter 9: Design of New Rigid Pavements

Chapter 10: Comparison of Designs

Chapter 11: Implementation of Design and Collection of Feedback

Appendices

1

INTRODUCTION

1 INTRODUCTION

1.1 Scope of the Guide

The Austroads *Pavement Design Guide* contains procedures for the design of the following forms of road pavement construction:

- flexible pavements consisting of unbound granular materials;
- flexible pavements that contain one or more bound layers;
- rigid pavements (i.e. concrete pavements); and

this Guide represents a major revision of the 1992 Austroads *Guide to the Structural Design of Road Pavements*. It incorporates substantial improvements that have resulted from Australian and overseas research into materials characterisation, traffic assessment, pavement performance studies and design methodology.

This Guide may be used for the:

- design of flexible pavements for conventional road traffic;
- development of design charts for flexible pavements for specific conditions as required by the user (example charts are included in the Guide for specific design input and performance parameters);
- design of rigid pavements for conventional road traffic.

Terminology used in this Guide is defined in *Appendix 1.1*, whilst the components of flexible and rigid road pavement structures are shown in *Figure 1.1*.

Chapter 2 presents a condensed description of the pavement design systems contained in the Guide and the sections referring to each system component. The Guide also contains detailed discussion of subgrade evaluation, pavement materials evaluation, analysis of traffic loading and structural design in addition to other factors relevant to pavement design.

An integral part of the pavement design process is an assessment by the designer of how well the outcome of the design – the constructed pavement – will perform. Because of the many factors which must be evaluated to design pavements, there is no absolute certainty that the desired performance will be achieved. Chapter 2 of the Guide provides guidance on how to design projects to a desired reliability of outlasting the design traffic.

Designers should note the following limitations of the Guide in terms of its scope:

- The procedures in this Guide are intended for the design of pavements, the primary distress mode of which is load associated. Where other modes of stress, such as environmental distress, have a significant effect on pavement performance, their effect should be separately assessed. In this regard, APRG Report No. 21, *A Guide to the Design of New Pavements for Light Traffic – a Supplement to Austroads Pavement Design Guide* provides useful guidance.
- The design procedures described in this Guide apply for pavements subjected to a minimum of 10^5 ESA over the life of the pavement. For the design of lightly-trafficked roads (<10^5 ESA), designers are again referred to APRG Report No. 21, *A Guide to the Design of New Pavements for Light Traffic – a Supplement to Austroads Pavement Design Guide*.
- Whilst the mechanistic procedures presented in this Guide can be applied to any pavement type and traffic load, the pavement types addressed in this Guide relate to public roads subjected to normal highway traffic only – and not to industrial pavements subjected to off-road vehicle loads such as fork lifts and straddle carriers.
- The procedures in the Guide are applicable to traffic with normal transverse load distribution (wander) within traffic lanes. For example, standard deviation of truck traffic wander of 200 mm to 350 mm have been reported (Jameson, Sharp and Vertessy 1992). Caution is advised in using the Guide for pavements with truck wander different from normal highway loading.

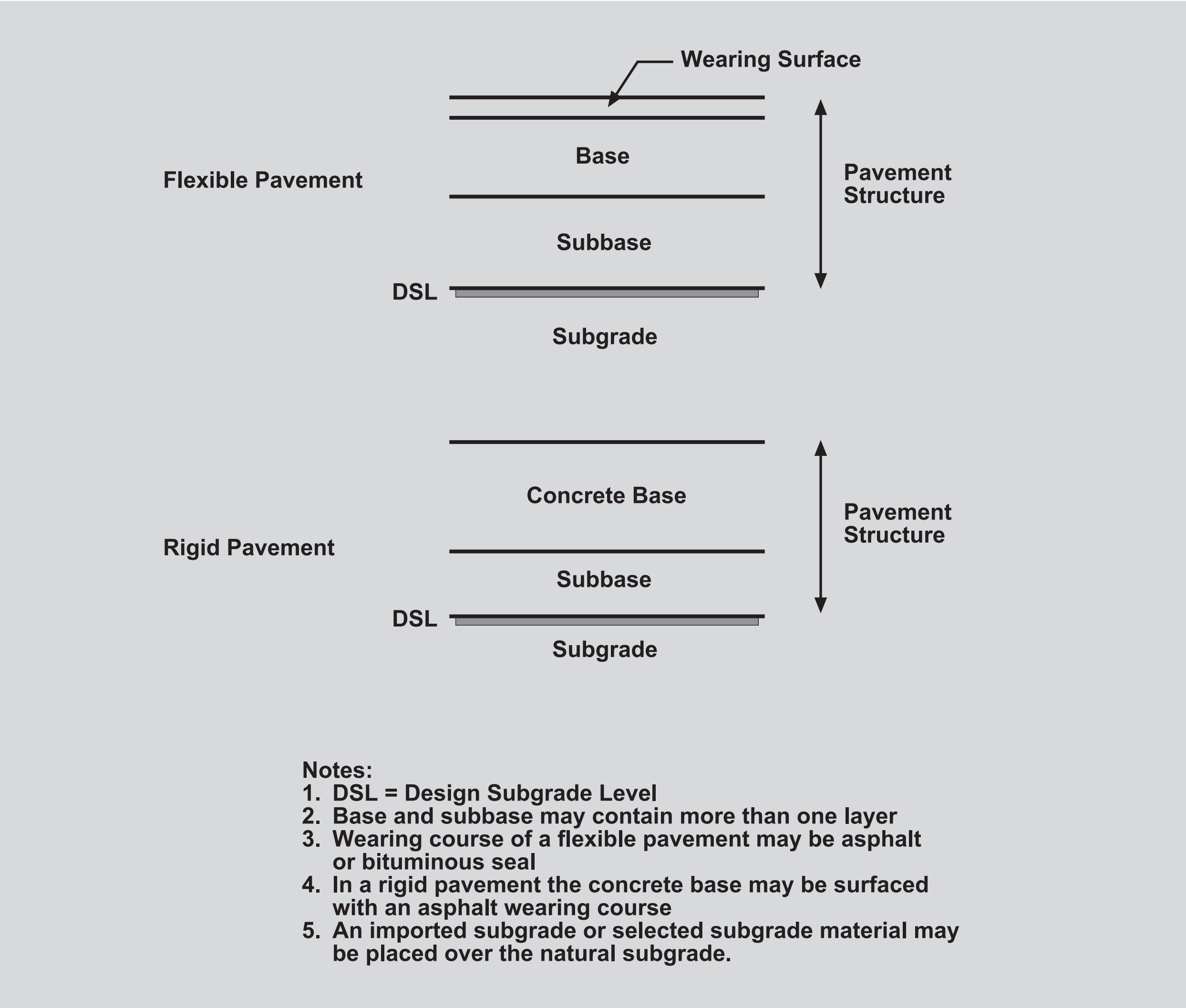


Figure 1.1 Components of flexible and rigid road pavement structures

- Pavements are assumed to be constructed to the usual quality standards specified by Austroads Member Authorities.
- Unsurfaced pavements are not considered in the Guide because the performance of these pavements is heavily dependent on the performance of local materials, local environmental conditions and maintenance policies. The *Unsealed Roads Manual*, published by ARRB Transport Research, provides useful guidance in this regard.
- The design and selection of pavement rehabilitation treatments is not considered in this Guide. Such treatments, including structural overlays, are provided in the *Austroads Pavement Rehabilitation Guide*.
- The Guide provides information associated with the structural design of pavements rather than structural detailing or design detailing.

It is emphasised that this document should be used as a **guide only**; it should not be referred to as a design specification. Judgement must be exercised by the designer in arriving at decisions regarding the parameters that are incorporated into particular designs.

It should also be emphasised that pavement design is only one aspect associated with the achievement of sound pavement performance. Sound pavement performance depends on a number of factors. The primary factors are illustrated in *Figure 1.2*.

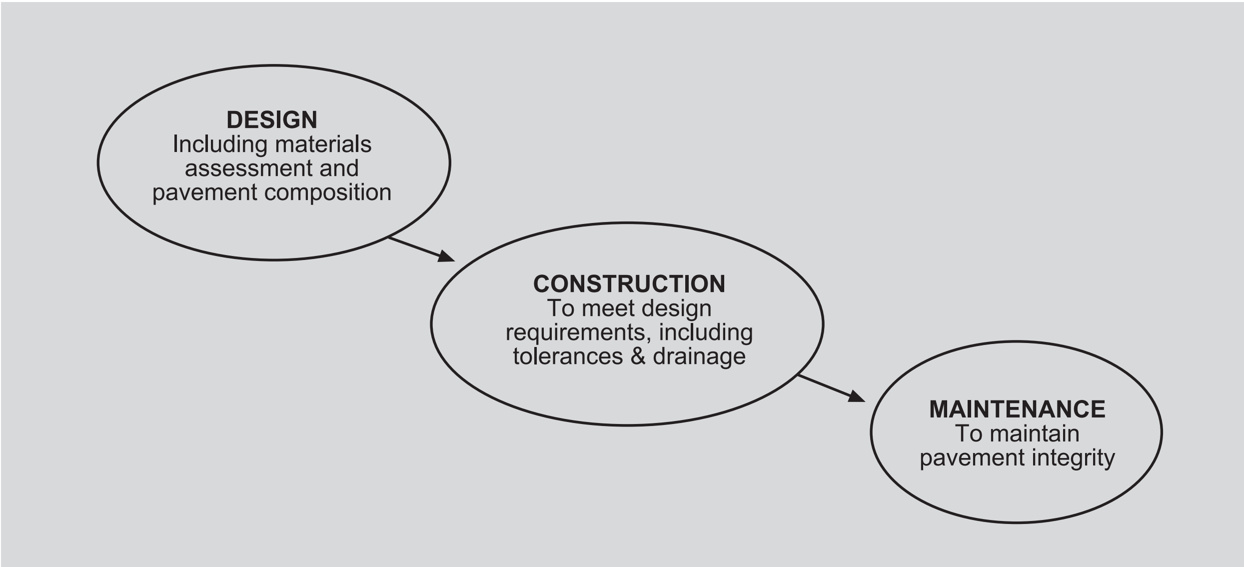


Figure 1.2 A global and integrated approach is required if high levels of pavement performance are to be achieved

1.2 Project scope and background data requirements for design

The first stage in any engineering endeavour is to gain an understanding of the nature and scope of the problem under consideration. In some instances a project will have a comprehensive brief that outlines the problem, constraints and expectations. On other occasions, it may be necessary to enter into discussions with the project initiator to determine or define these matters.

Clearly, the selection of the most appropriate pavement type must take into consideration a number of potentially conflicting issues, any of which may limit the range of options that can be considered. For example, the budget for the works (investigation, design and construction) must be determined because this will control many factors in the process, including the type and extent of the site investigation works conducted and the type of pavement that can be finally adopted.

One of the most important issues that needs to be resolved early in the process is the purpose of the proposed pavement. Whatever the desired function, the desired life of the pavement must be determined in order that resources are not wasted on providing a costly, long-life pavement when less expensive, short-term, or interim, solution may be all that is required. In some cases it may be necessary to develop a ‘staged’ construction plan if funding is not immediately available to allow the most desirable pavement type – from the structural and functional viewpoint – to be immediately implemented.

Some of the issues, or questions, that might need to be considered when scoping the project are listed in *Table 1.1*. In working through this list a more complete understanding of the nature and extent of the project should be obtained, including the client expectations, and the constraints on any remedial treatments. Often it may also be necessary to collect additional information in order to better understand the project context. As such, a preliminary background data search and understanding may be prudent at the time of scoping the project. Alternatively, background data may not be considered, if at all, until the pavement investigation phase. Some of the more common background data sets are also listed in *Table 1.1*.

Several pavement-related issues are discussed in more detail in the following sections of this Chapter of the Guide.

1.2.1 Investigation and design proposal

Prior to undertaking a site investigation and conducting the design, it is recommended that some work be conducted to formulate the methodology to be followed and/or the extent of engineering works to be undertaken. This may be presented as a project proposal.

A key function of a project proposal is to identify the areas of risk and uncertainty and to clarify ownership or responsibility for various elements or issues. A proposal may be relatively vague – based more on principles and objectives – or more detailed and prescriptive. The nature of a proposal is usually determined by the level

Table 1.1 Project scope and required background data

Project objectives	■ Level of service ■ Project reliability ■ Design period ■ Structural capacity ■ Level of future maintenance/rehabilitation
Funding	■ What funding is available for the investigation and design? ■ What funding is available for the construction works? ■ Are there any restrictions on availability and usage of funds? ■ Is there scope for additional funds? ■ Economic considerations – initial costs, future maintenance costs, service life, user benefits, etc.
Timing	■ Timing and duration of investigations ■ Timing and duration of construction works ■ Staging of investigation, design and construction
Critical success factors	■ What are the critical success factors for this project e.g. timing, funding, practicality, innovation, public relations?
Pavement options	■ Road agency policy or preferences ■ Alternatives designs and their evaluation ■ Alternative materials and their evaluation ■ Need for field trials or laboratory evaluation
Usage	■ Likely users and future trends ■ Required levels of usage: volume, load, time distribution, future trends ■ Management of users during investigation and construction ■ Significance of project in terms of the network ■ Other uses: flood levee, floodway, stabilising berm, etc.
Characteristics of the site	■ Climate ■ Geomorphology – terrain, geology, hydrology, soils ■ Land use: industrial, commercial, residential or rural ■ Access ■ Geometry – overhead heights, levels, widths, alignment, cross-section ■ Foundations and stability ■ Drainage ■ Hazards ■ Regional characteristics ■ Future changes in site environment
Environment	■ Planning regulations ■ Energy and resource conservation ■ Potential for use of recycled materials ■ Hazards ■ Pollution: air, noise, water, visual, vibratory, waste disposal, erosion, etc.
Safety	■ Ability to undertake investigations ■ Ability to construct pavement ■ Levels of service – past, current, future, rate of change, standards – skid resistance, ride quality, geometry, visibility, wet and dry road characteristics ■ Driver and public behaviour
Pavement	■ Required condition/performance – functional and structural ■ Configuration ■ Composition ■ Cross-section ■ Future maintenance/rehabilitation practices

of information available from the project scoping and background search activities and the importance of the project in terms of the overall management of the pavement asset in the region/State, etc.

A detailed project proposal might contain some or all of the following general headings:

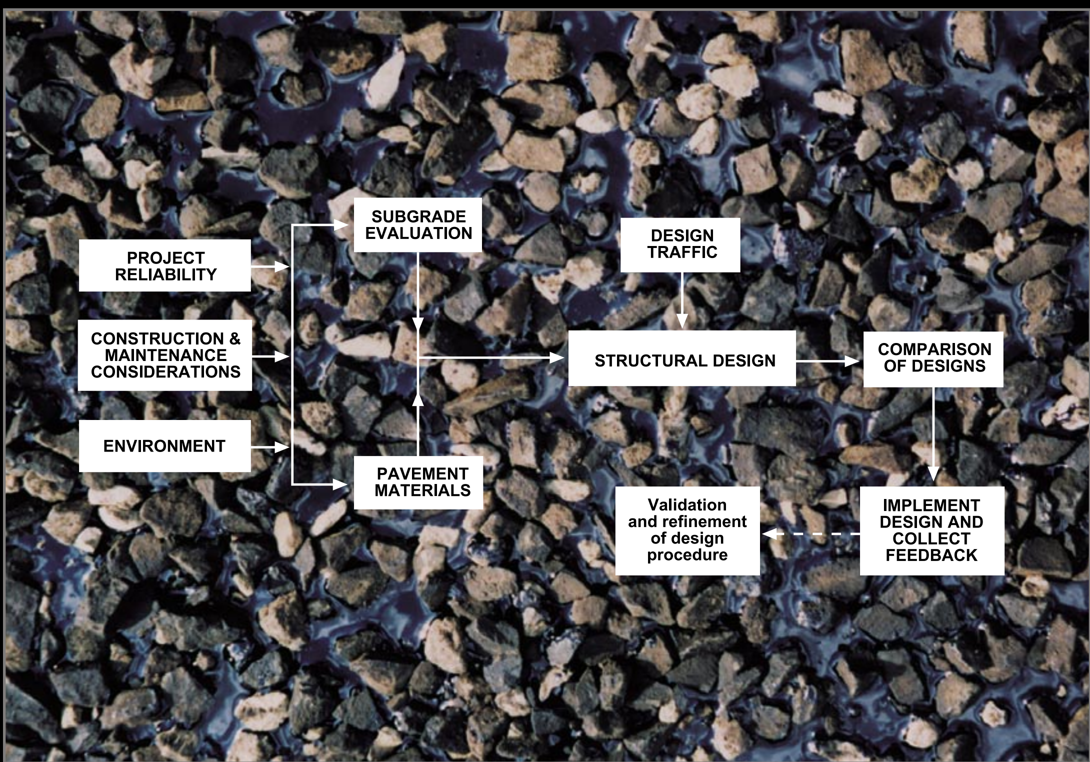
- Project title
- Client
- Project description – location, extent, nature
- Project objectives
- Considerations or issues and the extent of current understanding of these
- Scope of services
- Methodology
- Information supplied by the client
- Quality requirements and standards
- Hold points and client liaison
- Deliverables
- Timeframe
- Resources
- Costs.

Reference

Jameson, G.W., Sharp, K.G, Vertessy, N.J. (1992). *Full-depth asphalt pavement fatigue under accelerated loading: The Mulgrave (Victoria) ALF trial, 1989/1991. ARRB Research Report ARR 224*.

2

PAVEMENT DESIGN SYSTEMS



2 PAVEMENT DESIGN SYSTEMS

2.1 General

The aim of pavement design is to select the most economical pavement thickness and composition which will provide a satisfactory level of service for the anticipated traffic.

To achieve this goal, the designer must have sufficient knowledge of the materials, the traffic, the local environment – and their interactions – to be able to predict the performance of any pavement composition. In addition, the designer must have knowledge of what level of performance, and what pavement condition, will be considered satisfactory in the circumstances for which the pavement structure is being designed.

Because of the many variables and interactions which influence the result, it is appropriate to adopt a systematic approach to pavement design. Depending on the amount of data which has to be provided or, conversely, on the number of assumptions which have to be made, a pavement design procedure may be very complex at one extreme or very simple at the other.

Sound pavement performance depends on a number of factors and relies on a "cradle to grave" approach, managed by experienced professional staff. The primary factors are:

- Design – including materials assessment and pavement composition.
- Construction – to meet design requirements, including tolerances and drainage.
- Maintenance – to maintain pavement integrity.

2.2 Overview of pavement design systems

The system for the design of pavements is shown in flow chart form in *Figure 2.1*. Although in a practical design procedure some of the parts of the system may be omitted or combined with others, it is convenient to use *Figure 2.1* to demonstrate the relationships between input variables, analytical methods and the decision processes which comprise pavement design.

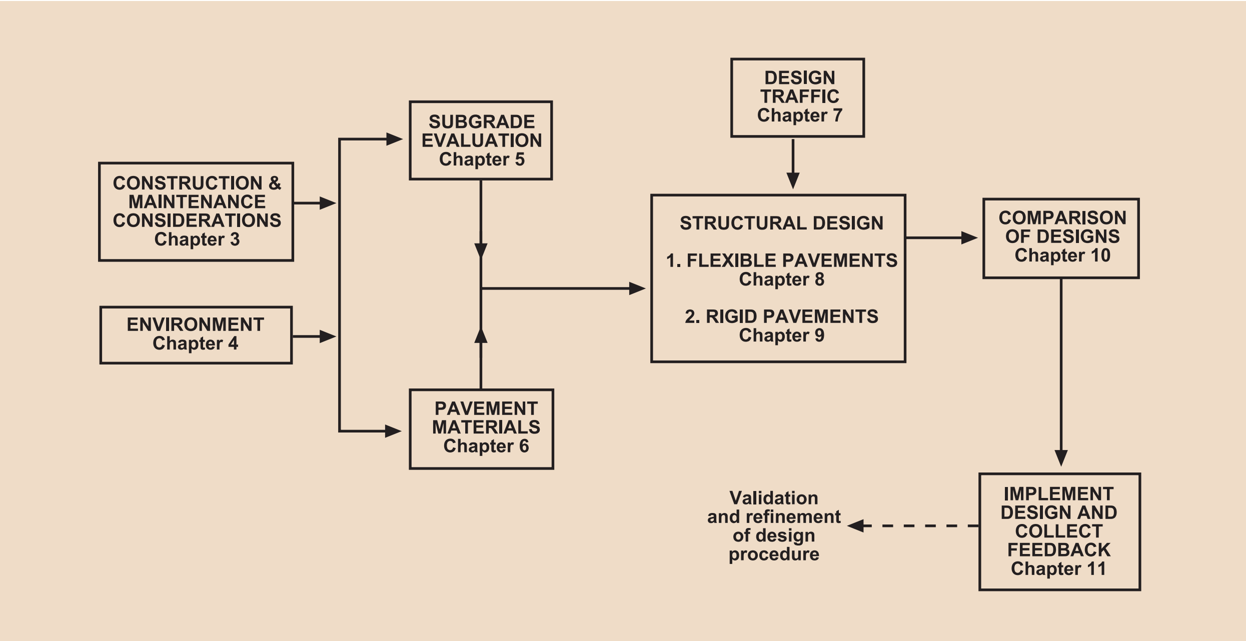


Figure 2.1 Pavement design system

2.2.1 Input variables

2.2.1.1 Design traffic

Axle numbers, load distribution, loading rate (speed) and tyre pressures can all have a significant influence on pavement performance. Not only must the current traffic be taken into account, but also the change in volume, mass and composition must be estimated during the design period. Detailed consideration of traffic is presented in Chapter 7 of the Guide.

2.2.1.2 Project reliability

An integral part of the pavement design process is an assessment by the designer of how well the outcome of the design – the constructed pavement – will perform.

It is unreasonable to expect that a pavement design process can guarantee, with absolute certainty, that a subsequently-constructed pavement will perform to design expectations. The reasons for this are as follows:

- No design process perfectly models how a specific pavement will perform in a controlled environment with a specified traffic loading, let alone in its allotted environment when subjected to its actual traffic.
- The design values chosen for material properties are, at best, gross simplifications of the complex and variable properties of pavement and subgrade materials.
- No construction process can produce a pavement in complete conformance with a design configuration, both in terms of layer thicknesses and (simplistic) material properties.

Because of this lack of certainty, an appropriate measure of the anticipated performance of the proposed pavement is its project reliability, which is defined as follows:

> *The Project Reliability is the probability that the pavement, when constructed to the chosen design, will outlast its Design Traffic before major rehabilitation is required. In regard to these reliability procedures, a project is defined as a portion from a uniformly designed and (nominally) uniformly constructed road pavement which is subsequently rehabilitated as an entity.*

The desired project reliability is the chance that the pavement being considered will outlast its design traffic, assuming that:

- the pavement is designed in accordance with the procedures in this Guide;
- the pavement is constructed in accordance with standard specifications; and
- the materials used meet standard specification requirements.

The desired project reliability is chosen by the road agency or designer. Typical project reliability levels are given in *Table 2.1*.

Table 2.1 Typical project reliability levels

Road class	Project reliability (%)
Freeway	95–97.5
Highway: lane AADT>2000	90–97.5
Highway: lane AADT≤2000	85–95
Main Road: lane AADT>500	85–95
Other Roads: lane AADT≤500	80–90

To achieve the desired project reliability in the mechanistic design of flexible pavements it is necessary to use an appropriate performance relationship to estimate allowable loading from the calculated strains for each of three distress modes. For asphalt fatigue and cemented materials fatigue, Reliability Factors (RF) are used in the performance relationships (Chapter 6) appropriate to the desired project reliability. The performance relationship for rutting and shape loss (equation 5.1) was derived from the empirical design chart for unbound granular materials with thin bituminous surfacings (*Figure 8.4*). This chart, and hence the performance relationship for rutting and shape loss (equation 5.1), is expected to result in appropriate levels of project reliability across the range of design traffic levels covered in the Guide. Consequently, RF values are not explicitly provided within this performance relationship.

In the development of reliability guidelines (Austroads 2004), it was assumed – based on the best available information – that when pavements are:

- designed using the procedures presented in Chapter 8 of this Guide;
- constructed to Austroads Member Authority standard specification requirements; and

- maintained to Austroads Member Authority standards

then, on average, the project reliabilities associated with the RF were applicable. However, average project reliabilities may vary with environment, loading and Austroads Member Authority standards and practices.

As stated above, the empirical design chart (*Figure 8.4*) for unbound granular pavements with thin bituminous surfacing includes appropriate levels of project reliability across the range of design traffic levels covered in the Guide. Hence there is no need to adjust granular thicknesses derived from this chart for project reliability. Material quality is a critical factor affecting the reliability of these pavement configurations.

In the design of rigid pavements, the axle loads are multiplied by a Load Safety Factor (LSF), which enables the design to be conducted to a selected reliability level. As well as depending on the desired reliability for the specific project under consideration, the LSF also depends on the specific pavement type and design method being considered. The procedures are detailed in Chapter 9 of this Guide.

The above-mentioned procedures relate to the reliability of pavements outlasting the Design Traffic. No specific detailed guidance is given in this Guide with respect to reliability procedures for outlasting the chosen Design Period, which has the additional uncertainty associated with the estimation of traffic loading during the design period. However, this uncertainty can be reduced by using conservative parameters in the design traffic calculations (Chapter 7).

2.2.1.3 Subgrade and pavement materials

Ideally, the designer's knowledge of the pavement and subgrade materials should include:

- the strength/stiffness parameters which can be used to quantify their load bearing properties;
- the variations in these parameters which result from changes in moisture and temperature, ageing, or cumulative distress during the design period;
- the manner in which they deteriorate and the significant reaction to load (stress or strain) which can be used to quantify the rate of distress (refer to *Table 2.2*); and
- the limiting value(s) of stresses or strains at which a given degree of distress will occur, commonly known as the performance criteria.

Some of the input parameters apply to the analysis phase of the design system. For example, parameters such as elastic stiffness are used in analytical models to determine load-induced stresses and strains.

Performance criteria (e.g. roughness, rutting and cracking), on the other hand, are used only to predict when distress will occur.

Asphalt and cemented materials are complex in that their performance criteria are a function of their stiffness. These relationships enable materials design to be incorporated into the overall pavement design system, providing added flexibility for the designer.

A detailed consideration of subgrades and pavement materials is contained in Chapters 5 and 6 respectively of this Guide.

2.2.1.4 Construction and maintenance considerations

Construction and maintenance policies can influence the type of pavement structure which is adopted. In addition, the properties of many materials are dependent on construction influences, including the level of compaction, the method of curing concrete or cemented materials, the type of equipment used for placing granular materials (such as crushed rock), and the extent of both sub-surface and surface drainage incorporated in the design.

Construction and maintenance considerations are discussed more fully in Chapter 3 of the Guide.

2.2.1.5 Environment

Variations in material properties due to changes in moisture and temperature may be measured by testing. Temperature changes occur on both a diurnal and seasonal basis, whilst moisture changes may occur as a result of seasonal changes or as a result of some local occurrence or long-term variation in climatic conditions.

The values to be used in the analysis will depend on the actual moisture and temperature existing in service. Because this is a complex issue it is usually necessary to characterise a particular site environment to some extent. The significance of environmental effects depends on the materials that are selected, but it can also depend on the temporal distribution of traffic loading (both on a daily and seasonal basis).

A detailed consideration of environmental effects is presented in Chapter 4 of the Guide.

Table 2.2 Distress modes for flexible and rigid pavements

Pavement type	Distress mode	Likely causes	Materials affected
Flexible	rutting	*traffic associated:* ■ densification, shoving	all but sound cemented materials
	cracking	*traffic associated:* ■ single or low repetitions of high load ■ many repetitions of normal loads *non-traffic associated:* ■ thermal cycling ■ reflection of shrinkage cracks from underlying materials ■ swelling of subgrade materials	asphalt, cemented materials, granular materials
	roughness	variability of density, material properties	
Rigid	fracture or cracking	*traffic associated:* ■ repeated loading (fatigue) ■ spalling at joints (excessive slab movement) *non-traffic associated:* ■ thermal stresses ■ reflection of shrinkage cracks from underlying materials ■ swelling of subgrade materials	
	faulting at joints and slab tilting	*traffic associated:* ■ loss of fines from under slab *non-traffic associated:* ■ slab warping ■ moisture variation (shrinkage/swelling of subgrade) ■ consolidation settlement	
	disintegration	associated with material deficiency or reinforcement corrosion rather than structural considerations	

2.2.2 Selecting a trial pavement configuration

The design process consists of selecting a trial pavement configuration and analysing its performance when subjected to the input design parameters described in Section 2.2.1.

A trial pavement configuration may often be selected by judgment or by using a simple published design procedure. Many such procedures are empirical. Therefore, if one is used, it is desirable that it has been derived from experiences and observations which are compatible with the design task at hand. It is important, however, that these empirical procedures are not extrapolated to traffic conditions outside the scope of the procedure.

If such a simple design procedure is assumed to be sufficiently reliable for the designer's needs, then the design process is complete, since the following phases (analysis, distress prediction and design modification) are all assumed to have been taken care of.

The example design charts in Chapters 8 and 9 of this Guide have been derived from the design system for flexible and rigid pavement respectively, but in each case for a specific set of input parameters. These parameters are listed on, or adjacent to, each chart. Where the designer faces circumstances which are different, the most applicable example chart could be used to select the first trial section. It can then be analysed as the next step to obtaining a more appropriate pavement configuration.

2.2.3 Structural analysis

The purpose of structural analysis is to quantify the critical strains and/or stresses which are induced by the traffic loading in the trial pavement configuration.

In structural analysis, it is usual to represent pavements as a series of layers of different strengths/stiffnesses. The pavement layers may be considered to be fully elastic or viscoelastic, uniform in lateral extent, or variable, and with full friction, or no friction, between the layers. These variations have been used in an attempt to obtain theoretical estimates which agree with observed reactions to traffic loading.

The traffic loading which can be applied varies from a single vertical load having a uniform contact stress to multiple loads with multi-directional components and non-uniform stress distribution. The rate of loading will also vary with traffic speed.

Care must be taken to ensure that the sophistication of the analysis method is compatible with the quality of the input data. If not, then so many assumptions must be made to fill the gaps that the results of the analysis can be misleading, if not worthless.

The method of structural analysis presented in this Guide is consistent with the extent of knowledge of pavement materials and their performance which exists within the Austroads Member Authorities and industry. Information required as input to the analysis method can already be obtained with some reliability or is currently being developed. The results obtained provide predictions of pavement performance which are in reasonable agreement with Australasian experience of pavement performance.

It should be noted that all the performance relationships presented in this Guide have been developed on the basis of the damage caused by normal road traffic loadings on road pavements. If the analysis models in this Guide are used to analyse other loading spectra (e.g. container carriers) then the performance relationships may not be applicable.

Details of the methods of structural analysis which have been adopted for this Guide are given in Chapters 8 and 9 for flexible and rigid pavements respectively.

2.2.4 Distress prediction

The results of the structural analysis are used to estimate the allowable loading of the trial pavement configuration. Most of the performance criteria which are assigned to pavement materials, and to the subgrade, are in the form of relationships between the level of strain induced by the single application of a load and the number of such applications which will result in the condition of the material, or the pavement, reaching a tolerable limit.

Where pavements are composed of a variety of materials which have different distress modes – for example, granular pavements surfaced with asphalt – the allowable loading of the pavement as a whole will be determined by the mode of distress for which the allowable loading is the first to be exceeded by the design traffic loading. In the example quoted above, the allowable loading of the pavement may be taken as the traffic loading at the end of which permanent deformation of the pavement becomes intolerable, or, if it is less, then the loading at the end of which cracking of the asphalt surfacing becomes intolerable.

An exception to this general rule may be observed when a cemented subbase layer is used. In this case, although distress in the cemented subbase layer may reach its tolerable limit early in the pavement's life (i.e. the layer may crack and lose its tensile strength), the serviceability of the pavement as a whole may remain adequate for a considerable period of time, depending on the properties of the layers above the cracked cemented subbase and the residual life of the cracked (now "unbound") material (refer Section 8.2.4).

If all loads applied to the pavement are of identical type and magnitude, then the number of repetitions to "failure" can be obtained directly from the limiting strain versus repetitions criteria. The service life is then the period during which the number of repetitions is just sufficient to cause failure. However, in practical situations, the pavement is usually subjected to a range of loadings, and each magnitude of load produces its own level of strain and stress in the pavement.

Determining the service life in these circumstances is more complicated. There are two relatively common ways to deal with this problem. The first is to convert the numbers of loads of different magnitude to an equivalent number of loads of a standard magnitude – equivalent in the sense that they will cause the same amount of pavement damage. These equivalencies have, in the past, been determined by observing specifically-designed road tests, but they can also be derived theoretically from the performance criteria for the different pavement materials. This method is used in the design of flexible pavements.

The second method used to deal with loads of different magnitudes is to use the concept of cumulative damage. This method is used in the Guide to design rigid pavements. In this method, the proportion of damage caused by loads of a given magnitude is equal to the ratio of the number of such loads in the design period to the number of such loads which will cause failure as derived from the performance criteria.

The sum of these ratios for all load magnitudes indicates the total distress which will occur. If this sum is less than, or equal to, unity then the pavement configuration being analysed is assumed adequate. If not, then the trial pavement is unacceptable and it must be modified in such a way that the deficiency is overcome. Depending on the materials which it contains, and the form of the inadequacy, an increase in thickness or stiffness may have to be made. The new pavement configuration is then re-analysed and a new distress prediction made.

2.2.5 Comparison of alternative designs

When a satisfactory pavement configuration has been obtained it may be adopted or compared with other pavements of different composition, the adopted pavement being selected on the basis of an economic analysis, or other criteria.

Comparison of alternative designs is discussed in Chapter 10 of the Guide.

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3

CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

3 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

3.1 General

The design procedures presented in this Guide assume that appropriate standards of construction and maintenance practice will be adopted. Such standards are well-documented in specifications of individual Austroads Member Authorities and other Austroads and NAASRA publications.

3.2 Construction and maintenance considerations

Unless appropriate construction standards are met, material properties assumed during the design stage may not be achieved and pavement performance may fall well short of expectations.

However, several construction and maintenance considerations must be taken into account in pavement design because they can influence the type of wearing surface which is adopted, the base and subbase material requirements or even the fundamental choice of pavement type. The significant construction and maintenance factors are:

- extent and type of drainage;
- use of boxed construction;
- surfacing type;
- availability of equipment – especially material mixing, placing and compaction plant;
- use of staged construction;
- use of stabilisation;
- pavement layering considerations;
- transverse variations in pavement design;
- use of Strain Alleviating Membrane Interlayers (SAMIs);
- aesthetic and environmental requirements;
- social considerations;
- construction under traffic;
- maintenance strategy; and
- acceptable risk.

There are several Austroads and NAASRA publications which discuss these issues in some detail (e.g. Austroads 1998 and 2003, NAASRA 1983 and 1984).

3.3 Extent and type of drainage

Special drainage provisions may be provided, including sub-surface drains or porous drainage layers, particularly in cuttings. On the other hand, it may not be possible to provide them because of financial constraints, the lack of suitable drainage course materials or drainage outlets. In the latter situation the pavement should be constructed in such a way that water infiltration is minimised or by using materials which will not weaken unduly in the presence of water. These materials may include those stabilised with bitumen, cementitious and/or chemical stabilising agents (Austroads 1998).

In high rainfall regions, or areas subject to high ground-water levels, the use of a properly designed drainage layer underneath a pavement may be an effective means of removing water which has infiltrated through the surface, shoulders or from beneath the pavement. To be effective, such a layer needs to be constructed using a coarse filter material (e.g. graded macadam). In some situations, a layer of fine filter material is also required where the subgrade material is fine grained. Such materials are difficult to lay and compact – and to compact upon – and may be rendered ineffective as a result of ravelling and instability caused by construction traffic.

Unless a considerable quantity of water is likely to infiltrate under a head from beneath the pavement, such drainage layers should be omitted and a cementitious- or bituminous-bound material (which is less sensitive to the effects of water) used in the pavement as previously described.

In drier areas, consideration should be given to placing more emphasis on cross-section design details and materials selection which minimise the infiltration of water into both the pavement and the subgrade, rather than the provision of drainage to remove water after infiltration.

More detailed information is provided in the following sub-sections, whilst further information can be found in NAASRA (1983).

3.3.1 Purpose and details of drainage measures

Drainage measures can be installed in the pavement and subgrade for a variety of reasons, including:

- to provide local lowering of the watertable (drainage of subgrade);
- to cut off water ingress to the subgrade or pavement from water-bearing strata;
- to drain specific pavement layers;
- to control surface run-off; and
- to achieve a combination of some or all of the above.

As part of pavement and/or subgrade drainage design, each of the following steps should be undertaken:

- identify the drainage requirements and the available drainage measures;
- ensure that the total pavement design provides adequate pavement drainage;
- check that no other aspects of pavement configuration or cross-section detail inhibit any of the drainage measures from operating as intended; and
- ensure that the reasons for the drainage installation, and the purpose of each component, are conveyed to those who will construct the pavement.

Attention to detail in drainage design and construction is essential for optimum performance. Expensive drainage systems can be blocked or otherwise prevented from operating by inappropriate construction procedures or drainage design. Poor performance of a drainage system can, in turn, result in major deficiencies with respect to the performance of the pavement.

3.3.2 Drainage of pavement materials

Permeable pavement materials allow considerable longitudinal moisture movements which can result in a build-up of moisture within the pavement:

- in sag vertical curves;
- at changes of pavement type or thickness – where these occur on longitudinal grades; and
- along widenings or major patches where the introduction of new or different material may interrupt the drainage path towards the pavement edge.

Where pavement materials with a permeability greater than $3{\times}10^{-7}$ m/s are used, transverse pavement drains should be installed at the bottom of sags, along cut-to-fill lines, and at any point where a reduction of pavement permeability occurs on a downward longitudinal grade (e.g. at the limit of works or at bridge or culvert abutments). Road Construction Authority Victoria Technical Bulletin No. 32 (RCA 1984) gives indicative values of the permeability of various pavement materials.

Longitudinal pavement drainage is usually achieved by draining the pavement layers into drains below subgrade level. This type of drainage system should not be used where:

- the subgrade consists of expansive soils;
- it is difficult to connect the required layer to a drain below subgrade level (e.g. rock cuttings); and
- no adequate outlet is available (e.g. flat areas with little fall to drainage lines or water-tables).

In such cases the use of a pavement drain within the pavement itself should be considered.

In general, a philosophy of increasing the permeability of materials with increase in depth from the surface should be followed (e.g. see VicRoads 1998). This allows any moisture entering the pavement to flow as quickly as possible to the bottom, where stresses within the pavement due to traffic loading are at their lowest level. From there, the moisture can either migrate to the outer edge(s) of the pavement (if the subbase is more permeable than the subgrade) and be removed by a sub-surface drainage system, or be allowed to percolate through the subgrade (if the subgrade is more permeable than the subbase).

In some cases a situation may arise where a pavement layer is placed on a significantly less permeable material, resulting in a "permeability reversal". With unbound granular materials in particular a build-up of water may occur in the more permeable material, resulting in an adverse effect on pavement performance. This situation can be avoided by requiring the subbase material to be at least ten times more permeable than the base material. Alternatively a less desirable solution is to design the sub-surface drainage system to drain the less permeable material.

3.3.3 Use of a drainage blanket

In areas subject to high ground water levels, the use of a free-draining lower subbase layer may be an effective means of removing water which has infiltrated through the surface, shoulders or from beneath the pavement. Such a treatment is commonly referred to as a drainage blanket.

Drainage blanket material may consist of an open-graded 20 mm crushed rock (having no more than 3% of material finer than 75 µm), produced by blending size 20, 14 and 10 mm aggregates with coarse, washed sand. This material can also be cement stabilised.

Where drainage blankets are placed on fine-grained subgrades it is advisable to use a geotextile as a separation layer between the drainage blanket and the subgrade to limit contamination of the drainage blanket. A geotextile is also often used between the drainage blanket and the overlying pavement material.

3.3.4 Permeable pavements on moisture-sensitive subgrades

Moisture-sensitive subgrades such as silt, clay or silty sand can become unstable if saturated by rain during construction. In such cases the surface of the subgrade should be stabilised (using mixing equipment designed for this purpose) so as to render it less susceptible to the effects of water. Alternatively, a low-permeability granular subbase (permeability less than 10^{-8} m/s) could be provided between the permeable base and subgrade. The choice of treatment will depend on the relative costs and the type of soil (see also Section 3.6 of RCA [1984]).

3.3.5 Full depth asphalt pavements on moisture sensitive subgrade

Rainfall at critical stages of full depth asphalt construction can cause subgrade instability, resulting in excessive construction delays. As with permeable pavements, these delays can be prevented, or at least reduced, by providing a subbase (approximately 100 mm thick) of low permeability bound or unbound granular material with a permeability less than 10^{-8} m/s as a working platform, or by stabilisation of the top of the subgrade.

Construction of full depth asphalt pavements will generally be very difficult for a pavement with a design subgrade CBR of less than 5%. An insitu CBR in excess of 10% is required at the time of construction to achieve adequate compaction of the asphalt layers.

3.4 Use of boxed construction

Where pavement materials are expensive, or wide verges and flat batters are used, it may be more economical to adopt boxed instead of full-width construction. Extreme care must be taken with this form of cross-section to avoid softening of the subgrade because of poor drainage during construction, and to ensure that excessive moisture does not collect in the pavement during its service life. When it is not possible to effectively drain the pavement and/or subgrade, the pavement design should be based on subgrade strength values obtained from soaked test specimens. Verges should be shaped to lead water away from the pavement and, for high standards of performance, comprehensive sub-surface drainage is usually required. Pavement materials should be chosen to avoid the use of moisture-sensitive materials. The layer configuration should be chosen to avoid the creation of a permeability reversal in which moisture may be trapped in the upper pavement layers.

In addition, consideration should be given to the installation of longitudinal sub-surface drains or mitre drains through the verge.

Care should be exercised during the construction of full width pavements to avoid the creation of unintended boxed conditions such as windrows formed after trimming the subgrade.

3.5 Availability of equipment

The pavement type must be compatible with the equipment that is available for construction. For large projects it may be economical to import the required equipment, but in remote areas the locally-available equipment will affect the choice of pavement type and composition.

Sometimes, if a number of small jobs are to be constructed in a short period within the same region, then the number of available economical alternatives can be increased.

3.6 Use of staged construction

Many of the major roads in Australia have been constructed in stages as traffic loading demands and finances permit. If staged construction can be shown to be economical, then this policy is still appropriate, but the first stage of construction should be compatible with subsequent improvements. For example, it is known that the performance of asphalt surfacings are dependent on the stiffness of the underlying layer. Therefore, if it is proposed to provide asphalt surfacing at a later date, then the initial pavement should be of adequate stiffness.

Staged construction may also be employed where there are likely to be changes in traffic patterns as a result of the introduction of new traffic links or situations where settlement may occur due to poor ground conditions.

Particular care should be taken with staged construction of pavements using bound layers because, if fatigue cracking of this initial layer occurs, considerable additional expense may be incurred in the second construction stage to prevent reflection of this cracking, thereby significantly increasing whole-of-life costs.

In all cases the pavement should not be allowed to deteriorate to such an extent that extensive reconstruction, rather than simply strengthening, is required in the second stage.

Other factors which should be assessed when considering staged construction include:

- the economic and social consequences of pavement distress in the first stage resulting from delays in constructing the second stage;
- the cost of sidetracking and other provision for traffic during the second stage construction – this is particularly significant in mountainous or swampy terrain;
- the effects of raising the pavement level at the second stage on kerbs, culverts, guardrails and other road components;
- the relative permeabilities of the subgrade and paving materials and the effect on pavement drainage by omission of the second stage material; and
- the difficulties associated with achieving a high standard of construction with some paving materials if construction under traffic is necessary (see Section 3.12).

In some areas it is common practice to defer the placement of the wearing course on asphalt-surfaced roads. This practice can lead to a significant reduction in pavement life, depending on the length of delay in placing the wearing course. This delay should be allowed for in determining the design thicknesses of layers. For example, for a heavily-trafficked asphalt road, an additional 5 to 10 mm of asphalt is required to be added to the final thickness of the asphalt layers if the placement of the wearing course is deferred for two years. Where bound layer(s) are present, the mechanistic design process allows the likely life of each stage of a staged pavement proposal to be determined.

If placement of the wearing course is deferred some modifications to surface drainage may be required to ensure water does not accumulate on the road surface.

In addition, consideration needs to be given to ensuring that the temporary surface has adequate skid resistance.

3.7 Use of stabilisation

Where suitable equipment and expertise are available, stabilisation can be used to:

- increase the strength and improve the uniformity of subgrades and pavement materials;
- provide resistance to the effects of water ingress;
- provide a working platform for subsequent construction;
- optimise the use of available pavement materials; and
- reduce layer thicknesses compared to unbound materials.

Austroads (1998) contains comprehensive information on most forms of soil and pavement material improvement.

For the purposes of pavement design, using the CBR method of soil characterisation, subgrade material which has been stabilised should not generally be assigned a CBR value greater than 15%.

The introduction of heavy duty, purpose-built stabilisation equipment, and the introduction of slow-setting binders, has allowed the construction of thicker stabilised layers with longer working time and reduced shrinkage cracking. This has widened the range of applications to which stabilisation can be applied, particularly for rehabilitation works.

Shrinkage cracking of cemented materials tends to be unavoidable. If a stabilised base is constructed in more than one layer, then care must be taken to ensure that the layers are initially fully bonded and remain so in service; otherwise, the designer should consider two unbonded layers in the design model. Cracks which propagate to the pavement surface provide pathways for the infiltration of moisture which can lead to debonding of layer interfaces within the pavement and/or weakening of granular layers and subgrade. The extent and severity of cracking is influenced by factors such as binder type and content, material type, initial moisture content and drying and curing conditions.

The required thickness of asphalt or granular material that should be placed above a cemented layer to prevent reflective cracking will depend on many factors including traffic loading, environment, quantity and type of binder used in the cemented layer, degree of subgrade support, etc. Required thicknesses of cover to delay the onset of reflection cracking are discussed in Chapters 6 and 8.

If asphalt is to be placed on a cemented material, then it is suggested that a delay of at least one week before placement of the asphalt be effected to lessen the strain induced by the shrinkage of the cemented material.

3.8 Pavement layering considerations

The layering of pavements can have a significant influence on performance. For example, pavements with open-graded asphalt wearing surfaces may permit the ingress of moisture into the lower layers, particularly if these layers have a significant amount of interconnected voids. This can provide a mechanism that will promote saturation – manifest as stripping – within the asphalt.

To reduce the chances of rutting in heavily-trafficked, deep strength or full depth asphalt pavements, the use of rut resistant, dense-graded asphalt near the surface, often incorporating a modified binder, is increasing.

The use of a high bitumen content asphalt fatigue layer at the bottom of full depth asphalt pavements is also increasing. A minimum thickness of cover of 100–125 mm of dense-graded asphalt should be adopted to prevent the likelihood of instability and permanent deformation under heavy traffic.

Granular pavements with layers of differing permeabilities can allow the development of permeability reversals within the pavement and this should be considered during the design phase (Wallace 1974).

3.9 Use of Strain Alleviating Membrane Interlayers

The use of Strain Alleviating Membrane Interlayers (SAMIs) is common practice. SAMIs consist of a mixture of either crumb rubber or a polymer with bitumen and have proved to be effective in the reduction of reflective cracking (NAASRA 1984; Bell 1992).

A SAMI placed beneath an asphalt layer is normally applied at a rate of 1.5 to 2 L/m^2 and covered with a light coating of size 10 mm aggregate to prevent “pick-up” on the wheels of paving machinery.

SAMIs can be used effectively on structurally adequate pavements with extensively cracked surfaces, and over cement-stabilised bases where future cracking is likely to reflect through the asphalt surfacing.

The use of a SAMI for the treatment of general cracking in a sound pavement is more effective, and economical, than the use of an asphalt incorporating a polymer modified binder (PMB). A SAMI is not so effective where cracks are over 3 mm wide.

For isolated cracking, proprietary self-adhesive bituminous “bandage” products and poured overseal banding processes are available which act as localised SAMIs. Generally, however, they require a minimum 40 mm thickness of asphalt overlay to provide some benefit.

3.10 Environmental and safety constraints

The choice of surfacing type may be influenced by the need to consider issues such as skid resistance (high or low speed situations), noise, wheel spray, night-time visibility, etc. These issues, and the choice of appropriate surfacing types, are discussed in detail in Austroads (2003).

3.11 Social considerations

In very heavily-trafficked areas, or on roads adjacent to commercial developments (shops, etc.), rapid forms of construction may have to be adopted for social and political reasons. This requirement, which needs to be considered during the design stage, will preclude the use of certain pavement types. In addition, the presence of public utilities close to the surface of the pavement may influence the choice of pavement type (Austroads 2003).

The use of pavements by pedestrians and cyclists may also impact on the selection of the wearing surface in terms of texture, colour, etc.

3.12 Construction under traffic

In certain circumstances it is necessary to construct the pavement under traffic and this may also influence the choice of pavement type. When it is necessary to construct the pavement under traffic, it would be inappropriate to choose pavement types which require deep excavations or long curing periods.

However, there also appears to be both advantages and disadvantages associated with the early trafficking of pavements with cementitious layers that are undergoing shrinkage while developing initial strength. As very few field studies have specifically addressed this issue it is difficult to determine the approach that represents best practice. The introduction of micro cracking by early loading is thought to reduce the severity of regular shrinkage cracks, but may also cause a reduction in the strength of the bound layer. In one major overseas field trial (Williams 1986), less shrinkage cracking was reported without any strength loss from immediate trafficking for 2 days, compared to a 7-day delay in opening. A delay of 3 to 4 days in opening to traffic was concluded to be the worst alternative.

For new pavement construction where the design is based on achieving a particular fatigue life from a cemented layer, trafficking should generally not occur for at least 7 days after placement. This would reduce the risk of inadequate strength development that many practitioners associate with early trafficking.

However, where the binder is assumed to only modify granular materials, or deep insitu stabilisation is undertaken for pavement strengthening and rehabilitation, acceptable pavement performance has been obtained after allowing immediate access to traffic.

3.13 Maintenance strategy

High-speed, heavily-trafficked roads are usually designed for a longer life and to be composed of more durable materials, compared with lightly-trafficked roads and streets, because of the hazards and costs associated with closing lanes for maintenance. These types of pavements may be designed so that no deep-seated pavement failures, such as fatigue of cemented subbases, are expected and that all maintenance can be scheduled from the top down. Asphalt- or concrete-surfaced pavements are more suited to this type of application.

More durable pavements are also required for roads through commercial centres, to minimise disruption to businesses as a result of maintenance activities, and to delay maintenance overlays, which would interfere with property access.

Constraints on future overlays (e.g. due to kerbing levels) must be considered by the designer for all urban pavements.

3.14 Acceptable risk

An integral part of the pavement design process is an assessment by the designer of how well the outcome of the design – the constructed pavement – will perform. Because of the many factors which must be evaluated to design pavements, there is no absolute certainty that the desired performance will be achieved.

There are numerous pavement designs and rehabilitation treatments which, if constructed under ideal conditions, may meet design objectives. However, if they are constructed under less than ideal conditions then the pavement will be more susceptible to premature distress. Therefore, as the pavement is susceptible to issues relating to conditions at the time of construction there is a significantly higher risk that the design objectives will not be achieved.

A typical example of this is the performance of thin (<50 mm thick) asphalt-surfaced granular pavements compared with thick (>100 mm thick) asphalt pavements for high traffic loadings. The thin asphalt is more susceptible to adverse temperature conditions at the time of placement (e.g. insufficient density), the fatigue performance of the thin asphalt is highly dependent on the stiffness of the underlying granular materials and the fatigue life of the surfacing is highly dependent on its thickness, even within normal construction tolerances. Therefore, although the thin asphalt-surfaced granular pavement may achieve the design objectives, there are numerous factors that may affect its field performance. On that basis, its adoption may be considered to be a "riskier" treatment than that of the thick asphalt pavement for high traffic loadings.

The probability of design data resulting in a given level of performance should be considered and quantified if possible using statistical methods.

The risk of poor performance may then be varied in the selection of design parameters. It is appropriate that this risk vary with the function of the road for which the pavement is being designed.

Chapter 2 of the Guide provides guidance on how to design projects to a desired reliability of outlasting the design traffic.

3.15 Improved subgrades

3.15.1 Soft subgrades

Soft subgrades are often encountered in coastal areas, and in areas having moisture sensitive soils with high watertables and heavy seasonal rainfall. These require treatment to allow construction to proceed.

Subgrades with an insitu CBR less than 5% at the time of construction may require treatment to avoid delays in construction and assist in compaction of subsequent pavement layers.

Subgrades with an insitu CBR below 3% at the time of construction will usually require some form of treatment or, alternatively, the placement of an overlying working platform to enable construction to proceed.

Measures which may be employed to facilitate construction on soft subgrades include the following:

- draining and drying of the subgrade;
- excavation and replacement of soft material with stable material;
- provision of a gravel or rock fill working platform covered by an impermeable capping layer;
- stabilisation of the top layer of the subgrade;
- provision of a working platform of cemented material;
- provision of a lean concrete working platform; and
- use of geotextiles.

In designing pavements with a subgrade CBR below 3% at the time of construction, the effect of subgrade improvement or the introduction of a working platform placed primarily to allow construction to proceed is usually ignored and a design CBR of 3% is adopted at the new subgrade level. More substantial structural improvements to a very weak subgrade may be mechanistically modelled to achieve effective subgrade strengths in excess of 3%.

Refer to Section 9.3.2 of the Guide for the determination of subgrade strength beneath rigid pavements.

3.15.2 Improved layers under cemented layers

The provision of an improved subgrade – or lower subbase layer of at least 150 mm thick – under cemented layers has little effect in reducing the magnitude of horizontal tensile strain in the cemented layer (which determines the onset of fatigue cracking during its design life).

However, the provision of an improved subgrade provides the following benefits:

- reduces the likelihood of damage after construction to the cemented layer due to heavy construction traffic;
- protects subgrade from rainfall and trafficking during construction;
- improves the compaction of cemented layers;
- provides greater resistance to erosion and pumping of fines through shrinkage cracks in bound pavements;
- reduces local stresses around shrinkage cracks in the cemented layers, and the effects of local subgrade softening and swelling due to the ingress of moisture; and
- improves pavement life – by providing a longer "Stage 2" life – after the cemented layer has cracked.

3.16 Surfacing type

The desired characteristics of the wearing surface will impact on the selection and design of the pavement. Pavements incorporating an asphalt surfacing, or more particularly an asphalt base, are generally more expensive to construct than those composed of unbound granular pavement materials and a sprayed bituminous surfacing. The following sections give background information to aid in the selection of base and surfacing materials. Further guidance on surfacing types can be found in Austroads (2003).

3.16.1 Sprayed seals

For many rural roads, a prime or primerseal, followed by a single coat seal of bitumen and size 10 mm or 14 mm one-sized aggregate is used. A primerseal should be used when it is not practical to prime, due to adverse weather conditions or construction under traffic.

Multiple application seals consist of two or more applications of both bitumen and aggregate. They may be used under certain circumstances, such as:

- To apply a heavy sprayed seal coat in areas of high traffic loading, e.g. rural freeways, intersections (where asphalt is not warranted), steep and/or hilly country, very cold conditions and pavements having high bitumen absorption characteristics.
- To allow the use of soft or brittle sealing aggregates (e.g. soft limestone) which are not suited to a single application seal but which may be cheaper than importing a harder, more expensive aggregate.
- To enable the use of a large aggregate (size 14 or 20 mm) which may have poor adhesion characteristics.
- For work using unmodified bitumen emulsion at total binder rates of application exceeding 1.5 L/m^2.
- For pedestrian and parking areas, town streets and other areas where a reasonably smooth, fine textured surface is required.
- To attain a longer life.

3.16.2 Asphalt or concrete surfaces

These surfaces are used:

- on urban freeways to minimise disruption to traffic during maintenance and rehabilitation operations;
- when scuffing of tyres (e.g. turning traffic) or braking traffic (e.g. intersections) would damage seals; and
- where the use of an asphalt surface is required for vehicle tyre noise reduction.

3.16.3 Open-graded asphalt

Open-graded asphalt (OGA) surfacing has the following advantages over surfacings consisting of dense-graded asphalt, grooved or hessian-dragged concrete and sprayed seals:

- lower tyre/pavement interaction noise;
- less water spray;
- better night-time visibility in the wet;
- better line marking definition; and
- better high speed skid resistance in wet conditions.

OGA is being increasingly used as the surfacing on major urban arterial roads. A polymer modified binder is commonly specified to increase the service life of the surfacing. As it is a porous material, it is placed above the lip level of the kerb and channel or shoulder to facilitate its drainage.

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4

ENVIRONMENT

4 ENVIRONMENT

4.1 General

This Guide mainly describes procedures to assist in the design of pavements to withstand load-associated distress. While environmentally-induced distress is mentioned in a number of passages, pavements where environment is the major distress mode are not specifically discussed in this Guide. These types of pavements are usually lightly trafficked and their design is addressed in APRG Report No. 21 – *A Guide to the Design of New Pavements for Light Traffic* (Austroads and ARRB Transport Research Ltd 1998).

The environmental factors which significantly affect pavement performance are: (a) moisture, and (b) temperature. Freeze/thaw conditions are not discussed in this Guide as they rarely occur in Australia.

4.2 Moisture environment

The moisture regime associated with a pavement has a major influence on its performance. The stiffness/strength of unbound materials and subgrades is heavily dependent on the moisture content of the materials. Further guidance may be obtained from NAASRA (1983). The design, construction and maintenance of subsurface drainage systems for roads is described in Road Construction Authority Victoria (1984) Technical Bulletin 32.

The factors that influence the moisture regime within and/or beneath a pavement and which must be assessed at the design stage include:

- rainfall/evaporation pattern;
- reactivity of subgrade to variation in the moisture regime;
- permeability of wearing surface;
- depth of watertable or to water-bearing strata;
- relative permeability of pavement layers;
- whether or not to seal shoulders;
- type of vegetation to be used in medians or on verges, and their proximity to the pavement;
- the form of pavement construction (boxed or full width); and
- pavement drainage, e.g. availability of table-drains, sub-surface drainage, etc.

Moisture changes in pavements usually result from one or more of the following sources:

- seepage from higher ground to the road pavement;
- fluctuations in the height of the watertable;
- infiltration of water through the surface of the road pavement and the shoulders;
- an abrupt, significant decrease in the relative permeabilities of the successive layers in the pavement (permeability reversal) causing saturation of the materials in the vicinity of the permeability reversal;
- the transfer of moisture, in either the liquid (i.e. soil suction) or vapour states, as a result of moisture content or temperature differences within or beneath the pavement, including transfer due to the moisture content at construction differing from the equilibrium moisture content; and
- the transfer of moisture due to osmotic pressure in the vicinity of the root structures of large vegetation.

Of the above sources only the initial four can be controlled by the installation of properly-designed subgrade and pavement drains. Subgrade drains are only effective when subgrade moisture is subject to hydrostatic head (i.e. positive pore pressures). It is not uncommon for fine-grained subgrade materials to have an equilibrium moisture content above optimum moisture content yet, because they are still only partially saturated, they cannot be drained.

The moisture conditions in unbound granular pavement materials can also have a major effect on performance. When the degree of saturation of unbound granular materials exceeds about 70%, the material can experience significant loss of strength/stiffness.

Most of the above sources of moisture infiltration are illustrated in *Figure 4.1*.

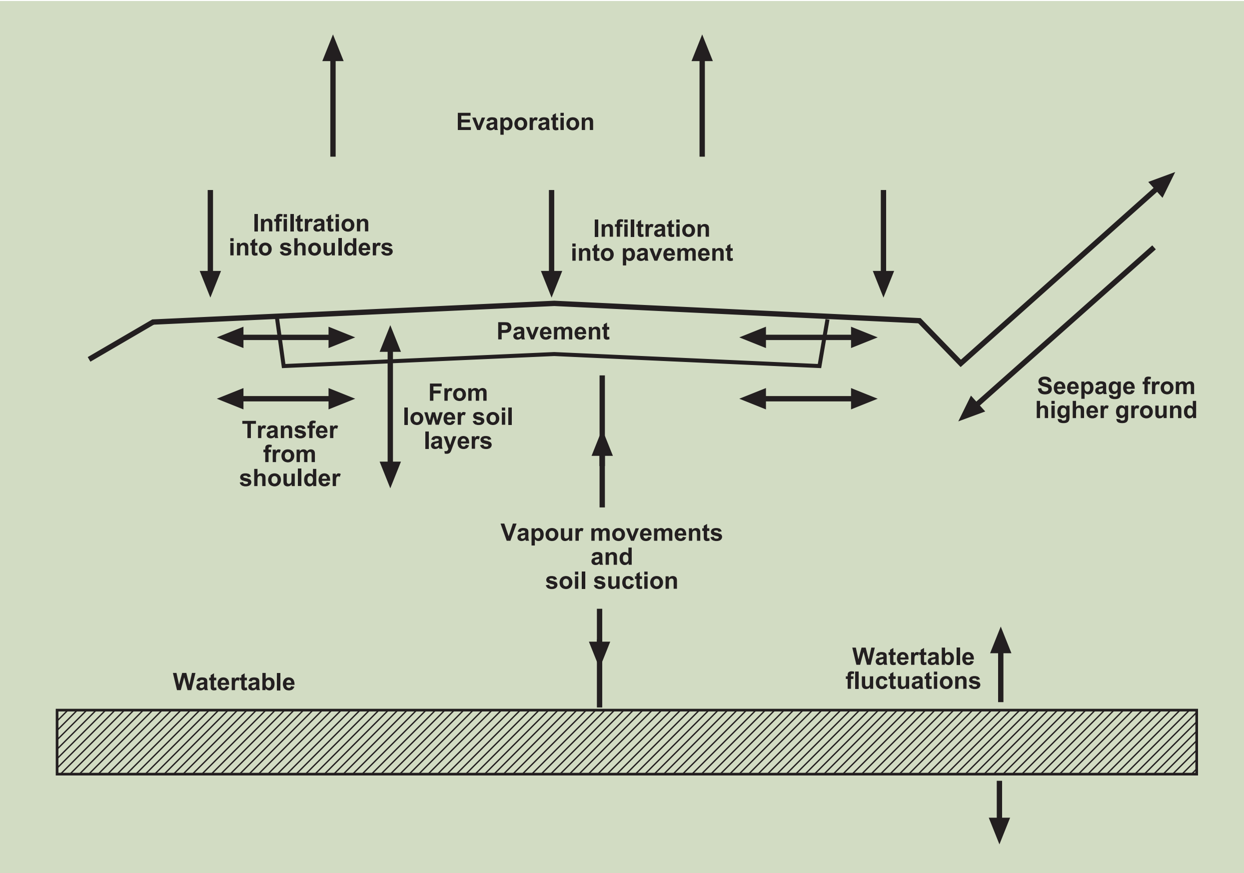

Figure 4.1 Moisture movements in road pavements

An alteration of the moisture content of the subgrade can result in a change in volume and/or a change in strength. The significance of these changes will depend on their magnitude and the nature of the subgrade material.

The sensitivity of the subgrade strength/stiffness to changes in moisture content should in all cases be assessed. In general the following can be stated:

- For sandy soils, small fluctuations in moisture content produce little change in volume or strength/stiffness.
- For silty soils, small fluctuations in moisture content produce little change in volume but may produce large changes in strength/stiffness.
- For clay, small fluctuation in moisture may produce large variations in volume and, if the moisture content is near optimum moisture content, then large changes in strength/stiffness may also occur.

Particular problems associated with expansive clays are discussed in Section 5.3.5.1.

In salt-affected areas and high watertables, salinity may adversely affect pavement performance. ARRB Transport Research *Environmental Practices Manual for Rural Sealed and Unsealed Roads* (McRobert *et al.* 2002) provides guidance on design practices for such areas.

The effect of changes in moisture content on the strength/stiffness of the subgrade is taken into consideration by evaluating the strength parameters (e.g. CBR or modulus) at the highest moisture content likely to occur during the design period. It is important that as accurate an estimate as practicable be made of the representative in service moisture conditions.

Volume changes are minimised if the required density of the subgrade is obtained by compaction at a moisture content representing the value that occurs most frequently. The moisture content, which is used to compact the soil initially may also influence the extent of volume change (refer Section 5.3.5.1).

The estimation of both the representative in service moisture conditions and the moisture content of minimum volume change is usually based on the use of the Equilibrium Moisture Content (EMC) concept where this is considered applicable.

4.2.1 Equilibrium Moisture Content (EMC)

The term Equilibrium Moisture Content (EMC) describes a condition concept that occurs in some situations where the moisture conditions under a sealed pavement at some stage after construction reach a state of equilibrium with the moisture regime of the local environment.

The principal variables which control this condition are:

- climate,
- soil type,
- depth to watertable,
- vegetation, and
- composition of the soil water.

In considering the influence of soil water on the behaviour of partially-saturated soils, the soil water can be quantified either:

- in terms of a ratio to another volumetric or gravimetric property of the soil (e.g. gravimetric moisture content); or
- in terms of its energy state, such as soil moisture suction. Soil moisture suction is used here in the common context of negative pore pressure (matrix potential). However, "suction" is sometimes used in the context of total potential, when the osmotic (solute) potential of the soil is added to the matrix potential.

For practical purposes, the soil water condition at which soil strength is to be determined must finally be expressed in terms of (gravimetric) moisture content. If soil moisture content has been expressed in terms of soil suction, then a conversion to moisture content is necessary before EMC can be used. However, since the measurement and monitoring of soil suction in the field, and the determination of the relationship between soil suction and moisture content, is difficult and not in general use, such methods are not described here. Some relevant references are NAASRA (1974 and 1983), OECD (1973), Richards (1969), Richards and Peter (1987), Wallace (1974) and Waters and Kapitzke (1974).

The conditions which lead to the formation of an EMC are generally those found towards the centre of the pavement. Within about 1 to 2 metres from each edge of the pavement, fluctuations in moisture conditions can result from the relatively rapid changes in moisture content that can occur in the shoulder. These changes can cause the critical moisture content (i.e. representative in service moisture conditions) for the outer wheelpath to be above the EMC estimated for the central portion of the pavement.

In situations where changes in moisture content in the shoulders can be large, treatment in the form of shoulder sealing could be considered to reduce the influence of these moisture content fluctuations. Where such treatment is not likely to be effective, or is considered inappropriate for other reasons, it will be necessary to adopt a representative in service moisture content which is greater than the EMC for the centre portion of the pavement. Determination of the representative in service moisture conditions is discussed in Section 5.6.2 of the Guide.

4.3 Temperature environment

The temperature environment has a major influence on the performance of pavements.

For asphalt-surfaced pavements, the asphalt becomes stiff and relatively brittle at low temperatures, when it is susceptible to fatigue cracking, but it is soft and viscoelastic at higher temperatures, when it is susceptible to permanent deformation. Permanent deformation in asphalt at high temperatures is a failure mode which is not quantified in the current design procedures. Rather, it is considered in the asphalt mix design procedure and, hence, it is assumed, for the purposes of pavement design, that asphalt mixes are sufficiently stable that the magnitude of permanent deformation is minimised. The only failure mode, therefore, considered in the Guide for asphalt is flexural (fatigue) cracking.

The distribution of temperature, both on a daily and a seasonal basis, has an important bearing on pavement performance (Dickinson 1981). The effect of temperature changes in asphalt on pavement performance is a complex matter which must be taken into account at the design stage. For example, if traffic loading occurs at night when temperatures are low, and the asphalt is relatively brittle, then a considerable reduction in the life of a thin asphalt surfacing may occur due to the onset of flexural cracking. On the other hand, if traffic loading occurs during periods of high temperature, then this may lead to increased strains in the lower layers of the pavement if these layers are composed of cemented material. If the lower layers are unbound materials, however, then the higher stresses induced in the unbound materials will result in higher moduli values being achieved.

Climatic effects, and particularly temperature, have a significant effect on the ageing of materials. Asphalt gains stiffness with time and, in the case of thick asphalt layers, increases of the order of four from the initial stiffness over the design life have been recorded without performance being affected (Butcher 1997; Chaddock and Pledge 1994; Nunn 1996; Nunn *et al.* 1997; Nunn 1998). Oxidation – which occurs near the surface of asphalt pavements – can lead to brittleness and ravelling, and hence a loss of performance of the surfacing.

For pavement designs, the temperature of the asphalt can be characterised in terms of the Weighted Mean Annual Pavement Temperature (WMAPT). The WMAPT takes into account the relationship between asphalt temperature and the fatigue life of thick asphalt pavements. It was derived using a method published by Shell in 1978. Procedures to calculate WMAPT, and calculated WMAPTs for selected sites throughout Australia and New Zealand, are presented in *Appendix 6.1*.

Temperature may also affect the properties and performance of cemented layers and concrete. Temperature can have a significant effect on the rate of strength gain of these materials and, if high temperatures occur during construction, drying out will result, impairing both the ultimate strength and fatigue characteristics of the materials.

Rigid pavements are subject to environmental movements that are related to the time of construction and also the short-term and long-term shrinkage characteristics of the concrete (RTA, NSW 1991). These movements can result in significant stresses in the pavement and joints which can impact on pavement performance. This needs to be considered during design.

Diurnal temperature changes over a 24-hour period can also influence the performance of rigid pavements because curling movements are induced in the slab at various stages throughout the day. Movements of slabs during (cold) night-time conditions, for example, can be related to the period of heaviest trafficking, especially near the edge (outer wheelpath) of the slab.

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5

SUBGRADE EVALUATION

5 SUBGRADE EVALUATION

5.1 General

The support provided by the subgrade is generally regarded as one of the most important factors in determining pavement design thickness, composition and performance. The level of support as characterised by the subgrade strength or stiffness is dependent on the soil type, density and moisture conditions at construction and during service.

The guidance provided in this Chapter is of a broad nature and covers the principles of subgrade evaluation. Many of the Austroads Member Authorities have developed more detailed procedures, which are based on local conditions of climate, traffic, topography and materials. These are included in the references.

Note that the subgrade is the trimmed or prepared portion of the formation on which the pavement is constructed. The subgrade may be prepared insitu materials but particularly for heavy-duty pavements, may also include selected subgrade materials that are placed above the insitu subgrade.

5.2 Measures of subgrade support

The measures of subgrade support used in this Guide are the California Bearing Ratio (CBR), and the elastic parameters – vertical modulus (E_V), horizontal modulus (E_H) and Poisson's ratio (ν). The use of these measures for designing various pavement types is given in *Table 5.1*.

Table 5.1 Use of subgrade support measures

Pavement type	Measure of subgrade support: CBR	Measure of subgrade support: Elastic parameters
Flexible	✓	✓
Rigid	✓	

5.3 Factors to be considered in estimating subgrade support

Many factors must be considered in determining the design support conditions, including:

- subgrade variability;
- consequences of premature distress;
- sequence of earthworks construction;
- target compaction moisture content and field density achieved;
- moisture changes during service life;
- pavement cross-section;
- sub-surface drainage and the depth to the watertable; and
- the presence of weak layers below the design subgrade level.

5.3.1 Subgrade variability

Subgrades are inherently variable in nature and reflect the changes in topography, soil type, and drainage conditions that generally occur along an existing or proposed road alignment. Hence the selection of a subgrade design value requires adequate consideration of the degree of variability within a particular project section, and the quantity and quality of data on subgrade properties available to the pavement designer.

5.3.2 Performance risk

The investigation methodology and the strength (or stiffness) assessment techniques adopted to determine the design support condition should be consistent with the required level of performance risk for the pavement under consideration. More comprehensive testing programs and/or conservative design values are commonly selected when the consequences of premature pavement distress are highly significant or considered unacceptable.

5.3.3 Sequence of earthworks construction

Pre-construction planning often allows the use of selected subgrade materials that result in significant construction savings. The pavement design can often be based on the CBR of the selected subgrade material at service moisture and density conditions. Where material selection is not feasible, or where uncertainty exists, a preliminary evaluation of subgrade materials may be necessary, with confirmation at the time of construction. Allowances must be made for any changes in subgrade moisture content that may occur after construction while the pavement is in service.

Even where the design can not be based on selected subgrade material, good management of the earthworks during construction will ensure that the best available material is used. The careful selection of subgrade materials during pre-construction will result in a more controlled subgrade.

5.3.4 Compaction moisture content used and field density achieved

As the strength of subgrade materials is influenced by compaction and moisture content, consideration should be given during design to the likely construction densities and moisture conditions specified for the construction of the subgrade. An indication of the likely effects of variations in relative density and moisture content on subgrades in shown in *Figure 5.1*.

While compaction of clay subgrades to specified density may be achieved at very low moisture contents, this practice results in an open soil structure which is likely to weaken considerably on wetting. This weakening will be compounded by any density loss due to swelling of the clay.

Volume changes are normally minimised if the subgrade is compacted to the required density at a moisture content consistent with the moisture regime that is expected to prevail most frequently during the design period. However, in areas where the most frequent conditions are very dry, but where wetting may occur, the comments above concerning open soil structure in clays should be noted, and compaction closer to Optimum Moisture Content (OMC) considered.

5.3.5 Moisture changes during service life

The placing of a sealed pavement surfacing isolates the subgrade from some of the principal influences which affect moisture changes, especially infiltration of large quantities of surface water and evaporation. Where these influences are the controlling ones (i.e. drier environments), the moisture conditions in subgrades generally tend to remain relatively uniform after an initial adjustment period. In such situations, the subgrade under the central region of the pavement is said to reach an equilibrium moisture condition. This region is flanked by two outer regions having moisture conditions that vary with time due to seasonal climatic influences, termed edge effects. Edge effects generally occur under the outer 1 to 2 metres of the sealed surfacing. The magnitude of these fluctuations generally increases with distance from the centre of the road towards the edge of the sealed surfacing.

In high rainfall areas, subgrade infiltration – particularly lateral infiltration through unsealed shoulders, through defects in wearing surfaces, or through joints – has a major influence on the subgrade moisture conditions. Specific action should therefore be taken to guard against this influence.

The proximity of the ground watertable or local perched watertable to the pavement wearing surface may also play a significant role in influencing the subgrade moisture conditions. In circumstances where the height of the watertable fluctuates seasonally, the subgrade moisture condition will reflect these fluctuations equally across the central and peripheral regions of the pavement.

Overall, moisture changes in the subgrade reflect variations in rainfall and temperature which cause changes in watertable levels, saturation or drying of the shoulders, changes in ground water seepage, etc.

The type of subgrade soil will often control the rate at which seasonal moisture changes occur, and their extent. For example, sand and silty–sand subgrades may reach their wettest condition a few days after heavy rain occurs, while clay or silty–clay subgrades may not reach their wettest condition until months after the end of the wet season. Similarly, sand and silty–sand subgrades will saturate readily, being more permeable, whereas long periods of access to water are required to substantially change the moisture condition of a highly plastic clay subgrade.

Wetting-up of subgrades can, however, be accelerated by an active head due to ponded water or a ground water seepage regime, or by cracking in the subgrade soils. Drying of subgrades, particularly of clays, can be delayed by surface tension effects and the availability of water vapour in the pores. A combination of this accelerated wetting and delayed drying often results in a progressive wetting up of the subgrade; commonly to or beyond optimum moisture content, even in ‘‘dry’’ areas.

5.3.5.1 Expansive soils

Loss of pavement shape due to moisture changes in expansive soils can be a significant factor in the need to rehabilitate pavements. A guide to the identification and qualitative classification of expansive soils is presented in *Table 5.2*. The potential swell test is preferred to the plasticity index test if facilities are available.

Volume changes in highly expansive soils can be minimised by adoption of some, or all, of the following strategies:

- Construct at a time when the value of the soil suction (the ability of a soil to attract moisture) for subgrade or fill materials is likely to be near the long-term equilibrium value.
- Compact the soil at its Equilibrium Moisture Content (EMC). This value occurs when a soil is at its equilibrium soil suction value (refer Section 4.2.1).

Table 5.2 Guide to classification of expansive soils

Expansive nature	Liquid limit (%)	Plasticity index	PI × % < 0.425 mm	Potential swell (%)*
very high	>70	>45	>3200	>5.0
high	>70	>45	2200–3200	2.5–5.0
moderate	50–70	25–45	1200–2200	0.5–2.5
low	<50	<25	<1200	<0.5

* Swell at OMC and 98% MDD using Standard compactive effort; 4-day soak. Based on 4.5 kg surcharge.

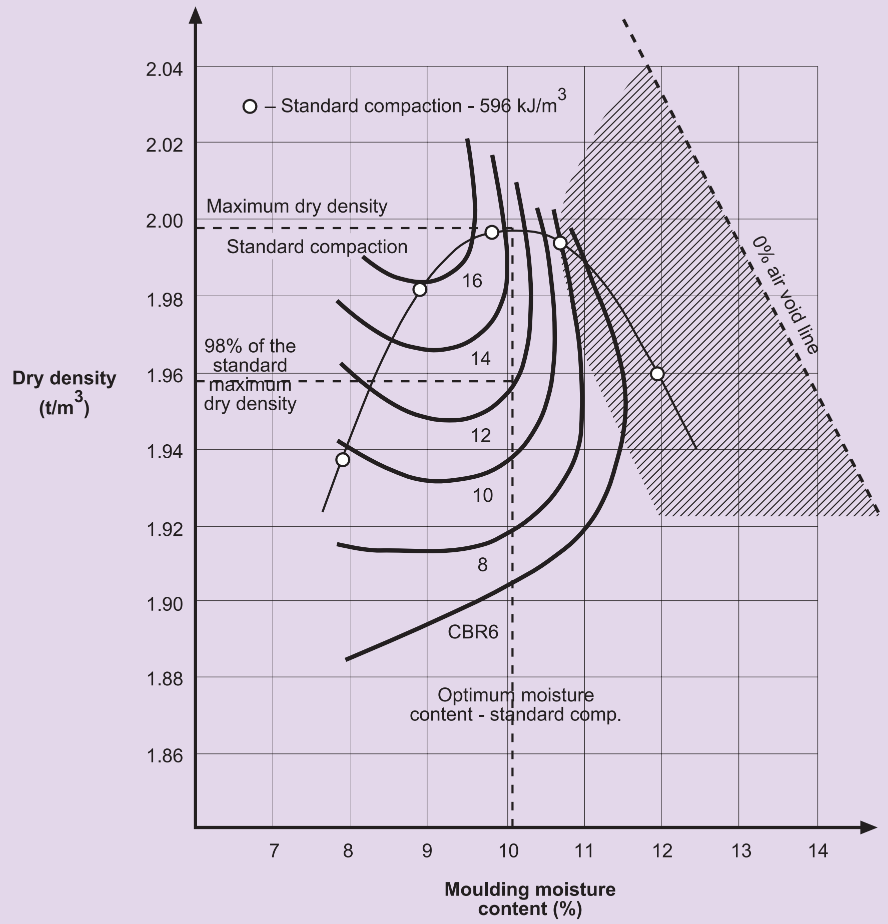

Figure 5.1 Variation of CBR with density and moisture content for a clayey sand

- Provide a low-permeability lower subbase or select fill capping layer above the expansive soil. The minimum thickness of this layer should be the greater of 150 mm or two-and-a-half times the maximum particle size.
- Provide a minimum cover of material over the expansive soil for all pavement types. Material used to provide this layer should have swells of less than 2.5% and be placed at an appropriate moisture content to remain within this limit. The required thickness of cover increases with the traffic loading to reflect the better ride quality required on higher traffic volume roads.
- Ensure that the location of pavement drains is confined to the impermeable subbase and does not extend into the expansive soils. Drains located within expansive soils will cause fluctuations in the moisture content of the soil.
- Restrict the planting of shrubs and trees close to the pavement.
- Provide – through appropriate design of the cross-section of the road – sealed shoulders and impermeable verge material. A seal width of 1 to 1.5 m is required outside the edge of the traffic lanes to minimise subgrade moisture changes under the outer wheelpath.
- Use appropriate construction techniques when placing the expansive soil.
- Incorporate lime stabilisation to reduce the plasticity and increase the volume stability of the upper layer of the expansive clay subgrade.

Further guidance may be found in RTA, NSW (1999) and VicRoads (1993).

5.3.6 Pavement cross-section and sub-surface drainage

Features such as width of sealing, boxed construction, relative permeability of pavement layers and the presence and extent of pavement drainage can all have a considerable effect on subgrade moisture conditions and strength.

As discussed in Section 5.3.5, the outer regions of the pavement and subgrade are subject to significant changes. If this zone of significant moisture fluctuation can be removed from the trafficked area by using sealed shoulders, the more stable moisture conditions may be allowed for in the selection of a design CBR.

Cross section types with relatively high permeability pavement materials either in “boxed” into the surrounding natural materials or flanked by less permeable shoulder materials can inhibit drainage unless appropriate pavement drainage is provided.

These factors must be considered when deciding how to divide the total road length into homogenous sub-sections for design purposes. The sub-sections should be selected on the basis that the condition and type of the subgrade material is essentially likely to be constant. They can then form the basis for determining the design subgrade conditions.

5.3.7 Presence of weak layers below the design subgrade level

Evaluation of the actual support provided to the pavement structure by the subgrade can be complicated by the strength variations that often occur with depth. It is essential that the potential effects of any weak layers below the design subgrade level are considered in the pavement design process, particularly for low-strength materials occurring to depths of about 1 metre.

Where strength decreases with depth, the subgrade may be sublayered for the purposes of the mechanistic pavement design of flexible pavements and when calculating the effective subgrade stiffness for rigid pavement design (Section 9.3.2). For subgrade strengths that are constant or improve with depth, the support at the design subgrade level governs the pavement design.

5.4 Methods for estimating subgrade support value

There are primarily two modes of testing available for estimating subgrade support values: field testing and laboratory testing.

Field testing is applicable to situations where the support values from the insitu subgrade soil conditions are expected to be similar to those of the proposed pavement.

Laboratory testing is applicable both in that situation and also when subgrade support is to be determined from first principles. Due consideration should be given to the sample density, moisture, and soaking conditions which simulate the expected pavement support while in service. The two modes are illustrated in *Figure 5.2*.

There is a range of direct and indirect testing methods that can provide the CBR of the subgrade. Many of these are based on empirical correlations that have considerable variability. For this reason, where possible, a combination of test methods should be used to allow appropriate checks and for confirmation of critical support determinations.

5.5 Field determination of subgrade CBR

This procedure may be used to determine the subgrade CBR in situations where soils similar to those of the subgrade of the road being designed have existed under a sealed pavement for at least two years and are at a

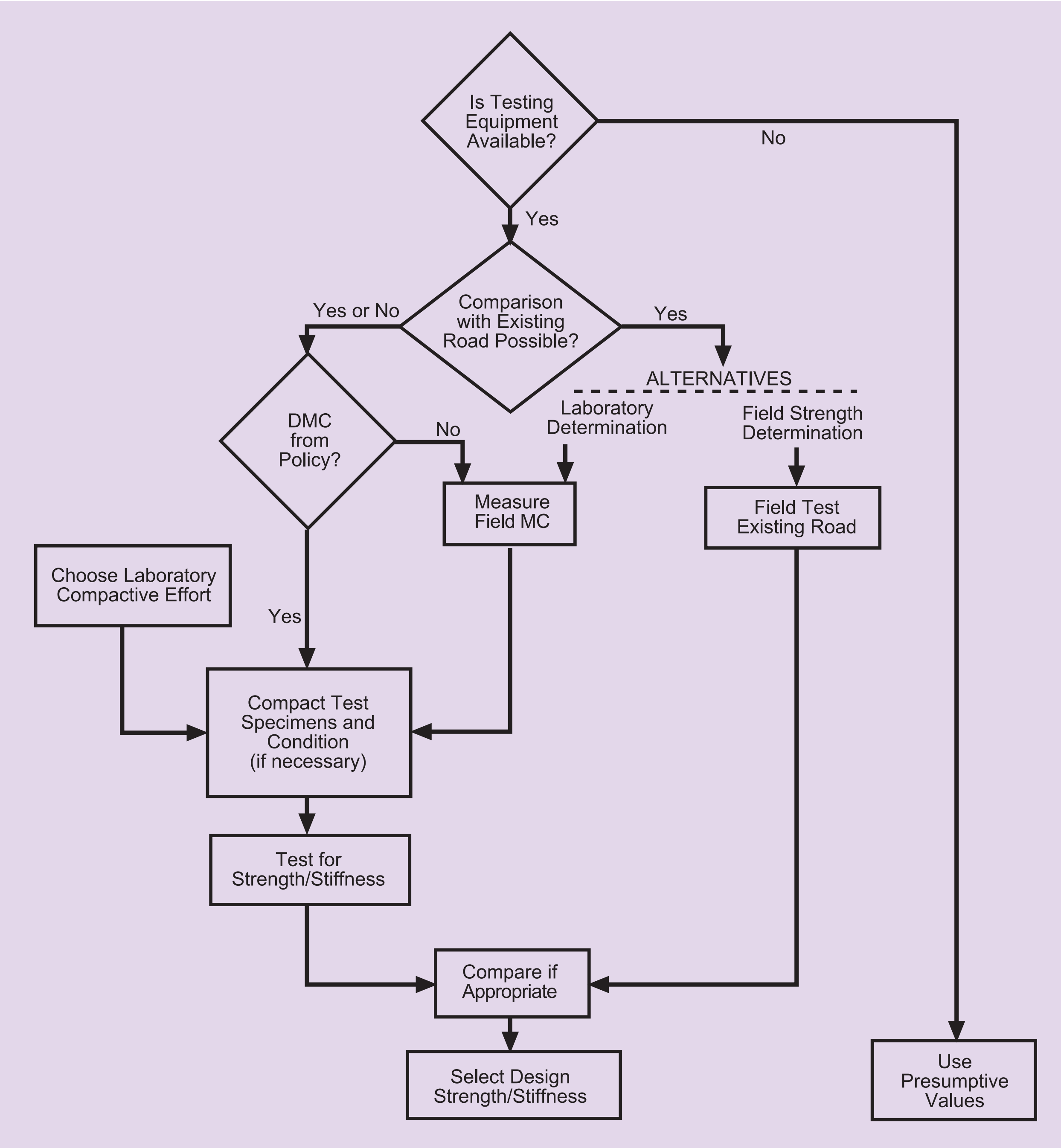

Figure 5.2 Methods for estimating subgrade support values

density and moisture condition similar to those likely to occur in service. Where further disturbance of exposed subgrade soils on new alignments is unlikely, field CBR testing may also be relevant. In both situations, care must be taken when carrying out the tests that the subgrade is in a critical moisture condition; otherwise, seasonal adjustments may need to be made. A number of field tests may be used to estimate subgrade CBR, e.g. insitu CBR test or cone penetrometer (see Sections 5.5.1 and 5.5.2).

If the testing interval and data is unbiased, and the variability of test results is low, then statistical analysis can be used to determine a design CBR at an appropriate percentile level. To ensure homogeneous sub-sections of subgrade, the CBR values should have a coefficient of variation (i.e. standard deviation divided by the mean) of 0.25 or less. The ten percentile level (i.e. mean – 1.3 × standard deviation) is commonly adopted for the design of highway pavements. For roads in arid climates, or roads of lesser importance, higher percentile values may be appropriate (VicRoads 1993 and 1996).

5.5.1 Insitu CBR test

The insitu CBR test should be carried out in accordance with Method 6.1.3 of AS1289 (Standards Australia 1998). This test is time-consuming and expensive and its best application is usually as a supplement to other forms of testing.

5.5.2 Cone penetrometers

Cone penetrometer tests should also be carried out in accordance with Method 6.3.2 of AS1289 (Standards Australia 1997). Its use should be restricted to fine-grained subgrades to avoid misleading results as a result of the influence of large particles. The CBR can be determined from the results of dynamic cone penetrometer testing using *Figure 5.3*. This is a general relationship that suits most fine-grained cohesive soil types.

Several relationships between CBR and penetration (in mm/blow) have been reported in the literature, e.g. Schofield (1986), Mulholland (1984) and Smith and Pratt (1983).

When using the cone penetrometer extensively for subgrade investigation, other CBR testing alternatives should be used to confirm the validity of the CBR/ penetration relationship adopted.

5.5.3 Deflection testing

Back-analysis of the surface deflection bowl data can be used to estimate the elastic modulus of the subgrade from software packages such as EFROMD2 (Vuong 1991) and ELMOD (Dynatest 1989), etc. However, the errors and uncertainty associated with these back-analysis procedures generally limit their use to the development of indicative pavement models that explain past performance and hence can assist in the design of rehabilitation treatments.

In most cases it would be inappropriate to use back-analysis for determining a design CBR for new pavements or verifying the constructed subgrade support conditions with a sufficient degree of confidence. In part, this is because back-analysis does not provide unique modulus solutions and the values may differ from the measured values traditionally used for the design of new pavements.

It is also common for the back-analysis of a data file representing many test locations to return only a limited number of useable solutions, which then introduces a bias or uncertainty about the true characteristic value. Furthermore, it is usually difficult to accurately represent the actual variability of insitu subgrades by a theoretical model that requires defined sublayer thicknesses.

The ELMOD user manual indicates that the back-calculated modulus values are generally within a factor of 2 of the true value, and that this may be unacceptable for many situations requiring only the determination of layer moduli. The EFROMD2 user manual similarly warns that even small average errors of 0.5% between measured and calculated deflections does not guarantee accurate predictions of moduli, critical stresses and strains.

Some Austroads Member Authorities have developed other methods for estimating subgrade moduli values from deflection data. These are generally empirically based and can provide indicative values for certain pavement types such as unbound granular pavements with thin bituminous surfacings (Main Roads Department, Queensland 1990; RTA, NSW 1992).

Results obtained in this manner should be treated with caution and supplemented with results from other means of investigation.

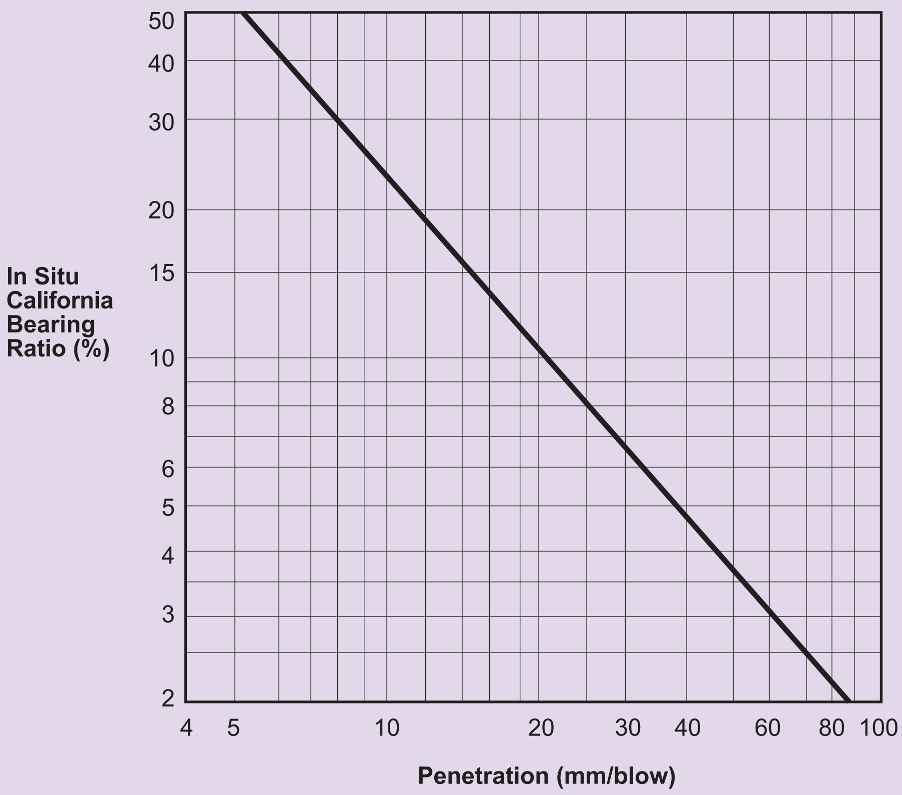

Figure 5.3 Correlation between dynamic cone penetration and CBR

5.6 Laboratory determination of subgrade CBR and elastic parameters

This procedure may be used to determine design CBR or modulus when sufficient samples of the subgrade material for the new pavement can be obtained for detailed laboratory investigations and where a reasonable estimate can be made of likely subgrade density and moisture conditions in service. The method is particularly useful where there is not a close similarity in material type, density and moisture content between the proposed subgrade and any existing site that may be available for insitu testing.

Laboratory tests may be undertaken on specimens tested at a density which correspond to those likely to occur in service or at a particular compaction standard and moisture as a characterising test. Alternatively, undisturbed samples can be obtained from the field by coring.

It may not always be practicable to prepare laboratory specimens at the selected density. In these cases, at least four specimens should be prepared at densities as close as possible to the characteristic value. The design subgrade CBR can then be determined from interpolation of the results for these specimens.

The test procedures for the laboratory CBR test are given in Australian Standard AS 1289 and Materials Testing Manuals of Austroads Member Authorities.

Special pre-treatments may be necessary or desirable when dealing with particular types of subgrade material. For example, for extremely weathered and highly weathered rocks such as siltstone and shale, the effects of construction should be simulated either by applying repeated cycles of compaction of the material to simulate construction, or by introducing other forms of pre-treatment prior to compacting the specimens for testing.

For thickness design purposes using mechanistic procedures (see Chapter 8), subgrade materials are assumed to be elastic and cross-anisotropic. A cross-anisotropic material is characterised by five parameters – two moduli (vertical, horizontal), and two Poisson's ratios (vertical and horizontal) and the additional stress parameter (f). Details of these parameters are provided by Wardle (1977) and Mincad Systems (2004). In this Guide, the ratio of vertical to horizontal modulus is assumed to be 2 and both Poisson's ratios are assumed to be equal. The stress parameter f can be determined using the following relationship:

$$f = \frac{\text{Vertical Modulus}}{(1 + \text{Poisson's ratio})} \quad (5.1)$$

Hence, the values of the five parameters can be determined from the vertical modulus and Poisson's ratio data.

The vertical modulus (in MPa) of a subgrade can be determined from laboratory testing of conditioned specimens (Thompson and Quentin 1976) or by using the empirical relationship:

$$\text{Modulus (MPa)} = 10 \times \text{CBR} \quad (5.2)$$

This equation is, at best, an approximation and modulus has been found to vary in the range 5 × CBR to 20 × CBR (Sparks and Potter 1982). A maximum value of 150 MPa is normally adopted for subgrade materials. Representative values of Poisson's ratio for subgrades are 0.45 for cohesive materials and 0.35 for non-cohesive materials.

5.6.1 Determination of density for laboratory testing

The density selected for testing should correspond to that which will occur in service and may be one of the following:

- insitu density of undisturbed or reworked subgrade as appropriate;
- minimum standard of compaction achieved in construction (embankments); and
- density after swelling has occurred (expansive soils).

5.6.2 Determination of moisture conditions for laboratory testing

The designer should ensure, either on the basis of knowledge of moisture conditions likely to occur in the locality, or by means of detailed field investigations, that the laboratory test conditions realistically represent in service moisture conditions. In many situations, testing under soaked conditions may be warranted.

Several combinations of specimen preparation moisture contents and soaking conditions used in laboratory CBR testing to achieve moisture contents similar to in service pavements are presented in *Table 5.3*.

5.7 Adoption of presumptive CBR values

This approach may be used when no other relevant information is available. It is particularly useful for lightly-trafficked roads where extensive investigations are not warranted, and also when conducting preliminary designs for all roads. Typical presumptive values of CBR are given in *Table 5.4*. However, such values should only be utilised on the basis that the information will be supplemented by taking account of local experience.

5.8 Limiting subgrade strain criterion

In the mechanistic design of flexible pavements (Chapter 8), the pavement is designed to limit the vertical compressive strain at the top of the subgrade to a tolerable level throughout the life of the pavement.

The strain induced is mostly elastic (i.e. recoverable). However, each vertical strain induced by traffic loading is not fully recoverable, and hence, after many load applications, permanent deformation accumulates at the subgrade level, and also, though generally to a much lesser extent, throughout all the pavement layers. These permanent deformations manifest themselves as rutting in the wheelpaths, although, due to the inherent variability of the subgrade and pavement materials and construction techniques, surface roughness increases as the magnitude of this deformation increases.

The limiting strain criterion for the subgrade is given in equation 5.3. It was derived (Austroads 2004) by applying the mechanistic procedure presented in Chapter 8 to a range of pavements selected using *Figure 8.4*. It represents a "best fit" relationship. Pavements designed in accordance with *Figure 8.4* have been found to provide satisfactory service under Australian conditions.

$$N = \left[\frac{9300}{\mu\varepsilon}\right]^7 \quad (5.3)$$

where $\mu\varepsilon$ is the vertical strain (in units of microstrain) at the top of the subgrade and N is the allowable number

Table 5.3 Typical moisture conditions for laboratory CBR testing

Median annual rainfall (mm)	Specimen compaction moisture content	Testing condition	
		Excellent to good drainage	Fair to poor drainage
<500	OMC	unsoaked to 4-day soak	1 to 4-day soak
500–800	OMC	unsoaked to 4-day soak	4 to 7-day soak
>800	1 to 1.15 × OMC	unsoaked to 4-day soak	4 to 10-day soak

Table 5.4 Typical presumptive subgrade design CBR values

Description of subgrade		Typical CBR values (%)	
Material	USC classification	Excellent to good drainage	Fair to poor drainage
Highly plastic clay	CH	5	2–3
Silt	ML	4	2
Silty clay	CL	5–6	3–4
Sandy clay	SC		
Sand	SW, SP	10–15	5–10

of repetitions of a Standard Axle at this strain before an unacceptable level of permanent deformation develops.

The use of this relationship in the mechanistic procedure presented in Chapter 8 will therefore produce designs that are generally consistent with observed performance of road pavements throughout Australia in terms of the development of surface deformation. However, its use for the design of pavements that carry significantly different loads to road pavements should be treated with considerable caution.

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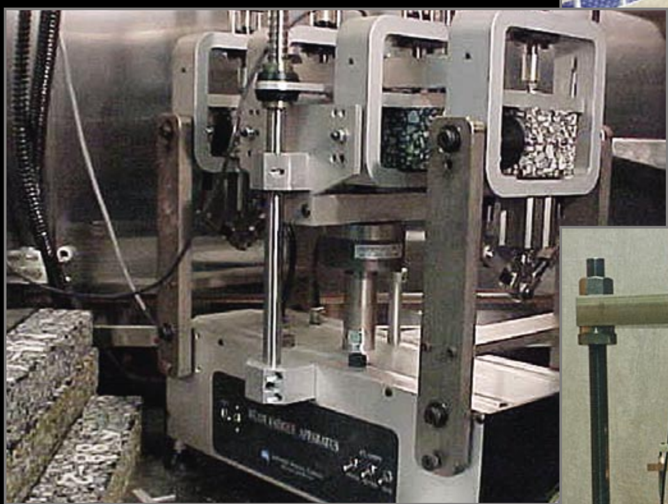
6

PAVEMENT MATERIALS



6 PAVEMENT MATERIALS

6.1 General

The design procedures in this Guide permit the use of a wide range of materials, provided pertinent information on their behaviour and likely performance is known. The choice of materials for any particular application should be based on considerations of structural requirements, economics, durability, workability and experience.

Pavement materials can be classified into essentially five categories according to their fundamental behaviour under the effects of applied loadings:

(a) unbound granular materials,

(b) modified granular materials,

(c) cemented materials,

(d) asphalt, and

(e) concrete.

Table 6.1 summarises the characteristics of the various categories of pavement materials. Specific comments relating to each material category are included in the text.

The manner of characterising the categories of material, other than concrete, is determined by the available design procedures (refer Chapter 8) of which there are currently two: an empirical method and a mechanistic-empirical, or structural, method. The former, which is limited to the design of pavements incorporating only unbound granular materials, requires materials to be characterised in terms of their strength. The latter, which is applicable to the design of pavements incorporating any of the category (a), (b), (c) or (d) materials, utilises a computerised analytical technique, named CIRCLY (Mincad Systems, 2004) that models the pavement as a series of elastic layers and requires the materials to be characterised in terms of their elastic properties – modulus and Poisson ratio. The manner of characterising these materials is discussed in Sections 6.2 to 6.5.

6.2 Unbound granular materials

6.2.1 Introduction

6.2.1.1 Material characteristics and requirements

Unbound granular materials consist of gravels or crushed rocks which have a grading that makes them mechanically stable, workable and able to be compacted. Requirements for such materials are given in NAASRA (1976, 1980, 1982*a* and 1982*b*). Their performance is largely governed by their shear strength, stiffness and resistance to material breakdown under construction and traffic loading. The most common modes of distress of granular base layers are rutting and shoving due to insufficient resistance to deformation through shear and densification, and disintegration through breakdown.

6.2.1.2 Construction considerations

The major construction considerations relating to granular materials are the level of compaction, the uniformity of its placement in the pavement and the extent of the dry-back from the compaction moisture condition to the moisture condition at sealing and in-service. The level of compaction and the in-service moisture condition determine the resistance to permanent deformation and the stiffness of the granular material in service.

It is important that the level of compaction for granular base materials be as close to the maximum dry density as possible. For low-permeability and high-plasticity crushed rocks, from which water is not readily squeezed out during construction, it is critical that the moisture content at placement is as close as possible to the optimum moisture content (OMC) value to achieve the maximum dry density (MDD) value. For highly permeable, non-plastic crushed rocks, from which water can be squeezed out during construction, the moisture content at placement is not as critical and the maximum dry density is usually achieved at the saturated condition. In both cases, a dry-back period is required to reduce the moisture content to an acceptable level before sealing. Different specifications

Table 6.1 Pavement material categories and characteristics

Characteristics	Pavement material category				
	Unbound granular (Section 6.2)	**Modified granular (Section 6.3)**	**Cemented (Section 6.4)**	**Asphalt (Section 6.5)**	**Concrete (Section 6.6)**
Materials types	■ Crushed rock ■ Gravel ■ Soil aggregate ■ Granular-stabilised materials	■ Bitumen-stabilised materials ■ Chemically-modified materials ■ Cement, lime, lime/fly ash or slag-modified materials	■ Lime-stabilised materials ■ Cement-stabilised materials ■ Lime/fly ash-stabilised materials ■ Slag-stabilised materials ■ Slag/lime-stabilised materials	■ Asphalt	■ Concrete
Behaviour characteristics	Development of shear strength through particle interlock. No significant tensile strength.	Development of shear strength through particle interlock. No significant tensile strength.	Development of shear strength through particle interlock and chemical bonding. Significant tensile strength.	Development of shear strength through particle interlock and cohesion. Significant tensile strength. Properties are temperature sensitive.	Development of shear strength through chemical bonding and particle interlock. Very significant tensile strength.
Distress modes	Deformation through shear and densification. Disintegration through breakdown.	Deformation through shear and densification. Disintegration through breakdown.	Cracking developed through shrinkage, fatigue and over-stressing. Erosion and pumping in the presence of moisture.	Cracking developed through fatigue, overloading. Permanent deformation.	Cracking developed through shrinkage, fatigue and erosion of subbase/subgrade.
Input parameters for design	■ Modulus ■ Poisson's ratio ■ Degree of anisotropy	■ Modulus ■ Poisson's ratio ■ Degree of anisotropy	■ Modulus ■ Poisson's ratio	■ Modulus ■ Poisson's ratio	■ 90 or 28 day flexural strength or 28 day compressive strength.
Performance criteria	Current materials specifications (not covered in this Guide).	Current materials specifications (not covered in this Guide).	Fatigue relationships (see Section 6.4.5.4).	Fatigue relationships (see Section 6.5.6).	Fatigue and erosion relationships – see Section 9.4.2)

of moisture condition at sealing can be adopted for different material types to maximise the performance of both the seal and the base.

6.2.1.3 Characterisation for pavement design

For the empirical design procedure presented in Chapter 8, unbound granular materials are characterised in terms of their California Bearing Ratio (CBR). The required CBR is governed by the depth of the material below the wearing surface and is specified in the design chart (see *Figure 8.4*) (refer to NAASRA 1976, 1980, 1982*a* and 1982*b*).

When the mechanistic design procedure is to be used, granular materials are characterised by their elastic parameters (modulus and Poisson's ratios). In this case, it is assumed that all the material and construction requirements as adopted in the empirical design procedures are met – and hence so are the shear strength and permanent deformation requirements.

For unbound granular materials, there is strong evidence that the modulus in the vertical direction is different from that in the horizontal direction (i.e. they are anisotropic). In the mechanistic design procedure the vertical modulus of unbound granular materials is taken as being equal to twice the horizontal modulus.

Physically this effect may be attributed to the fact that pavement materials are generally compacted in horizontal layers and so exhibit a preferred particle orientation. While there is not a universal method for characterising anisotropic behaviour, it has been found that allowing for anisotropy in the elastic model CIRCLY produces a better fit between calculated and measured deflections. However, it is not known whether this is due solely to the anisotropic behaviour, or the stress-dependent behaviour, of these materials.

It is universally recognised that the modulus of unbound granular materials is stress dependent. It has been found that the stress dependency of vertical modulus can be modelled using the elastic model CIRCLY by dividing the granular layers into several sublayers, with each assigned a vertical modulus to reflect the stress level at which it operates (see Section 8.2.3). However, the horizontal component of stress dependency cannot be directly modelled using CIRCLY. The use of finite element models, which allow for stress dependent and anisotropic behaviour in both the vertical and horizontal directions, would enable an exact fit between calculated and measured deflections to be derived.

6.2.2 Factors influencing modulus and Poisson's ratio

The modulus of unbound granular materials must be appropriate for the range of stresses under which they are likely to operate. The modulus of unbound granular materials is largely independent of the rate at which they are loaded (Hicks and Monismith 1971). Poisson's ratio values between 0.1 and 0.5 have been shown to have little influence on pavement thickness requirements within this range. Commonly, Poisson's ratio is assumed to be 0.35.

The main factors that affect the modulus of granular materials are given in *Table 6.2*. Some of these factors are briefly discussed below.

6.2.2.1 Dry density and moisture content

The modulus depends on both density and moisture content. *Figures 6.1* and *6.2* show examples of relationships between modulus and dry density and moisture content for a typical granular base and a typical subbase material. It is essential that the design modulus be estimated for conditions which approximate those likely to occur in service.

Where samples are to be tested for the determination of design modulus, testing should be conducted at conditions of anticipated field moisture content and density.

Table 6.2 Factors affecting modulus of granular materials and effect of increasing factors

Factor	Effect of increasing factor
proportion of coarse angular particles	increase
density	increase
compaction moisture content	increase up to an optimum value, then decrease
stress level	
mean normal stress	increase
shear stress	decrease
in-service moisture content	decrease
age	no change
temperature	no change
rate of loading	no change

In the absence of more reliable site-specific information, in severe environments, such as the following, soaked conditions may be adopted:

- floodways, causeways and other pavements likely to be regularly inundated;
- cuttings below the water table or when seepage is likely; and
- other situations where the water table is within 1 metre of the subgrade level.

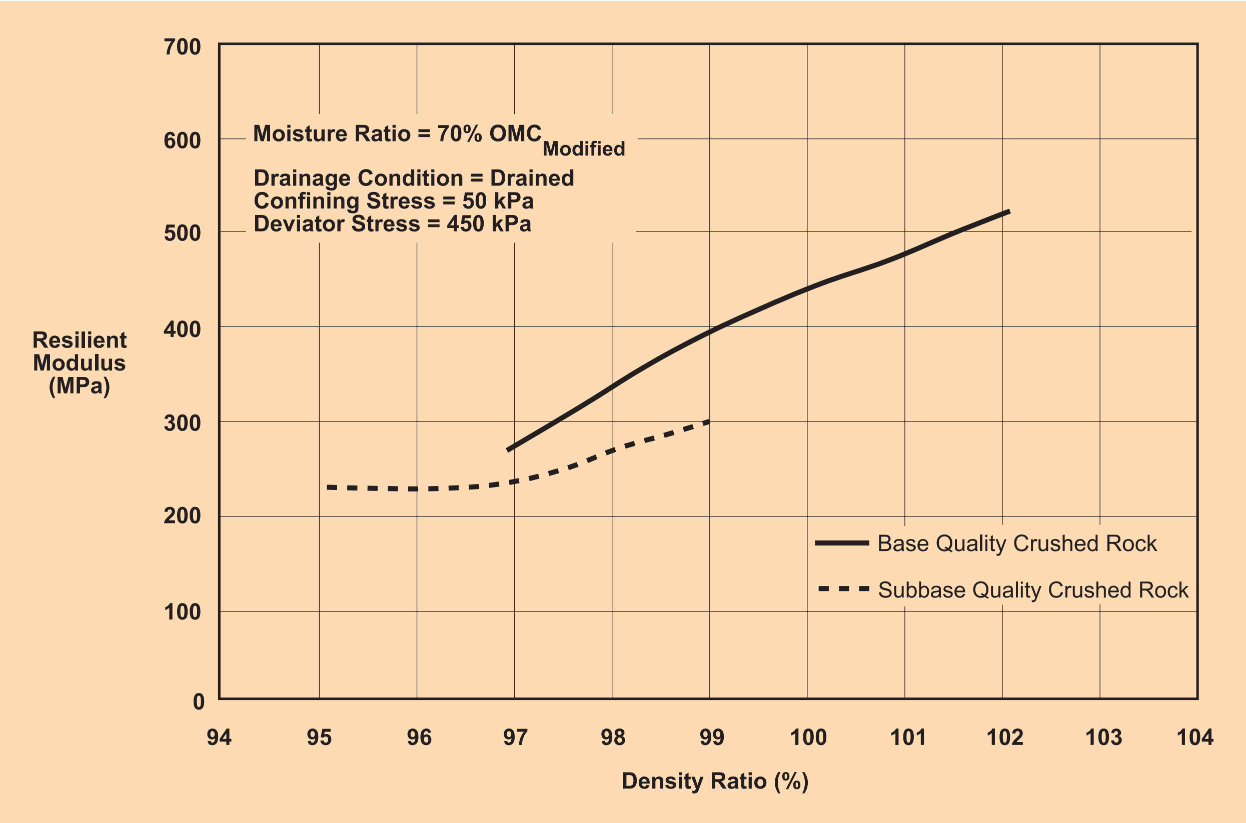


Figure 6.1 Typical relationships between modulus and dry density

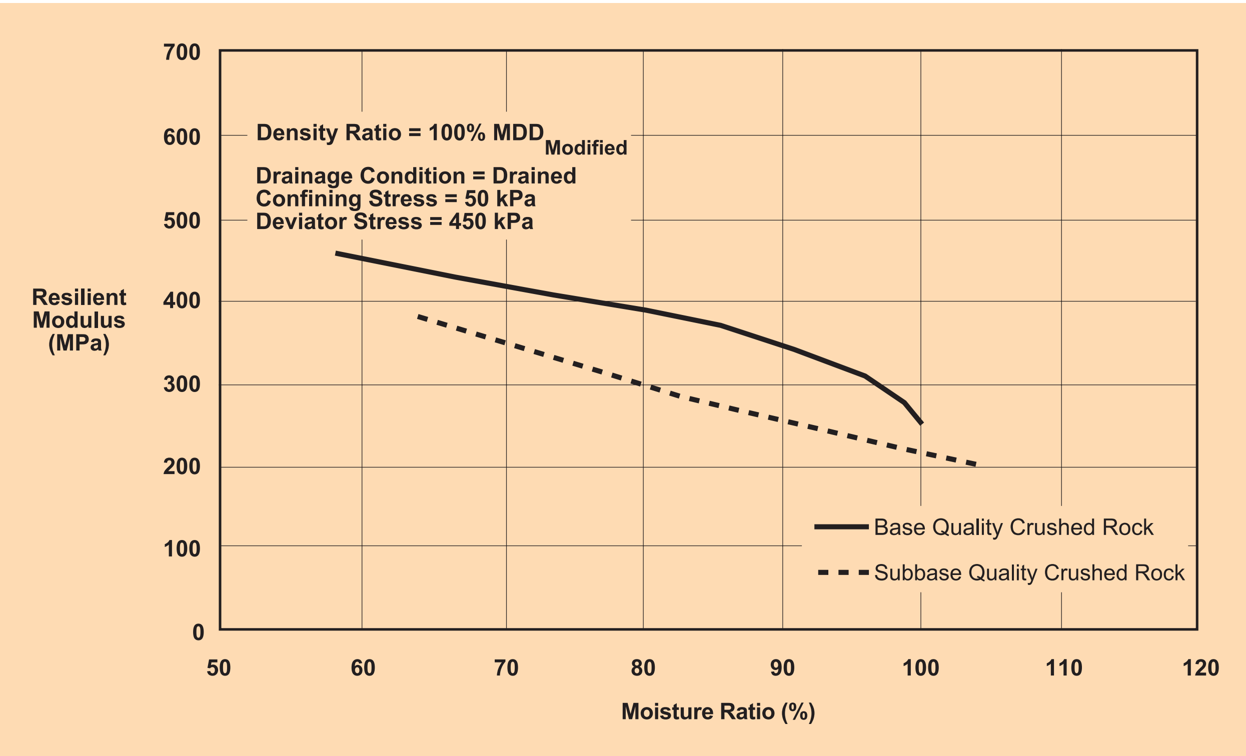


Figure 6.2 Typical relationships between modulus and moisture content

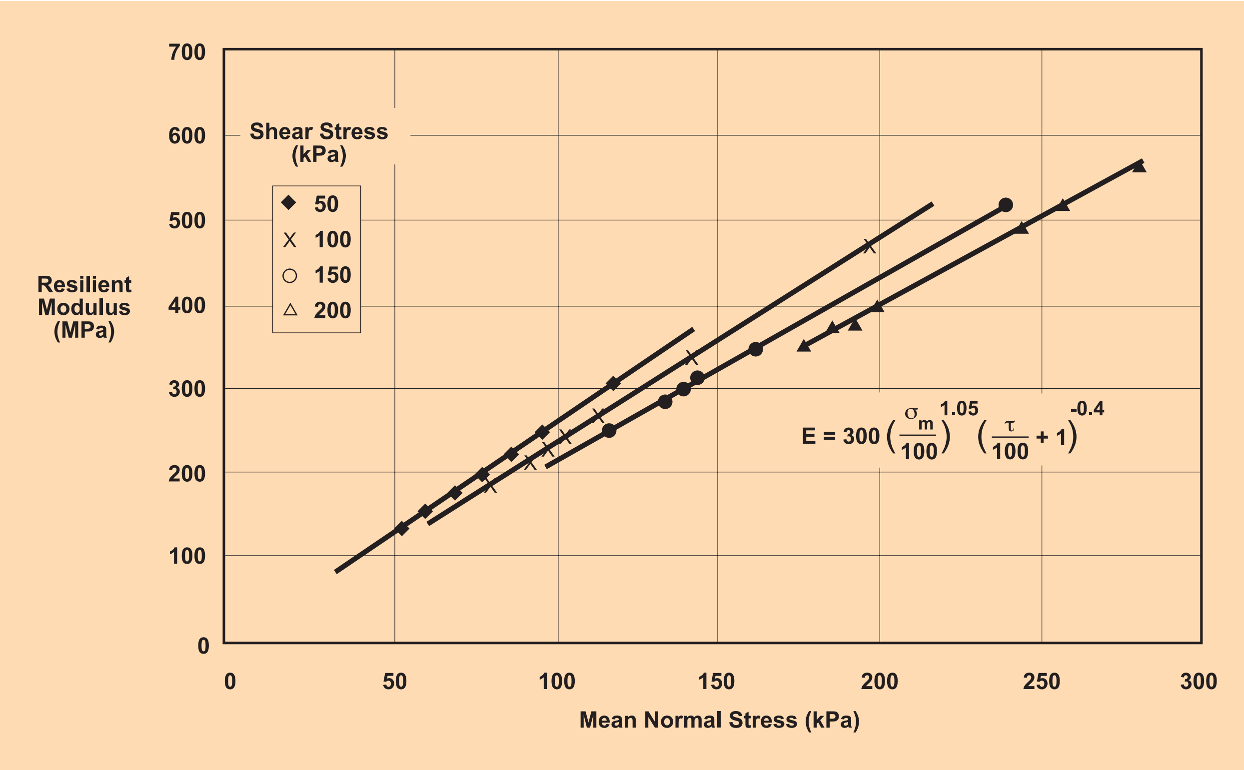


Figure 6.3 Typical relationships between modulus and stress level

6.2.2.2 *Stress level*

Modulus is influenced strongly by stress level, to an extent and nature dependent on material type. For unbound granular materials, modulus increases markedly with increasing mean normal stress and decreases with increasing shear stress, as shown typically by the following relationship (Uzan 1985):

$$E = K_1 \times \left(\frac{\sigma_m}{\sigma_{ref}}\right)^{K_2} \times \left(\frac{\tau}{\sigma_{ref}} + 1\right)^{K_3} \qquad (6.1)$$

where

E = modulus (MPa),

σ_m = mean normal stress (kPa),

τ = shear stress,

σ_{ref} = reference stress (atmospheric pressure = 100 kPa), and

K_1, K_2, K_3 are experimental test constants.

However, fine-grained soils commonly display a different response as modulus does not increases with increasing mean normal stress, but decreases with increasing shear stress level until a certain value is reached, after which it tends to increase slightly.

Figure 6.3 provides examples of relationships between modulus and stress level for an unbound material obtained from laboratory repeated load triaxial testing. It is apparent that the design modulus needs to be determined at stress conditions consistent with those which will occur in service.

6.2.3 Determination of modulus of unbound granular materials

6.2.3.1 *Definition of design modulus*

For pavement design purposes the appropriate value of the modulus of granular materials is the modulus obtained from laboratory repeated load triaxial testing at the material's insitu density, moisture content and stress levels under a Standard Axle.

In the procedures contained in this Guide it is necessary only to assign a modulus to the top portion of the granular layer. In determining values for other sublayers, it must be recognised that, in many cases, modulus will be controlled by the stiffness of underlying layers rather than the intrinsic characteristics of the granular material. A procedure for assigning these values is given in Section 8.2.3.

6.2.3.2 *Determination of modulus of top granular layer*

There are two recommended methods for determining the modulus of the top granular layer, which in order of preference are:

- direct measurement, and
- assigning presumptive values, e.g. *Table 6.3, 6.4(a)* and *6.4(b)*.

Direct measurement

In this, the preferred procedure, modulus is measured in a triaxial cell under conditions of repetitive loading. The recoverable portion of the axial deformation response is used in calculating the modulus.

As modulus is sensitive to stress level, moisture and density conditions, it is essential that laboratory test conditions approximate quite closely those which will occur in service.

Current technology for repeated load triaxial testing can, at best, only approximate the dynamic stress conditions in a pavement layer under a rolling wheel load. For routine material characterisation, a simplified test procedure has been developed (Vuong and Brimble 2000). In this simplified test, recently-developed, low-cost pneumatic equipment is used. The test is drained to avoid pore pressure build up. A cylindrical sample 100 mm in diameter and 200 mm high is used, which allows for testing of material up to 20 mm maximum particle size. The approximated stress conditions include static confining pressure and a range of stress levels for base, upper subbase and lower subbase materials. For simplicity, off-sample vertical strain measurement is adopted. This test has been found to produce acceptable accuracy for modulus determination up to approximately 700 MPa to 1000 MPa.

More sophisticated equipment and procedures may be required in circumstances where pore pressure build-up is anticipated, or for consideration of the effects of dynamic confining pressure, to more closely simulate field stress conditions. In addition, for stiffer materials, on-sample strain measurement is usually required to reduce system effects in the measurement of low strain values.

The assumptions and simplifications inherent in the test equipment and test protocol selected must be considered in the determination of associated design moduli from the laboratory results. For the routine test procedure and equipment described by Vuong and Brimble (2000), relationships are being developed to relate the laboratory-based modulus with expected design/field moduli. Until this work is completed a conservative approach should be adopted in translating the laboratory-based moduli into design moduli.

Table 6.3 Presumptive values for elastic characterisation of unbound granular materials under thin bituminous surfacings

Elastic property	Base quality materials			Subbase quality materials
	High standard crushed rock	Normal standard crushed rock	Base quality gravel	
Range of modulus (vertical MPa)	300–700	200–500	150–400	150–400
Typical modulus (vertical MPa)	500	350	300	250*
Degree of anisotropy†	2	2	2	2
Range of Poisson's ratio (vertical, horizontal and cross)	0.25–0.4	0.25–0.4	0.25–0.4	0.25–0.4
Typical value of Poisson's ratio	0.35	0.35	0.35	0.35
f (see CIRCLY Manual)	Given by formula $f = \dfrac{\text{Vertical Modulus}}{(1 + \text{Poisson's ratio})}$			

* The values are those at typical subbase stress level in unbound granular pavements with thin bituminous surfacings.

† $\text{Degree of Anisotropy} = \dfrac{\text{Vertical Modulus}}{\text{Horizontal Modulus}}$

If stress levels in service are likely to be different from those used during testing, then the modulus value should be obtained by extrapolation. Laboratory-determined modulus-stress relationships may be used to assign moduli within an unbound granular layer. However, as this area of pavement modelling is still under development, specific procedures are not presently available.

Presumptive values

Because the moduli of unbound granular materials are stress dependent – and dependent on moisture and compaction levels in the road bed – caution must be exercised in applying published data. *Table 6.3* may be used as a guide when assigning maximum values to granular materials under thin bituminous surfacings when other, more reliable, information is unavailable.

Table 6.3 includes values for two qualities of base quality crushed rock: high standard and normal standard. Normal standard crushed rocks are those commonly used by road authorities. High standard base crushed rocks are those that:

- Are manufactured from sound and durable igneous and metamorphic rock.
- Have high durability, strength and shear strength and specified in a way that includes clay type and quantity, permeability, modulus and performance under repeated loading with the in-service moisture content.
- Are manufactured in a highly processed and controlled manner to tight tolerances with respect to durability, hardness, grading, Atterberg limits etc.
- Are placed to very high standards with respect to density, degree of saturation, level, thickness, shape, rideability etc. For example, insitu density a minimum of 100% characteristic (modified compaction).
- Have a very high level of quality control using on-site testing facilities and quality assurance based on lot testing of stockpiled materials.
- Are part of an overall design that addresses essential issues including,
  - protection from the infiltration of water from all sources (side, below and surface);
  - the construction platform; and
  - the surface course.

Table 6.4 may be used as a guide when assigning maximum values to base quality crushed rocks materials under asphalt surfacings when other, more reliable, information is unavailable. For thick asphalt pavements where the underlying granular material of subbase quality has been selected, the assigned moduli varies with material quality. The following may be used as a guide to selecting the maximum moduli of subbase materials: for high quality crushed rock subbases, which have a laboratory soaked CBR greater than 30%, an assigned maximum moduli of 210 MPa may be used, otherwise a value of 150 MPa may be assigned.

Some pavements comprise layers of different moduli of bound material (asphalt or cemented material) overlying granular layers. For these pavements an equivalent modulus (E_e) of the bound material may be calculated using the following equation:

$$E_e = \frac{\sum_i t_i E_i}{T} \qquad (6.2)$$

where

E_e = equivalent modulus of total thickness of bound material (MPa),

E_i = modulus of layer i (MPa),

t_i = thickness of layer i (mm), and

T = total thickness of overlying bound materials (mm).

Based on the values of E_e and T so calculated, the vertical modulus of the top sublayer of granular material may then be determined from *Table 6.4*.

6.2.4 Permanent deformation

Permanent deformation of granular material – manifest as rutting and shoving, particularly along the outer wheelpath near the pavement shoulder – results from the material having insufficient stability to cope with the prevailing loading and environmental conditions.

While it is acknowledged that permanent deformation is a primary distress mode for granular layers, it is not included in the procedures detailed in Chapter 8 of the Guide. At this stage, there is not a suitable model to reliably predict the development of rutting in a granular material under traffic. However, a number of models are available which can be used to estimate rutting potential (e.g. VESYS) for material ranking (comparative) purposes.

Table 6.4(a) Suggested vertical modulus (MPa) of top sublayer of normal standard base material

Thickness of overlying material	Modulus of cover material* (MPa)				
	1000	2000	3000	4000	5000
40 mm	350	350	350	350	350
75 mm	350	350	340	320	310
100 mm	350	310	290	270	250
125 mm	320	270	240	220	200
150 mm	280	230	190	160	150
175 mm	250	190	150	150	150
200 mm	220	150	150	150	150
225 mm	180	150	150	150	150
≥250 mm	150	150	150	150	150

* Cover material is either asphalt or cemented material or a combination of these materials.

Table 6.4(b) Suggested vertical modulus (MPa) of top sublayer of high standard base material

Thickness of overlying material	Modulus of cover material* (MPa)				
	1000	2000	3000	4000	5000
40 mm	500	500	500	500	500
75 mm	500	500	480	460	440
100 mm	500	450	410	390	360
125 mm	450	390	350	310	280
150 mm	400	330	280	240	210
175 mm	360	270	210	210	210
200 mm	310	270	210	210	210
225 mm	260	210	210	210	210
≥250 mm	210	210	210	210	210

* Cover material is either asphalt or cemented material or a combination of these materials.

To estimate rutting potential using models such as VESYS, the simplified test procedure developed by the Austroads Working Group of Repeated Load Triaxial Test Users (AWGRU 2000) (Vuong and Brimble 2000) can be used. This permanent deformation testing procedure characterises the vertical permanent deformation at three stress conditions, viz. using three levels of dynamic vertical stress of 350, 450 and 550 kPa and a static lateral stress of 50 kPa, each stress condition consisting of 10 000 repetitions. Based on the test results, stress-dependent characteristics of permanent strain for the material can be determined. Multiple tests at different density and moisture conditions may be required to assess the sensitivity of moisture and density of the material.

Figure 6.4 shows typical examples of relationships between permanent deformation and loading cycles for base and subbase quality crushed rocks. Several test values (e.g. deformation rate per 1000 cycles, and maximum deformation after each loading stage) are extracted from the test results for use in assessing the potential for permanent deformation in the field. Relationships are currently being developed to relate the laboratory-based permanent deformation with expected design/field pavement rutting and shoving. As this area of pavement modelling is still under development, specific procedures for translating the laboratory-based permanent deformation into design are not presently available.

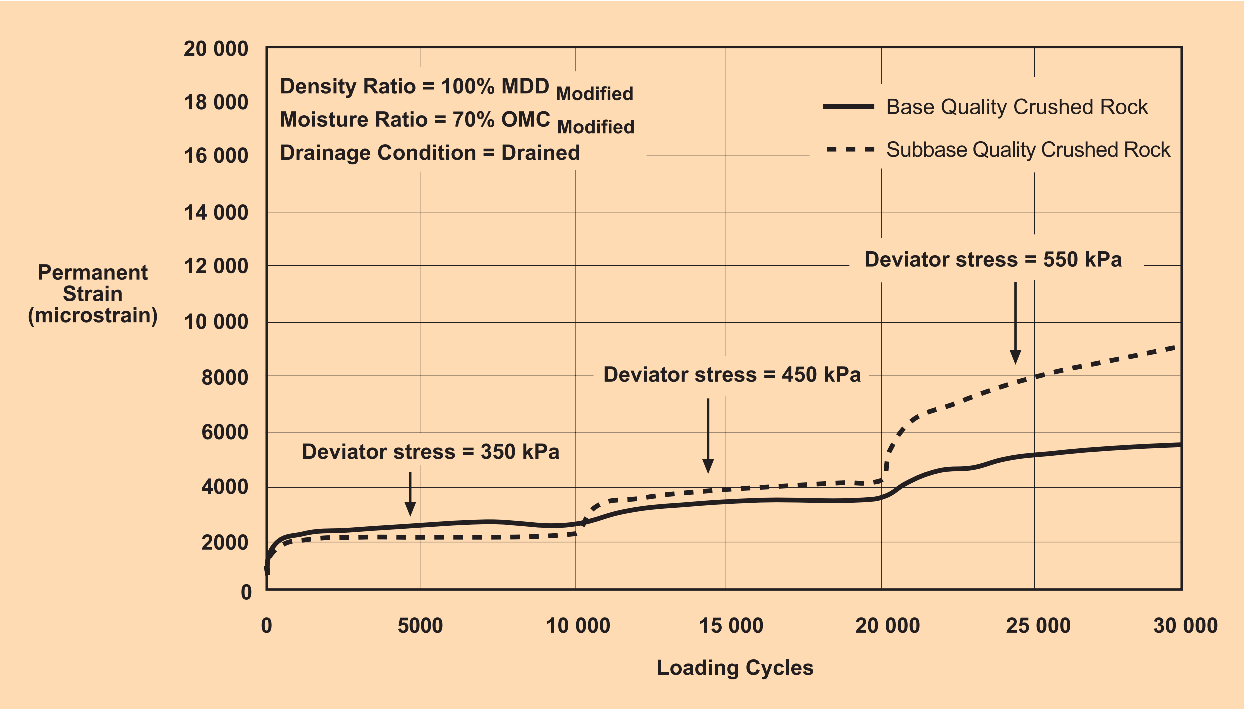


Figure 6.4 Typical relationships between permanent strain and loading cycles

More sophisticated equipment and procedures may be required in circumstances where pore pressure build-up is anticipated, or for consideration of the effects of dynamic confining pressure, to more closely simulate field stress conditions.

6.3 Modified granular materials

Modified granular materials are granular materials to which small amounts of stabilising agents have been added to improve stiffness or to correct other deficiencies in properties (e.g. by reducing plasticity) without causing a significant increase in tensile capacity (i.e. producing a bound material). Modified granular materials are considered to behave as unbound granular materials, i.e they do not develop tensile strain under load. In practice this condition can be difficult to ensure unless the stabilised material is reworked after most of the cementing action has occurred.

Modified materials are considered to behave as unbound granular materials and are characterised and modelled in the same manner. They are considered to:

- be cross-anisotropic (with a degree of anisotropy of 2);
- have a Poisson's ratio of 0.35; and
- require sublayering for modelling purposes, as for unbound granular materials.

The modulus of modified materials may be measured as described above for unbound granular materials. There are no firmly established criteria to differentiate between modified and bound materials. However, for reasonably well-graded base materials, values of 80 kPa for indirect tensile strength, 0.8 MPa for UCS (7 days moist curing) or a resilient modulus of 700–1500 MPa are values that could be used as a guide (Austroads 1998).

Bitumen stabilisation using low quantities of bitumen, typically less than 2%, can produce a modified granular material having a resilient modulus typically exceeding 700 MPa. The addition of cement or lime with bitumen may produce a bound material. In such cases laboratory testing should be undertaken to confirm the material has a modulus within the range for modified materials (700–1500 MPa).

6.4 Cemented materials

6.4.1 Introduction

6.4.1.1 Main characteristics

Cemented materials may be described as a combination of a cementitious binding agent, water and granular material which are mixed together and compacted in the early stages of the hydration process to form a pavement layer which is subsequently cured. The cementitious binder may consist of Portland cement, blended cement, lime, or other hydraulically-binding agent and may include one or more supplementary cementitious materials such as fly ash or ground granulated blast furnace slag. The binder should be added in sufficient quantity to produce a bound layer with significant tensile strength.

General categories and characteristics of cemented materials are given in *Table 6.1*. Further guidance can be gained from the Austroads (1998).

6.4.1.2 Characterisation for pavement design

Cemented materials are considered to be isotropic. Their elastic response may be regarded as linear in the normal operating stress ranges of pavements. They are characterised by an elastic modulus and Poisson's ratio. Due to similarities with the loading regime in service, flexural modulus is the preferred design input. The Poisson's ratio may be shown to have relatively little influence on pavement thickness requirements within the normal range. Commonly, Poisson's ratio is assumed to be 0.20.

In selecting appropriate parameters for design, the importance of each of these factors must be considered for the particular case. For further guidance, Austroads (1998), Jameson *et al.* (1995) and Ingles and Metcalf (1972) should be consulted.

6.4.2 Factors affecting modulus of cemented materials

Factors affecting the modulus of cemented materials – and the effects of increasing these factors – are shown in *Table 6.5*.

Table 6.5 Factors affecting modulus of cemented materials and effect of increasing factor values

Factor	Effect of increasing factor
proportion of coarse angular particles	increase
density	increase
compaction moisture content	increase up to an optimum value and then decrease
stress level	no change
cementitious binder content	increase
in-service moisture content	slight decrease
age	increase
extent of cracking	decrease
efficiency of mixing	increase
temperature	no change
rate of loading	no change

6.4.2.1 Mix composition

Fine-grained materials will normally have a lower modulus than well-graded materials. The modulus of cementitious materials is not particularly temperature sensitive, in contrast to asphalt.

The binder content significantly affects the modulus of material compacted to a specific density. Generally, for bound materials, the binder content is in excess of 3% by mass. Whilst the modulus increases with increased binder content, as the binder content increases so does the potential for drying/shrinkage cracking.

6.4.2.2 Density and moisture

These factors are inter-related: varying the moisture content (from optimum) will generally result in a decrease in density for a given compactive effort. Adequate compaction greatly increases the stiffness and improves the performance of cemented materials. Increased resistance to compaction occurs as a result of the rapid formation of cementitious bonds that resist the applied compactive effort for rapid-setting binders such as General Purpose cement. Compaction must be completed as soon as possible after the addition of the binder and water within the early stages of the hydration process. Slower a setting binders and retarders can extend the working time of cemented materials.

Cemented materials are usually placed and compacted in single layers to eliminate the early pavement deterioration that can result when sublayers are not bound together. For layer thicknesses in excess of

200 mm, consideration needs to be given to the lower density of the material in the lower half of the layer (e.g. see Moffatt *et al.* 1998). In such cases, consideration must be given to the effects of any density gradient in the adoption of a representative modulus for the characterisation of the full cemented layer. Alternatively, different moduli should be selected for the top and bottom halves of the layer.

Despite the likelihood of density gradients in thick cemented layers, this is generally preferable to having two thin layers, as any debonding between these layers can lead to substantial reductions in pavement performance (e.g. see Kadar, Baran and Gordon 1989).

6.4.2.3 Ageing and curing

The modulus of cemented materials increases rapidly in the first one or two days, after which the stiffness increases slowly, providing curing is sustained. The variability of properties over time is dependent on both the granular material and binder type (e.g. see Moffatt *et al.* 1998).

Curing is necessary to ensure that there is adequate water for the hydration reactions to proceed and that drying shrinkage is limited while the hydration reactions are proceeding and the material is strengthening. The design flexural modulus is normally adopted at 28 days' curing.

6.4.3 Determination of design modulus

6.4.3.1 Definition of design modulus

For pavement design purposes the appropriate value of the modulus of cemented materials is an estimate of the insitu flexural modulus after 28 days' curing in the road-bed.

Note that for some slow-setting binders, there can be significant increase in strength after 28 days (Jameson *et al.* 1995). For such binder, cognisance should be taken of this strength increase in selecting a design modulus for the test results at 28 days.

6.4.3.2 Alternative methods

Design moduli may be estimated from:

- the results of laboratory flexural beam tests on the cemented material to be placed in the pavement;
- correlations to estimate flexural beam values from other laboratory tests; and
- presumptive values.

Issues to be considered with the laboratory determination of the design modulus of cemented materials include:

- the availability of test equipment and test protocols, and the suitability of this equipment and protocols for the determination of flexural modulus;
- material properties, sample preparation procedures and curing environment;
- binder type and rate of strength development with time, sample age at testing;
- the time available for laboratory characterisation with respect to the required curing time and the construction schedule; and
- relationships between the results of the laboratory characterisation and the properties of the material cured insitu.

As there is not currently a standardised test for the determination of flexural modulus for design purposes, laboratory characterisation is often used simply to verify assumed presumptive design values or to optimise binder/host material combinations in terms of stiffness and/or working time. Samples prepared and cured under laboratory conditions may not accurately reflect the insitu placement process. Similarly, the age at testing may not allow adequate consideration of longer-term strength gains, particularly with slow-setting binders. Forward planning of laboratory testing is essential due to the long curing times required, sometimes in excess of 28 days. Accelerated curing may be adopted – involving increased curing temperatures – provided a clear correlation with longer-term, normal cure strength gains over time are established for the material/binder combination.

6.4.3.3 Laboratory measurement

A number of laboratory tests can be used to measure the modulus of cemented materials, including: flexural test, direct tension test, longitudinal vibration test, indirect tensile test and the direct compression test.

Third-point loading of flexure (beam) specimens (with a span/depth ratio greater than 3) is favoured for modulus characterisation as the test conditions are considered to simulate the stress/strain gradients generated within a pavement layer. The modulus is obtained from the straight line portion of the load-deflection plot using beam loading theory. Alternatively, direct tension testing may be used, with the modulus determined from the linear portion of the stress strain plot.

Dynamic methods involve the measurement of resonant frequency and have the advantage that testing is non-destructive. However, it must be recognised that, due to the small strains associated with these measurements, the moduli obtained are typically 10–15% higher than those determined by static loading tests. The indirect tensile test can also be used to estimate the flexural modulus. The test may be conducted on readily-available equipment developed for asphalt characterisation (refer to Section 6.5.3.3).

The remaining test methods may be used to estimate the modulus but the results should be viewed with caution and considered as a guide only (particularly compression testing).

6.4.3.4 Modulus correlations

Relationships have been developed between elastic modulus and other parameters such as Unconfined Compressive Strength (UCS). A typical relationship for cemented crushed rock and cemented natural gravel is:

$$E_{FLEX} = k\ UCS \tag{6.3}$$

where

E_{FLEX} = flexural modulus of field beams at 28 days moist curing (MPa), and

UCS = Unconfined Compressive Strength of laboratory specimens at 28 days (MPa).

k = values of 1000 to 1250 are typically used for General Purpose cements, the value depending on laboratory testing practices and construction specifications for cemented materials

It should be noted that this relationship was based on laboratory test results obtained for overseas materials with a range of binder contents (Alderson 1999, Moffatt and Yeo 1998; Yeo 1997; Otte 1978; Mitchell 1976). The equation should be used as a guide only as there was significant scatter in the data because a range of materials and binder types were tested. This linear relationship is applicable to 28 day UCS values up to 5 MPa.

As the Indirect Tensile Strength of cemented materials is generally approximately $\frac{1}{8}$ to $\frac{1}{10}$ of the UCS value, this parameter may also be used to estimate modulus.

6.4.3.5 Presumptive values

The moduli of cemented materials are dependent on a number of factors, two of the most important being binder content and density. Presumptive values cannot account for variations in these important parameters and thus should be treated with caution. The modulus values presented in *Table 6.6* are considered appropriate for 100% standard compactive effort and may be used as a guide if no other more reliable information is available.

6.4.3.6 Presumptive modulus: post-fatigue cracking

Following the initial fatigue cracking, further cracking and degradation of the cemented layer may occur, resulting in a reduction in the stiffness to a value similar to that of the unbound granular material from which the cemented material was derived. In situations where the cemented layer is a subbase beneath a granular or asphalt base of thickness greater than 175 mm (see

Table 6.6 Presumptive values for elastic characterisation of cemented materials

Property	Lean mix concrete	Base 4–5% cement	Subbase quality crushed rock 2–4% cement	Subbase quality natural gravel 4–5% cement
Range of modulus (MPa)	5000–15 000	3000–8000	2000–5000	1500–3000
Typical modulus (MPa)	7000 (Rolled) 10 000 (Screeded)	5000	3500	2000
Degree of anisotropy[†]	1	1	1	1
Range of Poisson's ratio (vertical, horizontal and cross)	0.1–0.3	0.1–0.3	0.1–0.3	0.1–0.3
Typical value of Poisson's ratio	0.2	0.2	0.2	0.2

* Although figures are only quoted for cement, other cementing agents such as lime, lime fly ash, cement fly ash and granulated slag may be used. The stiffness of such materials should be determined by testing (refer to *Austroads Guide to Stabilisation in Roadworks*).

† Degree of Anisotropy $= \dfrac{\text{Vertical Modulus}}{\text{Horizontal Modulus}}$

Section 8.2.4), the post-fatigue cracking life may be estimated as per Section 8.2.4.

For the purpose of mechanistic modelling in the post-fatigue phase, cemented materials may be assumed to have a presumptive vertical modulus of 500 MPa and a Poisson's ratio of 0.35. The layer is not sublayered and is considered to be cross-anisotropic, with a degree of anisotropy of 2.

6.4.4 Factors affecting the fatigue life of cemented materials

The principal factors affecting the fatigue life of cemented materials include: material strength, material modulus, density, moisture content, mixing efficiency, and cracking pattern.

Some of these factors are in turn dependent on binder type and content, etc.

6.4.4.1 Material strength and modulus

For a given strain level, fatigue resistance decreases with increasing strength, i.e. stronger/stiffer (brittle) materials may tolerate fewer repetitions of load at a given strain level than weaker (more flexible) materials. However, as the strains generated in a cemented pavement having a high modulus are generally quite low, a longer life is realised compared with a cemented pavement having a low modulus.

6.4.4.2 Dry density and moisture content

As a general rule, an increase in density results in an increase in the fatigue life of cemented materials.

As an increase in moisture content beyond optimum results in a decrease in modulus, it would be expected that an increase in moisture content would result in a decrease in fatigue life.

6.4.4.3 Mixing efficiency and uniformity of binder content

Mixing efficiency plays an important part in ensuring the strength and modulus of cemented materials. Inefficient mixing may result in pockets of material which are not mixed with binder, thereby resulting in a zone of weakness. Stress concentrations may occur around these areas and hence lower the fatigue life. Significantly better uniformity of mixing can be achieved by the use of purpose-built batching plants or purpose-built insitu stabilisation equipment for binder application and mixing (Austroads 1998).

6.4.4.4 Cracking

Cracking in cemented materials is normal, and inevitable, due to thermal and shrinkage stresses resulting from hydration of the binder. The extent of cracking is significantly influenced by the plasticity of the material to be stabilised and the binder type and content.

The effect of cracking in cemented materials on pavement performance depends upon factors such as the:

- durability of the cemented materials;
- presence, and type, of subbase;
- location of the cemented layer in the pavement structure;
- type and thickness of material overlying the cemented layer;
- width of cracks (narrow cracks are less likely to reflect through to the surface than wide cracks and are easier to bridge and keep sealed if reflection does occur); and
- effectiveness of crack sealing methods.

Any cracking in pavement surface layers has the potential to allow water entry. This frequently accelerates distress through weakening of the pavement and subgrade layers, erosion of cemented material or pumping of fines from below and between cemented layers. Often asphalt or granular material is placed over cemented materials to minimise reflection cracking. VicRoads (1993) and Queensland Transport (1990) provide lists of measures that may be considered to reduce shrinkage cracking in cemented layers. A selection of these measures is as follows:

- Minimise the total cementitious binder content – the lower the binder content, the lower the moisture required, and the less the shrinkage. However, this renders the material susceptible to erosion when subjected to moisture ingress under loading.
- Use slow-setting binders, which promote slightly less shrinkage than General Purpose (GP) cement. These binders are also likely to require less moisture for compaction which also reduces shrinkage.
- Minimise the clay content of the material to be cemented by controlling the amount and plasticity of fines in the aggregate. This can be achieved by limiting the fines content to less than 20% passing

the 75 µm sieve and the plasticity index to values not greater than 20.

- Treat the existing pavement materials which have an excess of plastic fines by:
  - (a) pre-treating with lime or lime and cement, followed by stabilisation with fly ash blend cement;
  - (b) mixing in gravel or crushed rock with little or no fines, the amount of material varying with the plasticity and fines content of the existing pavement compared to the desirable levels and the proposed depth of stabilisation;
  - (c) applying both (a) and (b); or
  - (d) using the existing material as a subbase only or, alternatively, programming for an early overlay.
- Place a bituminous curing coat as soon as possible after construction to inhibit rapid drying out of the cemented layer and delay surfacing as long as possible so that cracking occurs before surface placement.

In addition, whilst the following two measures do not serve to minimise shrinkage cracking they do ameliorate the influence of shrinkage cracking on overlying layers:

- In situations where the final seal is to be placed immediately following curing, apply a SAM (or a SAMI) or geotextile seal to inhibit potential shrinkage cracking of the surfacing.
- Use an appropriate polymer modified binder (PMB) asphalt surfacing in preference to conventional asphalt (Austroads 2003).

The benefits of these treatments are not reflected by the design process because the design model is not capable of predicting the onset and development of reflection cracking. Therefore, similar pavement compositions and structures will result regardless of the presence of these treatments. The benefits, however, can be shown in terms of an improved reliability of the design by providing a surfacing less prone to the onset and development of reflection cracking. The use of SBS or crumb rubber modifiers has been shown to provide a more elastic response and hence, provide a surface with a greater capacity to resist reflection cracking. Further discussion on the selection of appropriate PMBs for this type of application can be found in the *Austroads Specification for Polymer Modified Binders* (Austroads, 2001).

6.4.5 Means of determining the fatigue characteristics of cemented materials

6.4.5.1 Alternative methods

Fatigue characteristics of cemented materials may be determined through laboratory testing, preferably in conjunction with field trials, or by adopting relationships contained in the literature.

6.4.5.2 Laboratory fatigue measurement

As already discussed, laboratory fatigue testing of cemented materials can be carried out using flexure testing or, to a lesser degree, direct tension testing and indirect tensile testing. The flexure test, in particular, is favoured as it is considered to simulate field stress/strain gradients. However, the test procedure is still under development.

6.4.5.3 Correlations

Static testing may be used to obtain an approximate estimate of fatigue life. Research work has indicated that, for common material types and binder contents, approximately 50% of the strain at break corresponds to a fatigue life of some one million load repetitions. In the absence of more accurate data, this relationship may be used to estimate the fatigue life by determining strain at break from a static test and linearly interpolating the fatigue life for strain levels greater than 50% of the breaking strain. The fatigue life thus obtained should be considered as indicative only.

6.4.5.4 Fatigue criteria

Fatigue relationships have been derived for cemented materials having various modulus values and they may be used to give an indication of fatigue life. The relationships have been derived from overseas research work and may be used where more reliable information is unavailable.

Limited information is available on the in-service fatigue of cemented materials used in Australia, except for several Accelerated Loading Facility (ALF) trials (e.g. Jameson *et al.* 1992, 1995 and 1996; Vuong *et al.* 1996).

These relationships given in the following general equation, and shown in *Figure 6.5*, generally have been found to be in accordance with observed performance:

$$N = RF\left[\frac{(113\,000/E^{0.804} + 191)}{\mu\varepsilon}\right]^{12} \qquad (6.4)$$

where

N = allowable number of repetitions of the load;

$\mu\varepsilon$ = tensile strain produced by the load (microstrain);

E = cemented material modulus (MPa); and

RF = reliability factor for cemented materials fatigue (*Table 6.7*).

Table 6.7 Suggested Reliability Factors (RF) for cemented materials fatigue

Desired project reliability				
80%	85%	90%	95%	97.5%
4.7	3.3	2.0	1.0	0.5

This fatigue criteria is valid for cemented materials with moduli within the range 2000–10,000 MPa.

Guidance on the selection of desired project reliability levels is provided in Section 2.2.1.2 of this Guide.

6.5 Asphalt

6.5.1 Introduction

Asphalt is a mixture of bituminous binder and several, typically, single-sized aggregate fractions which is spread and compacted while hot, to form a pavement layer. While the binder is usually a conventional bitumen, for special applications it may be modified by the addition of specific polymers to the bitumen.

The strength/stiffness of asphalt is derived from friction between the aggregate particles, the viscosity of the bituminous binder under operating conditions and the cohesion within the mass resulting from the binder itself, and the adhesion between the binder and the aggregate. The most common modes of distress for asphalt layers are: (1) rutting and shoving due to insufficient resistance to permanent deformation, and (2) cracking due to fatigue.

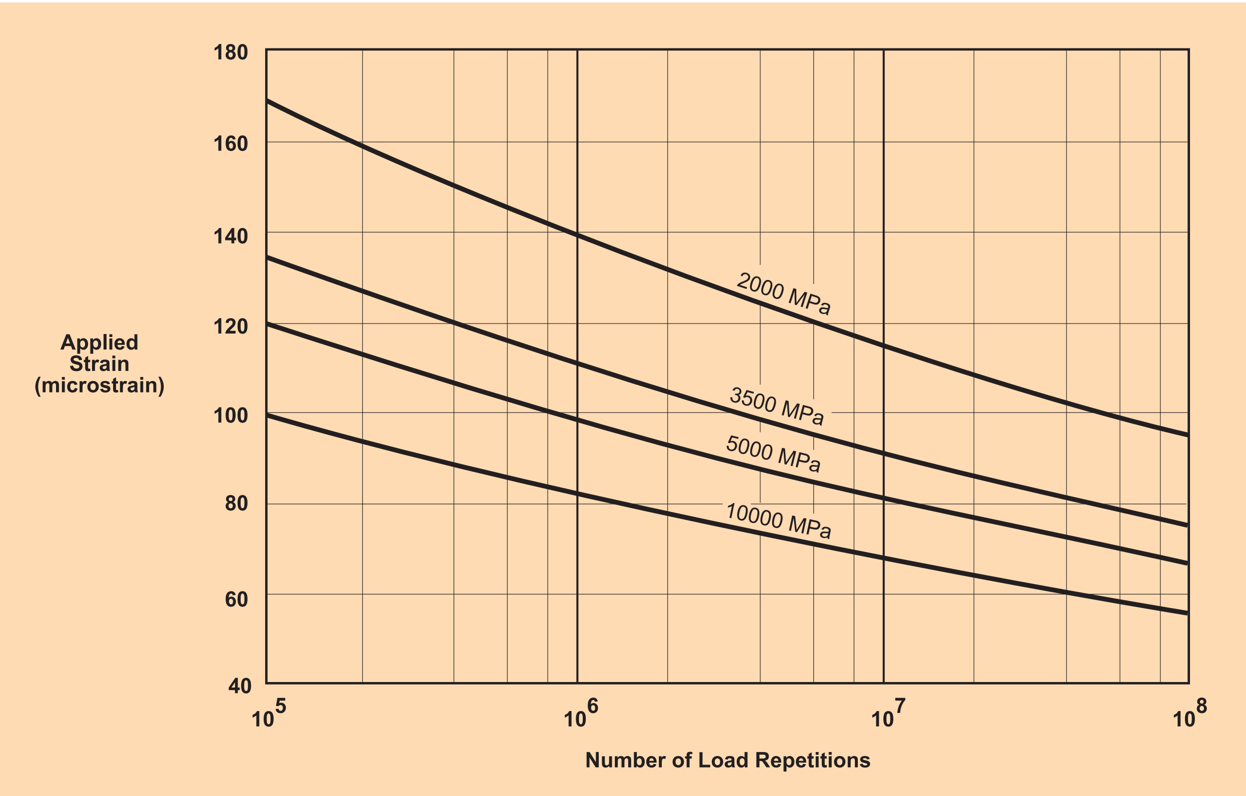


Figure 6.5 Fatigue relationships for various types of cemented materials for a desired Project Reliability of 95%

General categories and characteristics of asphalt are given in *Table 6.1*. Further details on asphalt mix design and asphalt technology are contained in APRG Report No. 18 (*Selection and Design of Asphalt Mixes: Australian Provisional Guide*) (APRG 1998) and the *Austroads Asphalt Guide* (Austroads 2003*a*).

The use of polymer modified binders and multigrade bitumen in heavy duty asphalt wearing course applications has progressed over the past two decades to become common practice. While the modulus and fatigue properties of these improved binders and asphalt mixes can be determined in a controlled laboratory facility, the actual field performance in a range of environments and loading regimes has proven relatively difficult to characterise.

6.5.1.1 Characterisation for pavement design

Although in reality the bituminous binder provides the mix with its viscoelastic properties, at the normal operating temperatures and rates and magnitudes of loading which are applicable to road pavements, asphalt may be approximated, for structural analysis purposes, by an elastic solid, the stiffness of which depends on temperature and loading rate (traffic speed). Asphalt is considered to be isotropic.

The stiffness of asphalt when it is subject to compressive loading is greater than when it is subject to tensile loading. This property is of relevance in selecting appropriate laboratory test procedures for its characterisation. With appropriate characterisation, asphalt may be modelled as a linear elastic material.

6.5.2 Factors affecting stiffness of asphalt

Factors affecting the stiffness of asphalt – and the effects of increasing these factors – are shown in *Table 6.8*. Some of these factors are briefly discussed below.

6.5.2.1 Mix composition

Aggregate angularity and grading

The more angular the aggregates, the greater will be the mix stiffness. Aggregate angularity is particularly important in determining the stiffness and stability of the mix at high temperatures because the binder stiffness is lower.

In general, densely-graded aggregates produce a mix which has a greater stiffness than more open-graded or gap-graded aggregates.

Table 6.8 Factors affecting stiffness of asphalt and effect of increasing factor values

Factor	Effect of increasing factor
proportion of coarse angular particles	increase
density	increase
stress level	no change
age	increase
extent of cracking	decrease
efficiency of mixing	increase
bitumen content	increase then decrease
bitumen class	increase
bitumen viscosity	increase
per cent air voids	decrease
temperature	decrease
rate of loading	increase

Binder type

The characteristics of the binder are determined by the source of the bitumen, the refining process and the type and amount of any binder additives. As well as affecting the actual properties of asphalt under any given set of conditions, the binder characteristics also affect the sensitivity of properties such as asphalt stiffness to changes in temperature and rate of loading. Australian Standard AS2008 (Standards Australia 1997) classifies different types of conventional road-making bitumens on the basis of specific key characteristics. Similarly, the Austroads (2000 and 2001) classify types of multigrade and modified bitumens, respectively.

If the Shell nomographs (Shell 1978) are used to determine a value for asphalt modulus as described in Section 6.5.3.4, then bitumen is characterised by its Penetration Index (PI) and $T_{800\ pen}$ as defined in Section 6.5.3.4. These nomographs have been developed only for conventional bitumens and are not applicable to mixes containing multigrade bitumens or modified binders.

Binder content

The effect of binder content on modulus may be likened to the effect of moisture content on the strength of soil. At low percentages, added binder increases the mix cohesion and strength. Beyond a certain value, further increases in binder content reduce the frictional contact between aggregate particles and the overall stiffness and stability of the asphalt. The optimum binder content depends on the type and grading of aggregate, the degree of compaction of the mix and the operating temperature. It can best be determined by testing. The binder content at which the maximum modulus occurs is usually below that used in normal production mixes and thus, across the common range of binder contents, an increase in binder content is usually associated with a decrease in modulus.

Air voids

At binder contents below the optimum, increased air voids are associated with reduced stiffness. The effect is interrelated with the amount of aggregate and the level of compaction of the mix. As the air voids decrease with binder contents above the optimum, there is usually a decrease in mix stiffness.

6.5.2.2 Construction considerations

The major construction considerations relating to asphalt are the level of compaction and the uniformity of both the asphalt mix and its placement in the pavement. The level of compaction determines:

- the resistance to permanent deformation and the stiffness of the mix in service; and
- the percentage of air voids in the asphalt mix.

A typical relationship between modulus (a measure of stiffness) and air voids is shown in *Figure 6.6* which is based on laboratory test data. The effect of decreasing the air voids in a compacted asphalt layer is to increase fatigue life and decrease the rate of oxidation of the binder.

6.5.2.3 Temperature

A most important factor in determining the modulus of asphalt is temperature. Its effect, based on data obtained by testing typical Australian asphalt mixes, is shown in *Figure 6.7*. It can be seen that the modulus of asphalt can vary by up to an order of magnitude in the range of temperatures applicable to pavements, all else being equal. The temperature environment of asphalt must therefore be taken into account in pavement design and analysis.

Commonly, the effect of temperature variations is taken into account by estimating the asphalt modulus at the Weighted Mean Annual Pavement Temperature (WMAPT). WMAPT values for Australian and New Zealand cities are presented in *Appendix 6.1*, together with the method for calculating the WMAPT.

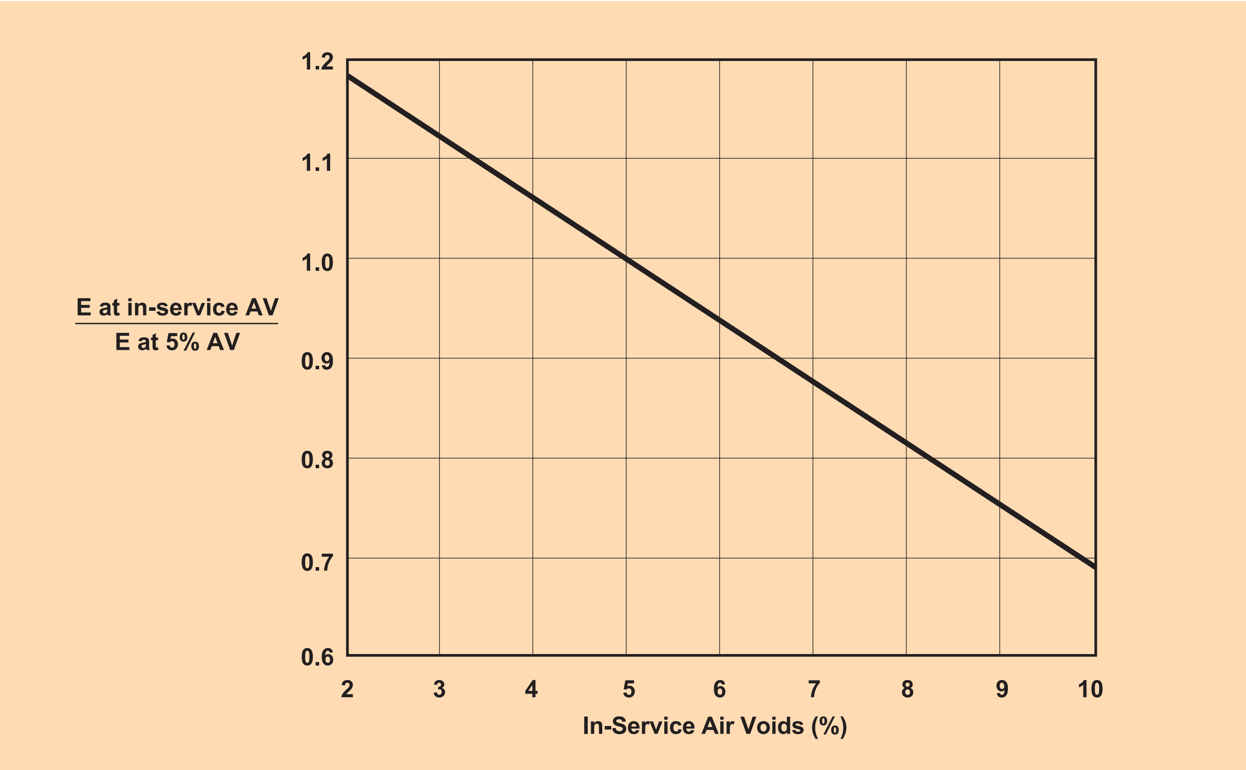


Figure 6.6 Variation of ratio of modulus at in-service air voids to modulus at 5% air voids with air voids content

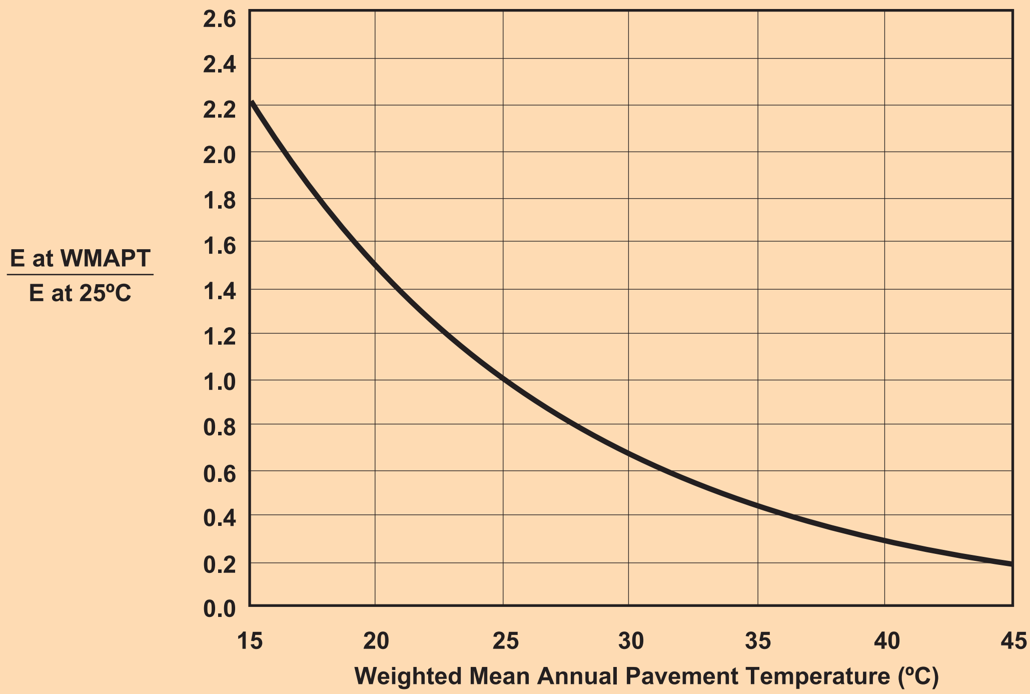


Figure 6.7 Variation of ratio of modulus at Weighted Mean Annual Pavement Temperature (E_{WMAPT}) to modulus from standard indirect tensile test (E_{test}) with Weighted Mean Annual Pavement Temperature (conventional mixes only)

6.5.2.4 Rate of loading (traffic speed)

Because of the viscoelastic nature of the bituminous binder, the stiffness of asphalt is dependent on the rate at which it is loaded – the slower the rate, the lower the modulus. This effect can be very significant, especially in pavement areas such as intersection approaches, bus stops and parking areas. When determining the modulus for a given traffic speed, the loading time used will depend on the type of testing device and the shape of the load pulse as well as the depth below the pavement surface at which the modulus is being sought.

The effect of rate of loading on modulus is indicated in *Figure 6.8*.

6.5.2.5 Age

The stiffness of asphalt has generally been found to increase with age. This is due to the cumulative loss of volatiles, steric hardening of the binder and the oxidation of constituents of the bituminous binder. The rate of increase with age is difficult to quantify as it depends on the binder type and binders, the percentage of air voids, binder content of the mix, depth below the surface and the local environment. Increases in modulus of the order of 400% after 20 years have been reported in overseas studies. The best methods for determining modulus at a given age are to sample and test the asphalt layer, to derive the modulus by back-calculation from loading tests on the pavement, or by sampling and testing the bitumen and using the Shell nomographs which are described in Section 6.5.3.4.

In terms of pavement thickness design, the effect of age on modulus and hence performance is taken into account in the asphalt fatigue relationship (see Section 6.5.6).

6.5.3 Determination of design asphalt modulus and Poisson's ratio

6.5.3.1 Definition of design modulus

For pavement design purposes, the appropriate value of asphalt modulus is an estimate of the value obtained from either:

- the resilient modulus measured using the standard indirect tensile test (ITT) adjusted to the in-service temperature (WMAPT) and for the rate of loading in the road-bed (Section 6.5.3.3), or

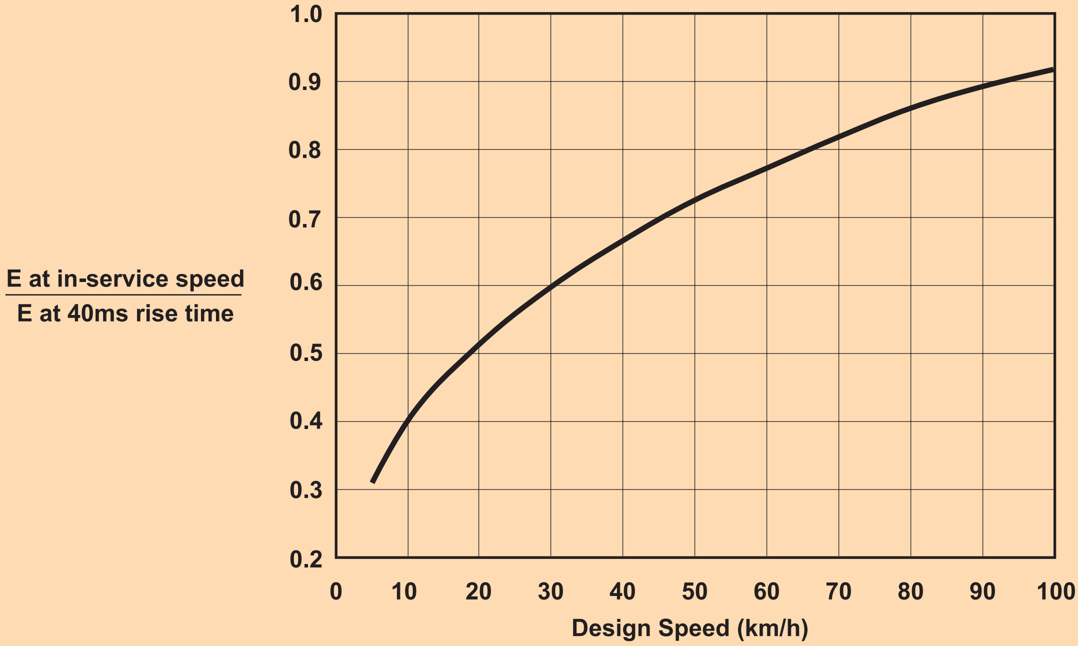


Figure 6.8 Variation of ratio of modulus at vehicle speed V (E_V) to modulus from standard indirect tensile test (40 ms rise time) with design speed (conventional mixes only)

- estimated from the bitumen properties and mix volumetrics using Shell nomographs and the in-service temperature (WMAPT) and rate of loading in the road-bed (Section 6.5.3.4).

If both of the above options are precluded, then the design moduli may be estimated by selecting a representative value of modulus from available published data. However, considerable care is needed in selecting a value which will represent the proposed asphalt mix in its field situation.

6.5.3.2 Poisson's ratio

Determination of a value for Poisson's ratio from laboratory testing is difficult. There is a further difficulty in interpreting the results obtained in the context of elastic characterisation of asphalt. While there is some evidence to suggest that its value is (to an extent) temperature-dependent, because its effect on the performance of asphalt in a pavement structure is secondary to the effect of asphalt modulus, a fixed value is commonly adopted for design purposes. Based on published data, it is recommended that a value of 0.4 be used.

6.5.3.3 Determination of design modulus from measured modulus

The indirect tensile test (Standards Australia 1995*a*) is the most commonly used laboratory test in Australia for the determination of asphalt modulus, because the testing equipment is relatively inexpensive and the test easy to conduct. Also, the laboratory preparation of test samples has become standardised (Standards Australia 1995*b*).

In this test, a pulsed load is applied to the diametral plane of a cylindrical specimen, while recording the extension of the perpendicular diametral plane. The rate of load application is pre-set by the user. Peak load is controlled to produce a nominal strain of 50 microstrain on the perpendicular diametral plane.

Standard Reference Test Conditions are 40 ms Rise Time (time for the applied load to increase from 10% to 90% of its peak value) and 25°C temperature, with a pulse repetition period of 3 seconds. While the stress and strain conditions developed within the specimen are complex and somewhat unrelated to those developed under traffic loading, pulsing of the load provides good simulation of loading produced by a succession of wheel loads.

Specimens should be prepared at the design binder content and grading and compacted to about 5% air voids in accordance with APRG 18 (1998) and tested in indirect tension. The resilient modulus assigned to a mix is typically the mean of triplicate specimens.

The results of resilient modulus tests can vary appreciably even between specimens of essentially the same composition tested on the same apparatus. Further variability is introduced due to variations in mix constituents and the limits of reproducibility of the test (i.e. the variability arising from testing the same specimen in two different laboratories). Due to this variability, designers are advised not to assign a high level of accuracy or precision to a design modulus determined from the mean of a single set of triplicate specimens. Consideration needs to be given to the number of resilient modulus results required to achieve a representative and statistically significant design modulus.

The steps involved in the determination of design modulus from laboratory indirect tensile test modulus testing are as follows:

1. Select the appropriate in-service air voids (AV) level representing the air voids level in service and consistent with the volume of binder used in the fatigue life calculations (Section 6.5.6).
2. Determine (from project information) a representative value for heavy vehicle traffic speed (V km/h).
3. Select the WMAPT for the project location (from *Appendix 6.1*).
4. Conduct the standard indirect tensile test on a laboratory compacted specimen at 5% air voids using a rise time of 40 ms and a test temperature of 25°C.
5. Using the following relationship, calculate the ratio of the modulus at the in-service air voids to the modulus of laboratory test specimen:

$$\frac{\text{Modulus at In-Service Air Voids}}{\text{Modulus at Test Air Voids}} = \frac{(21 - AV_{\text{In-Service}})}{(21 - AV_{\text{test}})} \tag{6.5}$$

   (This relationship is shown in *Figure 6.6*.)

   Correct the measured modulus for air voids by multiplying the measured modulus by this modulus ratio.

6. Using the following relationship, calculate the ratio of the modulus at the in-service temperature (WMAPT) to the modulus at the laboratory test temperature (25°C):

$$\frac{\text{Modulus at WMAPT}}{\text{Modulus at Test Temperature (T)}} = \exp(-0.08[\text{WMAPT}] - \text{T}]) \tag{6.6}$$

   (This relationship is shown in *Figure 6.7*.)

   Correct the measured modulus for temperature by multiplying the measured modulus by this modulus ratio.

7. Using the following relationship, calculate the ratio of the modulus at rate of loading in-service to the modulus at the laboratory loading rate (40 ms rise time):

$$\frac{\text{Modulus at Speed V}}{\text{Modulus at Test Loading Rate}} = 0.17V^{0.365} \tag{6.7}$$

   (This relationship is shown in *Figure 6.8*.)

   Correct the measured modulus for speed by multiplying the measured modulus by this modulus ratio.

6.5.3.4 Design modulus from bitumen properties and mix volumetric properties

A method developed by Shell (1978) may be used to obtain reasonable estimates of modulus of mixes which include conventional bituminous binders. There are two distinct stages in the estimation process:

1. the modulus of the bituminous binder is determined for the specific bitumen in the specific pavement design situation (traffic speed, operating temperature); and
2. the asphalt modulus is determined from the bitumen modulus, together with the volumetric composition of the mix.

The nomographs produced by Shell to undertake these two steps are reproduced as *Figures 6.9* and *6.10*. A software package – BANDS – is also available (from Shell) which undertakes these two steps.

In this method, the nomograph developed by van der Poel (1954) and reproduced in *Figure 6.9* is used to estimate the modulus of the bitumen at the required temperature and loading rate. The information required as input to the nomograph is:

Time of loading	The duration (s) of a step load for which the bitumen modulus equals the modulus under traffic loading. It may be taken as 1/V where V (km/h) is the design heavy vehicle speed.
Operating temperature	The effective temperature (°C) of the asphalt (WMAPT).
$T_{800\ pen}$	The temperature (°C) at which the penetration (100 g, 5 s) of the bitumen is 800 (0.1 mm).
PI	The Penetration Index of the bitumen which is an index of the temperature susceptibility of the penetration.

$T_{800\ pen}$ and PI may be determined from bitumen penetration or viscosity data by means of the relationships in *Table 6.9*.

During the mixing and laying of asphalt, the bitumen undergoes considerable hardening and decrease in temperature susceptibility. The Rolling Thin Film Oven (RTFO) test (AS2341.10) (Standards Australia 1994) has been developed to simulate these effects. Hence, it is important to note that the penetration and viscosity data used in *Table 6.9* refer to values obtained on bitumen which has been subject to the RTFO test.

Once the van der Poel nomograph has been used to estimate the modulus of the bitumen, the nomograph developed by Bonnaure *et al.* (1982) and reproduced in *Figure 6.10* is used to estimate the modulus of the asphalt mix.

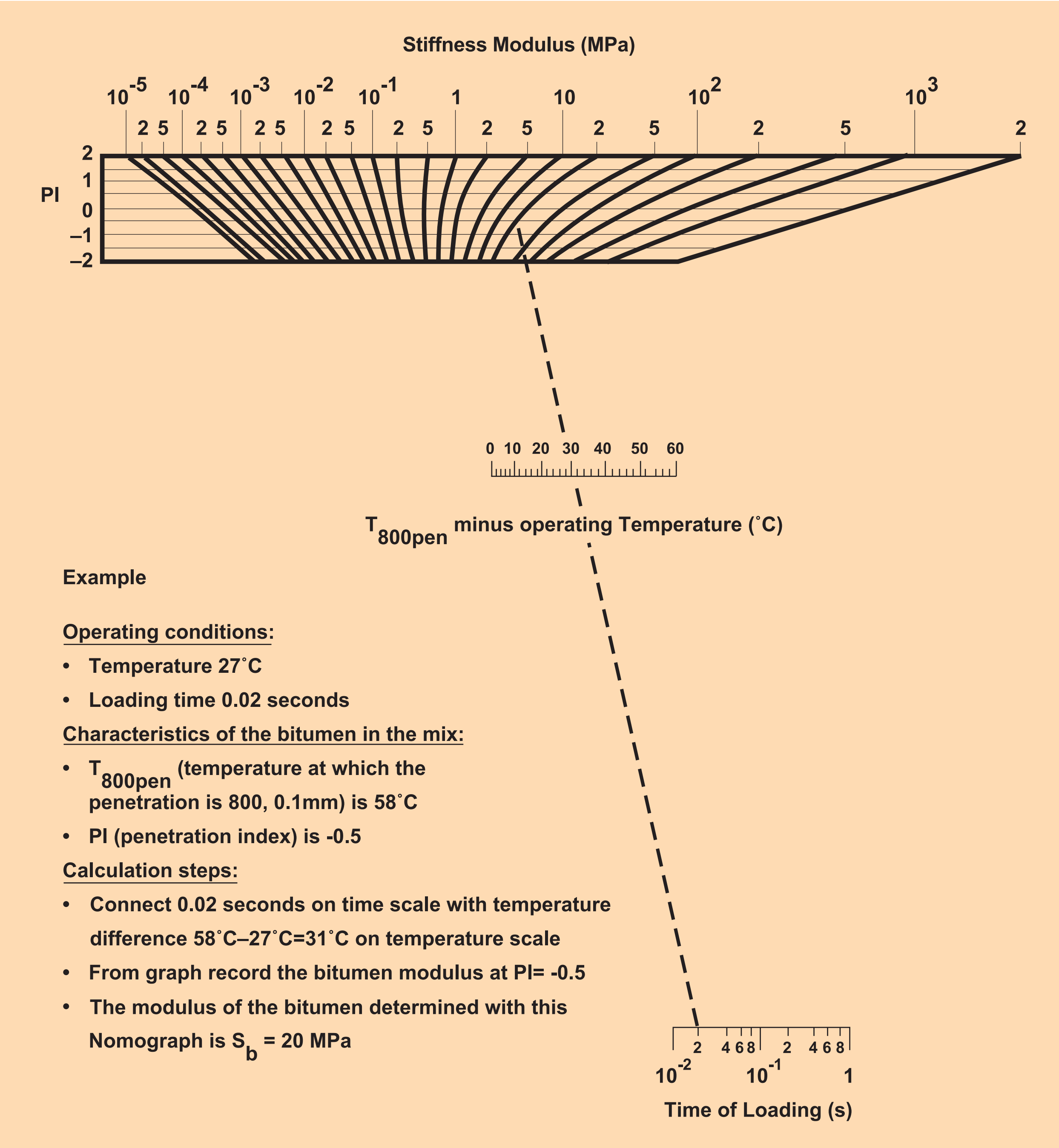
The information required as input to this nomograph is:

E_{bit}	modulus of the bitumen at the assumed temperature and loading rate as derived from the van der Poel nomograph (*Figure 6.8*);
V_b	the percentage by volume of bitumen in the asphalt. For a typical mix containing 5% bitumen by mass, V_b may be taken as 11%; and
V_{agg}	the percentage by volume of aggregate in the mix. For a typical mix containing 5% bitumen by mass and compacted so that it contains 6% air voids, V_{agg} will be approximately 83%.

Table 6.9 Relations for determining PI and $T_{800\ pen}$ from bitumen penetration and viscosity data

	BITUMEN PROPERTIES KNOWN		
PROPERTIES FOR BITUMEN STIFFNESS NOMOGRAPH	**Pen 1 at T_1 °C Pen 2 at T_2 °C**	**Pen 1 at T_1 °C Vis 2 at T_2 °C**	**Vis 1 at T_1 °C Vis 2 at T_2 °C**
A	$\frac{1}{T_2 - T_1}\log\frac{\text{Pen 2}}{\text{Pen 1}}$	$\frac{1}{T_2 - T_1}\left[\log\frac{800}{\text{Pen 1}} + \frac{5.42\log\frac{1300}{\text{Vis 2}}}{8.5 - \log\frac{1300}{\text{Vis 2}}}\right]$	$\frac{1}{T_1 - T_1}\left[\frac{46.07\log\frac{\text{Vis 1}}{\text{Vis 2}}}{\left[8.5 - \log\frac{1300}{\text{Vis 1}}\right]\left[8.5 - \log\frac{1300}{\text{Vis 2}}\right]}\right]$
PI	$\frac{20 - 500\,A}{1 + 50\,A}$		
T_{800PEN}	$T_1 + \frac{1}{A}\log\frac{800}{\text{Pen 1}}$		$T_1 - \frac{1}{A}\left[\frac{5.42\log\frac{1300}{\text{Vis 1}}}{8.5 - \log\frac{1300}{\text{Vis 1}}}\right]$

Notes:
1. **Pen 1, Pen 2 are penetrations (0.1mm) determined using 100g mass and loading time of 5 seconds**
2. **Vis 1, Vis 2 are dynamic viscosities (Pa.s).**
3. **A is the change in log (penetration) per °C change in temperature**



Example

Operating conditions:

- **Temperature 27˚C**
- **Loading time 0.02 seconds**

Characteristics of the bitumen in the mix:

- **T_{800pen} (temperature at which the penetration is 800, 0.1mm) is 58˚C**
- **PI (penetration index) is -0.5**

Calculation steps:

- **Connect 0.02 seconds on time scale with temperature difference 58˚C–27˚C=31˚C on temperature scale**
- **From graph record the bitumen modulus at PI= -0.5**
- **The modulus of the bitumen determined with this Nomograph is S_b = 20 MPa**

Figure 6.9 Nomograph for determining modulus of conventional bituminous binders

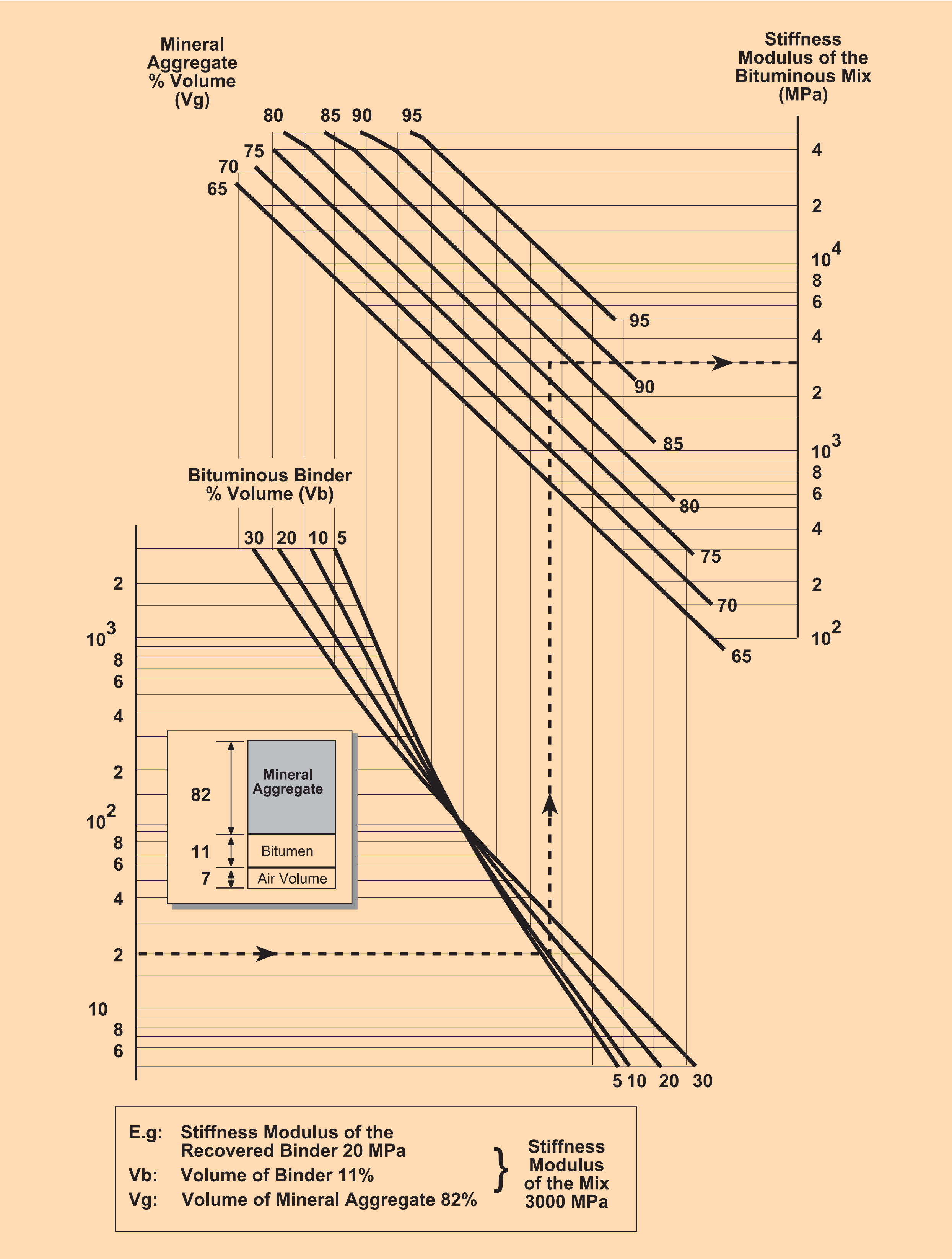


Figure 6.10 Nomograph for predicting the modulus of asphalt

In situations where the nomograph for mix modulus requires extrapolation (e.g. low bitumen modulus), the actual mix modulus depends largely on the aggregate properties, particularly the angularity of the aggregates.

The modulus of asphalt mixes incorporating polymer modified binders may differ significantly from those using conventional bitumen binders. Modulus testing of laboratory samples indicates that the relativity between polymer modified asphalts and Class 320 binder asphalt can be represented by the proportional adjustment factors shown in *Table 6.10* (RTA NSW 2000). These adjustment factors should be used with caution in the design model, and limited to asphalt wearing courses until they have been evaluated for other asphalt mix and pavement layer applications.

6.5.3.5 *Design modulus from published data*

The use of published data to assign a design modulus should only be adopted when testing facilities are unavailable; and EITHER the binder is not a conventional bitumen (i.e. it is a Multigrade or a polymer-modified binder) OR the data on bitumen and mix volumetric properties required in Section 6.5.3.4 are not available.

The information in preceding sections should be used to provide guidance in selecting, from published data, a modulus value which is appropriate for the proposed asphalt mix and the proposed field conditions (traffic speed, operating temperature). *Table 6.11* presents modulus values obtained using the indirect tensile test procedure under standard test conditions for typical dense-graded asphalt mixes used by Austroads Member Authorities. The results are for laboratory-manufactured samples and indicate the likely range of values, together with typical values. Values are provided for a range of mix sizes and binder types.

The use of the Table to determine a design stiffness value involves the following steps:

Table 6.10 Factors to estimate the modulus of polymer modified binder asphalts for modulus estimated from the Shell nomographs using a Class 320 bitumen

Pre-1990 description of binder type	Austroads binder grade*	Modulus adjustment factor
6% SBS	A10E	0.70
5% SBS	A15E	0.75
25 parts crumb rubber	A40R	0.75
3% SBS	A20E	0.80
PBD	A25E	0.90
Multigrade	Multigrade 1000/320	1
5% EVA	A30P	1.5
5% EMA	A35P	1.5

* Draft Austroads Modified Binder User Guide, January 2003

Table 6.11 Modulus (MPa) of typical Australian dense-graded asphalts determined on laboratory-manufactured samples using the indirect tensile test procedure and standard test conditions and 5% air voids

	Mix size (maximum particle size) (mm)					
	10		14		20	
Binder	Range	Typical	Range	Typical	Range	Typical
Class 170	2000–6000	3500	2500–4000	3700	2000–4500	4000
Class 320	3000–6000	4500	2000–7000	5000	3000–7500	5500
Class 600	3000–6000	6000	4000–9000	6500	4000–9500	7000
Multigrade	3300–5000	4500	3000–7000	5000	4000–7000	5500
A10E	1500–4000	2200	2000–4500	2500	3000–7000	3000

Note: Standard test conditions are 40 ms Rise Time and 25°C test temperature (Standards Australia, 1995*a*)

- Select from the *Table 6.11* a modulus value obtained from the standard indirect tensile test which is appropriate for the proposed mix.
- Estimate the design modulus using the procedures specified in Section 6.5.3.3.

The values given in *Table 6.11* are included only as a guide and it is recommended that designers establish presumptive values for design purposes based on average moduli of approved mixes. Presumptive values are often required as pavement design is usually carried out well before the source of the mix is known.

6.5.4 Factors affecting asphalt fatigue life

In broad terms, the factors which affect the fatigue life of asphalt are:

- the support (stiffness) provided to the asphalt by the underlying pavement structure;
- the contribution by the asphalt layer to the overall pavement stiffness;
- the modulus of the asphalt and the extent to which it changes with temperature;
- the type of binder in the mix;
- the spectrum of traffic loading; and
- the temperature environment in which the asphalt is operating.

Of the factors listed, only the last three are independent of other factors.

6.5.4.1 Effect of support provided by underlying structure

The fatigue life of an asphalt layer will be extended by increasing the stiffness of the pavement sub-structure supporting it. While this statement is true in general, it implies that this support remains throughout the life of the asphalt. Such may not be the case if, for example, the asphalt is placed directly onto a stiff cemented layer which develops fatigue cracks early in the pavement's life, thus significantly reducing the support provided and causing high strain levels in localised areas in the asphalt (above underlying cracks).

If the asphalt is placed on granular material, the level of support provided to the asphalt will depend on the contribution made by the asphalt layer to the overall pavement stiffness – the greater this contribution, the lower the level of support provided. (This occurs because the stiffness of granular material depends on the stress level it is subject to – the lower the stress, the lower its stiffness.)

6.5.4.2 Effect of asphalt layer's contribution to overall pavement stiffness

Consider, firstly, the extreme situations: the negligible contribution to overall pavement stiffness provided by a thin, very flexible, asphalt surfacing on a thick concrete base, and the 100% contribution provided by a thick asphalt pavement on a weak foundation. In the former case, when the pavement is loaded, the tensile strain in the asphalt is dominated by the stiffness of the pavement sub-structure supporting the asphalt and is largely independent of asphalt stiffness. In the latter case, when the asphalt is loaded, the tensile stress at the bottom of the asphalt is independent of asphalt stiffness, while the strain is inversely proportional to the stiffness. Hence, if the asphalt layer is the major contributor to the overall pavement stiffness, the asphalt layer is considered to operate in a 'controlled stress' mode, i.e. the level of tensile stress at the bottom of the asphalt layer is controlled by the dominance of the asphalt's contribution and is (relatively) independent of the stiffness of the layer.

Because the fatigue life of the layer depends on the levels of tensile strain produced in the layer by the traffic loads, and because these strain levels decrease as the asphalt stiffness increases, for the 'controlled stress' situation the fatigue life is increased by increasing the asphalt stiffness.

Consider, now, how the situation changes as we progress from the former case towards the latter. As the contribution by the asphalt layer to the overall pavement stiffness increases, the bottom of the asphalt layer will start to experience tensile stress and strain. If the asphalt's contribution to overall pavement stiffness is still relatively small, the level of strain in the asphalt will be (relatively) independent of the asphalt stiffness. In this situation (in practice, thin asphalt surfacings), the asphalt is considered to be operating in a 'controlled strain' mode because the tensile strain in the asphalt is 'controlled' by the dominance of the stiffness of the pavement sub-structure supporting the asphalt and, hence, is (relatively) independent of the asphalt stiffness.

Owing to the fact that a stiff asphalt layer can withstand less applications of a given strain than a soft asphalt layer, in the 'controlled strain' situation fatigue life is increased by decreasing the asphalt stiffness.

It should be noted that, in the above discussion, the asphalt is considered to be of one stiffness for the entire depth of asphalt. For the ‘controlled stress’ situation, where the asphalt is providing the bulk of the pavement stiffness, fatigue life can be further improved by using a lower modulus asphalt with a higher bitumen content in the bottom portion of the asphalt where it is subject to tensile strains. In such a manner, the versatile properties of asphalt are best utilised.

6.5.4.3 *Effects of asphalt stiffness and its temperature dependence*

The role that the contribution of the asphalt layer to overall pavement stiffness plays in determining the fatigue life of asphalt has been discussed above. This contribution increases with both the modulus of the asphalt and the thickness of the layer. In general, for a given type of asphalt subjected to a given level of strain, an increase in mix stiffness will result in a reduction in fatigue life.

The factors which, in turn, affect mix stiffness are discussed in Section 6.5.2. The direct effects that these factors have on fatigue life are indicated – for both the controlled stress and the controlled strain situations – in *Table 6.12*.

Table 6.12 Factors affecting the fatigue life of asphalt and the effect of increasing factor values

Factor	Effect of increasing factor value	
	Controlled stress	Controlled strain
bitumen viscosity	increase	decrease
bitumen content:		
up to optimum	Increase	increase*
over optimum	decrease	increase
% air voids	decrease	decrease
% of aggregate	increase	decrease
temperature	decrease	increase
rate of loading	increase	decrease

* The apparent contradiction in this effect is believed to be due to the bitumen film thickness increase over-riding the stress increase in the mix as a whole by actually reducing the stress in the bitumen itself.

The dependence of mix stiffness on temperature can be very relevant to asphalt fatigue life in locations where temperature variations are considerable.

6.5.4.4 *Effect of binder type*

For mixes using conventional binders, there is some increase in mix stiffness with increase in bitumen class number – with its resulting effect on fatigue life. The use of binders which have been modified by the incorporation of polymers (PMBs) or other binders (e.g. Multigrade binders) can result in considerably improved fatigue lives when tested in the laboratory (Baburamani and Potter 1996).

Note that there are currently no fatigue relationships available to confidently estimate the in-service fatigue life of mixes with other than standard grade conventional binders.

6.5.4.5 *Effect of temperature environment*

Because asphalt fatigue is dependent on mix stiffness which is in turn dependent on temperature, the distribution of load applications with time can be quite significant in determining fatigue life. For example, if all applications of load are applied at times of low temperatures, then a different fatigue life will result compared with the situation where the loading occurs at higher temperatures.

The actual difference depends on the magnitudes of the loads, the thickness of the asphalt layer, the pavement composition and the different proportions of traffic loads applied at low and high temperatures. Such a situation can exist for pavements subject to night-time usage by large numbers of heavy vehicles.

Commonly, the effect of temperature is taken into account by estimating the modulus at the Pavement Temperature (WMAPT). Values of WMAPT for locations throughout Australia and New Zealand – and the method used to calculate WMAPT values at a site – are given in *Appendix 6.1*.

6.5.5 Means of determining asphalt fatigue characteristics

6.5.5.1 *Alternative methods*

Asphalt fatigue characteristics may be determined by laboratory testing in conjunction with field trials or by adopting relationships contained in the literature.

6.5.5.2 *Laboratory fatigue testing*

The method adopted in Australia for characterising the fatigue properties of asphalt is flexural fatigue testing of asphalt beams. This method is preferred because the test reproduces the actual behaviour of an asphalt layer under wheel loading more closely than any other method. Both laboratory-compacted samples or specimens cut from the pavement can be tested.

A simply-supported beam of asphalt of rectangular cross-section (390 mm long, 50 mm high and 63 mm wide) is subjected to repeated applications of load applied at its third points until the modulus has decreased to half the initial value. The recommended mode of testing is controlled strain (appropriate for commonly used thin asphalt surfacings) with strain levels typically ranging between 200 and 1000 microstrain. A continuous haversine load is applied at a frequency of 10 Hz. The standard test temperature is 20°C.

The magnitude of load and the deflection at the beam centre are recorded. By applying simple elastic beam theory, the beam stiffness (asphalt stiffness) and the maximum strain level can be calculated to provide design data.

Fatigue characterisation of asphalt requires testing across the range of strains of likely relevance and, for each strain level, across the range of asphalt stiffnesses of likely relevance.

In estimating field performance from laboratory test results, consideration should be given to the variations in strain which will occur in the pavement at any point due to the lateral distribution of traffic at that location and to the number of applications required for crack propagation through the asphalt layer. There are also indications that some healing occurs in practice, i.e. intermittent loading has a less damaging effect than continuous loading. The actual number of load applications producing cracking in the field may be many times the number obtained by laboratory testing because of these factors. Hence, if using performance criteria developed from laboratory testing, a correlation with field performance must be made.

6.5.5.3 Published relationships

When appropriate testing facilities are not available, asphalt fatigue characteristics will have to be adopted from those published in the literature.

To ensure that appropriate relationships are selected, the following factors need to be considered:

- type of mix tested (type of binder and aggregate);
- controlled stress or controlled strain testing;
- adopted definition of failure;
- applicability to field performance (or only laboratory results);
- testing temperature; and
- duration of test load pulses.

6.5.6 Fatigue criteria

The asphalt fatigue relationship used in this Guide is the laboratory fatigue relationship published by Shell (1978) adjusted to predict fatigue life in the road-bed using a reliability factor according to the desired project reliability (Austroads 2004). (The Shell relationship was developed from laboratory test results for a broad range of mix types containing conventional binders. Testing was conducted using continuous sinusoidal [tension and compression] loading of beams in the controlled strain mode.)

For conventional bituminous binders, the general relationship between the maximum tensile strain in asphalt produced by a specific load and the allowable number of repetitions of that load is:

$$N = RF \left[\frac{6918(0.856\, V_B + 1.08)}{S_{mix}^{0.36}\ \mu\varepsilon} \right]^5 \qquad (6.8)$$

where

N	=	allowable number of repetitions of the load;
$\mu\varepsilon$	=	tensile strain produced by the load (microstrain);
V_B	=	percentage by volume of bitumen in the asphalt (%);
S_{mix}	=	asphalt modulus (MPa); and
RF	=	reliability factor for asphalt fatigue (*Table 6.13*).

Table 6.13 Suggested Reliability Factors (RF) for asphalt fatigue

Desired project reliability				
80%	85%	90%	95%	97.5%
2.5	2.0	1.5	1.0	0.67

Note that the reliability factors are transfer functions that relate a mean laboratory fatigue life, as determined by the relationship above (Shell, 1978) to the in-service fatigue life predicted using this Guide at a desired project reliability. In effect they comprise two components:

- a shift factor relating mean laboratory fatigue life to a mean in-service fatigue life, taking account of the differences between the laboratory test conditions and the conditions applying to the in-service pavement; and
- a reliability factor relating mean in-service fatigue life to the in-service life predicted using this Guide at a desired project reliability, taking into account those factors (e.g. construction variability, environment, traffic loading) discussed in Chapter 2.

Guidance on the selection of desired project reliability levels is provided in Section 2.2.1.2 of this Guide. Where the in-service fatigue life is governed by a wearing course that includes a modified binder, the project reliability is likely to be improved significantly.

Representative plots of tensile strain against allowable loading to asphalt fatigue are presented in *Figure 6.11*, indicating the effects of asphalt stiffness and bitumen content for a desired project reliability of 95%.

6.5.7 Permanent deformation of asphalt

Permanent deformation of asphalt – evidenced by rutting and shoving – is due to insufficient stability for the prevailing loading and environmental conditions.

While permanent deformation is well acknowledged as a distress mode of primary importance for asphalt, it is not included in the design procedures because no model is available which will reliably predict the development of rutting with the passage of traffic/time. The reason for this may be readily understood. During the service life of an asphalt layer in a road pavement, a very significant proportion of the accumulated permanent deformation in the asphalt layer will have occurred during the very rare times when the asphalt is at a highly elevated temperature. For asphalt layers to reach such elevated temperatures (throughout the layer) requires a succession of very hot, clear days and accompanying hot nights. Prediction of the occurrences of such weather patterns during the service life of the asphalt can be extremely difficult. Likely problem areas

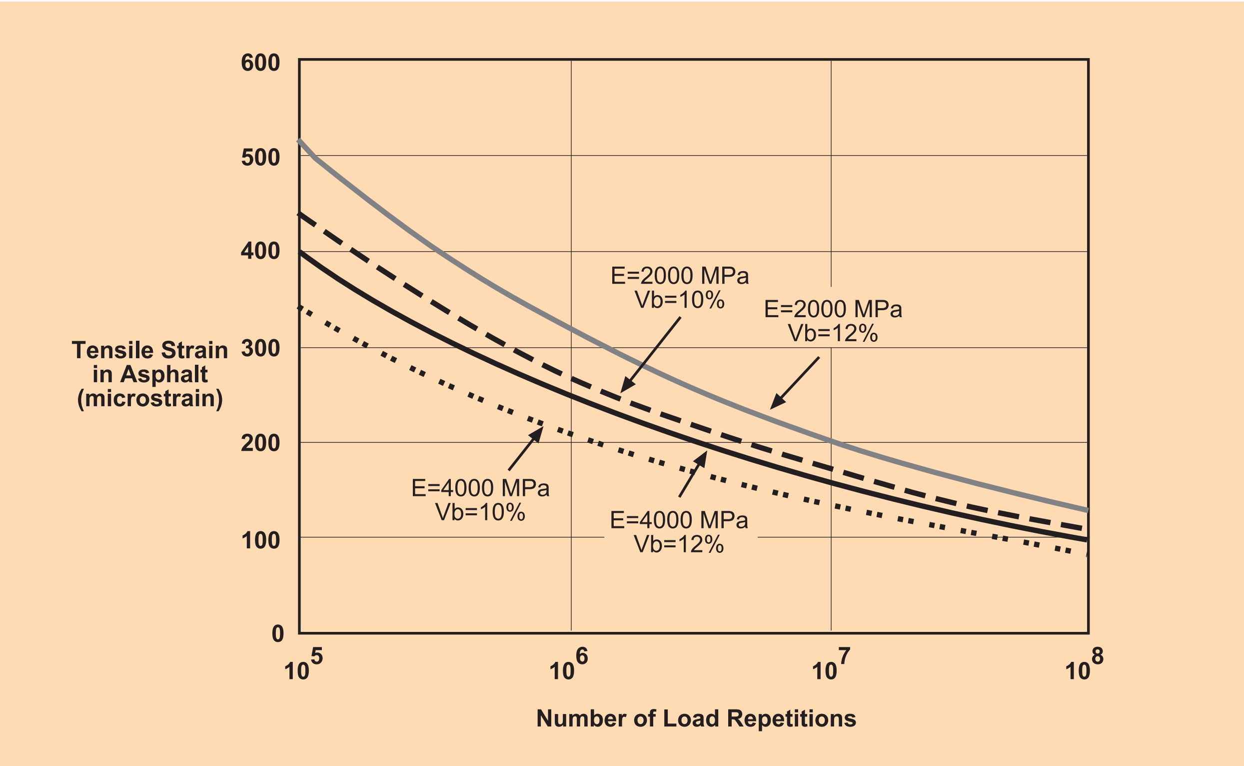


Figure 6.11 Representative plots of tensile strain against load repetitions to asphalt fatigue for a project reliability of 95%

are those associated with heavy vehicles travelling at low speed or accelerating or braking (climbing lanes, intersections, etc.).

If stability is considered in terms of the Mohr-Coulomb terminology of cohesion and friction, the significance of many of the factors which affect stability may be more easily classified as shown in *Figure 6.12*. The effects of varying the above factors are summarised in *Table 6.14*.

Adequate stability may be achieved by adopting an empirical approach during mix design. Laboratory tests are available to assist (at the mix design stage) in assessing the potential for permanent deformation of a candidate mix.

The laboratory wheel-tracking tests (in which an asphalt slab is repeatedly transversed by a loaded wheel) (Austroads 1999) has been found to correlate well with road performance and is widely accepted in a number of countries.

The wheel tracking is applied to an asphalt slab using a wheel load of 700 Newton and a test temperature of 60°C. The accumulation of permanent deformation is recorded with number of cycles of loading. Several test values (tracking rate [mm/kpass] and maximum deformation after 10 000 cycles) are extracted from the test results for use in assessing the potential for permanent deformation in the field.

Table 6.14 Factors affecting the stability of asphalt

Factor	Effect on mix stability of increasing factor
Surface roughness of aggregate	increase
Bitumen content	decrease
% Aggregate	increase
Angularity	increase
Viscosity of bitumen	increase
Age of bitumen	increase
Temperature	decrease
Rate of loading	increase

A simple test procedure (using the same test equipment as used for the indirect tensile test for determination of resilient modulus) in which a cylinder of asphalt is subjected to repeated loading pulses applied to its ends is under development.

6.5.8 Recycled asphalt

The recycling of asphalt (which has reached the end of its service life) by incorporating it with virgin asphalt during the asphalt production process is a relatively recent – but rapidly expanding – industry. The resulting product commonly contains 10–20% recycled material. To compensate for the hardening of the bitumen that

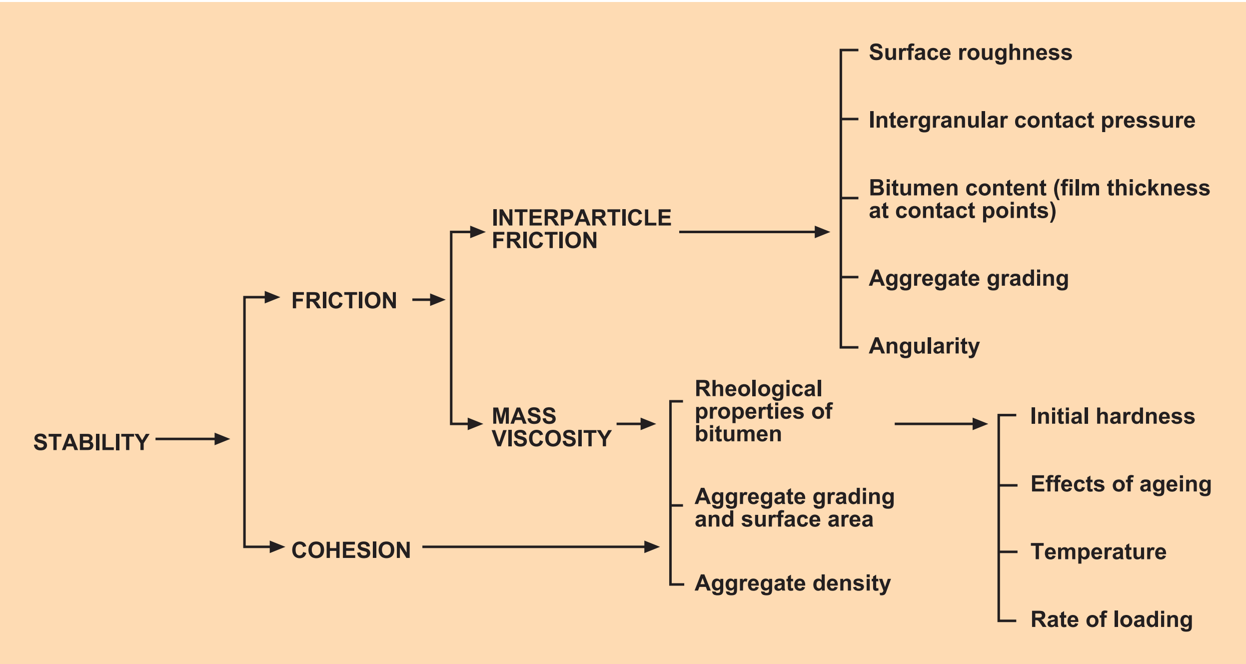


Figure 6.12 Factors affecting mix stability

has occurred throughout the service life of the asphalt, binders which lower the viscosity of the hardened bitumen may be incorporated in the mix. The Asphalt Recycling Guide, published by Austroads (1996) presents a comprehensive statement on all aspects of asphalt recycling.

With regard to the characterisation of recycled asphalt for pavement design purposes, the process is the same as that for virgin asphalt – the same properties are required and the same tests are appropriate, etc. While recycled asphalt is intrinsically a more variable product than virgin asphalt (because of the added variability associated with ageing effects), when the amount of recycled material in the mix is around 10–20%, the additional variability in the mix can be reduced by crushing and pre-screening RAP and monitoring the recovered binder viscosity of RAP so it can be regarded as a 'controlled component'.

6.6 Concrete

6.6.1 Introduction

Concrete refers to a homogeneous mixture of hydraulic cement, fine and coarse aggregate, water and chemical admixtures.

The cementitious portion of concrete may be of Portland cement or blended cement. Blended cements consist of Portland cement mixed with binders such as ground granulated blast furnace slag (slag) and/or pulverised fuel-ash (fly ash). Chemical admixtures may be used for set retardation, water reduction and air entrainment.

Concrete can be used as a subbase in either flexible or rigid pavements and as a base in rigid pavements.

General categories and characteristics of concrete are given in *Table 6.1*.

6.6.2 Subbase concrete

Lean-mix concrete which is used for subbase construction may contain a fly ash-blended cement and is required to attain a characteristic 28-day compressive strength of 5 MPa (with fly ash) and 7 MPa (without fly ash). The strength of concrete made using fly ash-blended cement increases at a slower rate up to 28 days. The determination of elastic parameters and performance characteristics should be carried out as discussed in Section 6.4.

The construction of both rigid and flexible bases over poor subgrades is facilitated by the adoption of a concrete subbase. For example, poor subgrades may preclude the achievement of adequate compaction in unbound or cemented granular materials or asphalt, which comprise the lower layers in flexible pavements.

Where subbase concrete is used in the design of a flexible pavement, the characteristics which must be known and evaluated for structural design purposes are modulus, Poisson's ratio and the response to repeated loading (see Section 6.4).

6.6.3 Base concrete

A rigid pavement is defined as a pavement having a base of concrete.

The 28-day concrete flexural strength is a key design parameter in predicting pavement performance. The 28-day design flexural strength of concrete suitable for road pavement construction is typically 4.0 to 5.0 MPa. Steel-fibre reinforced concrete should have a 28-day flexural strength in the range 5.0 to 5.5 MPa.

Since at the time of undertaking the thickness design the concrete will only have a nominal target strength, the design strength should be expressed in terms of the characteristic flexural strength to the nearest 0.25 MPa.

The durability of the concrete wearing surface requires a 28-day characteristic compressive strength of not less than 32 MPa (Standards Australia 2001).

A typical relationship for converting 28-day compressive strength to 28-day flexural strength for concrete with crushed aggregate is:

$$f_{cf} = 0.75 \sqrt{f_c} \tag{6.9}$$

where

f_c = 28-day concrete compressive strength (MPa), and

f_{cf} = 28-day concrete flexural strength (MPa).

The indirect tensile or splitting (Brazilian) test, has also been used for the control of concrete strength in pavement work. A typical relationship for converting splitting strength into flexural strength is:

$$f_{cf} = 1.37 \sqrt{f_{cs}} \tag{6.10}$$

where

f_{cs} = 28-day concrete splitting or indirect tensile strength (MPa).

The actual strength relationships for a given concrete mix will be dependent on the properties of its constituents, particularly the microtexture and particle shape of the coarse aggregate. For pavement thickness design purposes the above relationships are sufficiently accurate for concretes made with crushed aggregates possessing smooth microtexture.

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7

DESIGN TRAFFIC

7 DESIGN TRAFFIC

7.1 General

This Chapter contains procedures for determining traffic loading for the design of a broad range of flexible and rigid pavements. While the information presented in Sections 7.2 to 7.5 is pertinent to both pavement types, the manner in which it is used is dependent on the particular application being considered. Sections 7.6 and 7.7 provide more specific information on the design traffic for flexible pavements and rigid pavements, respectively.

7.2 Role of traffic in pavement design

A road pavement must be wide enough and of suitable geometry to permit all vehicles to safely operate at an acceptable speed. In addition, it must be strong enough to cater for both the heaviest of these vehicles and the cumulative effects of the passage of all vehicles. While the first of these requirements is in the province of geometric design, the second is the responsibility of the pavement designer[1].

Vehicular traffic consists of a mixture of vehicles ranging in the extreme from bicycles to triple road trains. The Austroads Vehicle Classification System, shown in *Table 7.1*, details the range of vehicles commonly using Australian roads, whilst the dominant vehicles in each of the 12 Vehicle Classification System classes in the Table are shown in *Figure 7.1*.

Because it has been well established that light vehicles (Austroads Vehicle Classes 1 and 2 in *Table 7.1*) contribute very little to structural deterioration, only heavy vehicles are considered in pavement design. Traditionally, the term 'commercial vehicle' has been used to denote these vehicles. In conformance with Austroads terminology, the term 'heavy vehicle' is now adopted as shown in *Table 7.1*.

The damage caused to a pavement by the passage of a heavy vehicle depends not only on its gross weight but also on how this weight is distributed to the pavement. In particular, it depends on:

- the number of axles on the vehicle;
- the manner in which these axles are grouped together – into axle groups; and
- the loading applied to the pavement through each of these axle groups – the axle group load.

For pavement design purposes, the following (heavy vehicle) axle group types are identified:

- single axle with single tyres (SAST);
- single axle with dual tyres (SADT);
- tandem axle with single tyres (TAST);
- tandem axle with dual tyres (TADT);
- triaxle with dual tyres (TRDT); and
- quad-axle with dual tyres (QADT).

All tyres referred to are conventional tyres. Axle groups fitted with wide super single tyres are not specifically considered in the pavement design process. McLean, Ramsay and Sharp (1995) and COST (2001) provide guidance on how to incorporate axle groups fitted with these tyres for flexible pavements.

1 The designer is cautioned that this Guide takes no explicit account of shear forces applied to the pavement (during acceleration and braking and travel on grades) or load transfer from the right to the left side of vehicles as they traverse roundabouts. For flexible pavements, the effect of shear forces can be modelled within the mechanistic design process (Chapter 8). The effect of load transfer can be readily accommodated at this design traffic stage by increasing the magnitudes of anticipated vehicle loads. Increases of up to 30% may be warranted in some circumstances.

Table 7.1 Austroads Vehicle Classification System

Level 1	Level 2		Level 3	Austroads classification	
Length (indicative)	Axles and axle groups		Vehicle type		
Type	Axles	Groups	Description	Class	Parameters
	Light vehicles				
Short Up to 6.6 m	2	1 or 2	Short Sedan, Wagon, 4WD, Utility, Light Van, Bicycle, Motorcycle, etc	1	d(1) ≤ 3.2 m and axles = 2
Medium 6.6 m to 14.5 m	3, 4 or 5	3	Short – Towing Trailer, Caravan, Boat, etc	2	groups = 3 d(1) ≥ 2.1 m, d(1) ≤ 3.2 m, d(2) > 2.1 and axles = 3, 4 or 5
	Heavy vehicles				
	2	2	Two Axle Truck or Bus	3	d(1) > 3.2 m and axles = 2
Long 11.5 m to 19.0 m	3	2	Three Axle Truck or Bus	4	axles = 3 and groups = 2
	4	2 or 3	Four Axle Truck	5	axles > 3 and groups = 2
	3	3	Three Axle Articulated Three axle articulated vehicle, or Rigid vehicle and trailer	6	d(1) > 3.2 m, axles = 3 and groups = 3
	4	>2	Four Axle Articulated Four axle articulated vehicle, or Rigid vehicle and trailer	7	d(2) < 2.1 m or d(1) < 2.1 m or d(1) > 3.2 m, axles = 4 and groups >2.
	5	>2	Five Axle Articulated Five axle articulated vehicle, or Rigid vehicle and trailer	8	d(2) <2.1 m or d(1) <2.1 m or d(1) >3.2 m, axles = 5 and groups >2
	6 >6	>2 3	Six Axle Articulated Six (or more) axle articulated vehicle, or Rigid vehicle and trailer	9	axles = 6 and groups > 2; or axles >6 and groups = 3
Medium combination 17.5 m to 36.5 m	>6	4	B Double B double, or Heavy truck and trailer	10	groups = 4 and axles > 6
	>6	5 or 6	Double Road Train Double road train, or Heavy truck and two trailers	11	groups = 5 or 6, and axles > 6
Long combination Over 33.0 m	>6	>6	Triple Road Train Triple road train, or Heavy truck and three trailers	12	groups > 6 and axles > 6

Definitions:
group: (axle group) – where adjacent axles are less than 2.1 m apart,
groups: number of axle groups,
axles: number of axles (maximum axle spacing of 10.5 m),
d(1): distance between first and second axle.
d(2): distance between second and third axle.

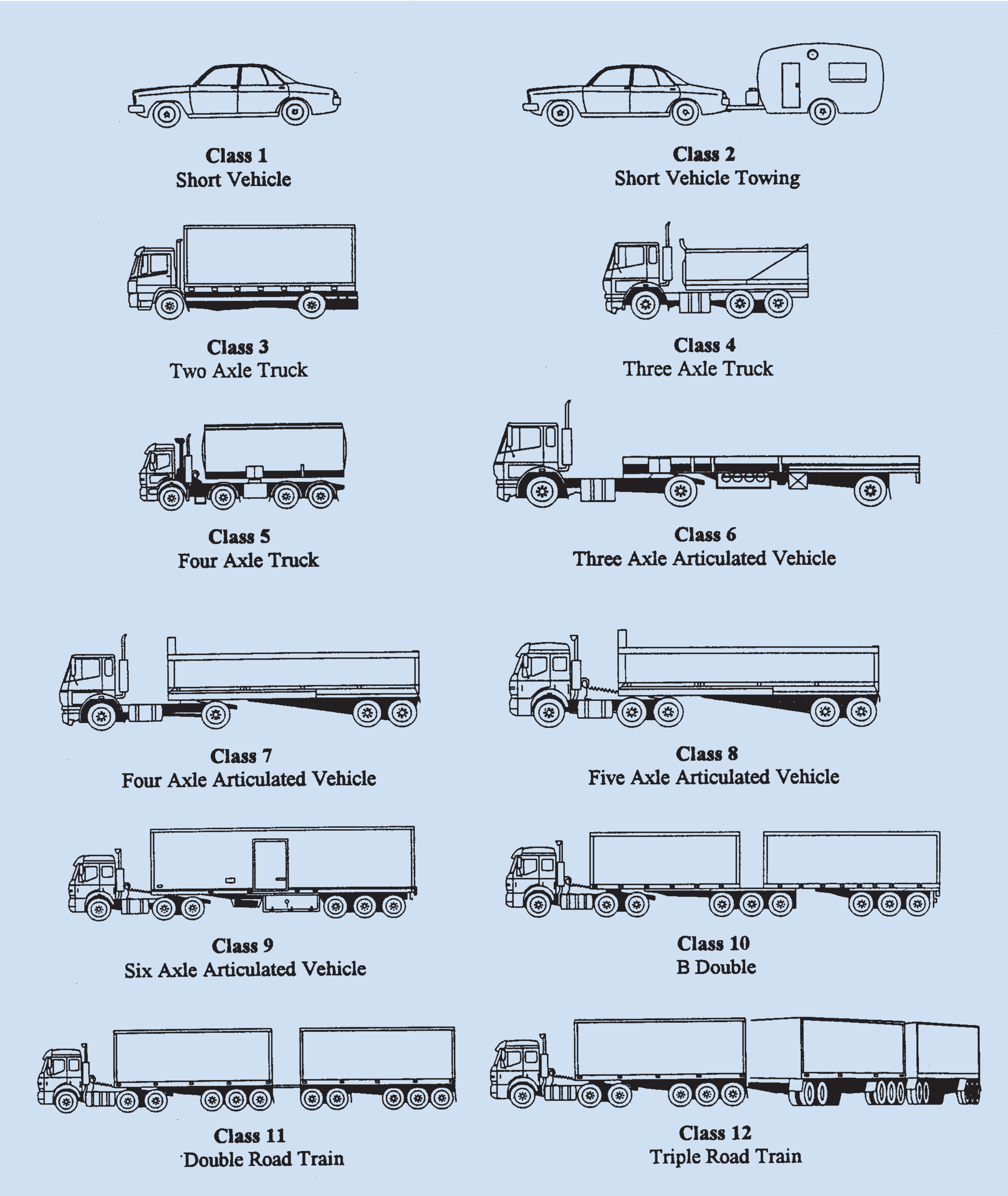


Figure 7.1 Dominant vehicles in each Austroads vehicle classification system class

The design tyre–pavement contact stress is taken as 750 kPa (Chapter 8). However, data collected in Tasmania by Chowdhury and Rallings (1994) indicates that tyre pressures vary widely – from 500 to 1200 kPa.

The cumulative loading on a pavement over a period of time is, in essence, an account of every axle group traversing the pavement during this time period, together with its type and its load. This cumulative loading is specified by:

- the cumulative number of axle groups traversing the pavement during the period;
- the proportions of each axle group type in this total; and
- for each axle group type, the frequency distribution of the axle group loads.

7.3 Overview of procedure for determining design traffic

The pavement design task is to select a suitable pavement configuration for the trafficked portion of a carriageway. This trafficked way may vary from a single lane catering for (with appropriate shoulders) traffic travelling in both directions, ranging up to six or more lanes for single-direction traffic. For multi-lane carriageways, the same pavement configuration is usually adopted for all lanes. The main reasons for this are:

- avoidance of steps in the finished surface of the subgrade – with the associated risk of water becoming trapped at the base of the step;
- avoidance of vertical planes of weakness formed within the pavement at the vertical interfaces between distinct material types; and
- construction expediency.

Selection of the pavement configuration is on the basis that the pavement will provide adequate service for the cumulative traffic expected over a designer-specified time-span – the design period. Adequate service implies that the pavement will not require major rehabilitation during the design period.

The sequence of steps leading to the specification of the design traffic for a project is as follows:

- Select a design period (Section 7.4.2).
- Identify the most heavily-trafficked lane in the carriageway – designated the design lane (Section 7.4.3).
- Estimate the average daily number of heavy vehicles during the first year of the project's life (Section 7.4.4).
- Estimate heavy traffic growth throughout the design period (Section 7.4.5).
- Estimate the average number of axle groups per heavy vehicle (Section 7.4.6).
- Coalesce the above three estimates to arrive at the cumulative heavy vehicle axle groups over the design period (Section 7.4.1).
- Estimate the proportion of axle group types and the distribution of axle group loads (Section 7.5).
- Express the cumulative traffic loading in a form suitable for the specific pavement design procedure to be used (Sections 7.6 and 7.7).

A flow chart of the sequence of all steps involved is provided in *Figure 7.2*.

If the pavement design is being undertaken for a re-alignment or a re-construction, then the opportunity exists to base the estimate of initial traffic loading on the (directly observable and measurable) existing traffic. If the pavement design is being undertaken for a greenfields situation, then reliance must be placed on the observation and measurement of traffic on nearby roads of a similar nature, coupled with an estimation of the additional traffic generated by the 'creation' of the new road. This latter aspect involves consideration of the land development likely to occur alongside or near the new link and the traffic which will be generated by such development.

7.4 Procedure for determining total heavy vehicle axle groups

7.4.1 Introduction

The first requirement of a pavement design is that it be adequate for the cumulative traffic loading anticipated in the design lane over the design period. Estimation of this loading firstly requires the calculation of the cumulative number of heavy vehicle axle groups (HVAG) over the design period. Calculation of the cumulative HVAG is commonly broken down into the following three distinct elements:

- estimation of the average daily traffic loading during the first year of the project's life;
- estimation of how the daily traffic loading may change throughout the design period; and

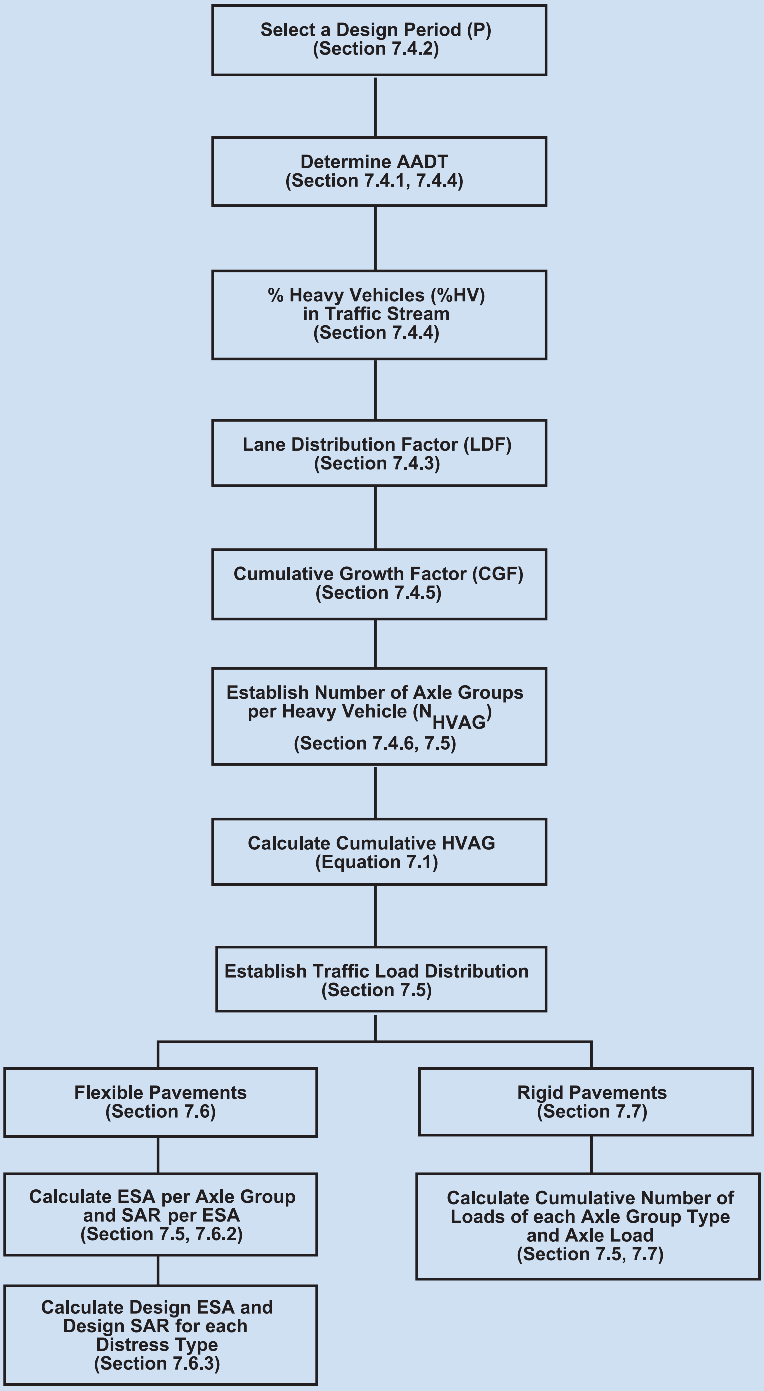


Figure 7.2 Procedure for determining design traffic

- a combination of these two estimates to arrive at an estimate of the cumulative traffic loading (in the design lane) over the design period.

The equation to derive the Design Traffic (N_{DT}) – in cumulative heavy vehicle axle groups (HVAG) – traversing the design lane during the specified period (N_{DT}) is:

$$N_{DT} = 365 \times (AADT{*}DF) \times \%HV/100 \times N_{HVAG} \times LDF \times CGF \quad (7.1)$$

where

AADT	=	Annual Average Daily Traffic[2] in vehicles per day in the first year (Section 7.4.4);
%HV	=	average percentage of all traffic comprising Heavy Vehicles (Section 7.4.4);
N_{HVAG}	=	average number of Axle Groups per Heavy Vehicle (Section 7.4.6);
DF	=	Direction Factor is the proportion of the two-way AADT travelling in the direction of the design lane
LDF	=	Lane Distribution Factor (Section 7.4.3); and
CGF	=	Cumulative Growth Factor (Section 7.4.5).

It is important to note that the Design Traffic (N_{DT}) is applicable for both flexible and rigid pavements, and additional calculations are required to derive standard axles of loading for flexible pavements (refer to Section 7.6).

7.4.2 Selection of design period

The design period adopted by the pavement designer is the time span considered appropriate for the road pavement to function without major rehabilitation or reconstruction. As such, it is a fundamental parameter in the entire pavement management process. In addition to its direct role in estimating the quantum of design traffic for the pavement design exercise, it also forms the basis for expectations of how the constructed pavement will perform. Further, it provides an initial input into the long-term network-wide programming of future major rehabilitation or reconstruction works. In selecting a design period, the following issues are relevant:

- The funds available for the project.
- The importance of the road.
- The likelihood of a future re-alignment to improve the geometric standard of the road. If likely, it would be prudent to adopt a design period consistent with the estimated time for the future re-alignment.
- The likelihood that major future upgrading will be required to improve the capacity of the road. If likely then it would be prudent to adopt a design period consistent with the time at which it is estimated that traffic capacity will be reached
- The likelihood that factors other than traffic (e.g. reactive subgrades, consolidation of imported fills or compressible soil strata) will cause distress necessitating major rehabilitation or reconstruction, in advance of any load-related distress. If likely, it may be prudent to opt for a shorter design period than would otherwise be the case.
- The likelihood of existing fixed levels (kerb and gutter, clearance under overhead structures, etc.) constraining the selection of rehabilitation treatments to more costly options. If likely, it may be prudent to opt for a longer design period than would otherwise be the case.

The design period adopted for a specific pavement type typically lies within the range indicated in *Table 7.2*.

It is to be stressed that, although a pavement is designed to provide satisfactory service over a specified design period, this service can only be expected if actual cumulative traffic over the period does not exceed the estimated cumulative traffic. Hence, the likely period of satisfactory service is controlled by the value adopted for the design traffic and not by the value adopted for the design period.

Table 7.2 Typical pavement design periods (years)

Flexible pavements	20–40
Rigid pavements	30–40

[2] The total yearly two-way traffic volume divided by 365, expressed as vehicles per day.

7.4.3 Identification of design lane

As discussed in Section 7.3, for new construction it is common practice to adopt the same pavement design for all lanes of the carriageway and to base this design on the traffic loading in the most heavily-trafficked lane. This most heavily-trafficked lane is termed the design lane.

For a two-lane, two-way road (i.e. one lane in each direction), the design lane is readily identified as that lane which is more heavily trafficked. For multi-lane single-direction carriageways, the design lane is – in the vast majority of cases – the left (or outermost) lane. For carriageways of this type, estimates of traffic loading are usually only available for the entire carriageway, i.e. there is no lane-specific information on traffic loading.

The distribution of traffic loading across the lanes is dependent on:

- the number of traffic lanes;
- the presence of parked vehicles in the left lane;
- the proximity of intersections; and
- the primary and secondary functions of the road.

Weigh-in-motion (WIM) systems are available which provide lane-specific traffic loading data across a carriageway. If project-specific information is not available, then *Table 7.3* provides guidance on the proportion of the cumulative carriageway traffic loading to be assigned to this left (design) lane. The proportion assigned to a specific lane is termed the Lane Distribution Factor (LDF) for the lane.

7.4.4 Initial daily heavy vehicles in the design lane

To calculate the cumulative HVAG in the design lane (eqn 7.1), an estimate is required of the average over the first year (of the project's operation) of the daily number of heavy vehicles in the design lane. This averaging over an entire year is conducted to ensure that the estimate is unaffected by day-to-day (or, often of more significance, season-to-season) fluctuations in daily traffic loadings.

Consistent with the broader discussion in Section 7.3, this average daily number of heavy vehicles in the design lane is specified by the following parameters:

- the cumulative number of heavy vehicles traversing the design lane during the average day; and
- the average number of axle groups per heavy vehicle.

Any of the following methods – listed in descending order of accuracy – may be used to estimate the initial daily number of heavy vehicles. The designer is encouraged to adopt a method commensurate with the importance of the project, availability of relevant data, and resources available for data collection.

1. Use of weigh-in-motion (WIM) survey data, either collected specifically for the project or recently collected for other purposes, to furnish both of the above parameters. WIM data also provides the number of axle groups per heavy vehicle required to estimate the cumulative number of HVAG (Section 7.4.6) and the distribution of axle group types and loads required to calculate the design traffic for flexible pavement and rigid pavements (Section 7.5).
2. Use of data obtained from Vehicle Classification Counters. Such data will directly furnish the above two parameters as well as data on the number of axle groups per heavy vehicle required to estimate the cumulative number of HVAG (Section 7.4.6). In addition, Classification Counters will provide information on the proportions of axle group types required to calculate the design traffic for both flexible and rigid pavements (Section 7.5). Classification counters, however, do not provide

Table 7.3 Typical Lane Distribution Factors for carriageways

Location	Lanes each direction	Lane Distribution Factor (LDF) Left lane	Centre lane	Right lane
Rural	2 lane	1.00*	N/A	0.50
	3 lane	0.95	0.65	0.30
Urban	2 lane	1.00*	N/A	0.50
	3 lane	0.65	0.65	0.50

* This value is the suggested limit for a lane. It may be reduced if there is sufficient traffic survey data that indicates a lower LDF is appropriate.

information on the distribution of axle loads within each axle group type (Section 7.5). Consequently, the use of Classification Counter data requires a traffic load distribution to be selected, as discussed in Section 7.5.

3. Use of data obtained from single tube axle counters, or manual traffic count surveys, together with an estimate of the proportion of heavy vehicles. However, as tube counters do not provide load or axle type data, the use of this data is very dependent on the engineering judgement of the designer.

The designer is reminded of the necessity to ensure that the data forming the basis for this estimation task is representative of the traffic loading for the entire year the project is opened to traffic. For example, corrections may be required to make allowance for seasonal variations in traffic, such as peaks associated with the movement of harvested produce. Caution is advised when considering heavy traffic loadings that are concentrated within short periods, as the normal pavement performance models adopted within this Guide may not be appropriate.

7.4.5 Cumulative traffic volumes

Part of the task of estimating the cumulative traffic (in the design lane) over the design period is to estimate the likely changes in daily traffic loading during this period. Once these changes have been estimated, their effects are then incorporated in the estimate of cumulative loading. Changes can occur both in the (daily) volume of traffic using the road and also in the sizes of loads carried by heavy vehicles. Because these two types of change have distinct traits and also distinct effects on the resultant cumulative traffic loading, it is appropriate to consider them separately. The growth in traffic volumes is considered in this Section. Consideration of changes in the magnitudes of axle loads is discussed in Section 7.4.7.

If the project is a re-alignment, a re-construction or an overlay, then it is appropriate to base the estimation of growth in traffic volumes on historical data for the existing road. If the project is a greenfields project, then the estimation should be based on the growths experienced by similar roads in the vicinity, coupled with consideration of the additional traffic to be generated by the ensuing land development in the corridor serviced by the new road.

Based on historical evidence, it is reasonable to expect that the daily volume of traffic (both light and heavy vehicles) will increase either for the entire design period or up to the stage where the traffic capacity of the road is reached. This evidence also indicates that the growth is geometric in nature, i.e. it can be modelled by conventional compound growth formulae.

The compound growth of traffic volumes is usually (and conveniently) specified as a percentage increase in annual traffic volumes – a typical statement being ‘the annual growth rate is R%’. Adopting this specification of growth, and with compound growth occurring throughout the design period, the cumulative growth factor (when constant) over the design period is readily calculated as follows:

Cumulative Growth Factor (CGF)

$$= \frac{(1 + 0.01R)^{P} - 1}{0.01R} \quad \text{for } R > 0 \qquad (7.2)$$

$$= P \quad \text{for } R = 0$$

where

R = Annual Growth Rate (%), and

P = Design Period (years).

For this case of below-capacity traffic volumes throughout the design period, values of the Cumulative Growth Factor (CGF) for a range of Annual Growth Rates (R) and Design Periods (P) are presented in *Table 7.4*. Where a change in annual growth rate is expected during the design period, then the method described in *Appendix 7.1* should be used to calculate the CGF. Note also that the growth in the number of heavy vehicles may not be the same as the growth in AADT.

7.4.6 Estimating axle groups per heavy vehicle

As seen from equation 7.1, to estimate the cumulative heavy vehicle axle groups (in the design lane) over the design period, the average number of axle groups per heavy vehicle (N_{HVAG}) is required.

Either of the first two methods – listed in descending order of accuracy in Section 7.4.4 – may be used to estimate N_{HVAG}. The designer should adopt a method commensurate with the importance of the project, the availability of relevant data, and the resources available for data collection and analysis.

Table 7.4 Cumulative Growth Factor (CGF) values for below-capacity traffic flow

Design period (P) (years)	Annual growth rate (R) (%)							
	0	1	2	3	4	6	8	10
5	5	5.1	5.2	5.3	5.4	5.6	5.9	6.1
10	10	10.5	10.9	11.5	12.0	13.2	14.5	15.9
15	15	16.1	17.3	18.6	20.0	23.3	27.2	31.8
20	20	22.0	24.3	26.9	29.8	36.8	45.8	57.3
25	25	28.2	32.0	36.5	41.6	54.9	73.1	98.3
30	30	34.8	40.6	47.6	56.1	79.1	113.3	164.5
35	35	41.7	50.0	60.5	73.7	111.4	172.3	271.0
40	40	48.9	60.4	75.4	95.0	154.8	259.1	442.6

In the absence of WIM or Classification Counter data, a presumptive value for N_{HVAG} needs to be selected. *Appendix 7.2* provides a list of N_{HVAG} values based on WIM data obtained at sites throughout Australia. In addition, typical N_{HVAG} values for rural roads, urban moderate-to-heavily-trafficked roads have been calculated using this WIM data. These presumptive values are given in *Table 7.5*.

Table 7.5 Presumptive numbers of heavy vehicle axle groups per heavy vehicle (N_{HVAG})

	N_{HVAG}
Rural roads	2.8
Urban roads	2.5

7.4.7 Increases in load magnitude

If the designer anticipates that during the design period there will be increases in load magnitudes for some or all axle groups, then it is necessary to incorporate these anticipated increases in the estimation of design traffic. The procedures to incorporate increase in load magnitude are described in *Appendix 7.3*.

7.5 Estimation of Traffic Load Distribution (TLD)

In Section 7.4, the procedures to estimate cumulative HVAG over the design period were described. In addition to the cumulative HVAG, the traffic load distribution (TLD) for the project is required to calculate the design traffic loading.

The TLD provides information necessary to evaluate the pavement damage caused by the HVAG, specifically:

- the proportions of all axle groups that are a particular axle group type, and
- for each axle group type, the proportion of axles applied at each load magnitude.

The use of the TLD to calculate pavement damage is discussed in Sections 7.6 and 7.7.

WIM survey data, either collected specifically for the project or recently collected for other purposes, may be used to estimate the TLD for a project . The designer is encouraged to adopt a method commensurate with the importance of the project, availability of relevant data, and resources available for data collection.

In the absence of WIM data, a presumptive TLD needs to be selected. The TLDs included in *Appendix 7.2* are representative of the results from WIM surveys undertaken throughout Australia. Details of the surveys undertaken, and the TLDs derived, are available in Koniditsiotis and Cropley (1998). It is recommended that the pavement designer use all available information (project-specific, local, regional, etc.) before an appropriate TLD is selected from this survey list.

Using this representative WIM data, presumptive TLD values for rural roads and urban moderate-to-heavily-trafficked roads have been calculated. These presumptive TLDs are presented in *Appendix 7.4*.

7.6 Design traffic for flexible pavements

7.6.1 Damage to flexible pavements

All information presented thus far has been relevant to both the design of flexible pavements and rigid pavements. In this section, the Design Traffic as assembled above is converted into a form suitable for use in flexible pavement design.

To appreciate the nature of the traffic requirements for flexible pavement design, it is first necessary to understand, in broad terms, the flexible pavement design process and how this process reflects the types of damage that can occur in flexible pavements.

This Guide provides two separate and distinct design procedures as follows:

1. empirical design applicable to new flexible pavements consisting of a thin bituminous surfacing (sprayed seal or less than 40 mm of asphalt) over granular material, using *Figure 8.4*; and
2. mechanistic design applicable to new flexible pavements which contain one or more layers of bound material (asphalt or cemented material).

In the design of granular pavements with thin bituminous surfacings, only one type of damage is considered, namely the overall deterioration of the pavement, reflecting increased levels of roughness and rutting. However, for pavements containing one or more bound layers up to three distinct types of damage are considered:

- fatigue damage to asphalt;
- rutting and loss of surface shape; and
- fatigue damage to cemented material.

Hence, overall there are *four distinct types of damage* which are considered in the design of flexible pavements – one for the empirical design of granular pavements and up to three for the design of bound pavements.

Design traffic is commonly described – especially for feasibility or preliminary design or when broad comparisons are made between road projects – in terms of the number of Equivalent Standard Axles (ESA). While this is a well-entrenched and useful practice, design traffic so described does not, by itself, contain enough information for the design of pavements containing one or more bound layers. For this reason, design traffic is expressed in terms of Standard Axle Repetitions as discussed below.

7.6.2 Pavement damage in terms of Standard Axle Repetitions

In the flexible pavement design process, the first step in assessing the ability of a pavement configuration to withstand the design traffic is to determine the extent of damage caused to the configuration by a reference Axle Group termed the Standard Axle. [The Standard Axle is a single axle with dual tyres (SADT) applying an axle load of 80 kN to the pavement.]

The candidate pavement is analysed to determine the allowable number of Standard Axle Repetitions (SAR) for each of the relevant damage types (Chapter 8). These allowable SAR are compared with and need to equal or exceed the number of Standard Axle Repetitions (SAR) that cause the same damage as the design traffic.

To calculate the SAR of damage, a procedure is required to calculate the damage associated with each axle group of each axle type in the traffic load distribution relative to the damage caused by a Standard Axle. Loads on axle configurations that cause the same amount of damage as the Standard Axle are given in *Table 7.6*.

Denoting this axle group load (which causes the same damage as a Standard Axle) as the Axle Group's Standard Load (SL_i), SAR of damage is evaluated as follows:

$$SAR_{ijk} = \left(\frac{L_{ij}}{SL_i}\right)^m \tag{7.3}$$

where

SAR_{ijk}	=	number of Standard Axle Repetitions (or passages of the Standard Axle) which causes the same amount of type *k* damage as a single passage of axle group type i with load L_{ij};
SL_i	=	Standard Load for axle group type i (from *Table 7.6*);
L_{ij}	=	load on the axle group;
k	=	damage designation for damage type (from *Table 7.7*); and
m	=	an exponent which is specific to the type *k* damage (from *Table 7.7*).

Table 7.6 Axle group loads which cause same damage as standard axle*

Axle group type	Load (kN)
Single axle with single tyres (SAST)	53
Single axle with dual tyres (SADT)	80
Tandem axle with single tyres (TAST)	90
Tandem axle with dual tyres (TADT)	135
Triaxle with dual tyres (TRDT)	181
Quad-axle with dual tyres (QADT)	221

* The axle group loads which cause equal damage are taken to be those loads which produce equal maximum deflection of the pavement surface.

For granular pavements with a thin bituminous surfacing designed using the empirical design chart (*Figure 8.4*), the Standard Axle Repetitions (SAR) – calculated with a damage exponent *m* of 4 – are commonly referred to as Equivalent Standard Axles (ESA). This damage exponent was derived from field studies of pavement performance.

The Damage Exponents 5 and 12 (for fatigue of asphalt and cemented material respectively) are derived from the fatigue relationships for these materials presented in Chapter 6. The Damage Exponent 7 (for rutting and loss of surface shape of bound pavements) is derived from the subgrade strain criterion presented in Chapter 5. The Damage Exponent 4 (for overall damage to granular pavements with thin bituminous surfacing) is derived from field studies of pavement performance.

The first step in calculating the SAR of damage due to the design traffic is the calculation of the design traffic in ESA from the cumulative number of heavy vehicles axle groups (eqn 7.1). This calculation requires the estimation of the average number of ESA per heavy vehicle axle group (ESA/HVAG) from the Traffic Load Distribution (TLD, Section 7.5) for a project. The TLD may be based on:

- project-specific WIM data;
- data selected from the WIM data given in *Appendix 7.2* ; or
- presumptive average TLDs presented in *Appendix 7.4*.

The ESA/HVAG for a range of selected WIM sites are provided in *Appendix 7.2* whilst the ESA/HVAG values for presumptive TLDs in *Appendix 7.4* are given in *Table 7.8*.

The design traffic calculated in terms of ESA loading is converted to SAR of loading using the average SAR/ESA value for each distress type. These factors

Table 7.7 Damage exponents for each specific type of damage

Design method	Pavement type	Damage type	Damage designation k	Damage exponent m
Empirical (*Figure 8.4*)	Granular pavement with thin bituminous surfacing	Overall pavement damage	e	4
Mechanistic	Pavement containing one or more bound layers	Fatigue of asphalt	a	5
		Rutting and shape loss	s	7
		Fatigue of cemented materials	c	12

Table 7.8 Characteristics of representative Traffic Load Distributions (TLDs) for urban and rural roads

Pavement type	Damage type	Damage Index	Presumptive rural TLD	Presumptive urban TLD
Granular pavements with thin bituminous surfacings, designed using *Figure 8.4*	Overall damage	ESA/HVAG	0.9	0.7
		ESA/HV	2.5	1.8
Pavement containing one or more bound layers, mechanistically designed	Fatigue of asphalt	SAR_a/ESA	1.1	1.1
	Rutting and shape loss	SAR_s/ESA	1.6	1.6
	Fatigue of cemented materials	SAR_c/ESA	12	12

are determined from the TLDs that characterise actual or presumptive traffic data. The SAR/ESA for a range of selected WIM sites are provided in *Appendix 7.2* whilst the SAR/ESA values for the presumptive TLDs in *Appendix 7.4* are given in *Table 7.8*.

Details of the procedure to estimate ESA/HVAG and SAR/ESA from a selected TLD are given in *Appendix 7.6*.

7.6.3 Definition of design traffic and its calculation

The design traffic requirements for flexible pavement design can be formally stated as follows:

> The Design Traffic for flexible pavement design is – for each relevant damage type – the total number of Standard Axle Repetitions (SAR) during the design period which cause the same damage as the cumulative traffic.

Firstly, the design number of Equivalent Standard Axles of traffic loading (DESA) is calculated:

$$\mathrm{DESA} = \mathrm{ESA/HVAG} \times \mathrm{N_{DT}} \quad (7.4)$$

where

ESA/HVAG = average number of Equivalent Standard Axles per Heavy Vehicle Axle Group (see Section 7.6.2); and

N_{DT} = cumulative number of Heavy Vehicles Axle Groups over the design period (from eqn 7.1).

Denoting the design number of Standard Axle Repetitions for each damage type by $DSAR_k$, then:

$$\mathrm{DSAR_k} = \mathrm{SAR_k/ESA} \times \mathrm{DESA} \quad (7.5)$$

where

SAR_k/ESA = average number of Standard Axle Repetitions per Equivalent Standard Axle for damage type *k* (see Section 7.6.2);

DESA = design traffic loading in ESA (from eqn 7.4); and

k = damage designation for damage type (from *Table 7.7*).

Where increases in heavy vehicle axle loads are likely to occur during the design period, refer to *Appendix 7.3* for guidance.

7.7 Design traffic for rigid pavements

For the design of rigid pavements, the cumulative HVAG (Section 7.4), together with the load safety factor and the traffic load distribution (TLD), are required to characterise the Design Traffic for all rigid pavement types.

The Design Traffic for rigid pavement design is the cumulative number of Heavy Vehicle Axle Groups over the Design Period (N_{DT}), classified according to the type of axle group and the load on the specific axle group type, i.e. the cumulative number of HVAG applied at each axle group load for each axle group type. The proportions of the cumulative HVAG applied at each load type and load are obtained using the TLD (Section 7.5).

Where it may be anticipated that an increase in axle loading is likely to occur, refer to *Appendix 7.1* for guidance.

The design of rigid pavements is discussed in Chapter 9 of the Guide.

7.8 Example of design traffic calculations

An example of design traffic calculations is given in *Appendix 7.6*.

References

Austroads (1988). *Roadway Capacity – A Guide to Traffic Engineering Practice, Part 2*. AP-11.2/88. (Austroads: Sydney.)

COST (2001). *COST 334 Effects of Wide Single Tyres and Dual Tyres*. Final Report. European Commission Directorate General Transport.

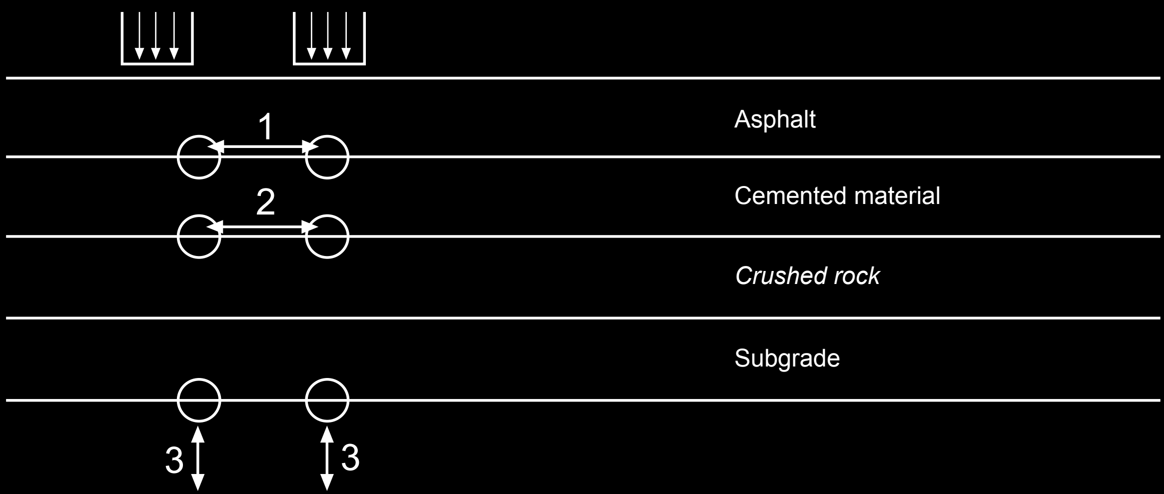
Chowdhury, F. and Rallings, R.A. (1994) *A Survey of Truck Tyre Pressures in Tasmania*. Road & Transport Research, Vol.3, No.3 September.

Koniditsiotis, C. and Cropley, S. (1998). *Update of Traffic Design Chapter in the Austroads Pavement Design Guide. Lightly and Heavily Trafficked Roads Traffic Load Distribution Data Set*. ARRB-TR Working Document R98/023.

McLean, J.R., Ramsay, E. and Sharp, K.G. (1995). *Review of Pavement Wear Effects of Wide Single Tyres*. ARRB-TR Contract Report TE 95/013. Prepared for NRTC Mass Limits Review.

8

DESIGN OF NEW FLEXIBLE PAVEMENTS



1. Tensile strain at bottom of asphalt – **asphalt fatigue**
2. Tensile strain at bottom of cemented material – **cement mat fatigue**
3. Compressive strain at top of subgrade – **rutting and shape loss**

   Denotes likely locations of critical strains due to applied loading

8 DESIGN OF NEW FLEXIBLE PAVEMENTS

8.1 General

The general design procedure which is contained in this Guide is mechanistic in nature. In addition to this, a specific empirical procedure is provided for the design of granular pavements with thin bituminous surfacings. This empirical procedure has been used extensively by Austroads Member Authorities, and has been found to give results consistent with the mechanistic procedure. Further, a set of example design charts for specific input parameters, which have been derived from the mechanistic procedure, is included.

Layer thicknesses derived from any of the procedures contained in this Guide should be considered to be minimum requirements and take no account of construction tolerance. This is particularly critical for bound pavement materials.

8.2 Mechanistic procedure

The detailed procedure presented here provides the designer with the capability of designing a broad range of pavement types, for a broad range of loading types and configurations.

A flow chart of the procedure is shown in *Figure 8.1*.

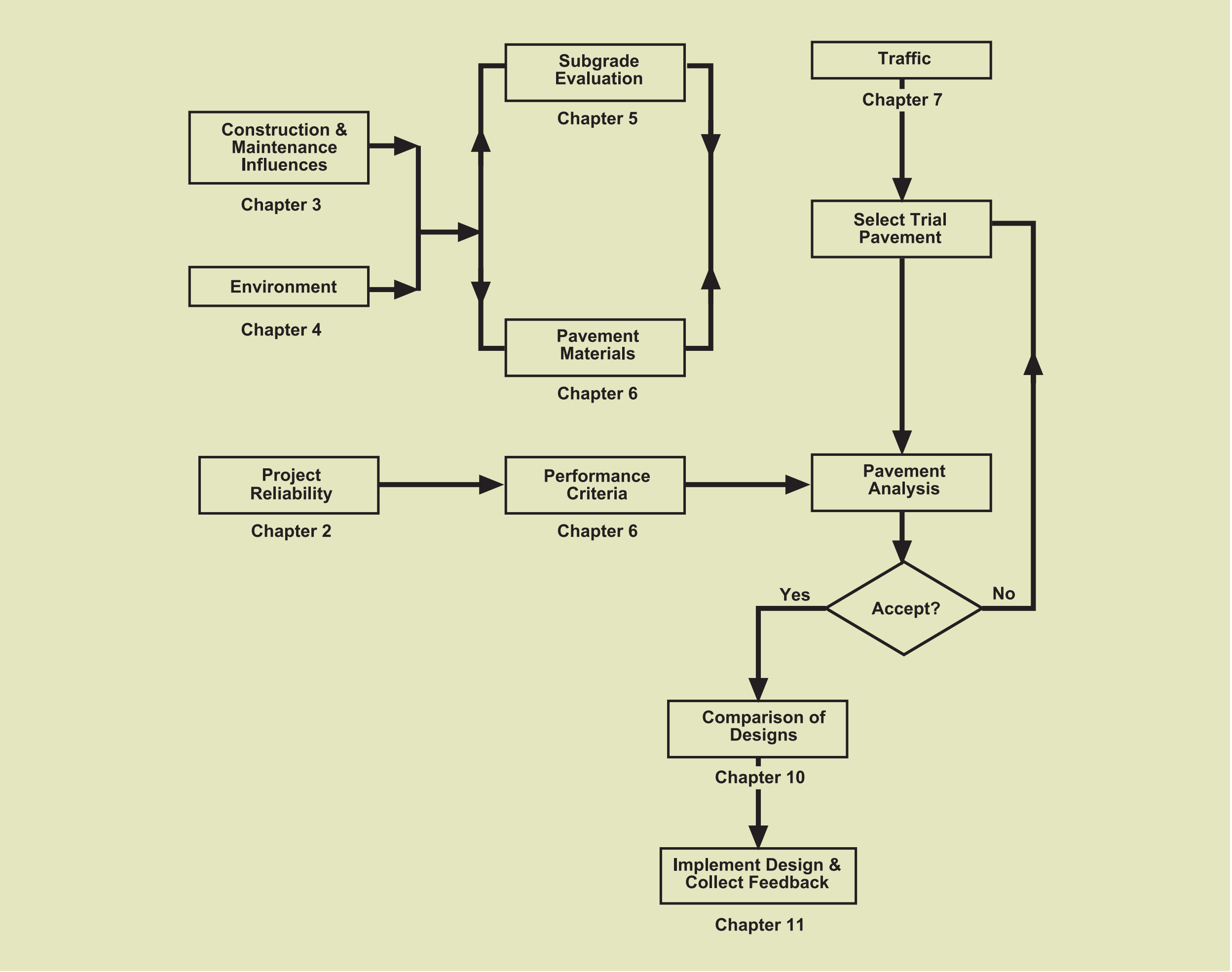


Figure 8.1 Design procedure for flexible pavements

In summary the procedure consists of:

- evaluating the input parameters (materials, traffic, environment etc.);
- selecting a trial pavement;
- analysing the trial pavement to determine the allowable traffic;
- comparing this with the design traffic; and
- finally, accepting or rejecting the trial pavement.

The appropriate design inputs are:

- desired project reliability (Chapter 2)
- construction and maintenance policy influences (Chapter 3);
- environment (Chapter 4);
- subgrade (Chapter 5);
- materials and performance criteria (Chapter 6); and
- design traffic (Chapter 7).

The design procedure is based on the structural analysis of a multi-layered pavement subject to normal traffic loading. The critical locations of the strains within a pavement model – and the idealised loading situation – are shown in *Figure 8.2*.

Significant features of the assumed model are as follows:

1. Pavement materials are considered to be homogeneous, elastic and isotropic (except for unbound granular materials and subgrades which, as discussed in Sections 5 and 6, are considered to be anisotropic).
2. Response to load is calculated using linear elastic theory and specifically the computer program CIRCLY (Mincad Systems 2004).
3. The critical responses assessed for pavement and subgrade materials are:
   - Asphalt – horizontal tensile strain at bottom of layer;
   - Unbound Granular – not considered in model;
   - Cemented – horizontal tensile strain at bottom of layer; and
   - Subgrade and Selected Subgrade Material – vertical compressive strain at the top of the layer.
4. Standard Axle loading consists of a dual-wheeled single axle, applying a load of 80 kN. For flexible pavements, the critical responses within the pavement occur either along the vertical axis directly below the inner most wheel of the dual wheel group or along the vertical axis located symmetrically between a pair of dual wheels – see *Figure 8.2*.
5. Standard Axle loading is represented by four uniformly-loaded circular areas of equal area separated by centre-to-centre distances of 330 mm,

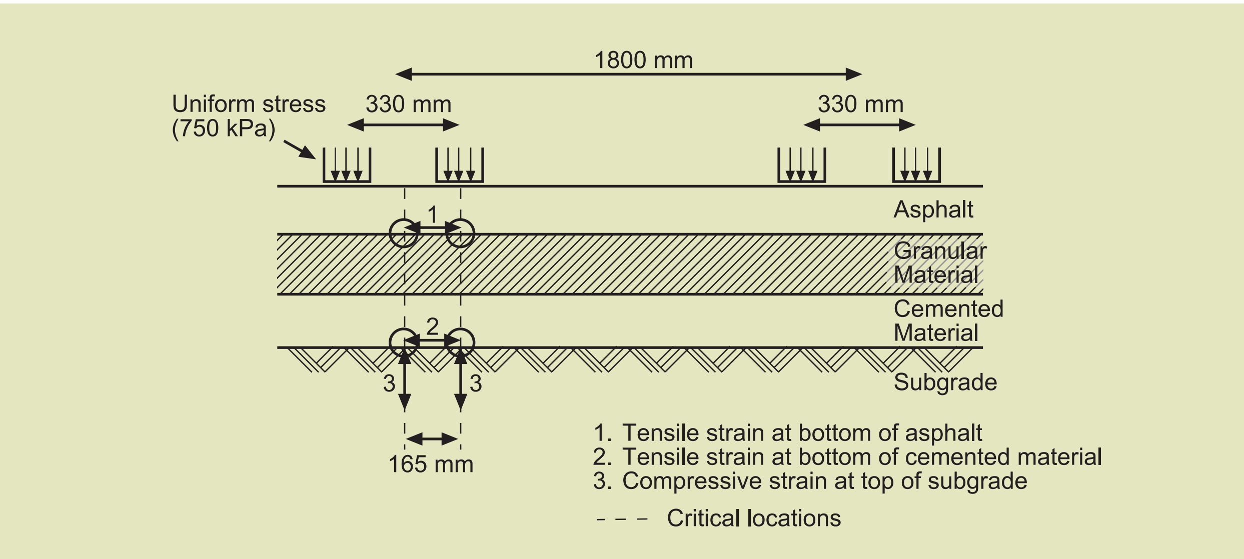


Figure 8.2 Pavement model for mechanistic procedure

1470 mm and 330 mm respectively as illustrated in *Figure 8.2*.

6. The contact stress is assumed to be uniform over the loaded area and, for the purpose of design, is taken to be 750 kPa. The contact stress is related to the tyre pressure which for highway traffic is assumed to be in the range 500–1000 kPa.
7. Some variations to the above may be appropriate for other than normal axle types and loadings; for example, where sharp turning movements or acceleration or braking occur. A model which more closely corresponds to the actual axle configuration and loading should be adopted in such cases. The computer program CIRCLY can accommodate these variations. However, this is rarely undertaken for most pavement design situations and there is little case study experience to relate the calculated CIRCLY responses to pavement performance.
8. For some projects, the mechanistic modelling may indicate that both a thin (<40 mm) and thick asphalt surfaced pavement can be adopted. Caution is advised in adopting the thin asphalt surfaced pavement option because the dominant damage types are not necessarily those addressed by the design model and as a consequence mechanistic modelling of asphalt layers less than 40 mm thick is less certain than for thicker asphalt layers (refer Section 8.2.5).

Tables 8.1(a), *(b)*, *(c)* and *(d)* list in detail the steps which are required to carry out the Mechanistic Design Procedure.

- *Table 8.1(a)* deals with inputs.
- *Table 8.1(b)* deals with the analysis.
- *Table 8.1(c)* deals with the interpretation of the results of the analysis.
- *Table 8.1(d)* sublayering design equations.

8.2.1 Selection of trial pavement

The selection of a trial pavement involves specifying the pavement materials to be used, the thicknesses of each material and the relative positions of these materials in the pavement.

8.2.2 Procedure for elastic characterisation of selected subgrade materials

The modulus of selected subgrade materials is dependent not only on the intrinsic characteristics of these materials, but also on the stiffness of the underlying insitu subgrade. As a result in mechanistic designs, the total thickness of all selected materials is divided into five sublayers and each assigned a modulus value according to the following guidelines:

a) Divide the total thickness of selected subgrade materials into 5 equi-thick sublayers.

Table 8.1(a) Mechanistic design procedure: input requirements

Step	Activity	Reference
1	Select a trial pavement and a desired project reliability	Section 8.2.1 Section 2.2.1
2	Determine the following elastic parameters for the insitu subgrade and selected subgrade materials: E_V; $E_H = 0.5\ E_V$; $\nu_V = \nu_H$; $f = E_V/(1+\nu_V)$	Chapter 5 Section 8.2.2
3	Determine the elastic parameters (as above) of the top sublayer of the granular layer (if relevant)	Section 6.2 & Table 6.4
4	Determine the elastic parameters and thickness of the other granular sublayers (if relevant)	Section 8.2.3
5	Determine the elastic parameters for cemented materials, pre and post fatigue cracking (if relevant)	Section 6.4
6	Determine the elastic parameters for asphalt (if relevant)	Section 6.5
7	Adopt the subgrade strain criterion	Section 5.8
8	Determine fatigue criteria for cemented materials (if relevant)	Section 6.4
9	Determine fatigue criteria for asphalt (if relevant)	Section 6.5
10	Determine design number of Standard Axle Repetitions (SAR) for each relevant distress mode	Section 7.6.3

Table 8.1(b) Mechanistic design procedure: analysis

Step	Activity
11	Approximate the Standard Axle wheel loading as four uniformly loaded circular areas at centre-to-centre spacings of 330 mm, 1470 mm and 330 mm; a vertical load of 20 kN is applied to each circular area at a uniform vertical stress distribution of 750 kPa. Radius of each loaded area $R = 2523p^{-0.5}$ (about 92.1 mm for highway traffic), where R = radius (mm) and p = vertical stress (kPa).
12	Determine critical locations in the pavement for the calculation of strains as follows: ■ bottom of each asphalt or cemented layer, and ■ top of insitu subgrade and top of selected subgrade materials. on vertical axes through the centre of an inner wheel load and through a point midway between the two wheel loads at a centre-to-centre spacing of 330 mm.
13	Input the above values into CIRCLY and determine the maximum vertical compressive strain at the top of the subgrade and top of selected subgrade materials and the maximum horizontal tensile strain at the bottom of each cemented and/or asphalt layer. If the post-fatigue cracking phase of cemented materials life is being considered (Section 8.2.4), it is necessary to calculate critical strains for both pre-cracking and post-cracking phases of life.

Table 8.1(c) Mechanistic design procedure: interpretation of results

Step	Activity	Reference
14	Determine using the criteria selected in Steps 7, 8 and 9 the allowable number of Standard Axle Repetitions for each of the relevant distress modes. If the post-cracking phase of cemented materials life is being considered, calculate the total allowable loading of the pre-cracking and post-cracking phases of life. In this case the total allowable loading is expressed in terms of ESA rather than Standard Axle Repetitions.	Section 8.2.4
15	For each distress mode, compare allowable number of Standard Axle Repetitions with the design number of Standard Axle Repetitions.	Chapter 7
16	If, for all distress modes, the allowable number of Standard Axle Repetitions exceeds the design number of Standard Axle Repetitions, the pavement is acceptable. If not, it is unacceptable.	
17	If the pavement is unacceptable or additional pavement configurations are required for comparison, select a new trial pavement, return to Step 1 and repeat Steps 1 to 16.	
18	Compare alternative acceptable designs.	Chapter 10

b) The vertical modulus of the top sublayer of selected subgrade is the minimum of the 10 times the design CBR of the selected subgrade material and that determined using equation 8.1 in *Table 8.1(d)*.

c) The ratio of moduli of adjacent sublayers is determined using equation 8.2 in *Table 8.1(d)*.

d) The modulus of each sublayer may then be calculated from the modulus of the adjacent underlying sublayer, beginning with the insitu subgrade, the modulus of which is known.

e) where the trial pavement configuration includes more than one type of selected subgrade material, a check needs to be made that the vertical modulus calculated for each sublayer (step d) does not exceed 10 times the design CBR of each selected subgrade material within the sublayer. If this condition is not met, an alternative trial selected subgrade configuration needs to be selected.

For all selected subgrade materials, the other stiffness parameters required for each sublayer may be calculated from the following relationships:

$E_H = 0.5\ E_V$ – refer to Section 5.6, and

$f = E_V/(1 + \nu_V)$ – refer to CIRCLY Manual.

Table 8.1(d) Sublayering design equations

Selected subgrade materials

$$E_{V\ \text{selected subgrade top sublayer}} = E_{V\ \text{insitu subgrade}} \times 2^{(\text{total selected subgrade thickness}/150)} \quad (8.1)$$

$$R = \left[\frac{E_{V\ \text{selected subgrade top sublayer}}}{E_{\text{insitu subgrade}}}\right]^{\frac{1}{5}} \quad (8.2)$$

Granular materials

$$E_{V\ \text{top granular sublayer}} = E_{V\ \text{subgrade}} \times 2^{(\text{total granular thickness}/125)} \quad (8.3)$$

$$R = \left[\frac{E_{\text{top granular sublayer}}}{E_{\text{subgrade}}}\right]^{\frac{1}{5}} \quad (8.4)$$

8.2.3 Procedure for elastic characterisation of granular materials

The modulus of granular materials is dependent not only on the intrinsic characteristics of these materials, but also on the stress level at which they operate and the stiffness of the underlying layers. As a result, the modulus of pavement materials subjected to vertical loading will decrease with depth to an extent influenced by the stiffness of the subgrade. Iterative analyses with a finite element model would permit allowance to be taken of the stress-dependant nature of the modulus of granular material; however, it would not make allowance for the degree of support provided by underlying layers.

In addition, as such models are not readily available to pavement designers, the procedure in this Guide utilises the elastic layer model CIRCLY, with the granular layers partitioned into several sublayers and each assigned a modulus value according to the following guidelines:

1. For granular materials placed directly on to a stiff cemented subbase, no sublayering is required. The modulus is determined using the procedures discussed in Chapter 6.
2. For granular materials placed directly on the insitu subgrade or selected subgrade material, sublayering is required and should be conducted as follows:
   a) Divide the total thickness of unbound granular materials into 5 equi-thick sublayers.
   b) The vertical modulus of the top sublayer is the minimum of the value indicated in *Table 6.4* and that determined using equation 8.3 in *Table 8.1(d)*.
   c) The ratio of moduli of adjacent sublayers is determined using equation 8.4 in *Table 8.1(d)*.
   d) The modulus of each sublayer may then be calculated from the modulus of the adjacent underlying sublayer, beginning with the subgrade or upper sublayer of selected subgrade material as appropriate, the modulus of which is known.
3. For all granular materials, the other stiffness parameters required for each sublayer may be calculated from the following relationships:

   $E_H = 0.5\ E_V$ – refer to Section 6.2, and

   $f = E_V / 1 + \nu_V)$ – refer to CIRCLY Manual.

8.2.4 Consideration of post-cracking phase in cemented materials

Where a pavement incorporates a cemented layer which reaches its fatigue life, the pavement may then enter a post-cracking phase whereby other layers continue to further the life of the pavement structure. Typically a thick (≥ 175 mm) asphalt on a cemented material subbase pavement would be modelled by taking account of the fatigue lives of both the cemented layer and the asphalt layer.

Where granular material is used over a cemented layer, a permeability reversal may be introduced into the pavement, in which case specific drainage measures need to be incorporated into the pavement to avoid saturation of the granular material.

A post-cracking phase of the design life can only be considered if cracking from the fatigued cemented material does not reflect through to the surface. To inhibit such reflective cracking the pavement should provide a minimum cover equivalent to 175 mm of asphalt over the cemented material. Granular material can be used as cover either solely (i.e. any sprayed seal or thin asphalt surfacing is not considered to be part of the cover), or in conjunction with asphalt, subject to the following criterion:

$$(0.75 \times \text{thickness of granular material cover}) + (\text{thickness of asphalt cover}) \geq 175\text{ mm}$$

Cracked cemented material should be modelled as a cross-anisotropic material with a vertical modulus of 500 MPa, and a Poisson's ratio of 0.35. No sublayering of the material is required.

The subgrade strain criterion (Section 5.8), cemented materials fatigue relationship (Section 6.4) and the asphalt fatigue relationship (Section 6.5) calculate allowable loading in Standard Axle Repetitions. To calculate the total allowable loading of the pre-cracking and post-cracking fatigue phases, the allowable loading needs to be expressed in terms of Equivalent Standard Axles (ESA) rather than Standard Axle Repetitions.

The equations for taking account of the post-cracking phase of the cemented material are as follows:

Asphalt Fatigue:

$$N_A = N_C + \left(1 - \frac{N_C}{N_{1stA}}\right) \times N_{2ndA} \qquad (8.5)$$

where

N_A = total allowable loading to asphalt fatigue (ESA);

N_C = allowable number of load repetitions (ESA) to cemented material fatigue (1st phase life);

N_{1stA} = allowable number of load repetitions (ESA) to asphalt fatigue prior to cemented material fatigue (1st phase life); and

N_{2ndA} = allowable number of load repetitions (ESA) to asphalt fatigue after cemented material fatigue (2nd phase life).

Permanent Deformation:

$$N_S = N_C + \left(1 - \frac{N_C}{N_{1stS}}\right) \times N_{2ndS} \qquad (8.6)$$

where

N_S = total allowable loading to unacceptable permanent deformation (ESA);

N_C = allowable number of load repetitions (ESA) to cemented material fatigue (1st phase life);

N_{1stS} = allowable number of load repetitions (ESA) to unacceptable permanent deformation prior to cemented material fatigue (1st phase life); and

N_{2ndS} = allowable number of load repetitions (ESA) to unacceptable permanent deformation after cemented material fatigue (2nd phase life).

Note that eqn 8.5 is only applicable if N_C exceeds N_{1stA} and eqn 8.6 is only applicable if N_C exceeds N_{1stS}.

For thick asphalt base on a cemented subbase, pavements may also be designed without consideration of the asphalt life in the post-cracking phase, that is, pavements designed such that the cemented layer outlasts the design traffic. It should be noted that for some pavements, particularly thin cemented subbase layers on low strength subgrades, the required asphalt thickness to inhibit cemented materials fatigue may exceed the asphalt thickness in a full depth asphalt design. For such pavement configurations, designers are advised to give consideration to the post-cracking phase of the design life.

8.2.5 Design of granular pavements with thin bituminous surfacings

The mechanistic design procedures may be used to design granular pavements with thin bituminous surfacings.

Designers are cautioned, however, that the mechanistic design model has not been validated for granular pavements having asphalt surface layers less than 40 mm thick and that there is considerable uncertainty associated with the use of the model for these pavements. In particular, while the design model may suggest that pavements with thin asphalt surfacings can perform comparably to thick asphalt pavements at high traffic loadings, it does not adequately account for the impact of traffic loads on these thin surfacings. These inadequacies include:

- the assumption that the tyre loading is applied as a uniform, vertical stress distribution;
- the assumption that the interface between the surfacing and the underlying pavement is fully bonded;
- the omission of horizontal loads due to braking, accelerating, turning and climbing movements;
- the assumed moisture levels of the granular base courses;
- construction variability; and
- the omission of environmental effects.

Tyre loading – Accurate measurements of tyre–pavement contact stress patterns (de Beer 1997) have shown that stresses at the edge of the tyre can be up to double the stress at the centre of the tyre. While the effect of this stress variation has relatively little impact on the performance of thick asphalt pavements, it can have a pronounced effect on thin surfacings.

Interface bond – The assumption of a fully bonded interface is often not achieved for thin asphalt surfacings placed on granular substrates, where the substrate is not primed prior to placing the asphalt. There is a tendency to omit the prime for reasons of expediency but it is critically important to the achievement of an adequate bond between the substrate and the surfacing. In some cases, even where the prime is applied, a build-up of vapour pressure as a result of higher moisture levels in the pavement can cause a debonding of the surfacing. Hence, the importance of controlling moisture build-up in pavements.

Horizontal loads – During certain manoeuvres, such as braking, turning and travelling uphill, heavy vehicles apply horizontal loads to the pavement, which are currently not taken into account in the design model. For thin surfaced granular pavements, the stresses generated by these loads are concentrated in the upper pavement layers and can have a significant impact on the performance of the surfacing. In thick asphalt pavements, constructed in a continuous operation, the interface bonding and the strength/stiffness of the layers allows these stresses to be dissipated throughout the pavement and, hence, to have relatively little effect on performance.

Moisture of granular layers – The performance of thin surfaced granular pavements is highly dependent on the properties of upper granular layers, which in turn depend on moisture levels within the granular layers (Section 6.2.2). Designers need to be aware that the predicted performance of thin surfaced granular pavements may not be achieved in situations where the moisture within granular layers is higher than anticipated.

Construction variability – It is often difficult to lay thin layers of asphalt accurately and achieve a uniform thickness and standard of compaction. Thin layers of asphalt cool quickly and so may be less uniform than thicker layers. This can lead to greater variability in performance for thin asphalt layers, particularly under heavy traffic.

Environmental effects – Asphalt oxidises and hardens over time due to exposure to the atmosphere and becomes more prone to ravelling and surface cracking. The rate of oxidation is very closely related to its exposure to the atmosphere and hence the level of insitu air voids in the asphalt layer, which in turn is dependent on the degree of compaction achieved at construction. Better compaction is achieved where the heat in the asphalt can be retained for longer periods, which, for given weather conditions (temperature, wind speed) is achieved with thicker layers.

8.3 Empirical design of granular pavements with thin bituminous surfacing

Pavement types addressed in this sub-section are those which are comprised of unbound layers of granular material and which are surfaced with either a bituminous seal or asphalt less than 40 mm thick. The design procedure for these pavements is illustrated in *Figure 8.3*.

The design procedure is based on an empirical design chart (*Figure 8.4*), which provides the allowable design traffic in terms of rutting and shape loss of these pavements. This design chart does not make any provision for a limitation on the allowable design traffic caused by the fatigue cracking of an asphalt surfacing. The use of mechanistic procedures to assess the fatigue life of such surfacings is discussed in Section 8.2.5.

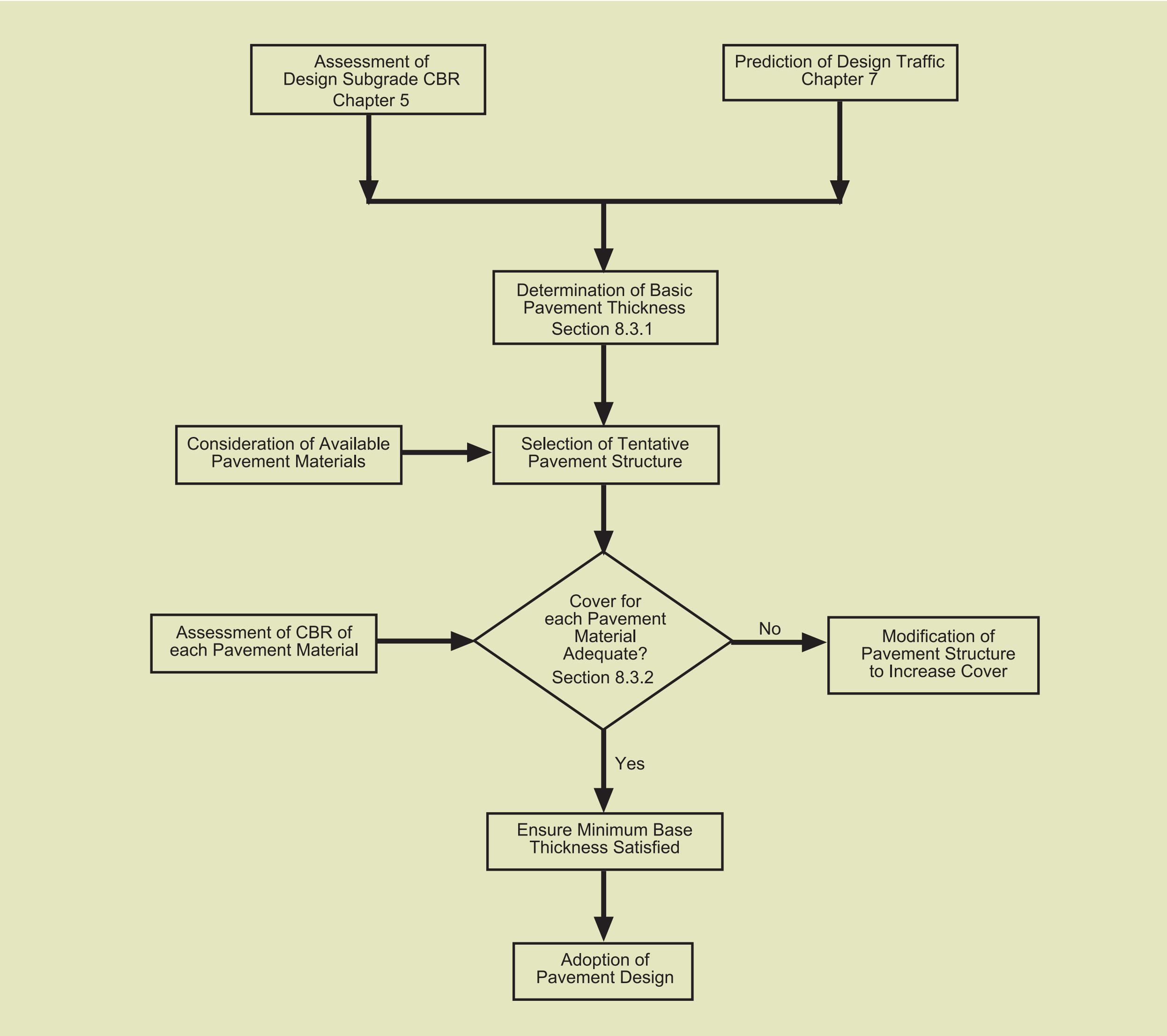


Figure 8.3 Flexible pavement design system for granular pavements with thin bituminous surfacing

8.3.1 Determination of basic thickness

The thickness of unbound granular pavement required over the subgrade is determined using the empirical design chart given in *Figure 8.4* and design traffic in ESA (eqn 7.4).

Note the mechanistic design procedures, as described in Section 8.2, yield the similar total granular thickness as *Figure 8.4* using a top granular moduli of 350 MPa and a SAR_S/ESA factor of 1.2.

8.3.2 Pavement composition

The total thickness of a granular pavement may be made up of a base and any number of subbase courses. The composition of the pavement is made up by providing sufficient cover over the subgrade and each successive subbase course. The thickness of cover required over a subbase is determined from its design CBR. If CBR value of the subbase is less than 30, then the cover required is determined, as for a subgrade material, from *Figure 8.4*.

For a subbase course with a design CBR equal to or greater than 30, it is necessary to provide a minimum thickness of a suitable (CBR ≥ 80) base material. This minimum base thickness is the thickness of cover required over material having a CBR equal to or greater than 30 (*Figure 8.4*).

Note that the CBR test is not the sole measure used to assess adequacy of unbound granular materials (refer Section 6.2.1.1).

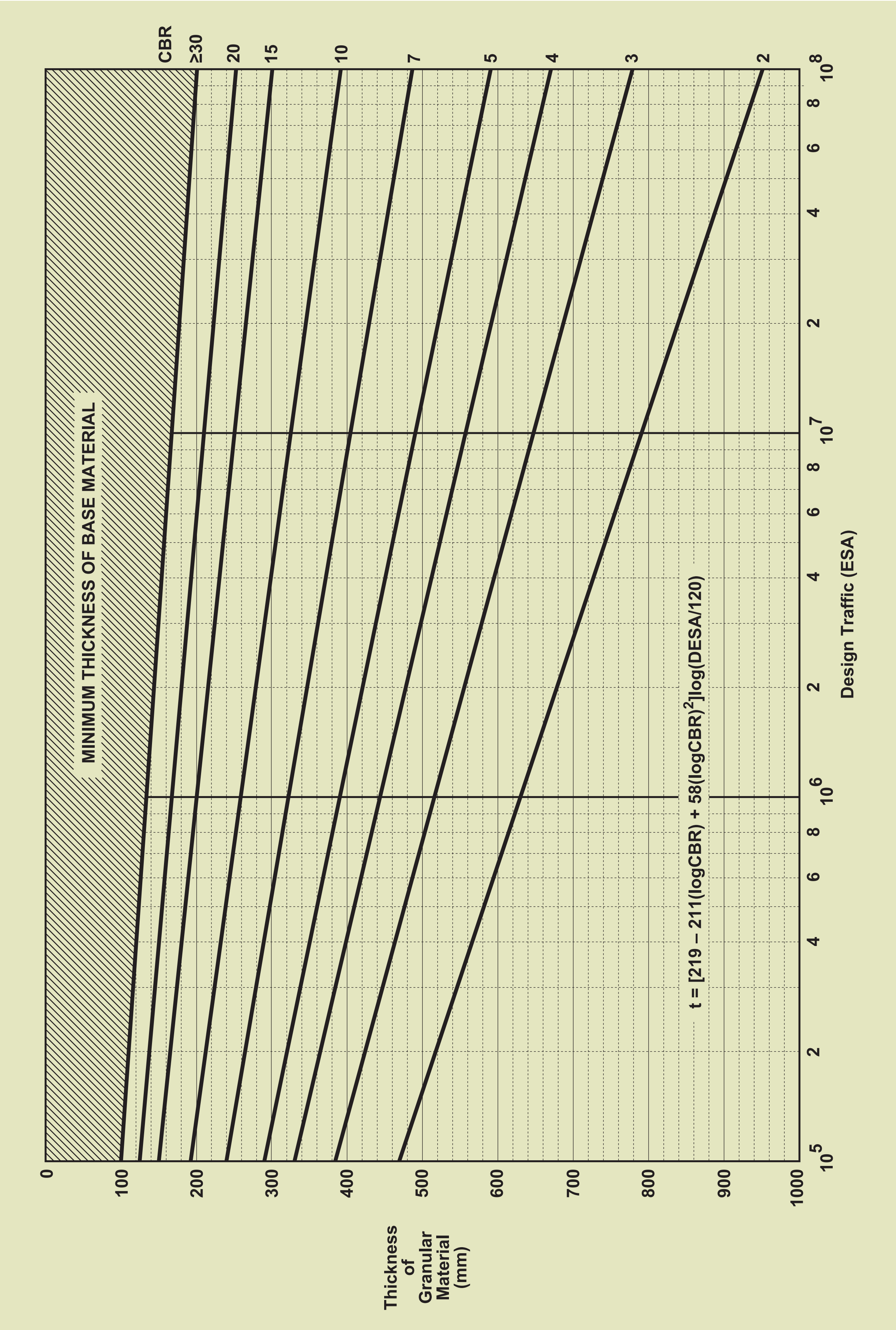


Figure 8.4 Design chart for granular pavements with thin bituminous surfacing

8.4 Mechanistic procedure – example charts

A primary application of this Guide is the provision of a basis for developing design charts for specific circumstances. As an example of this, a number of example design charts for flexible pavements are presented below. Examples of the use of the design charts are given in *Appendix 8.1*.

8.4.1 Values of input parameters implicit in design charts

(i) Design Period

For the example design charts, traffic loading is expressed in terms of the design number of Equivalent Standard Axles (ESA).

The example design charts are applicable to the normal range of design periods.

(ii) Traffic Load Distributions

In developing the example design charts, the presumptive Urban traffic load distribution was used. This distribution is detailed in *Appendix 7.4* and relevant design factors are summarised in *Table 7.8*.

(iii) Materials Characterisation

For asphalt surfaced pavements, an asphalt modulus value of 3000 MPa was adopted. A broader range of asphalt moduli has been used to derive the set of charts for full depth asphalt pavements.

The elastic characterisation of the granular materials was discussed in Section 8.2.3.

Table 8.2 lists the pavement types included in the example design charts.

(iv) Summary of Input Parameters

The charts have been developed using the mechanistic procedure described above for the specific input parameters which presented in *Table 8.3*. Before using these charts for the purpose of pavement design, designers should ensure that their use is appropriate to the design situation for which they are to be used.

8.4.2 Notes accompanying example design charts

Explanatory notes accompany all the example design charts. In addition, discussion about why more than one asphalt thickness is theoretically satisfactory (Charts EC01, EC02 and EC03) refer to *Appendix 8.2*.

8.5 Examples of use of mechanistic procedure

Design examples using the mechanistic procedure are contained in *Appendix 8.3*.

Reference

Mincad Systems (2004). *CIRCLY 5 Users' Manual*. MINCAD Systems Pty Ltd, Richmond, Vic., Australia (www.mincad.com.au).

Table 8.2 Catalogue of example design charts

Pavement layers	Thickness (mm)	Subgrade modulus (MPa)	Chart number
Asphalt – 3000 MPa modulus	varying	30	EC01
Granular	varying	50	EC02
Subgrade		70	EC03

Pavement layers	Thickness (mm)	Asphalt modulus (MPa)	Chart number
Asphalt – range of moduli	varying	1000	EC04
Subgrade		3000	EC05
		5000	EC06

			Cemented material phases	
Pavement layers	Thickness (mm)	Subgrade modulus (MPa)	Pre-cracking Chart No.	Pre & Post cracking Chart No.
Asphalt – 3000 MPa modulus	varying	30	EC07	EC08*
Cemented material – 2000 MPa modulus	varying	50	EC09	EC10*
Subgrade		70	EC11	EC12*

			Cemented material phases	
Pavement layers	Thickness (mm)	Subgrade modulus (MPa)	Pre-cracking Chart No.	Pre & Post cracking Chart No.
Asphalt – 3000 MPa modulus	varying	30	EC13	EC14*
Cemented material – 5000 MPa modulus	varying	50	EC15	EC16*
Subgrade		70	EC17	EC18*

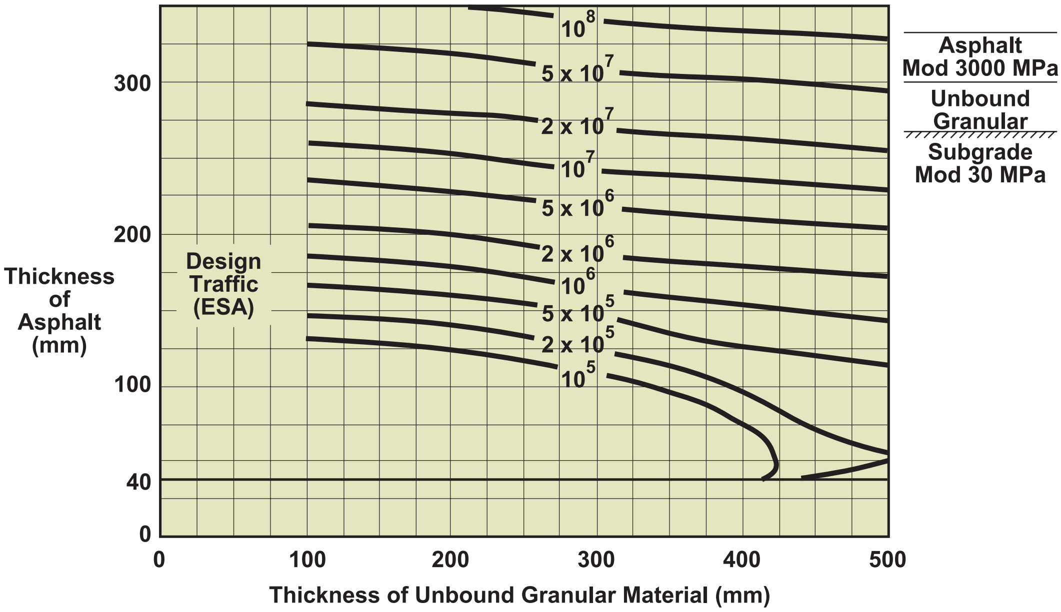
			Cemented material phases	
Pavement layers	Thickness (mm)	Subgrade modulus (MPa)	Pre-cracking Chart No.	Pre & Post cracking Chart No.
Asphalt – 3000 MPa modulus	varying	30	EC19	EC20*
Granular	100	50	EC21	EC22*
Cemented material – 5000 MPa modulus	varying	70	EC23	EC24*
Subgrade				

* Modulus of cemented material in post-cracking phase of life is 500 MPa. For guidelines see Section 8.2.4.

Table 8.3 Values of input parameters adopted for development of example design charts

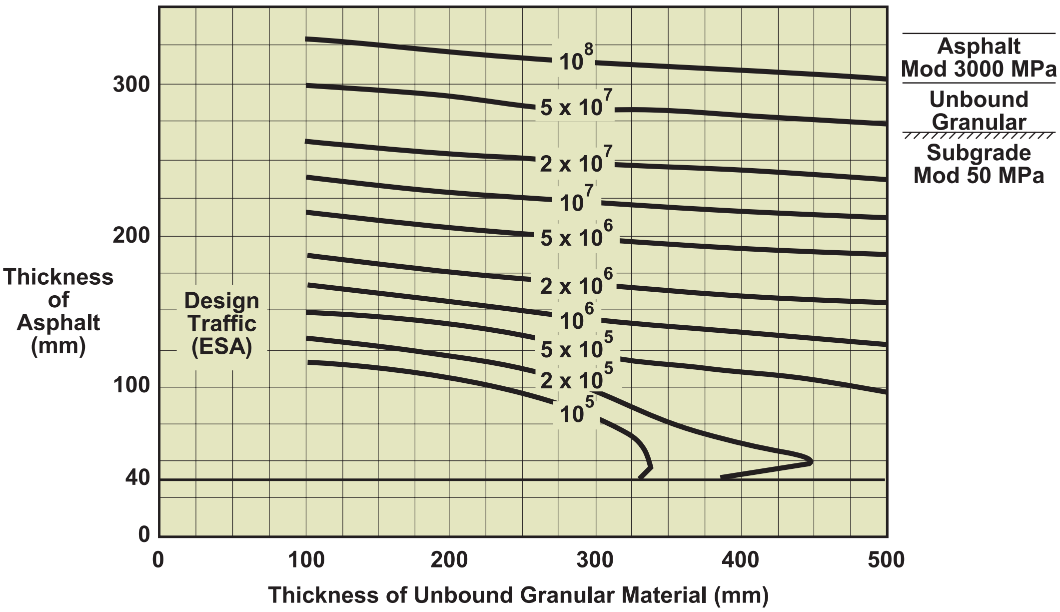
Input parameter	Value adopted for development of example design charts
Design period	Not applicable
Distribution of axle groups	Appendix 7.4 Table A7.4.3 Urban Roads
Distribution of loads on each type of axle group	Appendix 7.4 Table A7.4.3 Urban Roads
Modulus of asphalt	3000 MPa with the following exceptions: Chart EC04 – 1000 MPa; Chart EC06 – 5000 MPa
Poisson's ratio of asphalt	0.40
Elastic characterisation of granular material and need for sublayering	As per Section 8.2.3 and Table 6.4 (a) (normal standard granular) with $\nu_H = \nu_{VH} = 0.35$
Modulus of cemented material	2000 MPa or 5000 MPa, as indicated on specific charts (Phase 2 analysis conducted using 500 MPa for the vertical modulus of cracked cemented material.)
Poisson's ratio of cemented material	0.20
Vertical modulus of subgrade	Charts use modulus, not CBR; use E (MPa) = 10 CBR to convert
Poisson's ratio for subgrade	0.45
Additional anisotropic parameters for subgrade	As per Section 5
Standard axle loading input to CIRCLY	Four circular areas of radius 92.1 mm, uniform vertical stress of 750 kPa. Geometry as shown in Figure 8.2.
Relationship for asphalt fatigue	$N = RF\left[\frac{6918(0.856\,V_B + 1.08)}{S_{mix}^{0.36}\,\mu\varepsilon}\right]^5$ where S_{mix} = asphalt modulus (MPa) V_B = volume of binder in asphalt mix = 11% RF = 1, desired project reliability 95%
Relationship for fatigue of cemented materials	$N = RF\left[\frac{113\,000/E^{0.804} + 191}{\mu\varepsilon}\right]^{12}$ where E = modulus of cemented material (MPa) RF = 1, desired project reliability 95%
Relationship for permanent deformation	$N = \left[\frac{9300}{\mu\varepsilon}\right]^7$

EC01 – EXAMPLE DESIGN CHART 1



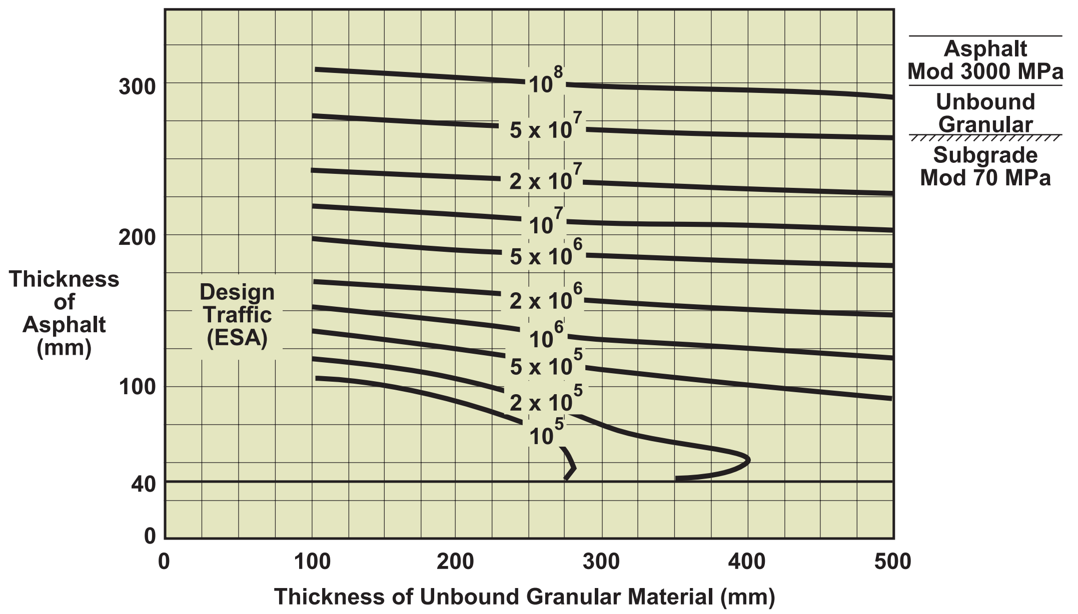
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. For explanation of why more than one asphalt thickness is satisfactory refer to Appendix 8.2
4. There is considerable uncertainty associated with the use of the mechanistic model for asphalt thicknesses less than 40 mm. Refer Section 8.2.5.

EC02 – EXAMPLE DESIGN CHART 2



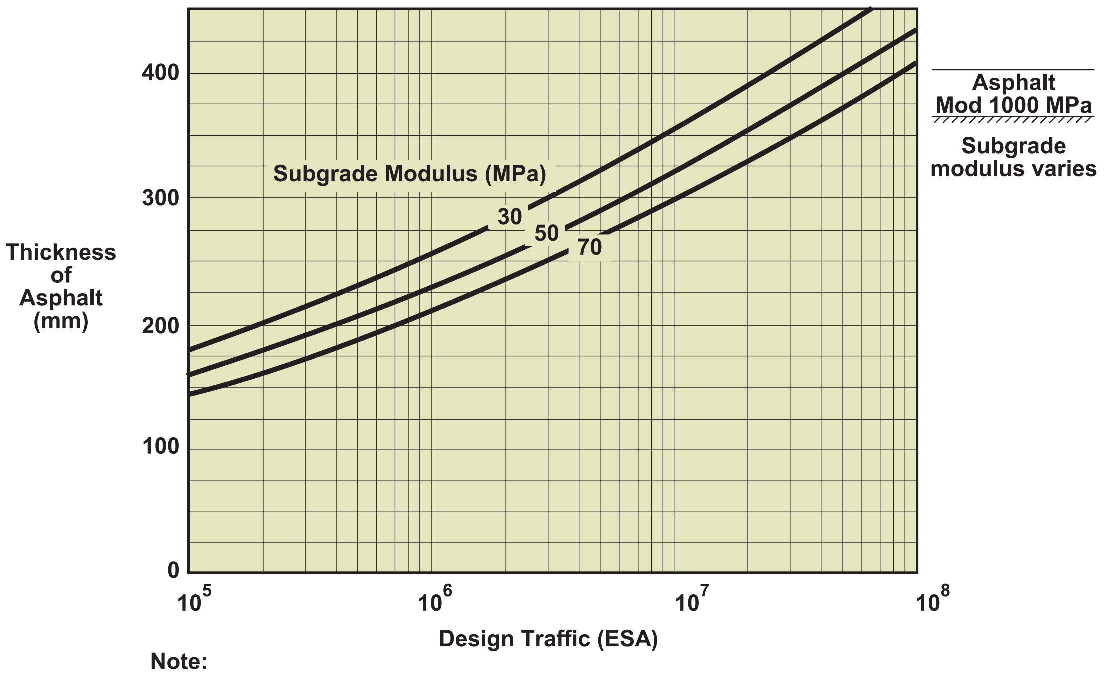
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. For explanation of why more than one asphalt thickness is satisfactory refer to Appendix 8.2
4. There is considerable uncertainty associated with the use of the mechanistic model for asphalt thicknesses less than 40 mm. Refer Section 8.2.5.

EC03 – EXAMPLE DESIGN CHART 3

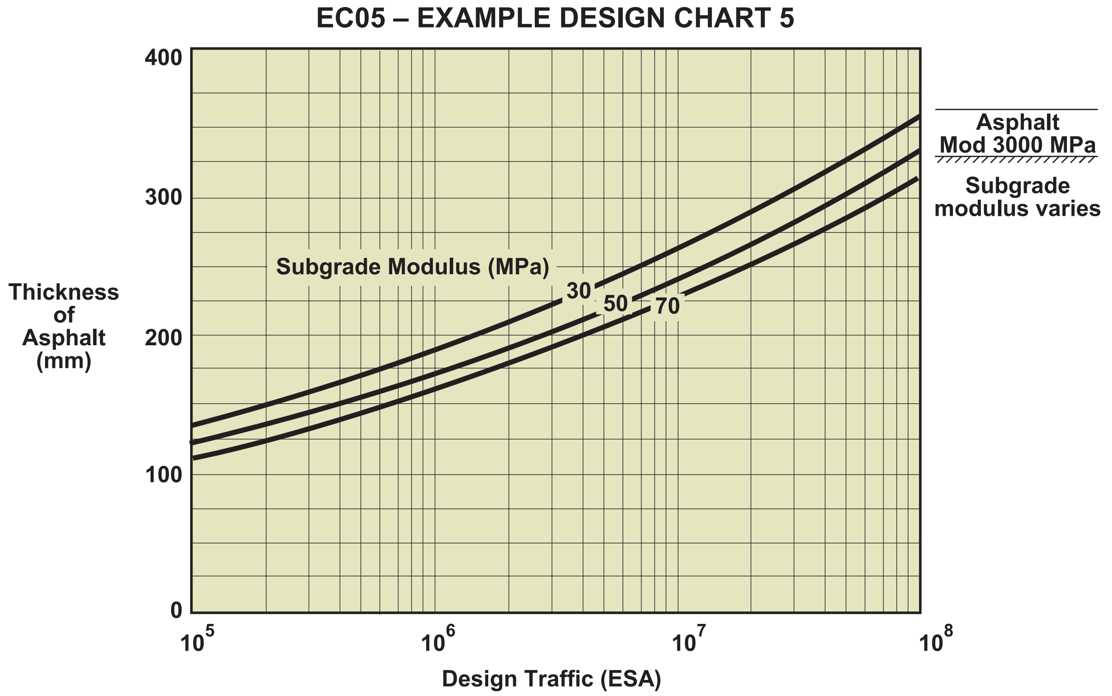


Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. For explanation of why more than one asphalt thickness is satisfactory refer to Appendix 8.2
4. There is considerable uncertainty associated with the use of the mechanistic model for asphalt thicknesses less than 40 mm. Refer Section 8.2.5.

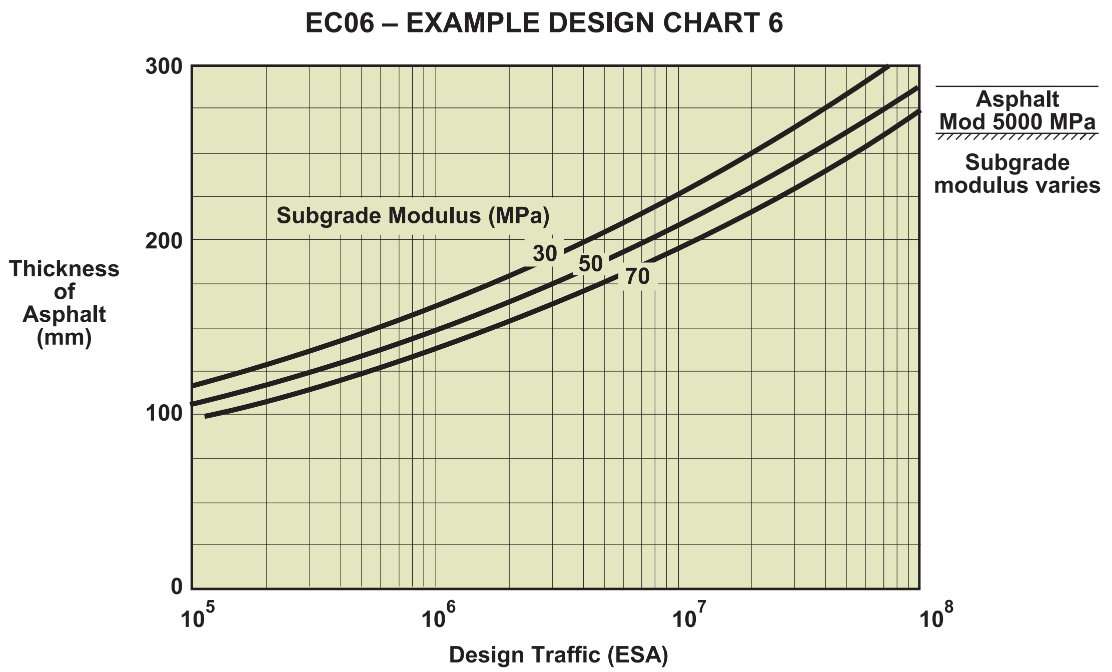
EC04 – EXAMPLE DESIGN CHART 4



Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.



Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.

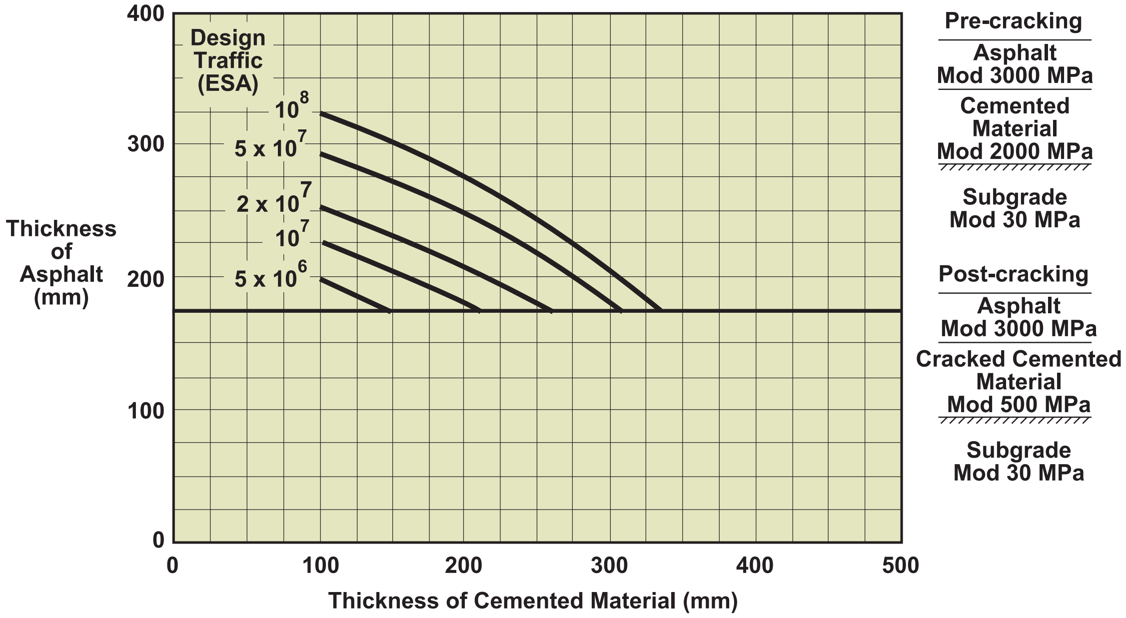


Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.

EC07 – EXAMPLE DESIGN CHART 7

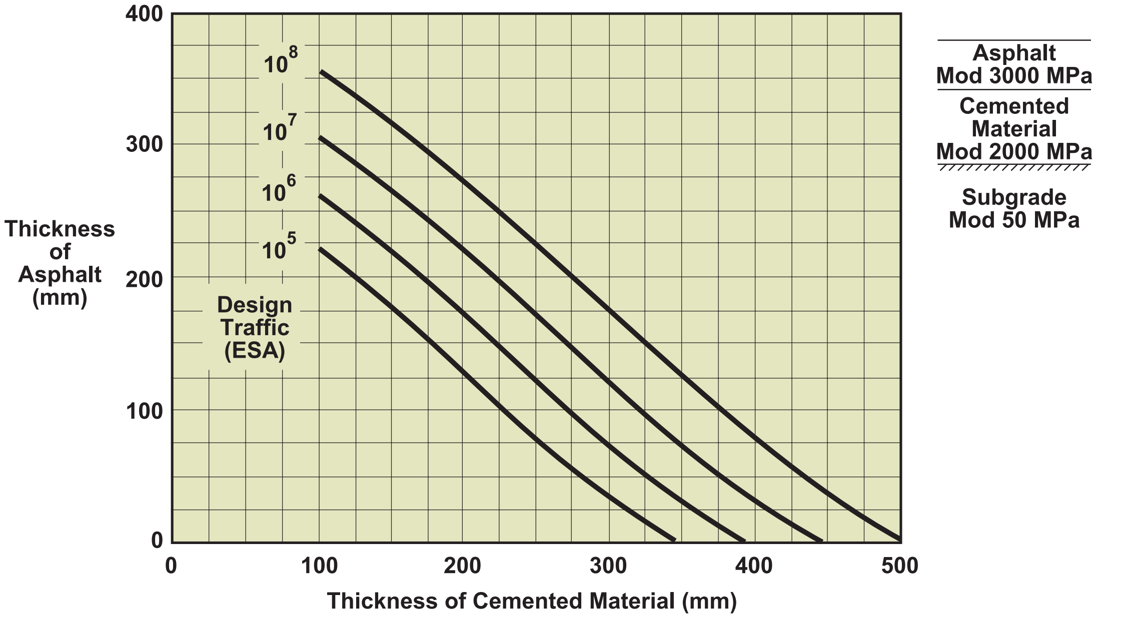
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of cemented material.
3. For pavements where the cover over the cemented material is 175mm or more, the second phase of life of the pavements after cracking of the cemented material may be considered, see Chart EC08. For guidelines see Section 8.2.4.

EC08 – EXAMPLE DESIGN CHART 8



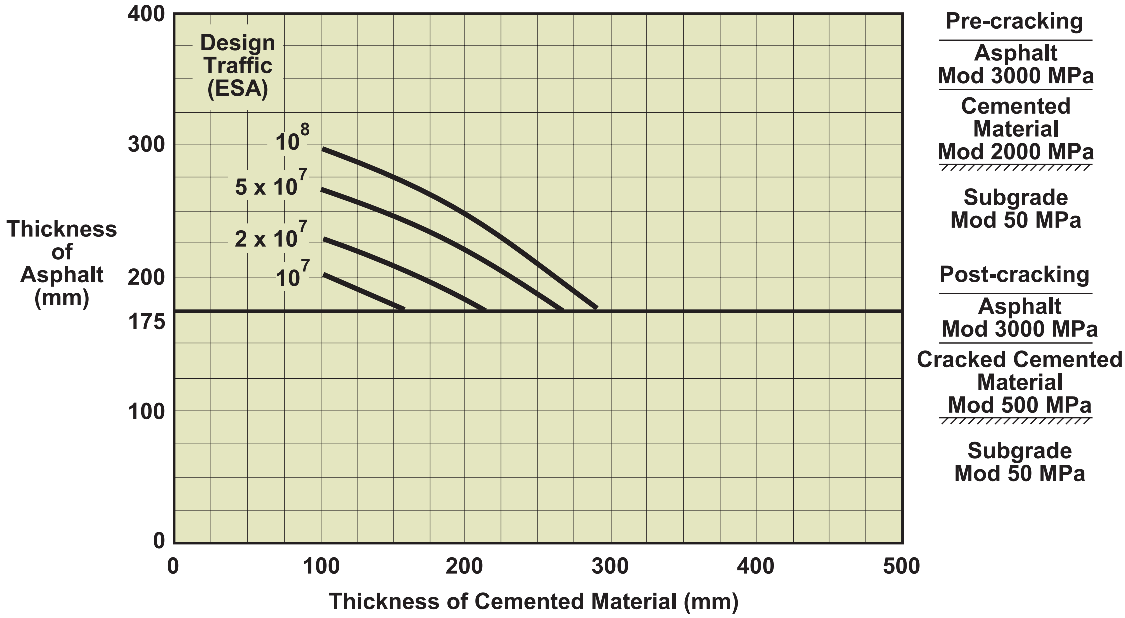
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. The second phase of life of the pavements after cracking of the cemented material has been considered. For guidelines see Section 8.2.4.

EC09 – EXAMPLE DESIGN CHART 9



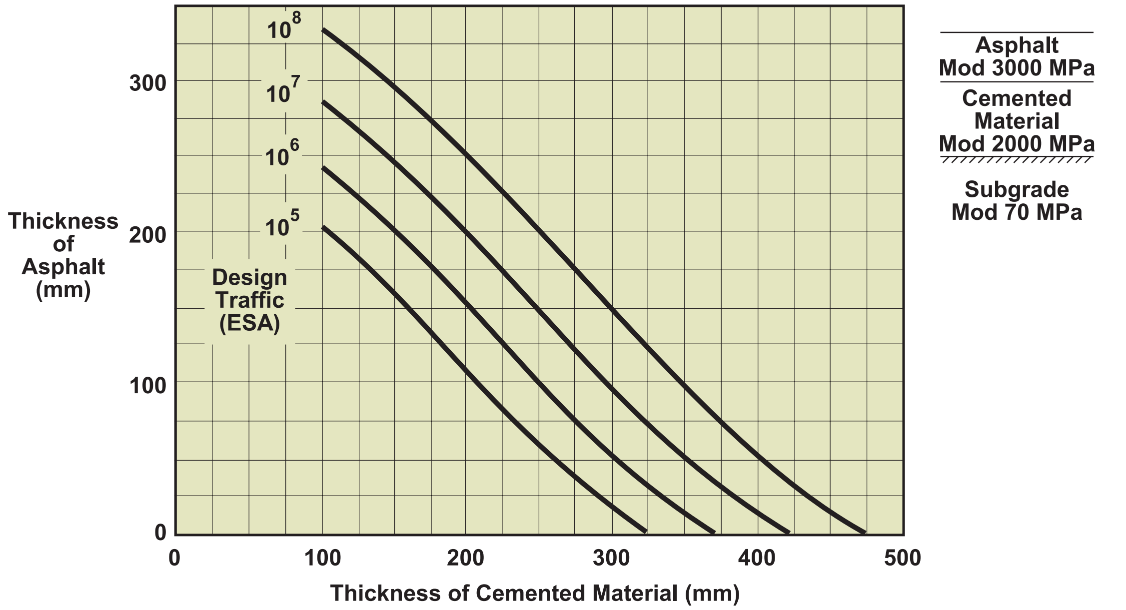
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of cemented material.
3. For pavements where the cover over the cemented material is 175 mm or more, the second phase of life of the pavements after cracking of the cemented material may be considered, see Chart EC10.
   For guidelines see Section 8.2.4.

EC10 – EXAMPLE DESIGN CHART 10



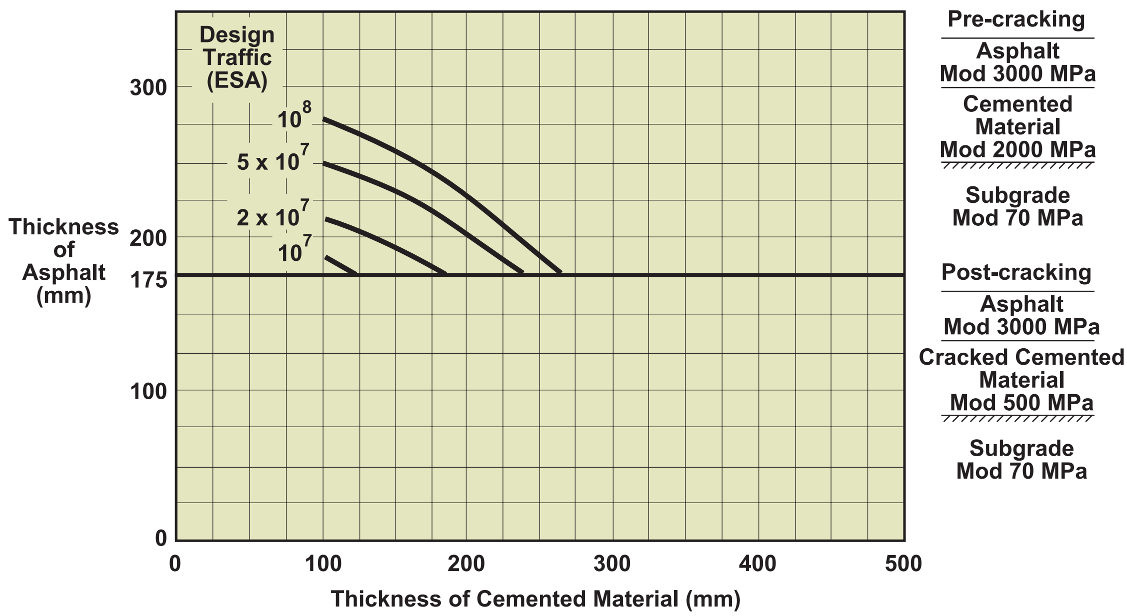
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. The second phase of life of the pavements after cracking of the cemented material has been considered.
   For guidelines see Section 8.2.4.

EC11 – EXAMPLE DESIGN CHART 11



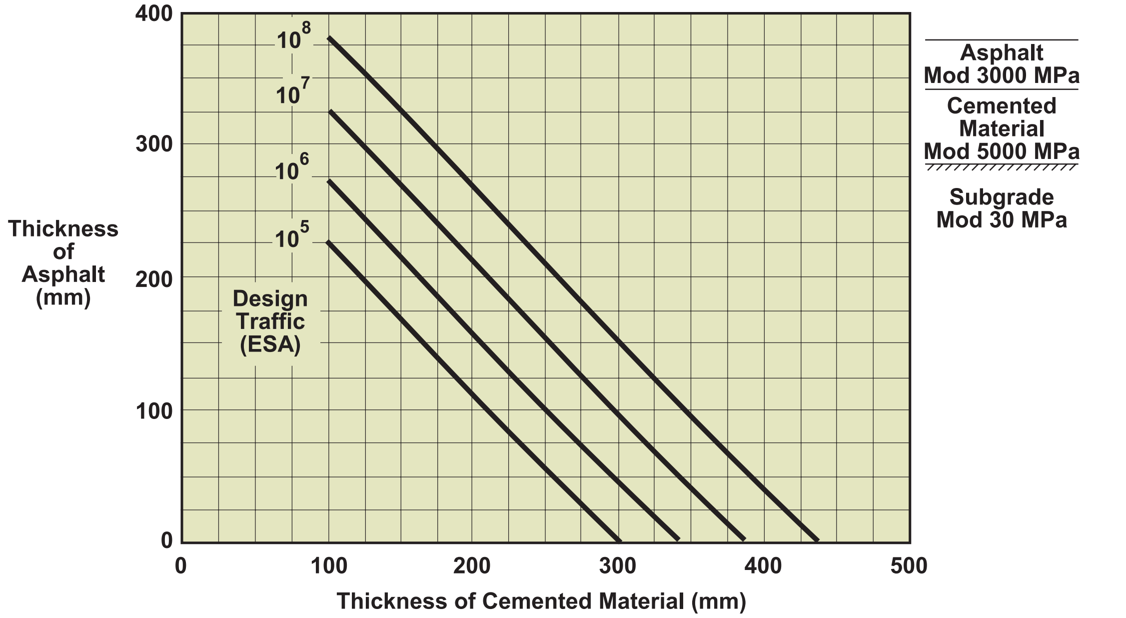
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of cemented material.
3. For pavements where the cover over the cemented material is 175 mm or more, the second phase of life of the pavements after cracking of the cemented material may be considered, see Chart EC12. For guidelines see Section 8.2.4.

EC12 – EXAMPLE DESIGN CHART 12



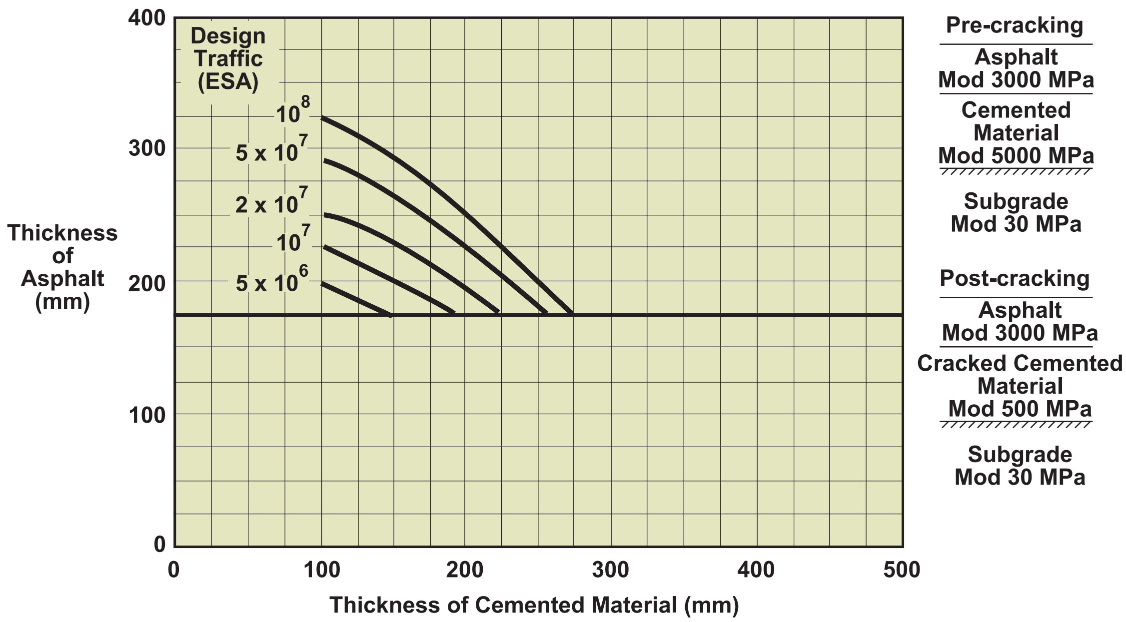
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. The second phase of life of the pavements after cracking of the cemented material has been considered. For guidelines see Section 8.2.4.

EC13 – EXAMPLE DESIGN CHART 13



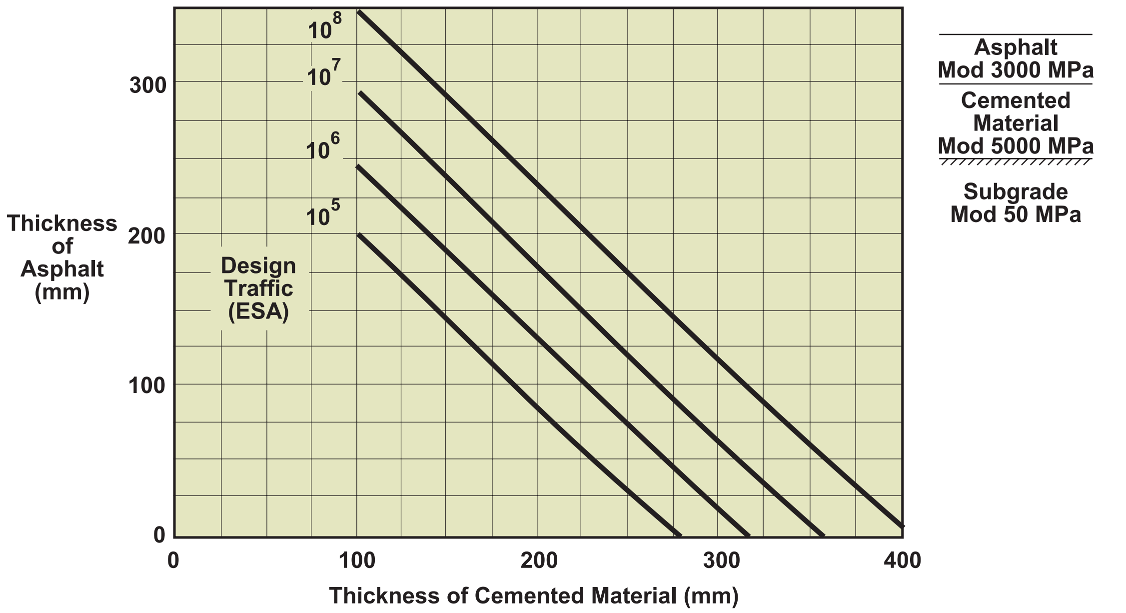
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of cemented material.
3. For pavements where the cover over the cemented material is 175mm or more, the second phase of life of the pavements after cracking of the cemented material may be considered, see Chart EC14. For guidelines see Section 8.2.4.

EC14 – EXAMPLE DESIGN CHART 14



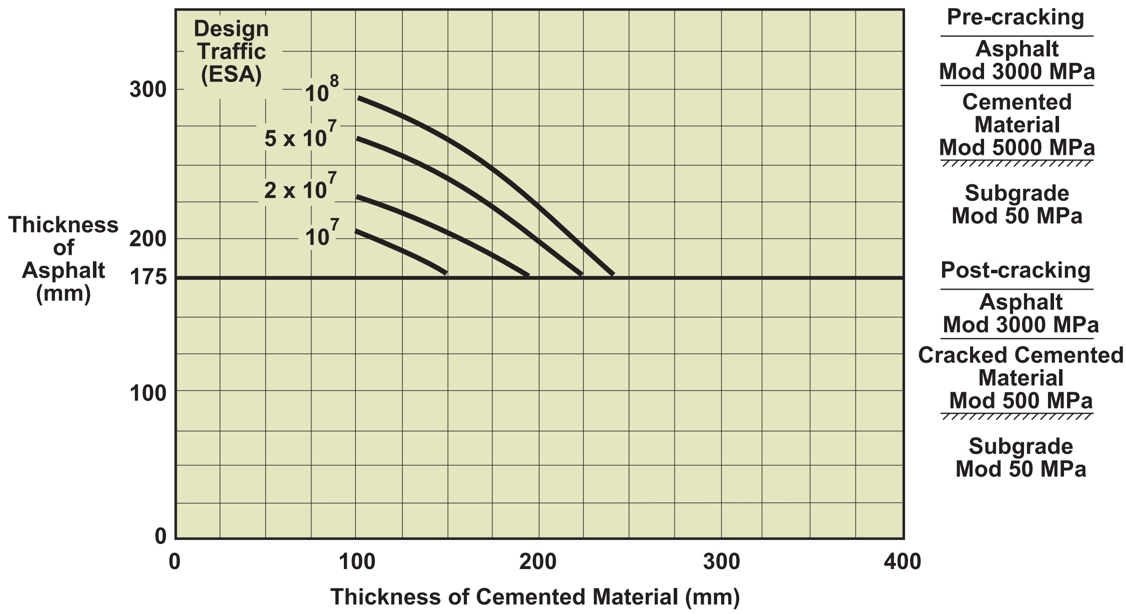
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. The second phase of life of the pavements after cracking of the cemented material has been considered. For guidelines see Section 8.2.4.

EC15 – EXAMPLE DESIGN CHART 15



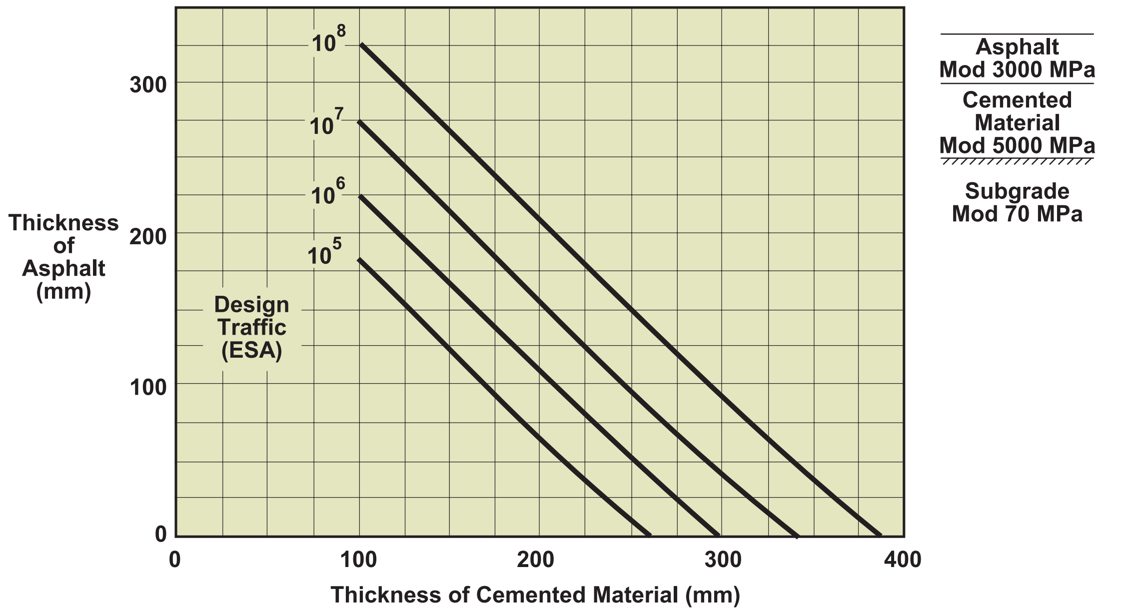
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of cemented material.
3. For pavements where the cover over the cemented material is 175 mm or more, the second phase of life of the pavements after cracking of the cemented material may be considered, see Chart EC16.
   For guidelines see Section 8.2.4.

EC16 – EXAMPLE DESIGN CHART 16



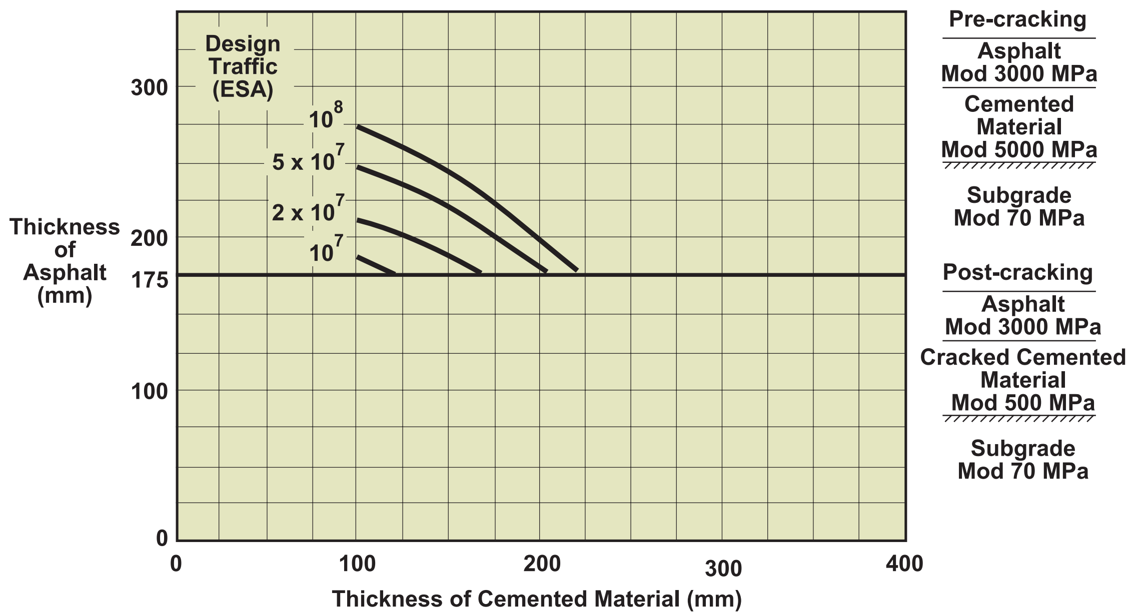
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. The second phase of life of the pavements after cracking of the cemented material has been considered.
   For guidelines see Section 8.2.4.

EC17 – EXAMPLE DESIGN CHART 17



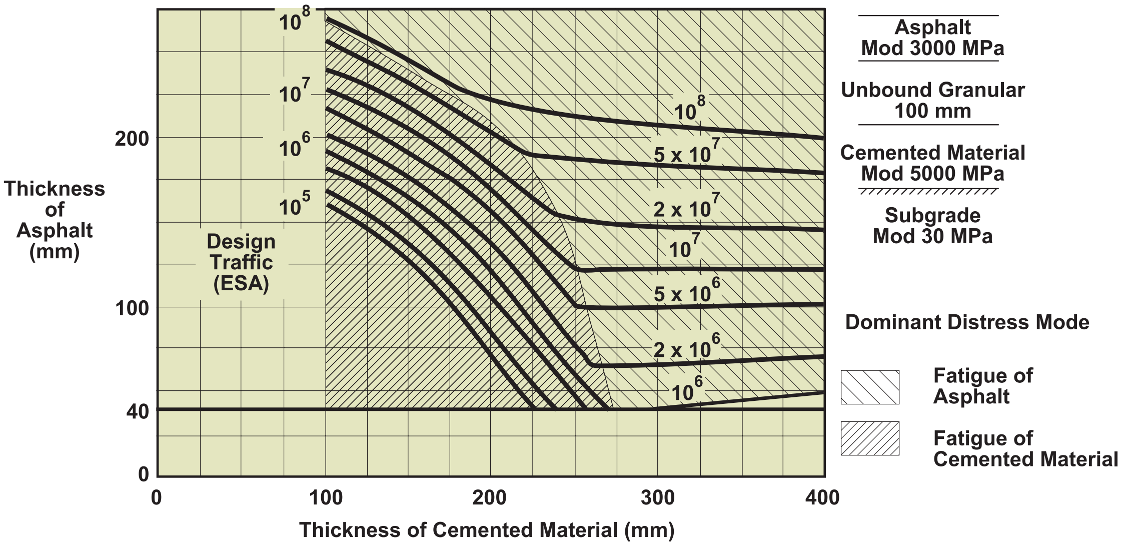
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of cemented material.
3. For pavements where the cover over the cemented material is 175 mm or more, the second phase of life of the pavements after cracking of the cemented material may be considered, see Chart EC18.
   For guidelines see Section 8.2.4.

EC18 – EXAMPLE DESIGN CHART 18



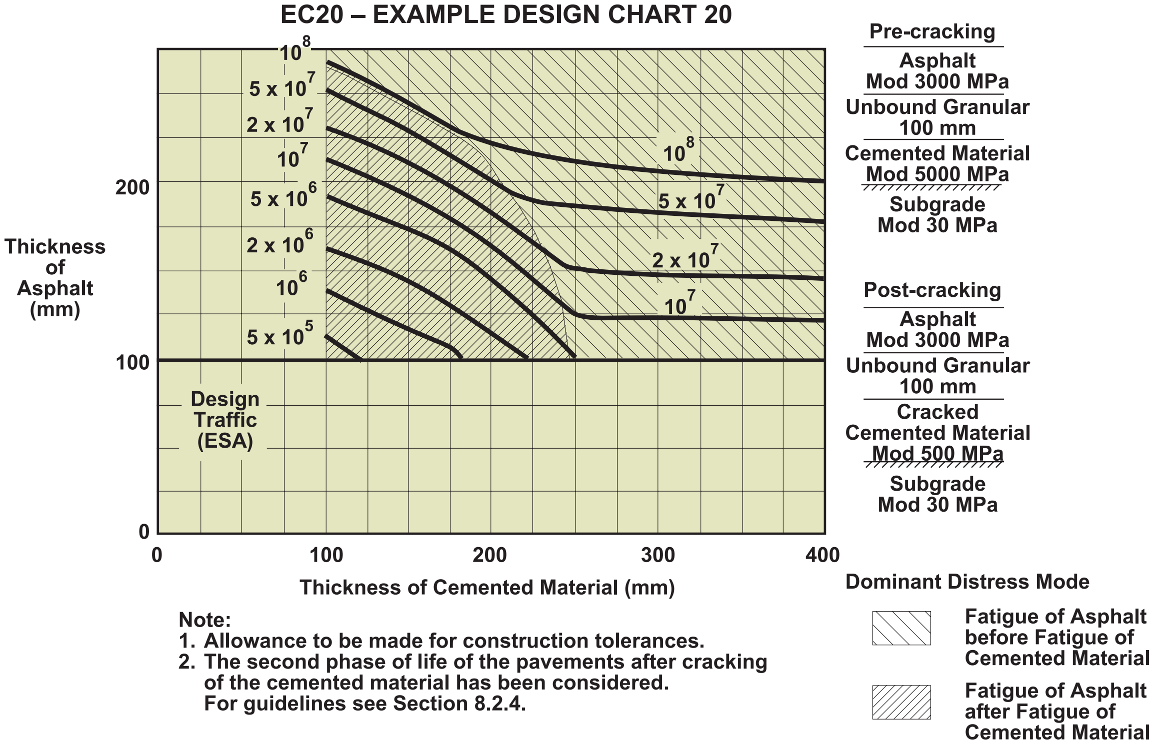
Note:
1. Allowance to be made for construction tolerances.
2. Dominant distress mode is fatigue of asphalt.
3. The second phase of life of the pavements after cracking of the cemented material has been considered.
   For guidelines see Section 8.2.4.

EC19 – EXAMPLE DESIGN CHART 19



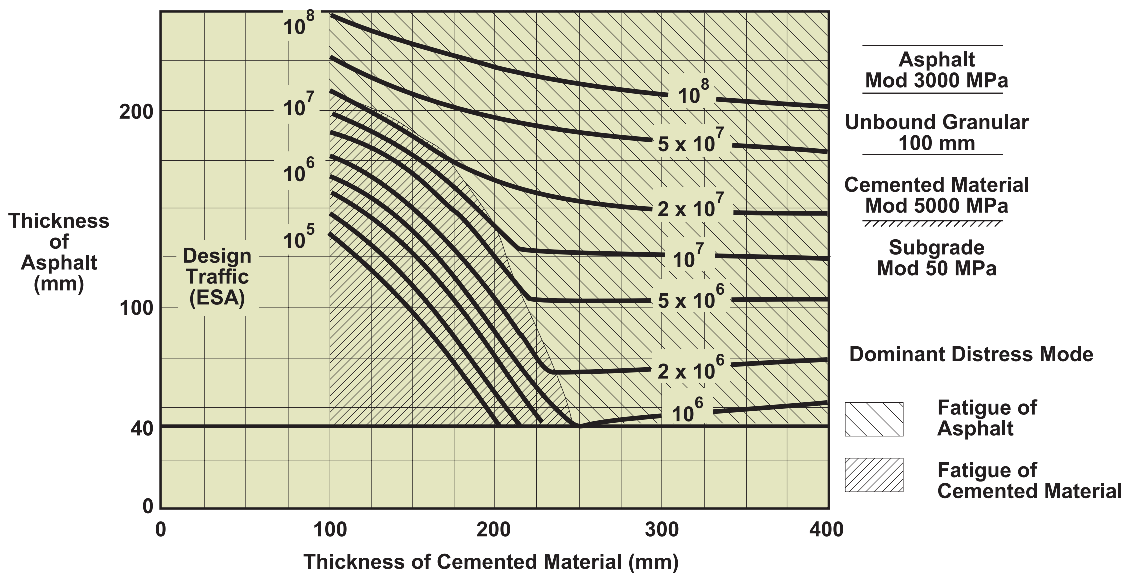
Note:
1. Allowance to be made for construction tolerances.
2. For pavements where the cover over the cemented material is 175 mm or more, the second phase of life of the pavements after cracking of the cemented material may be considered, see Chart EC20. For guidelines see Section 8.2.4.
3. There is considerable uncertainty associated with the use of the mechanistic model for asphalt thicknesses less than 40 mm. Refer to Section 8.2.5.

EC20 – EXAMPLE DESIGN CHART 20



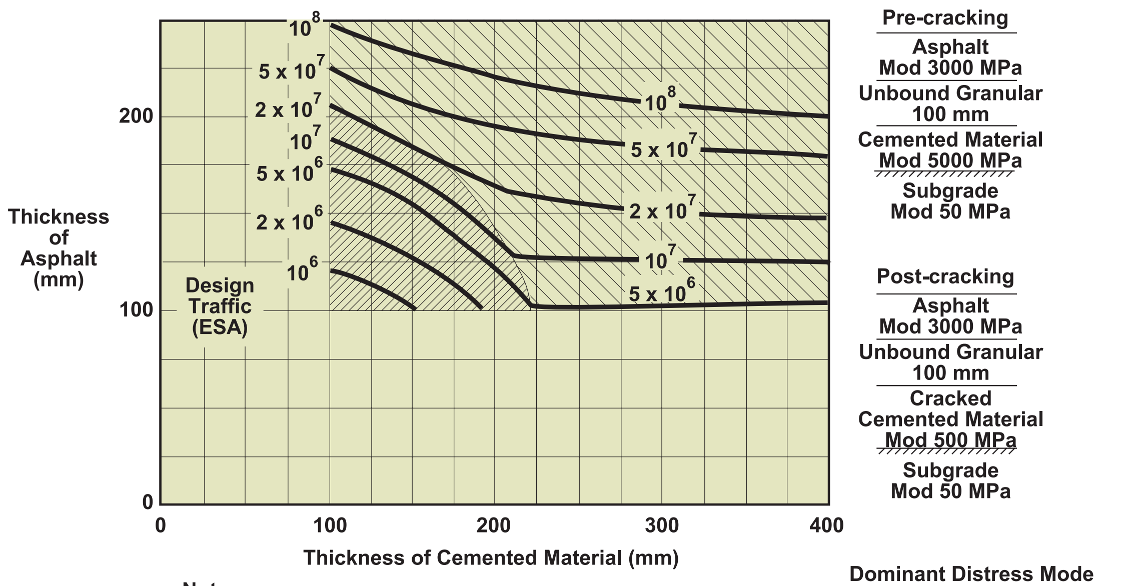
Note:
1. Allowance to be made for construction tolerances.
2. The second phase of life of the pavements after cracking of the cemented material has been considered. For guidelines see Section 8.2.4.

EC21 – EXAMPLE DESIGN CHART 21



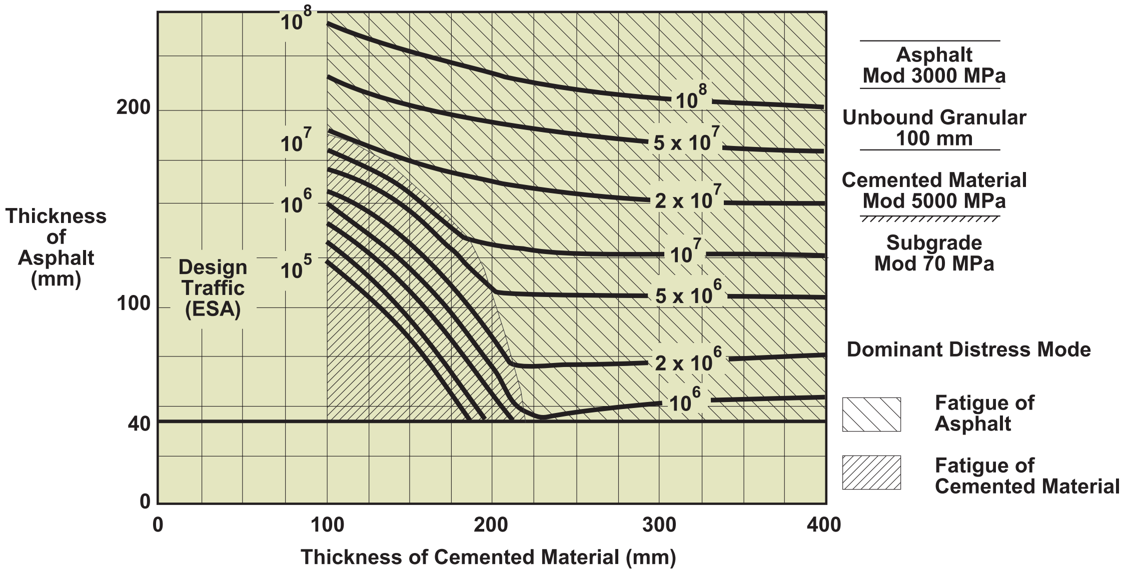
Note:
1. Allowance to be made for construction tolerances.
2. For pavements where the cover over the cemented material is 175 mm or more, the second phase of life of the pavements after cracking of the cemented material may be considered, see Chart EC22. For guidelines see Section 8.2.4.
3. There is considerable uncertainty associated with the use of the mechanistic model for asphalt thicknesses less than 40 mm. Refer to Section 8.2.5.

EC22 – EXAMPLE DESIGN CHART 22



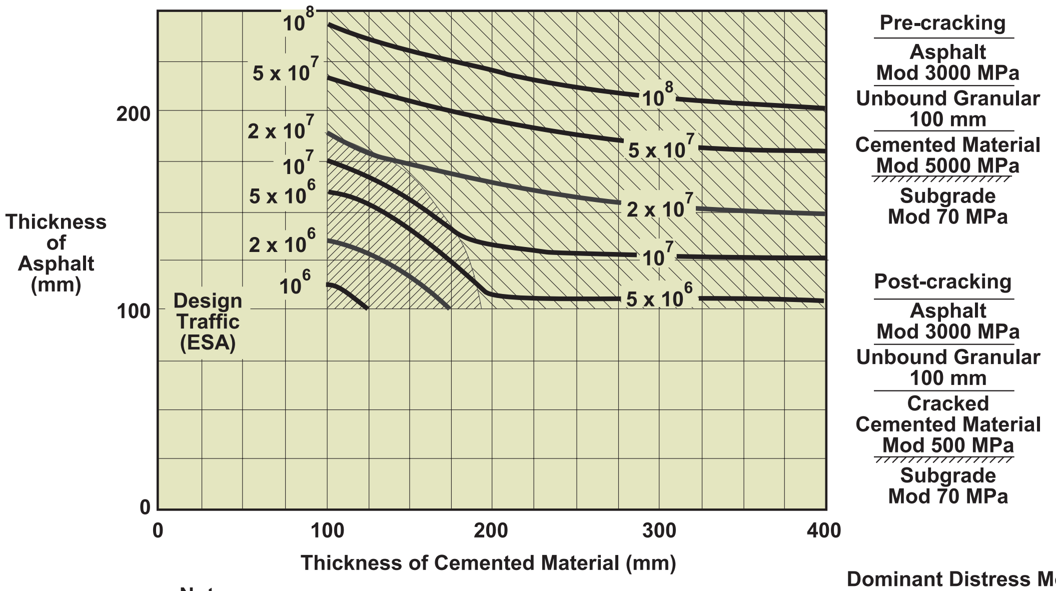
Note:
1. Allowance to be made for construction tolerances.
2. The second phase of life of the pavements after cracking of the cemented material has been considered. For guidelines see Section 8.2.4.

EC23 – EXAMPLE DESIGN CHART 23



Note:
1. Allowance to be made for construction tolerances.
2. For pavements where the cover over the cemented material exceeds 175 mm, the second phase of life of the pavements after cracking of the cemented material may be considered. For guidelines see Section 8.2.4.
3. There is considerable uncertainty associated with the use of the mechanistic model for asphalt thicknesses less than 40 mm. Refer to Section 8.2.5.

EC24 – EXAMPLE DESIGN CHART 24



Note:
1. Allowance to be made for construction tolerances.
2. The second phase of life of the pavements after cracking of the cemented material has been considered. For guidelines see Section 8.2.4.

Dominant Distress Mode

Fatigue of Asphalt before Fatigue of Cemented Material

Fatigue of Asphalt after Fatigue of Cemented Material

9

DESIGN OF NEW RIGID PAVEMENTS

9 DESIGN OF NEW RIGID PAVEMENTS

9.1 General

This Chapter provides guidance on the thickness design of rigid pavements consisting of cast insitu concrete and proposed for roads carrying heavy traffic vehicles as defined by Austroads Classes 3 to 12 (*Table 7.1*) with a design traffic loading exceeding 1×10^6 heavy vehicle axle groups (HVAG). The information in this Chapter is not intended for use in the design of residential, industrial or airport pavements.

Whilst this document is aimed at the determination of pavement configuration and concrete base thickness, it also provides guidance on various structural issues, such as dowels and joint detailing. Detailed information on these issues are available from Austroads Member Authorities.

This design method is based on the USA Portland Cement Association (PCA 1984) method with revisions to suit Australian conditions (Austroads 2004). The method assumes that the base and subbase layers are not bonded.

The design method is based on assessments of the:

- predicted traffic volume and composition over the design period (refer to Chapter 7);
- strength of the subgrade in terms of its California Bearing Ratio (refer to Chapter 5); and
- strength of the base concrete (refer to Chapter 6).

It should be remembered that all rigid pavements are designed on the presumption of uniform support and cannot be expected to perform as simply-supported structures. Therefore, careful consideration needs to be given to their use (and selection of type) in areas of potentially high differential settlement. Factors to be considered include the expected radius of curvature, the orientation of settlement relative to joints, the likely distress severity and the ease of undertaking rehabilitation.

A bound or lean mix concrete subbase is recommended under a concrete pavement for one or more of the following reasons:

- to resist erosion of the subbase and limit "pumping" at joints and slab edges;
- to provide uniform support under the pavement;
- to reduce deflection at joints and enhance load transfer across joints (especially if no other load transfer devices are provided, such as dowels); and
- to assist in the control of shrinkage and swelling of high-volume-change subgrade soils.

With these factors remaining constant, the concrete base thickness will vary according to the type of shoulder and joint/reinforcement details adopted. The selection of the overall pavement configuration is a matter for decision by the designer based on its suitability for a particular project and economics.

The thickness design approach outlined in this Chapter is based on analytical models and field testing of pavements with typical joint spacing and range of thickness for roads. In order to assist users, Section 9.2.1 provides guidance on typical joint spacing and other details for four concrete pavement types used commonly for roads in Australia.

9.2 Pavement types

9.2.1 Base types

The principal types of cementitious concrete pavements are:

- jointed plain (unreinforced) concrete pavements (PCP);
- jointed reinforced concrete pavements (JRCP);
- continuously reinforced concrete pavements (CRCP); and
- steel fibre reinforced concrete pavements (SFCP).

Another type of concrete pavement is the prestressed concrete pavement, but design procedures are not included in this Guide.

The following notes are indicative of current practices in Australia and provide a useful guide to the typical joint spacing of various pavement types.

There are two main categories of PCP suitable for Australian conditions, as follows:

- slabs 4.2 m long, with undowelled skewed joints, and
- slabs 4.5 m long, with dowelled square joints.

In urban areas, undowelled joints are typically constructed square in order to facilitate both design and construction.

Slab span (i.e. length and/or width) does not feature as an input parameter in the Austroads thickness design model. Nevertheless, designers should be aware that it will have an influence on fatigue life and so it is recommended that the above dimensions be adopted as upper limits.

So-called "plain" concrete pavement must be reinforced in discrete areas such as odd-shaped and mismatched slabs and in base anchor slabs.

JRCP is typically mesh reinforced, with square dowelled joints at spacings of 8 to 12 m.

In a CRC pavement, sufficient continuous longitudinal steel reinforcement is provided to induce transverse cracking at random spacings of about 0.5 to 2.5 m, and no contraction joints are required. Transverse reinforcement is provided to support the longitudinal steel and is designed in accordance with the subgrade drag theory. Transverse steel also provides "insurance" in the event of unplanned longitudinal cracking.

In PCP, JRCP and CRCP longitudinal joints should be provided to limit slab widths to about 4.3 m and should be tied up to a maximum total tied width of about 15 m. Untied longitudinal joints are unlikely to satisfy the "with shoulder" design condition unless they are located outside the zone of traffic influence.

Steel fibre reinforced concrete (SFRC) provides increased resistance to cracking in both odd-shaped and acute cornered slabs and so is ideally suited to areas with a high proportion of slabs of irregular shape. At intersections and roundabouts it will often be the only viable concrete pavement option because of the increased flexibility it allows in the design of the joint layout.

Slab corner angles should be maximised wherever possible but in SFCP they can be reduced to 65°. However, where possible, these low corner angles should be located away from commercial vehicle wheelpaths.

Transverse joints in SFCP are typically square (or radial in the case of roundabouts) and undowelled, at maximum spacings of 6.0 m.

Guidelines for tying of longitudinal joints are consistent with those for other base types, except that an increase in the total tied width appears to be warranted by the higher tensile and flexural strength of SFRC.

9.2.2 Subbase types

The purpose of the subbase is to provide uniform support to the base concrete layer and provide sufficient resistance to prevent erosion of subbase material under traffic and environmental conditions. Only lean-mix concrete or bound subbases are recommended for the traffic levels described in this document.

For the purpose of rigid pavement design, a bound subbase is defined as being composed of either:

- cement stabilised crushed rock with not less than 5% by mass cementitious content to ensure satisfactory erosion resistance (verifiable by laboratory erodability testing) – the cementitious content may include cement, lime/fly ash and/or ground granulated blast furnace slag; or
- dense-graded asphalt; or
- rolled lean concrete having a characteristic 28-day strength of not less than 5 MPa.

Lean-mix concrete (LMC) has a characteristic 28-day compressive strength of not less than 5 MPa and be designed to have low shrinkage, typically less than 450 microstrain.

Notes.

1. As a result of growing evidence from worldwide experience, increasing emphasis is being placed on the important role of the subbase in rigid pavement performance. Materials properties are therefore very important but are beyond the scope of this document.
2. Interlayer debonding is accomplished by the application of a bond-breaking layer to the surface of the subbase to provide a smooth surface with an appropriate level of uniform friction to avoid surface interlock (refer to *Table 9.10*).
3. Lean-mix concrete subbases are constructed as mass concrete without transverse joints and will therefore develop cracks. It is intended to achieve a pattern of relatively closely spaced and narrow

cracks that provide a degree of load transfer and which, in conjunction with a debonding layer, will not reflect into the base. Limiting both the upper strength and the shrinkage of the subbase concrete controls cracking. Current practice for longitudinal construction joints, if required by the construction process, is to offset these subbase joints by between 100 and 400 mm from the longitudinal joints in the base pavement in order to avoid reflective cracking.

Australian experience is that LMC subbase performs well with undowelled PCP base.

The selection of the minimum subbase type is determined from *Table 9.1* using the estimated design traffic.

Table 9.1 Minimum subbase requirements for rigid pavements

Design traffic (HVAG)	Subbase type
Up to 10^6	125 mm bound
Up to 5×10^6	150 mm bound or 125 mm LMC
Up to 1×10^7	170 mm bound or 125 mm LMC
Greater than 1×10^7	150 mm LMC*

* Under CRCP, a heavily bound subbase with an asphalt surfacing is an acceptable alternative.

9.2.3 Wearing surface

The wearing surface texture specified for the road should take into consideration the traffic speed, grade, cross-fall, carriageway width and rainfall. For further details refer to the *Austroads Guide to the Selection of Road Surfacings* (Austroads 2003).

For the purpose of the base thickness design, wearing surface layers of asphalt or concrete segmental paving are deemed not to contribute to the strength of the pavement.

9.3 Factors used in thickness determination

9.3.1 Strength of subgrade

For rigid pavement thickness design, the strength of the subgrade may be assessed in terms of CBR. Methods of assessing the design CBR are discussed in Chapter 5.

For thickness design purposes, all materials within 1 metre below the subbase must be assessed for determination of the design subgrade CBR. In other words, it is not permissible to adopt the CBR of a thin layer of high strength material immediately below the subbase.

The selected subgrade material should consist of a uniform volumetric stable material (refer to Chapter 5) that is constant for the service life of the pavement with a minimum CBR of 5%. The depth of the selected subgrade material below the subbase should be increased to 600 mm over expansive subgrades.

9.3.2 Effective subgrade strength

The recommended minimum subbase thickness and type for various levels of traffic loadings is described in Section 9.2.2.

For jointed undowelled bases a lean-mix concrete subbase is recommend with a minimum of 150 mm of select material below the subbase layer.

For guidance regarding the strength of subbases for use in lightly trafficked roads and roundabouts refer to APRG Report 21 (Austroads Pavement Reference Group, 1998), and RTA NSW (2003), respectively.

Research work and experience has identified that there is an increase in effective subgrade strength with the use of bound and lean-mix concrete subbases. *Figure 9.1* may be used to determine the increase in strength for use in base thickness design.

Other subbase types that can be adequately constructed may be used providing special investigations into the assessment of the effective design subgrade CBR are carried out.

Where the subgrade within 1 metre of the underside of the subbase shows (or is likely to show) vertical stratification, the determination of the design CBR must be based on a multi-layered subgrade system. The formula given in equation 9.1 provides a model that may be used to determine this equivalent design subgrade strength (CBR_m) based on the strength of the supporting soil depth (Japan Road Association, 1989).

$$CBR_m = \left[\frac{\sum_i h_i \, CBR_i^{0.33}}{\sum_i h_i} \right]^3 \quad (9.1)$$

where

CBR_i is the CBR value in layer thickness h_i., and

$\sum h_i$ is taken to a depth of 1.0 m.

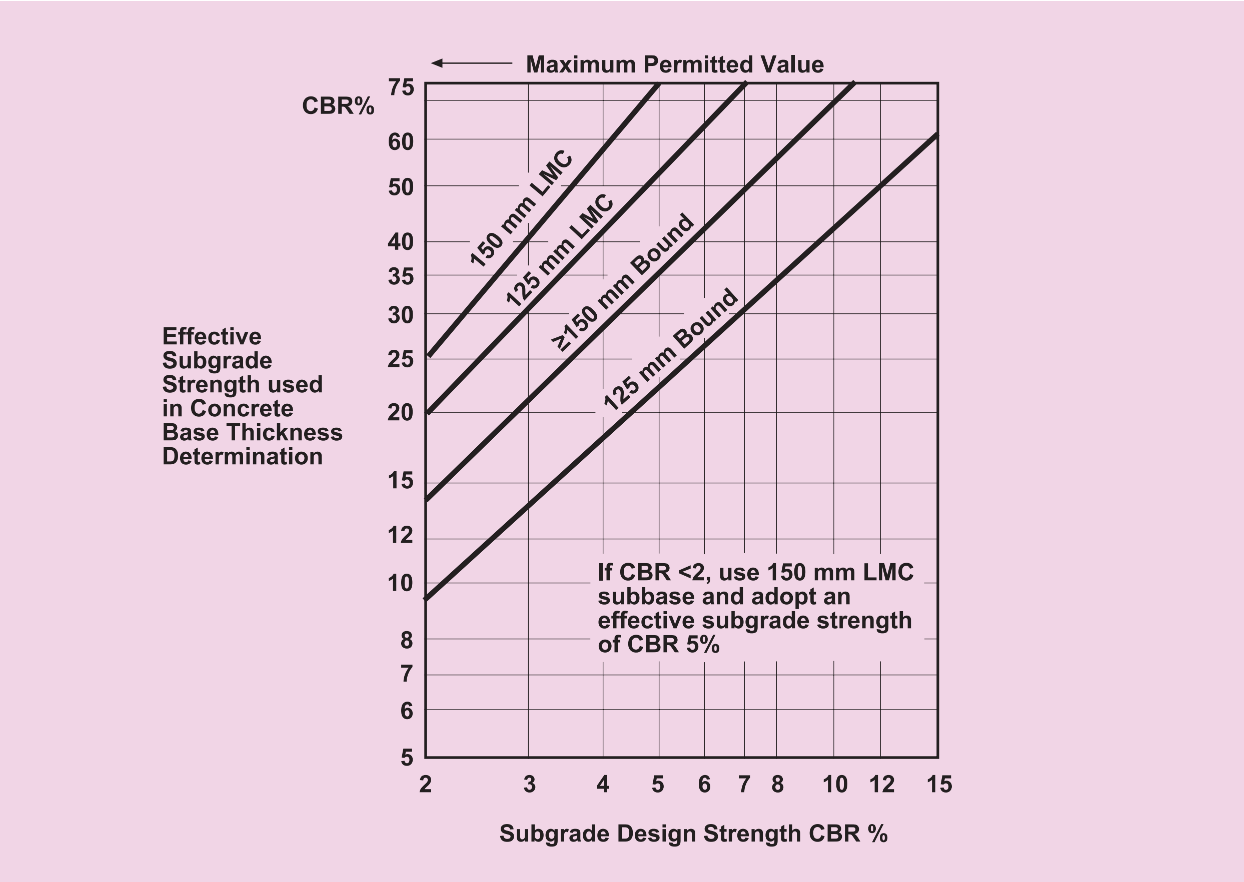


Figure 9.1 Effective increase in subgrade strength due to the provision of bound or lean-mix concrete subbase course (to be used for rigid pavement thickness design)

The following conditions apply to the use of this equation:

- Layers of thickness less than 200 mm must be combined with an adjacent layer. The lower CBR value must be adopted for the combined layer.
- It is assumed that higher CBR materials will be used in the upper layers. The formula is not applicable where weaker layers are located in the upper part of the subgrade.
- Filter layers must not be included in the calculation.
- The maximum CBR from the use of this formula is 20%.

9.3.3 Base concrete strength

The determination of concrete strength is discussed in Chapter 6 of the Guide. The flexural concrete strength used in the base thickness design procedure should reflect the time at which traffic is being applied to the road. Typically, the 28-day characteristic flexural strength (modulus of rupture) of the concrete is used as the design strength.

The minimum characteristic design concrete flexural strength for concrete pavements with a HVAG exceeding 1×10^6 is 4.5 MPa at 28 days.

Pavement construction techniques, such as fast-track paving, which allows traffic on the concrete within 24-hours of paving, is a viable option for new and rehabilitation pavement construction (FHWA, 1995; Grove, 1996).

9.3.4 Design traffic

The methods for estimating the design traffic loading for rigid pavement thickness design are included in Chapter 7.

Designers should be aware that rigid pavements are very sensitive to axle load magnitudes (such as overloads) but are relatively insensitive to axle load repetitions (i.e.

volumes). It is therefore recommended that a sensitivity analysis be undertaken for design traffic.

9.3.5 Concrete shoulders

Provision is made in the design procedure for the incorporation of concrete shoulders. Concrete shoulders enhance the pavement performance and enable a lesser base thickness to be adopted. For the purposes of this document, the concrete shoulder must be either integral or structural (both as defined) in order to satisfy the "with shoulder" criteria.

Integral concrete shoulders are made up of the same concrete and are the same thickness as the base pavement, and are cast integrally with the base pavement with a minimum width of 600 mm. The minimum width for integral cast shoulders in the median lane may be reduced to 500 mm.

A structural shoulder is a tied shoulder that is keyed by corrugating the joint and has a minimum width of 1.5 m, or is a 600 mm integral widening outside of the traffic lane (this may include integral channel or kerb/channel).

A tied concrete shoulder is made up of the same concrete and is the same thickness as the base pavement. It is formed, debonded and nominally tied to the base pavement.

Substantial kerbs (such as urban kerb and channel) can be considered to provide "with shoulder" support provided that:

- they are constructed of structural grade concrete of a strength consistent with the pavement; and
- they are effectively tied to the pavement and the joint has a corrugated face for load transfer.

Slipformed and fixed-form kerbs can satisfy these criteria, but extruded kerbs are not considered to comply.

9.3.6 Project reliability

In the design procedure, the axle group loads (Section 7.5) are multiplied by a load safety factor (LSF). The load safety factors used in the equations in Section 9.4.2 are derived from the values in *Table 9.2* according the desired project reliability (Section 2.2.1.2) for a specified pavement type.

Table 9.2 Load Safety Factors (LSF) for rigid pavement types

Pavement type	Project design reliability				
	80%	85%	90%	95%	97.5%
PCP	1.15	1.15	1.20	1.30	1.35
Dowelled & CRCP	1.05	1.05	1.10	1.20	1.25

The geometry of roundabouts usually dictates that traffic will travel through them at relatively low speeds. Where a lean-mix concrete subbase is provided, joint erosion is unlikely to be the controlling factor in their pavement life. Under these conditions, the thickness design for roundabouts is carried out only for the fatigue analysis. The load safety factors that should be adopted for rigid pavements for roundabouts in order to cater for radial/centripetal forces transmitted to the outer wheels is a value for a specific project reliability with the addition of 0.3.

9.4 Base thickness design

9.4.1 General

The procedure for the determination of the thickness of rigid pavements is based on the USA 1984 Portland Cement Association method (Packard 1984).

The two distress modes considered in this procedure are:

- flexural fatigue cracking of the pavement base; and
- subgrade/subbase erosion arising from repeated deflections at joints and planned cracks.

Account is taken of the presence or absence of dowelled joints and concrete shoulders. For design purposes, continuously reinforced pavements are treated as dowelled jointed pavements as studies and experience have found (Packard 1984) that the average stress is similar for these pavement types.

Information is required on both axle group types, and the distribution of each axle group type and the number of repetitions of each axle type/load combination expected to use the pavement during its design life.

The base thickness is calculated using the design procedure described in Section 9.4.2. The calculated base thickness should then be rounded up to the nearest 5 mm. The thickness so derived should be considered a minimum value and is referred to as the design base thickness.

In specifications which have a zero negative construction tolerance, the specified base thickness will typically be equal to the design base thickness.

In specifications where the lower bound value of the specified base thickness (after the application of construction tolerances) is less than the design base thickness, then consideration should be given to appropriately increasing the specified base thickness.

9.4.2 Base thickness design procedure

A trial base thickness is selected and the total fatigue and erosion damage is calculated for the entire traffic volume and composition during the design period. If either fatigue or erosion damage exceeds 100%, then the trial thickness is increased and the design process is repeated. The design thickness is the least trial thickness which has a total fatigue less than or equal to 100% per cent and a total erosion damage less than or equal to 100%.

The steps in the thickness design are detailed in *Table 9.3*.

The determination of the allowable axle load repetitions may be determined from the following equations (Austroads 2004):

9.4.2.1 Fatigue distress mode

Allowable load repetitions (N_f) for a given axle load is:

$$\log(N_f) = \left[\frac{0.9719 - S_r}{0.0828}\right] \quad \text{when } S_r > 0.55 \qquad (9.2)$$

$$N_f = \left[\frac{4.258}{S_r - 0.4325}\right]^{3.268} \quad \text{when } 0.45 \le S_r \le 0.55 \qquad (9.3)$$

where

$$S_r = \frac{S_e}{0.944\,f'_{cf}}\left[\frac{P.L_{SF}}{4.45\,F_1}\right]^{0.94}$$

S_e = equivalent stress (MPa);

f'_{cf} = design characteristic flexural strength at 28 days (MPa);

P = axle group load (kN);

L_{SF} = load safety factor;

F_1 = load adjustment for fatigue due to axle group:

= 9 for single axle with single wheel (referred to as SAST axle group)

= 18 for single axle with dual wheel (referred to as SADT axle group)

= 18 for tandem axle with single wheel (referred to as TAST axle group)

= 36 for tandem axle with dual wheel (referred to as TADT axle group)

= 54 for triaxle with dual wheel (referred to as TRDT axle group)

= 72 for quad axle with dual wheel (referred to as QADT axle group); and

N_f is infinite or commonly referred to as unlimited when S_r is less than 0.45.

The equivalent stress (S_e) is determined from equation 9.4 using the coefficients a to j in *Table 9.4*.

$$\begin{aligned} S_e \text{ or } F_3 &= a + b/T + c.\ln(E_f) + d/T^2 \\ &+ e.[\ln(E_f)]^2 + f.\ln(E_f)/T + g/T^3 + h.[\ln(E_f)]^3 \\ &+ i.[\ln(E_f)]^2/T + j.\ln(E_f)/T^2 \end{aligned} \qquad (9.4)$$

where

a, b, c, d, e, f, g, h, i, j are coefficients in *Tables 9.4* to *9.6*,

T = base thickness (mm), and

E_f = effective subgrade strength (refer Section 9.3.2)

9.4.2.2 Erosion Distress Mode

Allowable load repetitions (N_e) for a given axle load is:

$$\log(F_2\,N_e) = 14.52 - 6.77\left[\max\left(0, \left(\frac{PL_{SF}}{4.45\,F_4}\right)^2 \frac{10^{F_3}}{41.35} - 9.0\right)\right]^{0.103} \qquad (9.5)$$

where

P and L_{SF} are similar to previous definitions;

F_2 = adjustment for slab edge effects;

= 0.06 for bases without concrete shoulders

= 0.94 for bases with concrete shoulders;

F_3 = erosion factor; and

F_4 = load adjustment for erosion due to axle group:

= 9 for single axle with single wheel (referred to as SAST axle group)

= 18 for single axle with dual wheel (referred to as SADT axle group)

= 18 for tandem axle with single wheel (referred to as TAST axle group)

= 36 for tandem axle with dual wheel (referred to as TADT axle group)

= 54 for triaxle with dual wheel (referred to as TRDT axle group)

= 54 for quad axle with dual wheel (referred to as QADT axle group).

The erosion factor (F_3) is determined from equation 9.4 using the coefficients a to j in *Tables 9.5* and *9.6*.

There are no limits set for the axle load input and load safety factors to equations 9.2 and 9.4. However, designers should be cautious when the values of ($4.5PL_{SF}/F_1$) or ($4.5PL_{SF}/F_4$) exceed 65 kN.

Table 9.3 Design procedure for base thickness

Step	Activity	Reference
1	Select a rigid pavement type, either jointed undowelled, jointed dowelled or continuously reinforced concrete base.	9.2.1
2	Decide whether concrete shoulders are to be provided.	9.3.5
3	Using the design subgrade CBR and the predicted number of heavy vehicle axle groups over the design period, determine the subbase thickness and type from Table 9.1. Refer to the design subgrade CBR limit in Section 5.	9.2.2
4	Using the design subgrade CBR and the selected subbase, determine the Effective Subgrade Strength (CBR) from Figure 9.1.	9.3.1 and 9.3.2
5	Select the 28-day characteristic flexural strength of the concrete base f'_{cf}	6.6.3 9.3.3
6	Select the desired project reliability and hence the load safety factor.	2.2.1.2 9.3.6
7	Select a trial base thickness (appropriate trial base thickness may be governed by minimum base thickness from Table 9.7 or estimated from experience).	9.4.2
8	From the project Traffic Load Distribution (Section 7.5), obtain the highest axle load for the SAST axle group, and determine the allowable repetitions in the fatigue distress mode from equations 9.2 and 9.3.	9.4.2
9	Calculate the ratio of the expected fatigue repetitions (Section 7.7) to the allowable repetitions. Multiply by 100 to determine the percentage fatigue.	
10	Determine from equation 9.5 the allowable number of repetitions for erosion.	
11	Calculate the ratio of the expected erosion repetitions (Section 7.7) to the allowable repetitions. Multiply by 100 to determine the percentage erosion damage.	
12	Repeat steps 8 to 11 for each axle group load to a load level where the allowable load repetitions exceed 10^8, at which point further load repetitions are not deemed to contribute to pavement distress.	
13	Sum the percentage fatigue for all relevant loads of this axle group type; similarly, sum the percentage erosion for all relevant loads of this axle group type.	
14	Repeat steps 8 to 13 for each axle group type (i.e. SADT, TAST, TADT, TRDT and QADT).	
15	Sum the total fatigue and total erosion damage for all axle group types.	
16	Steps 7 to 15 inclusive are repeated until the least thickness that has a total fatigue less than or equal to 100% and also, a total erosion damage less than or equal to 100% is determined. This is the base concrete pavement design thickness.	
17	Ensure minimum base thickness requirement is met.	Table 9.7

Table 9.4 Coefficients for prediction of equivalent stresses

Coefficient	Without concrete shoulders				Concrete shoulders			
	Axle group type				Axle group type			
	SAST & TAST	SADT	TADT	TRDT & QADT	SAST & TAST	SADT	TADT	TRDT & QADT
a	0.118	0.560	0.219	0.089	–0.051	0.330	0.088	–0.145
b	125.4	184.4	399.6	336.4	26.0	206.5	301.5	258.6
c	–0.2396	–0.6663	–0.3742	–0.1340	0.0899	–0.4684	–0.1846	0.0080
d	26969	44405	–38	–10007	35774	28661	4418	1408
e	0.0896	0.2254	0.1680	0.0830	–0.0376	0.1650	0.0939	0.0312
f	0.19	19.75	–71.09	–83.14	14.57	2.82	–59.93	–61.25
g	–352174	–942585	681381	1215750	–861548	–686510	280297	488079
h	–0.0104	–0.0248	–0.0218	–0.0120	0.0031	–0.0186	–0.0128	–0.0058
i	–1.2536	–4.6657	3.6501	5.2724	1.3098	–1.9606	4.1791	4.7428
j	–1709	–4082	2003	4400	–4009	–2717	1768	2564

Table 9.5 Coefficients for prediction of erosion factors for undowelled bases

Coefficient	Without concrete shoulders				Concrete shoulders			
	Axle group type				Axle group type			
	SAST	SADT	TADT & TAST	TRDT & QADT	SAST	SADT	TADT & TAST	TRDT & QADT
a	0.745	1.330	1.907	2.034	0.345	0.914	1.564	2.104
b	533.8	537.5	448.3	440.3	534.6	539.8	404.1	245.4
c	–0.2071	–0.1929	–0.1749	–0.2776	–0.1711	–0.1416	–0.1226	–0.2473
d	–42419	–43035	–35827	–36194	–44908	–44900	–32024	–15007
e	0.0405	0.0365	0.0382	0.0673	0.0347	0.0275	0.0256	0.0469
f	27.27	26.44	0.64	15.77	20.49	16.37	–9.79	8.86
g	1547570	1586100	1291870	1315330	1676710	1654590	1150280	518916
h	–0.0044	–0.0039	–0.0060	–0.0084	–0.0038	–0.0032	–0.0052	–0.0075
i	–1.4656	–1.4547	1.0741	–1.2068	–1.3829	–0.9584	2.1997	1.5517
j	–1384	–1344	50	–625	–913	–765	469	–599

Table 9.6 Coefficients for prediction of erosion factors for dowelled or CRC bases

Coefficient	Without concrete shoulders				Concrete shoulders			
	Axle group type				Axle group type			
	SAST	SADT	TADT & TAST	TRDT & QADT	SAST	SADT	TADT & TAST	TRDT & QADT
a	0.072	0.643	1.410	2.089	–0.184	0.440	0.952	1.650
b	679.9	684.5	498.9	351.3	602.3	609.8	544.9	359.4
c	–0.0789	–0.0576	–0.1680	–0.3343	–0.0085	–0.0484	–0.0404	–0.1765
d	–58342	–58371	–39430	–25576	–50996	–52519	–47500	–28901
e	0.0179	0.0128	0.0322	0.0723	–0.0122	0.0017	0.0179	0.0435
f	6.70	4.61	13.80	29.58	8.99	9.62	–31.54	–15.97
g	2139330	2131390	1437580	923081	1874370	1949350	1719950	1085800
h	–0.0021	–0.0017	–0.0044	–0.0086	0.0008	–0.0007	–0.0051	–0.0084
i	–0.5199	–0.2056	–0.0380	–1.6301	–0.4759	–0.6314	3.3789	3.2908
j	–187	–185	–697	–1327	–374	–326	1675	758

Table 9.7 Minimum design base thickness

Pavement type (base)	Design traffic 1×10^6 < HVAG < 1×10^7	1×10^7 < HVAG < 5×10^7	HVAG $\geq 5 \times 10^7$
Plain concrete	150	200	250
Jointed reinforced and dowelled	150	180	230
Steel fibre reinforced concrete	125	180	230
Continuously reinforced concrete	150	180	230

9.4.3 Minimum base thickness

Irrespective of the base thicknesses determined in accordance with this procedure, the minimum allowable thickness of concrete base to be trafficked by heavy vehicles is noted in *Table 9.7*. The minimum values for design traffic exceeding or equal to 1×10^7 HVAG is to account for environmental factors, such as, warping and curling of slabs.

9.4.4 Example of the use of the design procedure

An example thickness design of a rigid pavement is given in *Appendix 9.1*.

9.4.5 Example design charts

Example design charts for rigid pavements are presented in *Figures 9.2* and *9.3*.

These charts are based on:

- presumptive Urban traffic load distribution (refer to Chapter 7, *Appendix 7.4*);
- the provision of concrete shoulders (refer to Section 9.3.5);
- design concrete flexural strength of 4.5 MPa; and
- load safety factors appropriate for desired project reliabilities of 80% (LSF = 1.05 for Dowelled & CRCP, LSF = 1.15 for PCP) and 95% (LSF = 1.2 for Dowelled & CRCP, LSF = 1.3 for PCP).

The charts allow designers to compare the design base thickness for different traffic volumes, effective subgrade strengths and load safety factors.

Table 9.8 lists the various configurations used to prepare the example charts. The effective subgrade strength values chosen in the design example charts reflect typical ranges used for new pavement designs. The design base thickness values shown in the example charts incorporate the minimum thickness limits detailed in *Table 9.7*.

Table 9.8 Design example charts for various traffic and pavement configurations

Base type	Effective subgrade strength (CBR)	Load Safety Factor	Chart
PCP	35% & 75%	1.15 & 1.3	EC01
Dowelled & CRCP	35% & 75%	1.05 & 1.2	EC02

9.4.6 Provision of dowels

The thickness design procedure provides for the option of dowelled or undowelled contraction joints. Dowel bars are to be plain round steel bars of Grade 250N and 450 mm long and placed at 300 mm centres. Dowels should be straight with the 'expansion/contraction end' free from burrs. Appropriate dowel diameters are given in *Table 9.9*.

More than half of the dowel should be coated with a debonding agent to ensure effective debonding from the concrete on that side of the joint. Dowels must be accurately aligned otherwise joint locking is likely to occur.

Table 9.9 Minimum dowel bar diameters for concrete pavements

Design base thickness (h) (mm)	Dowel diameter* (mm)
125 <h ≤ 160	24
160 <h ≤ 200	28
200 <h ≤ 250	32
> 250	36

* AS/NZ 4671.

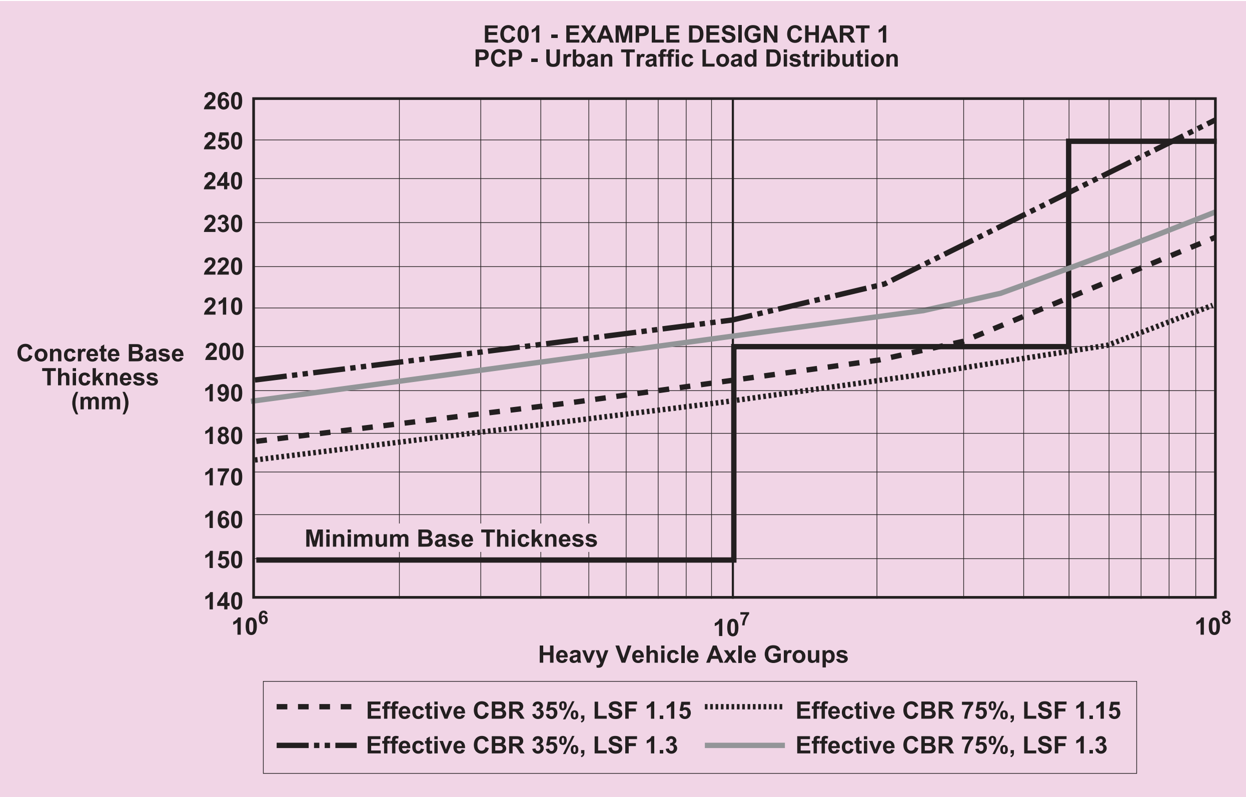


Figure 9.2 PCP example design charts

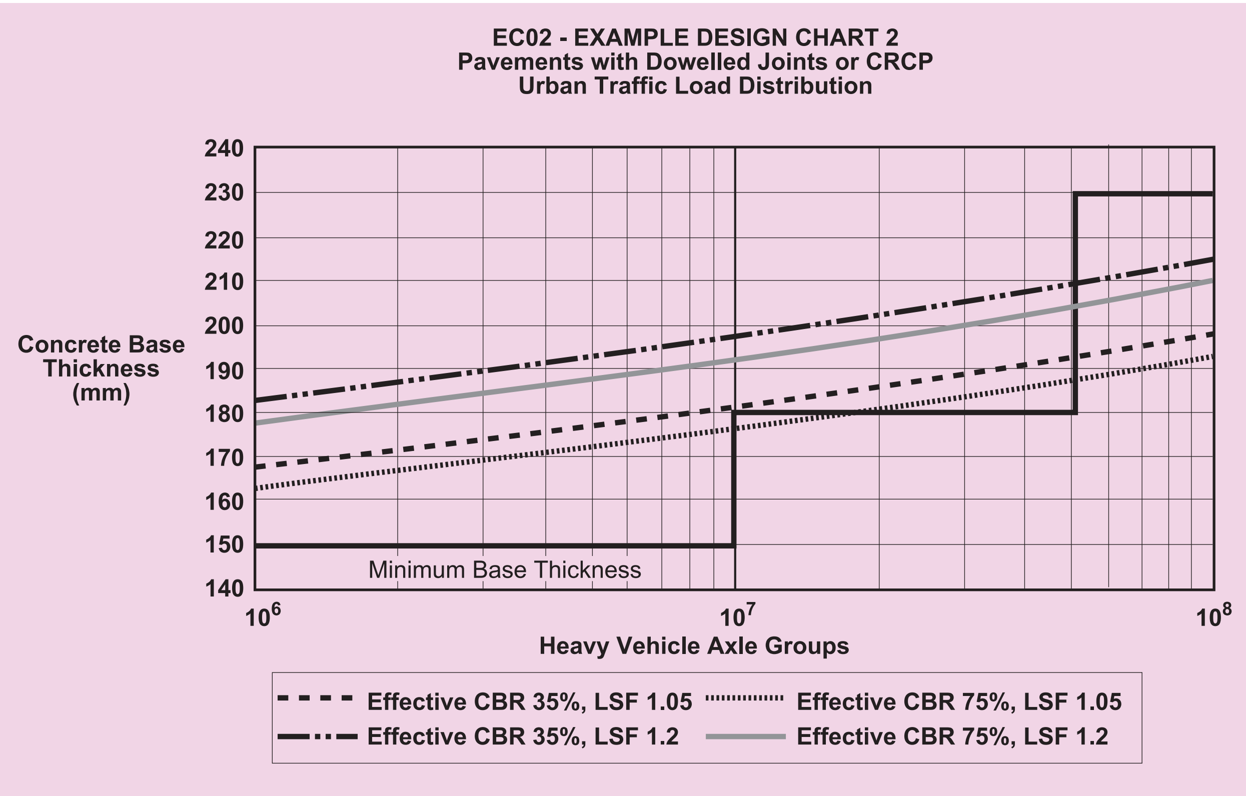


Figure 9.3: Dowelled jointed or CRCP example design charts

9.4.7 Provision of tie bars

Tie bars prevent separation of the pavement at longitudinal joints, whilst allowing warping or curling to occur without excessive restraint. Their design and long-term integrity is of utmost importance because their failure would create a “without-shoulder” loading condition which could reduce the pavement design life by up to 50%.

Ties bars are typically 12 mm in diameter, Grade 500N deformed steel bars, 1 m long, placed centrally in the joint. The spacing is determined in accordance with the subgrade drag theory (see eqn 9.6) and is influenced by parameters such as base thickness, interlayer friction, and distance to the nearest free edge of pavement.

In joints between CRC and jointed bases (such as at shoulders and ramp junctions), it is desirable to cluster tie bars to cater for the different longitudinal contractions in each base type.

In selecting the design friction coefficient, factors such as the subbase type (and its surface texture) and the interlayer debonding treatment need to be assessed.

It is beyond the scope of this Guide to provide a full design method for tie bars, and designers are referred to RTA, NSW (1991 and 1998) for further information.

9.5 Reinforcement design procedures

9.5.1 General

The purpose of reinforcing steel in rigid pavements is not to prevent cracking of the concrete, but to hold tightly closed any cracks that do occur in such manner that the load carrying capacity of the base is preserved.

In jointed pavements (JRCP) the amount of steel is governed by the spacing of contraction joints. In the case of continuously reinforced concrete pavements, sufficient steel is provided to eliminate the need for contraction joints by inducing fine transverse cracks at random spacings of 0.5 to 2.5 m.

9.5.2 Reinforcement in jointed unreinforced pavements

In jointed unreinforced pavements, reinforcement (usually in the form of welded wire fabric) is sometimes necessary to control cracking. Concrete slabs which are reinforced are those in which it is anticipated that cracks could occur due to stress concentrations which cannot be avoided by re-arrangement of the slab jointing pattern. Typical applications are:

- odd-shaped slabs;
- mismatched joints; and
- slabs containing pits or structures.

Guidance on the detailing of reinforcement is provided in the RTA NSW’s *Concrete Pavement Manual* (RTA, NSW 1991).

9.5.3 Reinforcement in jointed reinforced pavements

The required area of reinforcing steel in jointed reinforced pavements is calculated according to the subgrade drag theory using the equation:

$$A_s = \frac{\mu L \rho g D}{f_s} \tag{9.6}$$

where

A_S = the required area of steel (mm^2/m width of slab);

f_s = the allowable tensile stress of the reinforcing steel (MPa). Usually 0.6 times the characteristic yield strength (f_{sy}) from AS/NZS 4671 (that is $0.6f_{sy}$);

g = acceleration due to gravity (m/s^2);

D = thickness of the base (m);

L = distance to untied joints or edges of the base (m);

ρ = mass per unit volume of the base (kg/m^3); and

μ = coefficient of friction between the concrete base and the subbase. *Table 9.10* provides indicative values.

Experience has shown that the use of slab lengths between 8 and 12 metres provide an optimum balance of joint performance, costs and ride quality.

The use of steel fibre reinforced concrete is appropriate where increased flexural strength is required to control cracking in odd-shaped slabs and where increased abrasion resistance is required for durability. This type of pavement is often used for toll plazas, roundabouts and bus-stops. Steel fibres are typically between 15 mm and 50 mm in length with either enlarged ends that act as anchorages and/or crimping to improve bond.

Typically, fibres are added to the concrete at a rate of approximately 45 to 75 kg/m^3.

9.5.4 Reinforcement in continuously reinforced concrete pavements

9.5.4.1 *Longitudinal reinforcement*

The action of the longitudinal steel reinforcement is initially to induce transverse cracking (by providing restraint to shrinkage of the concrete) and finally to tie the planned cracks together.

Under the influence of thermal and drying contraction, combined with the restraint imposed by the reinforcement, tension builds up in the concrete until cracking occurs, after which local tension results in the steel and limits the opening of the crack. This tension is balanced by compression in the steel between cracks, until further cracking develops. Due to the stresses in the steel changing so rapidly, adequate bond strength between steel and concrete is essential.

The proportion of the cross sectional area of the pavement which is to be longitudinal reinforcing steel in CRCP is given by the equation:

$$p = \frac{(f'_t / f'_b)\, d_b\, (\varepsilon_s + \varepsilon_t)}{2W} \qquad (9.7)$$

where

p = required proportion of longitudinal reinforcing steel – this is the ratio of the cross-sectional area of the reinforcing steel to the gross area of the cross-section of the base;

f'_t/f'_b = the ratio of the direct tensile strength of the immature concrete to the average bond strength between the concrete and steel. The value of this ratio may be assumed to be 1.0 for plain bars or 0.5 for deformed bars complying with AS/NZS 4671, 2001;

d_b = diameter of longitudinal reinforcing bar (mm);

ε_s = estimated shrinkage strain – the shrinkage strain may be considered to be in the range 200 to 300 $\mu\varepsilon$ for a concrete with a laboratory shrinkage not exceeding 450 $\mu\varepsilon$ at 21 days when tested in accordance with AS 1012.13 after three weeks air drying (Standards Australia 1992);

Table 9.10 Estimated values for coefficient of friction

Subbase type	Base type	Recommended treatments: Lean-mix concrete curing	Recommended treatments: Debonding treatment	Estimated friction coefficient†‡
Lean-mix concrete	PCP & CRCP	wax emulsion	bitumen sprayed seal with 5–7 mm aggregate	1.5
	JRCP	(i) wax emulsion, or (ii) hydrocarbon resin	bitumen sprayed seal with 5–7 mm aggregate, or bitumen emulsion	(i) 1.5 (ii) 2.0
	SFCP	wax emulsion	wax emulsion	1.7
RLC & CSCR*	All	bitumen sprayed seal with 5–7 mm aggregate		2.5
Dense-graded asphalt	All	Note #		2.5–3.0#

* RLC = rolled lean concrete, CSCR = cement stabilised crushed rock.

† Friction values will vary depending on factors such as the surface smoothness of the lean mix concrete subbase, and the amount of residual curing compound present at the time of the debonding treatment. To guard against under-design of tiebars and other reinforcement, conservative (ie high) friction values have been adopted.

‡ The table is to be interpreted as follows. For JRCP, for example, wax emulsion curing followed by either of the debonding treatments can be assumed to yield a friction value of 1.5.

\# Friction values for asphalt could vary widely depending on factors such as age, stiffness and surface texture. An aged, stiff asphalt with an open-textured surface could yield a high friction level. By contrast, a new (and relatively flexible) asphalt is likely to have a lower effective friction level. See RTA, NSW (1991) for further information.

ε_t = estimated maximum thermal strain from the peak hydration temperature to the lowest likely seasonal temperature – a value of 300 με may be assumed, except when the average diurnal temperature at the time of placing concrete is 10°C or less, when a value of 200 με may be assumed (Standards Australia 1991); and

W = maximum allowable crack width (mm) – a value of 0.3 mm should be used in normal conditions, with 0.2 mm for severe exposure situations, such as adjacent to marine environments.

Equation 9.7 indicates that the proportion of steel is inversely proportional to the bond strength. In order to provide adequate bond capacity, the longitudinal reinforcing steel should be detailed as follows:

- Deformed bars should be used.
- The diameter of the bars should preferably be 16 mm and in any case not exceed 20 mm.
- The centre-to-centre spacing of the bars should not be greater than 225 mm.

For deformed bars, equation 9.7 may be simplified as:

$$p = \frac{0.25\, d_b\, (\varepsilon_s + \varepsilon_t)}{W} \tag{9.8}$$

To ensure against yielding of the steel, the actual steel reinforcement ratio should exceed the critical value given by the following equation:

$$p_{crit} = \frac{f_{ct}\,(1.3 - 0.2\mu)}{f_{sy} - m\, f_{ct}} \tag{9.9}$$

where

p_{crit} = minimum proportion of longitudinal reinforcement to match the specified (or target) concrete strength;

f_{ct} = concrete tensile strength (MPa) – a value equal to 60% of the 28-day specified concrete flexural strength (f_{ct}) may be assumed;

μ = coefficient of friction between the concrete base and the subbase – *Table 9.10* provides indicative values;

f_{sy} = the characteristic yield strength of the longitudinal reinforcing steel (AS/NZS 4671);

m = ratio of the elastic moduli of steel to concrete, (E_s/E_c) – a value of 7.5 may be assumed.

Equation 9.9 indicates that the critical proportion of longitudinal reinforcing steel increases more rapidly than the tensile strength of the concrete. The minimum proportion of longitudinal steel to be provided is 0.65%.

In the design of continuously reinforced pavements, it is important that an optimum amount of longitudinal steel of suitable type is provided so that crack spacing and crack width can be controlled.

If the spacing of the cracks is too wide, the cracks themselves will become wide with a consequent loss in aggregate interlock load-transfer and accelerated corrosion of the steel. If the spacing between cracks is too small, disintegration of the slab may commence. The function of the longitudinal steel is to keep the cracks in the concrete tightly closed, thereby ensuring load transfer across the cracks and also preventing the ingress of water and grit into the cracks.

The *theoretical* spacing of cracks in continuously reinforced pavements may be estimated by the equation:

$$L_{cr} = \frac{f_{ct}^{\,2}}{m\, p^2\, u\, f_b\, [(\varepsilon_s + \varepsilon_t)\, E_c - f_{ct}]} \tag{9.10}$$

where

L_{cr} = theoretical spacing between cracks (m);

f_{ct} = tensile strength of the concrete (MPa);

m = ratio of the elastic moduli of steel to concrete E_s/E_c – a value of 7.5 may be assumed;

p = area of longitudinal steel per unit area of concrete (i.e. steel proportion);

u = perimeter of bar per unit area of steel which may be simplified to 2 divided by radius of the bar (m^{-1});

f_b = bond stress (MPa) – for mature concrete, and when deformed bars are used this may be assumed as 2 f_{ct};

ε_s = estimated shrinkage strain – the shrinkage strain may be considered to be in the range 200 to 300 με for a concrete with a laboratory shrinkage not exceeding 450 με microstrain at 21 days when tested in accordance with AS 1012 Part 13 (after three weeks air drying);

ε_t = estimated maximum thermal strain from the peak hydration temperature to the lowest likely seasonal temperature – a value of 300 $\mu\varepsilon$ may be assumed, except when the average diurnal temperature at the time of placing concrete is 10°C or less, when a value of 200 $\mu\varepsilon$ may be assumed; and

E_c = modulus of elasticity of concrete (MPa).

This equation indicates that the spacing of cracks is inversely proportional to p, u and f_b; consequently, to ensure fine cracks and optimum crack spacings, the percentage reinforcement and perimeter to area relationship of the bars should be high. A closer spacing of cracks is also obtained when the bond stresses are high, therefore the use of deformed bars is preferred.

Experience with continuously reinforced pavements indicates that the optimum crack spacing is between 0.5 and 2.5 metres.

9.5.4.2 Transverse Reinforcement

The required area of transverse reinforcing steel (A_s) in continuously reinforced pavements is consistent with that provided in jointed pavements and is calculated using equation 9.6.

9.6 Base anchors

Base anchors are required to minimise the pavement from "creeping", thereby minimising interference with adjacent structures and flexible pavements. As a guide, base anchors should be constructed below the base pavement in the following situations:

1. Jointed Concrete Pavements:

   A single anchor is provided at all terminal ends (i.e. at bridge abutments and at flexible pavements). Additionally, on grades exceeding about 4%, a single intermediate anchor is provided at spacings of about 300 m to arrest downhill creep.

2. Continuously Reinforced Concrete Pavements:

   A set of three anchors is provided adjacent to all terminal ends (i.e. at bridge abutments and flexible pavements). Intermediate anchors are not used in CRC pavements.

Further details on anchors can be found in sources, such as the RTA, NSW (1991) Concrete Pavement Manual.

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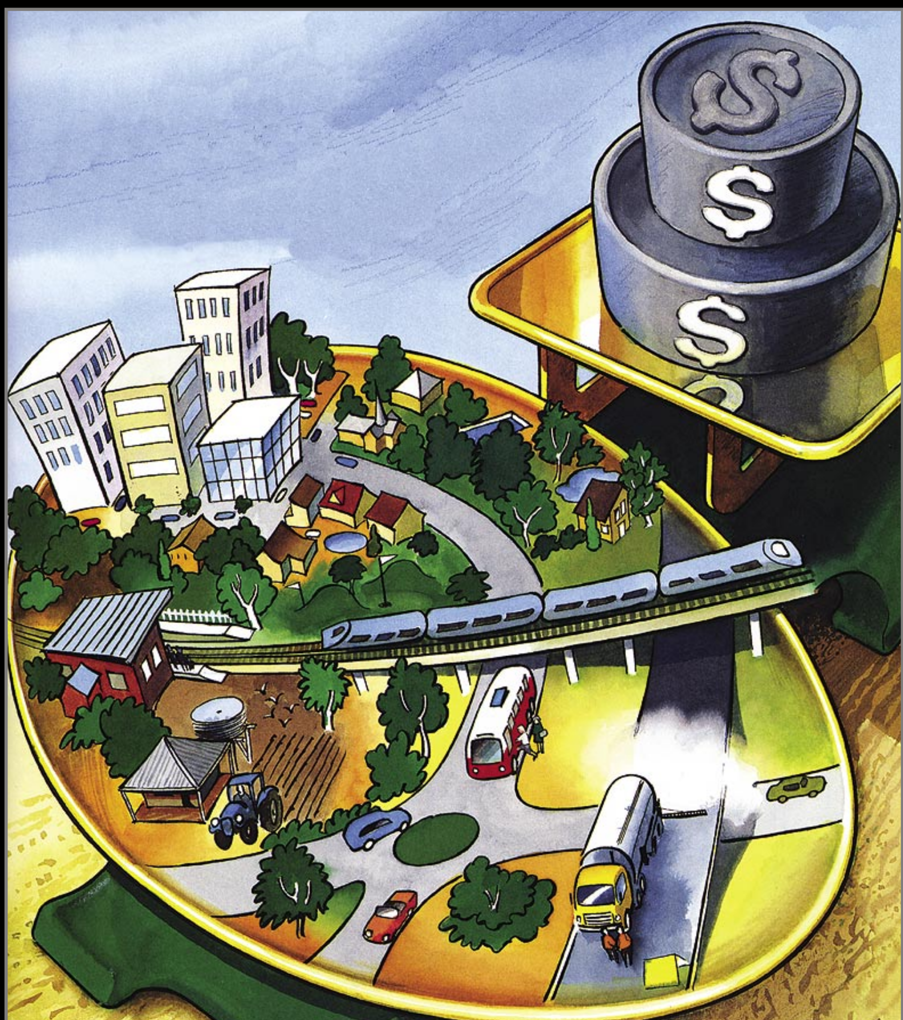
10

COMPARISON OF DESIGNS



10 COMPARISON OF DESIGNS

10.1 General

In comparing various alternative pavement types and configurations, cost is a prime consideration. This Guide provides the means for designing a range of feasible pavements for a given set of design parameters. To determine the most economical pavement, a cost comparison must be made.

Alternative projects should be evaluated primarily according to the criterion of minimum total (whole-of-life) cost, giving consideration also to the safety and service of road users and others that may be affected by the construction. In many cases, designers do not have information to reliably consider future maintenance and strengthening costs of various alternatives. However, these details are becoming available (e.g. VicRoads 1993; Porter and Tinni 1993; Bennett and Moffatt 1995). Road user costs are usually excluded from the analysis partly because of lack of reliable information but mainly because they are essentially similar for alternatives, provided minimum levels of serviceability are maintained. However, the exclusion of road user costs needs to be carefully considered, particularly for projects carrying high traffic volumes, as traffic disruption costs caused by maintenance activities can incur significant road user costs.

The purpose, in an economic comparison, is to evaluate alternative designs primarily according to the criterion of minimum total (whole-of-life) cost, giving due consideration also to the safety and service provided to road users and others who may be affected by the road or its construction. In particular, other criteria which may need to be considered are:

- the potential for differential settlement over the road alignment;
- the scale of the project;
- the requirement to construct under traffic;
- noise and spray effects; and
- maintenance requirements.

Despite the apparent simplicity of the models, designers are advised to consider carefully the results of any economic comparison of design alternatives and not to rely on it as the sole determinant of the most appropriate option.

10.2 Method for economic comparison

There are several methods for economic comparison of alternative designs. The "present worth" method is given here as it effectively allows for both uniform series and sporadic events (e.g. routine and periodic maintenance) which will occur during the service life of the pavement. With the present worth method, all costs are converted into capital sums of money which, invested now for an analysis period, would provide the sums necessary for construction of a project and subsequent maintenance during that period.

The present worth of costs (PWOC) can be calculated as follows:

$$\mathrm{PWOC} = C + \sum_i M_i\,(1+r)^{-x_i} - S(1+r)^{-z} \qquad (10.1)$$

where

PWOC	=	present worth of costs;
C	=	present cost of initial construction;
M_i	=	cost of the i^{th} maintenance and/or rehabilitation measure;
r	=	real discount rate;
x_i	=	number of years from the present to the i^{th} maintenance and/or rehabilitation measure, within the analysis period;
z	=	analysis period; and
S	=	salvage value of pavement at the end of the analysis period expressed in terms of present values.

In estimating present worth the principal elements are:

- construction costs;
- maintenance and rehabilitation costs, including routine periodic maintenance and structural rehabilitation;

- salvage value of the pavement at the end of the analysis period;
- real discount rate; and
- analysis period.

Two factors not directly accounted for in the model but which have an influence on the comparative costs are the growth in traffic over the analysis period and the availability of funds, which influences the duration of the analysis period. It is important that designers run the model on a range of traffic growths and investment periods to gauge its sensitivity to these factors.

It is important to recognise that the process of deflating future costs assumes that funds will be available over the analysis period at a level consistent with the adopted discount rate. As this may not occur, the sensitivity of the model to a range of discount rates should also be investigated.

The economic model also may not account for substantial differences in the social, political or environment impacts of future maintenance and rehabilitation activities associated with designs. For instance, one alternative may require reconstruction involving a total road closure at the end of the analysis period whereas another may be able to be rehabilitated under traffic but on a more frequent basis. Clearly both would involve some level of social disruption for the community that may not be properly reflected in the economic comparison.

10.3 Construction costs

Unit costs for alternative pavement designs will vary widely depending on locality, the availability of suitable natural and processed materials, scale of project and material standards. These may be assessed from experience with particular projects. There are, however, several other less obvious costs that warrant consideration.

For example, some alternatives will require more excavation or more fill (e.g. where surface levels are fixed by extraneous restraints), they may interfere with utility services, or require more shoulder material than others. Significant saving in shoulder material can be made, for equivalent performance, for example, by using a full-depth asphalt or cemented pavement, or an asphalt or cemented base pavement which is thinner than an unbound granular pavement. Consequently, comparing the cost per square metre of the pavement alone is often misleading. Comparative costs should be expressed as a cost per kilometre for the full pavement and shoulder cross-section or as a total cost per project (including all overheads).

Overheads, and other non-productive costs, which are not necessarily included in the pavement unit costs, may vary with the type of pavement used.

Such costs include:

- **Provision for traffic**: Alternatives which take longer to build usually incur higher traffic control costs. Some alternatives can be built under traffic while others may require traffic diversions.
- **Cost of time due to wet weather and the need to dry and rework material**: In wet climates these costs can be very significant and often unpredictable. Alternatives using bound material tend to be quicker to build and avoid the cost of reworking.
- **Establishment costs**: These include costs associated with the setting up and transport of plant, etc. These costs vary between alternatives.
- **Supervision costs**: Supervision costs are time-dependent: alternatives which are slower to construct tend to cost more to supervise.

There are significant economies of scale in some paving operations for larger projects, particularly for supply and laying of materials. In many cases, designers may not have sufficient information to make an accurate comparison of the construction costs of various structurally equivalent alternatives and, in such cases, it may be desirable to call alternative tenders.

10.4 Maintenance costs

The nature and extent of future maintenance is dependent on pavement type. For example, routine maintenance costs of rigid pavements are generally less than those of unbound flexible pavements because they are limited to joint repairs and minor structural repairs rather than resurfacing. In conducting cost comparisons based on present worth analyses, an assessment must be made of future annual routine maintenance requirements, periodic maintenance treatments such as resurfacing, and rehabilitation such as structural overlays or strengthening. In many cases, however, designers do not have information to reliably consider future maintenance and strengthening costs of various alternatives although these details are becoming available (e.g. VicRoads 1993; Porter and Tinni 1993; Bennett and Moffatt 1995, Austroads 2003 & 2004).

Road user costs are usually excluded from the analysis partly because of lack of reliable information but often because they are essentially similar for each of the alternatives, provided minimum levels of serviceability are maintained. Nonetheless, road user costs can have a significant impact on the selection of the optimum design where there are differences in the level and frequency of maintenance activities, in the duration of construction delays or in the levels of traffic safety, noise or ride quality all of which increase road user costs. As such, the exclusion of road user costs needs to be carefully considered, particularly for projects carrying high traffic volumes, as traffic disruption costs caused by maintenance activities can incur significant road user costs. If the duration of maintenance activities is different for each of the alternatives, road user costs should be included.

To reduce whole-of-life maintenance costs some road agencies ensure that, as far as possible, any deep-seated pavement failure modes, such as fatigue failure of cemented subbases, are minimised in the design phase. This ensures that any long-term periodic maintenance and rehabilitation treatments are limited to surface-based treatments which minimise traffic disruption and maintenance costs.

10.5 Salvage value

The salvage value of the pavement at the end of the analysis period is difficult to assess and is dependent on several factors, including the:

- continued use of an existing alignment;
- feasibility of upgrading or strengthening a pavement with an overlay;
- possibility of recycling existing paving materials, either in plant or insitu; and
- need to remove the pavement before reconstruction.

For some of these options there will be significant road user impacts and costs which will need to be accounted for in the economic modelling.

In cases where the existing pavement must be removed, the salvage value is negative.

While the salvage value is an item, which needs to be discounted to the evaluation year like all other items, for inclusion in the *PWOC* (Section 10.2), careful consideration should be given to the discount rate used.

Despite the challenges, some guidance can be given as to how to evaluate salvage value :

- For a project where the predicted condition of the pavement at the end of the analysis period is such that the base layer could serve as the subbase layer for the subsequent next project, then the salvage value is equal to the cost in current dollars (say year 2004) for construction, in say, year 2030 of a pavement to subbase level (less any costs for tidying up the works, scarification, compaction, drainage renovation etc.), discounted to the evaluation year (2004).
- Similarly, for a project where the predicted condition of the pavement at the end of the analysis period is such that the base layer could serve as the subgrade layer for the subsequent next project, then the salvage value is equal to the cost in current dollars (say year 2004) for construction, in say, year 2030 of a pavement to subgrade formation (less any costs for tidying up the works, scarification, compaction, drainage renovation etc.), discounted back to the evaluation year (say year 2004).
- However, for a project in which the pavement still has significant residual load carrying capacity at the end of the analysis period, it is necessary to pro-rata the value of the base layer only in proportion to the residual life as a proportion of the original life when new. In the calculation of the remaining salvage value of the rest of the pavement (subbase, subgrade, earthworks, drainage etc. this is added to the residual value of the base layer.
- For an analyses which takes account of road user costs in the evaluation, then the estimated road user costs beyond the end of the analysis period, which would have accrued if the pavement had served its full term, also needs to be discounted back as a credit to the overall road user costs.

10.6 Real discount rate

The real discount rate must be selected to express future expenditure in terms of present values and costs. Different discount rates may be used for specific commodities or processes if it is anticipated that such materials will be relatively more expensive in the future. However, most expenditure can be related with a uniform discount rate. For public sector project analysis in Australia, Austroads (1996) recommends that a discount rate of 7% is applicable for Austroads or national work. This rate is expressed in real terms, i.e. it excludes inflation. However, other jurisdictions may require the use of other rates for other purposes. In most cases, it is desirable to carry out a sensitivity analysis with the discount rate varying between 4 and 10%.

10.7 Analysis period

The analysis period is the length of time for which comparisons of total cost are to be made. It should be the same for all alternative strategies and should not be less than the longest design period of the alternative strategies.

It should be noted, however, that the duration of the analysis period is based on budgetary and financial considerations. Lengthening design lives and analysis periods may result in fewer more expensive projects which may not meet the investment approach or service delivery expectations of the road owner.

10.8 Road user costs

The road user costs for routine operations may be excluded from the analysis, as they are essentially similar for pavement alternatives, provided minimum levels of serviceability are maintained.

However, for alternatives involving frequent maintenance activities, the road user costs associated with delays and diversions may be significant on roads with high traffic volumes. Methods of analysis to assess road user costs may be found in Thoresen and Roper (1996) and Austroads (1999).

10.9 Surfacing service lives

Guidance on the range of expected service lives of surfacing can be found in *Austroads Guide to the Selection of Road Surfacings* (2003). Service conditions that affect the expected life include:

- traffic volume – high traffic volumes will tend to give a service life near the low end of the range, whereas lesser traffic volumes will result in longer service life; and
- climate – high service temperatures generally reduce service life. High rainfall may also reduce service life.

The economic modelling of the alternative treatments should consider a number of service lives, within the range given in the *Austroads Guide to the Selection of Road Surfacings* (2003) for each of the competing alternatives.

References

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11

IMPLEMENTATION OF DESIGN AND COLLECTION OF FEEDBACK

11 IMPLEMENTATION OF DESIGN AND COLLECTION OF FEEDBACK

11.1 Implementation of design

Once the final design has been selected and pavement construction is undertaken, it is necessary to ensure that design assumptions, such as layer thickness and material properties, are incorporated into the pavement construction.

Particular attention needs to be paid to variations in layer thickness, which can lead to large variations in performance. Reference to the example design charts will reveal the sensitivity of design life to pavement thickness.

Suitable construction tolerances need to be incorporated into specifications to ensure layer design thicknesses are achieved during construction. If possible, the dominant failure mode determined from the design process should be communicated to construction personnel to ensure that this receives adequate attention during the specification and construction phases.

If the design assumptions cannot be met due to some unforeseen constraints, then the pavements, as constructed, need to be analysed to ensure that adequate performance will still be achieved.

11.2 Collection of feedback

11.2.1 Need

The availability of well-documented, long-term pavement performance (LTPP) data is critical if existing design procedures – and associated laboratory testing protocols – are to be validated and refined. Historically, there is a paucity of well-documented data regarding the in-service performance of pavements in Australia over the past 30 years.

This situation is probably due to a number of factors including the following:

- Design and construction – and maintenance activities – are often carried out by different organisations.
- The generally relatively long period of time between construction and the onset of deterioration poses problems in terms of retention of records, the loss of key personnel and the commitment to the continuation of pavement assessment. This can result, for example, in the rehabilitation of sections which were being monitored before the final set of performance data is collected.
- A lack of understanding of failure mechanisms and terminal pavement condition.
- The lack of a perceived need to monitor pavement performance on a systematic basis.

This problem has recently been addressed with the establishment of an Austroads-sponsored project (see *Section 11.2.3*).

11.2.2 Benefits

There are a number of significant long-term benefits associated with the systematic collection of pavement design, construction and performance data – both structural (strength) and functional (roughness, rutting, cracking) data. These include the following:

- The validation and refinement of current design procedures, both through the monitoring of the sections themselves and – as an adjunct – the comparison of the in-service performance with that obtained through accelerated loading trials.
- The future analysis, on a statistical basis, of different pavement types, mix design procedures and construction processes.
- The provision of background data which can be used as input into the establishment of future rehabilitation needs.
- The evolution of local data on seasonal variations and the performance of specific materials and construction practices.
- The provision of data which can be used to validate, and enhance, pavement management systems and performance models.
- The provision of life cycle costs data, and the optimisation of them, in the longer term.

11.2.3 Current Australian LTPP program

In order to take advantage of the opportunity to be directly involved in the US Strategic Highway Research Program (SHRP), an Austroads-funded project has been established which has, as its primary aim, the monitoring of the performance of a range of Australian test sections in order that both the quantity and quality of pavement performance data could be enhanced and prediction models improved.

The overall objectives of the LTPP study are to:

- enhance asset management strategies through the use of improved pavement performance models based on an improved understanding of the behaviour of pavement structures (SHRP-LTPP program); and
- compare the results of accelerated pavement testing studies with actual road pavement performance (ALF-LTPP program).

Data analysis conducted to date has produced significant findings and clearly demonstrated the need to continue this long-term study (Clayton 2000*a*).

11.2.4 Data collection

Data should be comprehensive, but specific, easily accessible and sites readily found in the field. Generally, design and construction data are not retained for the design life of the pavement and so the establishment of a separate database for design and construction data is well warranted. Ideally, a site should be established near a weigh-in-motion (e.g. CULWAY) site to assist in the collection of high quality loading data.

Data collected should include:

- pavement geometry;
- pavement materials;
- pavement composition;
- construction costs and condition at construction;
- periodic condition measurement (say every one to five years);
- maintenance conducted, including costs; and
- traffic (heavy vehicle) volumes and load.

The number of sites monitored – and the level of sophistication of the monitoring – should be in balance with the resources available; it is generally preferable to properly maintain and monitor a small number of sites rather than to attempt to monitor a large number of sites. Clayton (2000*b*) has produced guidelines for the selection of new LTPP sites, recommendations for uniform and consistent data collection, and recommendations for systematic and readily accessible information management.

References

Clayton, B.M. (2000*a*). *Benefits of the LTPP Studies to Date*. ARRB TR Contract Report, RC 90256-2, June.

Clayton, B.M. (2000*b*). *Guidelines for Site Establishment and Data Collection for New Long Term Pavement Performance Sites*. ARRB TR Contract Report RC 90256-1, April.

APPENDICES

APPENDIX 1.1 GLOSSARY OF TERMS

accelerated loading facility (ALF)

A mobile loading frame which can apply power-driven, rolling wheel loads to sections of pavement in a manner which simulates the characteristics of traffic and which is used to study the response of the pavement to this loading.

additive

A substance added in small amounts for the purpose of aiding in the manufacture or handling of a product, or modifying the end properties.

adhesion

The action by means of which a fluid or plastic substance (e.g. bituminous binder) sticks to the surface of a solid body (e.g. aggregate).

aggregate

A material composed of discrete mineral particles of specified size or size distribution, produced from sand, gravel, rock or metallurgical slag, using one or more of the following processes: selective extraction, screening, blasting or crushing.

air voids

The spaces within the bulk material not occupied by solid matter.

anisotropic

Having properties that vary in different directions.

annual average daily traffic (AADT)

The total yearly two-way traffic volume divided by 365, expressed as vehicles per day.

asphalt

A mixture of bituminous binder and aggregate with or without mineral filler produced hot in a mixing plant which is delivered, spread and compacted while hot.

asphalt pavement

A pavement, the predominate structural strength of which is provided by asphalt layers.

atterberg limit(s)

A set of arbitrarily defined boundary conditions in soils related to water content. The limits are as follows:

Liquid limit: the moisture content at which a soil passes from the liquid state to the plastic state, as determined by a specific test procedure.

Plastic limit: the moisture content at which a soil passes from the plastic state to the semi-solid state, as determined by a specific test procedure.

Shrinkage limit: the moisture content from which a soil will continue to dry out without further change in volume.

Austroads

National Association of Road Transport and Traffic Authorities in Australasia. Previously known as NAASRA.

axle group

A set of closely spaced axles acting as a unit.

axle load

That portion of the total vehicle load transmitted to the road through a single axle.

base/basecourse

One or more layers of material usually constituting the uppermost structural element of a pavement and on which the surfacing may be placed, which may be composed of fine crushed rock, natural gravel, broken stone, stabilised material, asphalt or Portland cement concrete.

benefit/cost ratio (BCR)

A ratio that compares the benefits, accruing to road users and the wider community from constructing a project, with the cost of that project.

Benkelman beam

An instrument for measuring the deflection of the surface of a pavement caused by the passage of a dual-tyred single axle carrying a standard axle load.

binder

A material used for the purpose of holding solid particles together as a coherent mass.

bitumen

A very viscous liquid or a solid, consisting essentially of hydrocarbons and their derivatives, which is soluble in carbon disulphide, substantially non-volatile and softens gradually when heated, black or brown in colour and possesses waterproofing and adhesive properties, and obtained from native asphalt or by refinery processes from petroleum.

bound material

Granular material to which a binder of lime, cement, bitumen or similar additive, is added to improve the structural stiffness.

bound pavements

Pavements composed of materials incorporating sufficient amounts of chemical agent(s) to produce significant structural stiffness and improve the load bearing capacity.

bulk density

Mass per unit volume including voids in a material.

California Bearing Ratio (CBR)

The ratio expressed as a percentage between a test load and an arbitrarily defined standard load. This test load is then required to cause a plunger of standard dimensions to penetrate at a specified rate into a specifically prepared soil specimen.

cement

A general term for substances that bind aggregates. (Note: In concrete work the term normally refers to Portland cement.)

cement stabilisation

The controlled application of cement to improve the load-carrying capacity of a pavement layer (usually the basecourse) or of the subgrade.

cemented materials

Those produced by the addition of cement, lime or other hydraulically binding agent to granular materials in sufficient quantities to produce a bound layer with significant tensile strength.

cohesive soil

A soil whose relevant behaviour characteristics are derived largely or entirely from the cohesive bonds associated with the fine fraction.

compaction

Reduction in volume of a material by inducing closer packing of its particles by rolling, tamping, vibrating or other processes to reduce the air voids content.

concrete

An intimate mixture of aggregate, cement and water.

concrete pavement

A general term for a pavement in which the wearing course is concrete. (Note: It is usually implied that the concrete layer combines the functions of the base and of the surfacing.)

cost effectiveness

An economic measure defined as the effectiveness of an action or treatment divided by the present worth of life-cycle costs.

course

One or more layers of the same material within a pavement structure.

creep

The slow plastic deformation of a material under stress.

cross-anisotropic

A term used in relation to pavements having properties in the vertical plane different from those in the horizontal plane.

cross-section

A vertical section, generally at right-angles to the centreline showing the ground. (Note: On drawings it commonly shows the road to be constructed, or as constructed.)

cure

To facilitate the achievement of the desired end state of concrete or other material by providing a suitable environment.

deep strength asphalt pavement

A pavement structure in which the wearing course and a substantial portion of the base consist of asphalt.

deep-lift (stabilisation)

A pavement construction technique whereby insitu stabilisation is carried out to depths in excess of 250 mm.

deflection bowl

The depressed shape produced at the surface of a pavement when a load is applied.

design life

The period during which the performance of a pavement/structure is expected to remain acceptable.

design period

A period considered appropriate to the function of the road. It is used to determine the total traffic for which the pavement is designed.

design subgrade level (DSL)

The level of the prepared formation after completion of stripping and excavation or filling and upon which the pavement is to be constructed. (Design Subgrade Level = Finished Surface Level – Nominated Pavement Thickness.)

design traffic

The cumulative traffic, expressed in terms of equivalent standard axle loads, predicted to use a road over the structural design life of the pavement.

drainage

Natural or artificial means of intercepting and removing surface or sub-surface water (usually by gravity).

dry density

The ratio of the dry mass of a sample of material to its undried volume (including voids). See also *maximum density*.

dynamic cone penetration test (DCP)

A test in which the effort to push or drive a standard steel cone into soil under controlled impact is used as a measure of certain soil properties, such as the field CBR.

equilibrium moisture content

The stable moisture content that is reached in a soil in a particular environment after moisture movements have ceased.

equivalent standard axle(s) (ESA)

The number of standard axle loads which are equivalent in damaging effect on a pavement to a given vehicle or axle loading. (Note: The damage is calculated based on the 4th power of the ratio the axle load to the standard axle load.)

falling weight deflectometer (FWD)

A device to measure the surface deflection of a pavement under a dynamic load in order to evaluate its structural adequacy.

fatigue

The deterioration of a pavement or other structure caused by the action of repetitive vehicle loads.

fatigue cracking

A visible crack in the wearing course eventually resulting (in an alligator pattern) from the propagation of cracks caused by fatigue in, or lack of support from, the underlying pavement layer.

field density

The density of earthworks or pavement material measured insitu.

filler

A fine material, the majority of which passes a 0.075 mm sieve, derived from aggregate or other similar granular material and commonly used in slurry surfacing and asphalt.

fine crushed rock

A graded pavement material produced by selectively crushing and screening stone for use in pavement construction, normally 19.0 mm to 26.5 mm maximum size.

flexible pavement

A pavement that obtains its load-spreading properties mainly by intergranular pressure, mechanical interlock and cohesion between the particles of the pavement material. (Note: In the case of an asphalt pavement, this further depends on the adhesion between the bitumen binder and the aggregate, and the cohesion of that binder. Generally, any pavement in which high strength Portland cement concrete is not used as a construction layer.)

fly ash

A fine powder of pozzolanic material obtained from the combustion of pulverised coal in power stations.

gap graded material

Graded material in which one or more of the intermediate sizes are absent.

geotextile

A synthetic fabric, woven or non-woven used for various purposes including embankment reinforcing and stabilisation, as a filter layer between dissimilar materials and as a strain alleviating membrane.

geotextile reinforced seal (GRS)

An application of a bituminous binder into which both aggregate and geotextile are incorporated to provide a durable wearing surface.

graded aggregate

Aggregates having a distribution of sizes from coarse to fine, the largest size being several times larger than the smallest size.

granular material

Material with a particle size no smaller than sand.

heave

The upward movement of soil resulting from expansion or displacement caused by absorption, freezing of soil moisture and operations such as removal of overburden, pile driving and embankment construction.

heavy vehicle

A two-axle vehicle with the minimum axle spacing greater than 3.2 m, or a three-or more axle vehicle configured at least with two axle groups (excluding short towing vehicles, e.g., trailer, caravan, boats, etc.). Also a vehicle defined by Austroads as a Class 3 or higher classification vehicle.

impermeable

Cannot be penetrated by a fluid such as air or water, but commonly refers to water penetration.

improved subgrade

A subgrade whose load-bearing ability has been increased by the addition of suitable material, eg. bitumen, lime, cement or sand.

in situ (insitu)

A material or operation carried out on a material in its final position.

isotropic

Having properties that are equal in all directions.

layer

The portion of a pavement course placed and compacted as an entity.

lime stabilisation

The controlled application of lime to improve the load-carrying capacity of a pavement layer (usually the basecourse) or of the subgrade.

longitudinal profile

The shape of a pavement surface measured as vertical distances from same datum parallel to the traffic flow.

marginal aggregate

An aggregate which does not meet conventional aggregate specifications but is suitable for specific use in pavements.

maximum density

The maximum density that could be achieved in a sample of asphalt if it were possible to compact it so as to exclude all air voids between coated aggregate particles. (Also known as void-free density.)

maximum dry density

The dry density of soil obtained by a specified amount of compaction at the optimum moisture content.

mechanistic analysis

A design procedure based on stress analysis and on fundamental material behaviour in pavements.

mix

The proportions of ingredients in a batch of concrete or mortar or asphalt.

modification

The improvement of the properties of a material by the addition of small quantities of an additive, such as lime or cement, by the stabilisation process.

modified asphalt

An asphalt in which the binder has been modified by the incorporation of polymers, resins, rubber, or other material, to achieve specific physical properties.

modified materials

Granular materials to which small amounts of stabilising agent have been added to improve their performance (e.g. by reducing plasticity) without causing a significant increase in structural stiffness. (Note: Modified materials are considered to behave as unbound materials.)

moisture content

The quantity of water that can be removed from a material by heating to 105°C until no further significant change in mass occurs, usually expressed as a percentage of the dry mass.

open-graded asphalt

A bituminous mix using aggregate containing only small amounts of fine material, and providing a high percentage of air voids.

optimum moisture content (OMC)

That moisture content of a soil at which a specified amount of compaction will produce the maximum dry density under specified test conditions.

pavement

The portion of the road, excluding shoulders, placed above the design subgrade level for the support of, and to form a running surface for, vehicular traffic.

pavement distress

The deterioration of the pavement evidenced by visible surface defects.

pavement layer

A portion of the pavement placed and compacted as an entity.

pavement management system (PMS)

A systematic method of information collection and decision making, necessary to permit the optimisation of the use of resources for the maintenance and rehabilitation of pavements.

permeability

The property of a material by virtue of which a fluid such as water can pass through it.

permeability reversal

Occurs at a pavement layer interface when the coefficient of saturated water permeability of the upper layer is at least 100 times greater than that of the layer below it.

plain concrete pavement (PCP)

A concrete pavement that is unreinforced.

plant mix

A general term for mixtures of bituminous binder and aggregate produced in a central mixing plant.

plastic material

A material in a condition when it can be easily remoulded.

plasticity index (PI) (soil)

The numerical difference between the value of the liquid limit and the value of the plastic limit of a soil.

polymer

A predominantly organic substance comprising a very large number of chemical entities which may comprise identical segments (producing a homopolymer) or a combination of two or more different segments (producing a copolymer).

polymer-modified asphalt

An asphalt made using a polymer-modified binder.

polymer-modified binder (PMB)

A binder of polymeric materials dispersed in bitumen with enhanced binder performance for particular applications.

Portland cement

An artificial cement manufactured from Portland cement clinker. Usually referred to as cement.

Portland cement concrete

Concrete in which the binding material is Portland cement. Usually referred to as concrete.

prime (prime coat)

An application of a primer to a prepared base, without cover aggregate, to provide penetration of the surface, temporary waterproofing and to obtain a bond between the pavement and the subsequent seal or asphalt, which is a preliminary treatment to a more permanent bituminous surfacing.

pumping

The ejection by traffic action, or ground water pressure, of water and fine particles in suspension through transverse or longitudinal joints or cracks in a pavement.

rehabilitation

The restoration of a distressed pavement so that it may be expected to function at a satisfactory level of serviceability for a further design period.

relative compaction

1. The ratio between the field bulk density and the bulk density of the nominated mix when compacted in the laboratory.
2. The ratio between the field bulk density and the maximum density of the nominated mix.
3. The ratio between the field bulk density and the maximum theoretical density of the nominated mix.

resilient modulus

The ratio of stress to recoverable strain under repeated loading conditions; also referred to as elastic stiffness.

rigid pavement

A pavement of Portland cement concrete or having a Portland cement concrete base course.

road maintenance

The work required to keep a road at its specified level of service. It includes work on the road structure, furniture and drainage system.

road pavement stabilisation

The modification of any natural or prepared material to improve or maintain its load-carrying capacity.

roughness

The consequence of irregularities in the longitudinal profile of a road with respect to the intended profile.

rutting

The longitudinal vertical deformation of a pavement surface in a wheel path, measured relative to a straightedge placed at right angles to the traffic flow and across the wheel path.

seal (bituminous)

A thin surface layer of bituminous binder into which aggregate is incorporated. (See also *sprayed seal.*)

shoving

Lateral displacement of pavement structure by braking, accelerating or turning vehicles.

shrinkage

Interconnected cracks forming a series of large blocks usually with sharp corners or angles. Frequently they are caused by volume change in either the base or subgrade, or occasionally in the asphalt mix.

silt

All alluvial material intermediate in particle size between sand and clay. It is usually non-plastic.

soil

Unconsolidated material and organic material derived from weathering or breakdown of rock and decay of vegetation, which include organic matter, clay, silt, sand and gravel.

sprayed seal

A thin layer of binder sprayed onto a pavement surface with a layer of aggregate incorporated and which is impervious to water.

stabilisation (pavement)

The treatment of a road pavement material to improve it or to correct a known deficiency and thus enhance its ability to perform its function in the pavement.

standard axle

Single axle with two sets of dual wheels loaded to a total load of 80 kN.

subbase

The material laid on the subgrade below the base either for the purpose of making up additional pavement thickness required, to prevent intrusion of the subgrade into the base, or to provide a working platform.

subgrade

The trimmed or prepared portion of the formation on which the pavement is constructed. Generally taken to relate to the upper line of the formation.

subgrade design strength

This is the California bearing ratio (CBR) ascribed to the subgrade for the purpose of predicting the behaviour of a superimposed pavement. It is assigned after considering the significance of all layers in the formation.

surfacing

The uppermost part of a pavement specifically designed to resist abrasion from traffic and to minimise the entry of water, which may be a sprayed seal, asphalt or other material.

transverse profile

The shape of a pavement surface measured as vertical distances from a datum perpendicular to traffic flow.

triaxial test

A test to determine the stress–strain properties of a pavement material in which a cylindrical specimen of the material is subjected to a three-dimensional stress system, and the axial strain is related to the applied stress.

unbound base

A base comprised of granular or mechanically stabilised materials and without the capacity to resist significant tensile stresses.

viscoelasticity

The combined viscous and elastic response of a material to an applied stress. This applies notably to bituminous materials, as evidenced by their combined viscous and elastic response to an applied stress.

viscosity

The internal friction in fluids due to molecular cohesion, which is numerically assessed according to a standard method.

void content

The ratio of the volume of voids to the total volume of the material, expressed as a percentage.

watertable

The natural level at which water stands in a borehole, well, or other depression, under conditions of equilibrium.

wearing course

That part of pavement upon which the traffic travels.

APPENDIX 6.1 WEIGHTED MEAN ANNUAL PAVEMENT TEMPERATURE

The values contained in the following Tables can be used to select a Weighted Mean Annual Pavement Temperature (WMAPT) (ºC) for use in estimating asphalt moduli at in-service temperatures (Section 6.5).

The following method was used to calculate the WMAPT at each site:

1. Obtain from the Bureau of Meteorology the monthly average daily maximum air temperature and the annual monthly daily minimum air temperature – www.bom.gov.au/climate/averages.
2. Calculate the monthly average air temperatures by averaging the maximum and minimum air temperatures.
3. Using equation A6.1.1 and the monthly average air temperature, calculate the temperature Weighting Factors (WF) for each month.
4. For each site, average the 12 weighting factors obtained in step 3.
5. Using average WF from step 4 and equation A6.1.2, estimate the weighted mean annual pavement temperature (WMAAT) for each site.
6. Using the WMAAT and equation A6.1.3, estimate the WMAPT for each site.

Equation A6.1.1: Shell Weighting Factors (based on Chart W of Shell Pavement Design Manual)

$$\mathrm{WF} = 10^{(-1.224 + 0.06508T_{air} - 0.000145T_{air}^{2})} \qquad \text{(A6.1.1)}$$

Equation A6.1.2: WMAAT from average WF (based on Chart W of Shell Pavement Design Manual):

$$\begin{aligned}\mathrm{WMAAT} = 19.66 + 16.91 \log \mathrm{WF} \\ + 0.3117 * (\log \mathrm{WF})^2\end{aligned} \qquad \text{(A6.1.2)}$$

Equation A6.1.3: Estimating WMAPT from WMAAT (Chart RT Shell Pavement Design Manual, 100 mm asphalt):

$$\mathrm{WMAPT} = -12.4 + \frac{6.32(\mathrm{WMAAT})}{\ln(\mathrm{WMAAT})} \qquad \text{(A6.1.3)}$$

Victoria

Town	WMAPT
Bairnsdale	22
Ballarat	20
Benalla	26
Bendigo	24
Bright	22
Charlton	25
Dandenong	23
Dookie	25
Echuca	26
Frankston	23
Geelong	23
Horsham	24
Melbourne Region	24
Mildura	28
Nhill	24
Sale	23
Seymour	24
Swan Hill	27
Wangaratta	26
Warragul	22
Warrnambool	21
Wodonga	26
Yallourn	22

NSW and ACT

Town	WMAPT
Albury	26
Armidale	23
Bathurst	22
Bega	24
Bellingen	30
Blayney	19
Bourke	33
Braidwood	20
Broken Hill	30
Byron Bay	31
Campbelltown	27
Canberra	23
Casino	31
Cessnock	28
Cobar	31
Coffs Harbour	29
Cooma	20
Coonabarabran	26
Coonamble	31
Cowra	27
Deniliquin	27
Dubbo	29
Finley	27
Forbes	28
Gilgandra	29
Glen Innes	22
Gosford	27
Goulburn	22
Grafton	31
Griffith	28
Gundagai	26
Hay	28
Inverall	26
Katoomba	20
Kempsey	29
Kiama	27
Kiandra	12
Lismore	30
Lithgow	20

NSW and ACT (cont.)

Town	WMAPT
Liverpool	28
Merimbula	24
Mittagong	22
Molong	24
Moree	31
Moruya	25
Moss Vale	22
Mudgee	26
Murrurundi	26
Murwillumbah	31
Narooma	24
Narrabri	31
Narrandera	27
Newcastle	28
Nowra	26
Nyngan	31
Orange	20
Parkes	28
Parramatta	28
Port Macquarie	27
Queanbeyan	23
Richmond	28
Singleton	29
Sydney Region	28
Tamworth	28
Taree	28
Tenterfield	24
Thredbo	13
Tumut	23
Wagga Wagga	26
Walgett	33
Warialda	29
Wellington	28
Wentworth	29
Wilcannia	32
Wollongong	27
Wyong	26
Yass	24
Young	25

Queensland

Town	WMAPT
Brisbane Region	32
Bowen	36
Cairns	37
Caloundra	31
Charleville	34
Cunnamulla	34
Dalby	30
Emerald	35
Georgetown	38
Gympie	32
Ipswich	32
Kingaroy	29
Mackay	34
Maryborough	32
Miles	32
Monto	32
Mt Isa	39
Nambour	31
Rockhampton	35
Roma	33
Southport	31
St George	33
Toowoomba	27
Townsville	37
Warwick	28

Western Australia

Town	WMAPT
Albany	24
Broome	40
Bunbury	26
Cape Leeuwin	26
Carnarvon	34
Dampier	40
Esperance	26
Eucla	27
Fremantle	28
Geraldton	31
Kalgoorlie	30
Kunumurra	42
Manjimup	24
Meekatharra	36
Merredin	30
Morawa	32
Mt Magnet	35
Narrogin	26
Newman	39
Norseman	28
Northam	30
Ongerup	25
Paraburdoo	40
Perth	29
York	29

South Australia

Town	WMAPT
Adelaide	27
Bordertown	24
Ceduna	26
Keith	25
Mt Gambier	22
Murray Bridge	26
Port Augusta	30
Port Pirie	29
Renmark	28
Whyalla	29

Tasmania

Town	WMAPT
Burnie	20
Campbell Town	18
Devonport	20
Geevestown	18
Hobart	20
Launceston	20
New Norfolk	19
Queenstown	17
St Helens	20
Scottsdale	19
Swansea	20

Northern Territory

Town	WMAPT
Alice Springs	33
Barrow Creek	37
Daly Waters	40
Darwin	41
Katherine	40
Tennant Creek	39

New Zealand

Town	WMAPT
Auckland	23
Christchurch	19
Dunedin	18
Gisborne	21
Greymouth	19
Hamilton	21
Invercargill	17
Kaikoura	19
Masteron	20
Napier	21
New Plymouth	21
Nelson	21
Oamaru	18
Palmerston North	21
Queenstown	18
Rotorua	20
Taupo	19
Tauranga	21
Timaru	19
Wanganui	21
Wellington	20
Westport	19
Whangarei	22

APPENDIX 7.1 CALCULATING CGF FOR NON-CONSTANT ANNUAL GROWTH RATES

Where there is a change in annual growth rate expected during the design period, with the Annual Growth Rate being R_1% for the first Q years and then R_2% for the remainder of the design period (P), then the cumulative growth factor (CGF) can be determined from equation A7.1.1.

(This equation is only valid when the values of both R_1 and R_2 are non-zero. The case when R_1 is non-zero ($R_1 \neq 0$) and R_2 is zero ($R_2 = 0$) is considered below – see eqn A7.1.2.)

The possibility of reaching traffic capacity within the design period is a significant issue that must be considered when estimating traffic growth. The factors affecting the traffic capacity of a road/carriageway are:

- the number, types, and speeds of vehicles (both light and heavy vehicles);
- the type of road/carriageway (one-way or two-way flow, number of lanes, presence of signalised or unsignalised intersections, etc.);
- the widths of traffic lanes and shoulders; and
- the horizontal and vertical alignment of the road/carriageway.

Traffic capacity is one of the principal factors in the geometric design of road projects. If the pavement designer has not been apprised of the traffic capacity of the project, then the designer's attention is drawn to the Austroads publication *A Guide to Traffic Engineering Practice, Part 2: Roadway Capacity* (Austroads 1988), which presents methods for determining traffic capacity.

Where traffic capacity is expected to be reached in the Qth year during the design period, the CGF for the entire design period is given by equation A7.1.2.

$$\mathrm{CGF} = \frac{(1+0.01R_1)^{Q}-1}{0.01R_1} + (1+0.01R_1)^{Q-1}(1+0.01R_2)\left\{\frac{(1+0.01R_2)^{P-Q}-1}{0.01R_2}\right\} \qquad (A7.1.1)$$

$$\mathrm{CGF} = \frac{(1+0.01R)^{Q}-1}{0.01\,R} + (P-Q)\,(1+0.01R)^{Q-1} \qquad (A7.1.2)$$

APPENDIX 7.2 CHARACTERISTICS OF TRAFFIC AT SELECTED WIM SITES

As described in Chapter 7, the following characteristics are required to calculate the design traffic of a project:

- the average number of axle groups per heavy vehicle (N_{HVAG});
- the ESA of damage per heavy vehicle axle group (ESA/HVAG); and
- the SAR of damage per ESA (SAR_a/ESA, SAR_s/ESA and SAR_c/ESA).

These parameters are calculated from the Traffic Load Distribution (TLD) for the project. In the absence of project-specific WIM data, a presumptive TLD needs to be selected.

This Appendix provides characteristics of weigh-in-motion (WIM) data obtained at over 100 sites throughout Australia. It is recommended that the pavement designer use all available information (project-specific, local, regional etc.) before an appropriate TLD is selected from this survey list.

Table A7.2.1: Characteristics of traffic at selected WIM sites

Details of the WIM site										Characteristics of the traffic at the WIM site														
														Percentage distribution of axle group types						Traffic parameters for flexible pavements				
																					Standard Axle Repetitions (SAR) per ESA		
ID of WIM site	State	Road	Location	Speed limit	Direction	Lanes in this direction	Lane surveyed	Survey year	Days in the survey	Lane ADT	%HVs	HVs surveyed	Axle groups per HV, N_{HVAG}	SAST	SADT	TAST	TADT	TRDT	QADT	ESA per HVAG	Asphalt fatigue SAR_a/ESA	Rutting and shape loss SAR_s/ESA	Cemented material fatigue SAR_c/ESA
1	NSW	Hume Hwy	Coolac	100	N	1	1	96	297	2645	32.5	255308	2.93	33.9	4.9	0.2	34.0	27.0	0.0	1.387	1.16	1.65	4.85
2	NSW	Hume Hwy	Coolac	100	S	2	1	96	199	2777	24.6	135973	2.92	34.0	5.1	0.2	34.0	26.6	0.0	1.375	1.18	1.73	5.75
3	NSW	Hume Hwy	Holbrook	100	N	2	1	96	336	1917	33.5	215467	2.98	33.4	4.2	0.2	34.6	27.7	0.0	1.015	1.06	1.24	2.16
4	NSW	Hume Hwy	Holbrook	100	S	2	1	96	333	1878	34.7	217013	3.00	33.2	4.0	0.2	34.5	28.1	0.0	0.924	1.05	1.20	2.00
5	NSW	New England Hwy	Armidale	100	N	1	1	96	235	1380	19.3	62697	2.78	35.6	9.9	0.4	29.8	24.2	0.1	0.857	1.05	1.23	2.36
6	NSW	New England Hwy	Armidale	100	S	1	1	96	254	1383	18.8	66258	2.83	34.9	8.0	0.4	31.6	25.0	0.1	1.123	1.19	1.83	7.96
7	NSW	New England Hwy	Maitland	100	E	1	1	96	349	6288	14.5	317202	2.71	35.8	9.8	0.9	30.2	23.1	0.2	1.149	1.18	1.75	7.50
8	NSW	New England Hwy	Maitland	100	W	1	1	96	338	5993	12.6	254480	2.70	35.6	10.4	1.3	31.6	21.0	0.0	0.811	1.13	1.57	5.38
9	NSW	Newell Hwy	Dubbo	100	N	1	1	96	364	2630	16.6	158613	2.81	34.7	7.3	0.8	32.0	25.1	0.0	1.196	1.16	1.66	5.61
10	NSW	Newell Hwy	Forbes	100	N	1	1	96	186	1383	18.8	48288	2.89	34.3	5.6	0.2	33.2	26.7	0.0	1.419	1.17	1.70	5.75
11	NSW	Newell Hwy	Forbes	100	S	1	1	96	67	1155	16.6	12906	2.87	34.3	6.3	0.5	33.4	25.5	0.0	0.847	1.06	1.29	3.05
12	NSW	Newell Hwy	Jerilderie	100	N	1	1	96	86	824	22.9	16149	2.94	33.8	3.7	0.2	33.9	28.4	0.0	1.156	1.12	1.49	3.94
13	NSW	Newell Hwy	Jerilderie	100	S	1	1	96	329	813	12.5	33528	2.87	31.3	6.1	3.5	33.3	25.6	0.3	1.259	1.19	1.82	7.34
14	NSW	Olympic Hwy	Culcairn	100	N	2	1	96	348	958	19.3	64387	2.78	35.8	8.4	0.2	33.0	22.5	0.0	1.396	1.22	2.00	10.5
15	NSW	Pacific Hwy	Walker's Creek	100	N	1	1	96	361	3068	17.0	188138	2.66	37.3	11.6	0.4	29.7	21.0	0.0	1.079	1.12	1.49	3.95
16	NSW	Monaro Hwy	Michelago	100	N	1	1	96	312	1103	10.7	36931	2.56	38.5	13.0	0.6	29.4	18.6	0.0	1.109	1.18	1.77	7.00
17	NSW	Monaro Hwy	Michelago	100	S	1	1	96	223	943	11.3	23636	2.51	39.3	16.6	0.6	27.1	16.4	0.0	0.809	1.11	1.54	5.36
18	NSW	Princes Hwy	Batemans Bay	100	N	1	1	96	316	1463	10.2	47249	2.34	42.1	23.8	0.7	22.4	11.0	0.0	0.679	1.09	1.42	3.97
19	NSW	Princes Hwy	Batemans Bay	100	S	1	1	96	226	1367	9.6	29693	2.28	42.9	25.6	0.8	21.2	9.4	0.1	0.647	1.09	1.43	4.30
20	NSW	Cobb Hwy	Deniliquin	100	N	1	1	96	305	683	14.3	29722	2.64	37.0	13.5	0.9	28.1	20.4	0.1	1.048	1.21	1.96	11.1
21	NSW	Cobb Hwy	Deniliquin	100	S	1	1	96	321	692	14.0	31107	2.65	36.9	12.4	0.6	28.7	21.4	0.1	1.813	1.34	2.65	22.6
22	NSW	Denman-Merriwa Rd	Sandy Hollow	100	E	1	1	96	311	630	14.1	27687	2.71	36.3	11.7	0.5	29.0	22.4	0.0	1.419	1.23	2.01	10.3
23	NSW	Denman-Merriwa Rd	Sandy Hollow	100	W	1	1	96	316	648	14.9	30467	2.79	35.2	9.1	0.6	30.5	24.6	0.0	1.151	1.15	1.61	5.00
24	NSW	Olympic Hwy	Culcairn	100	S	2	1	96	340	967	19.1	62808	2.80	35.4	7.6	0.3	34.3	22.5	0.0	0.781	1.05	1.24	2.50
25	VIC	Calder Hwy	Loddon	100	N	2	1	96	80	1219	12.8	12438	2.54	36.2	12.1	0.5	28.5	22.8	0.0	0.787	1.10	1.45	4.20
26	VIC	Calder Hwy	Loddon	100	S	2	1	96	101	1162	13.7	16112	2.60	36.2	12.0	0.4	28.5	22.9	0.0	0.858	1.10	1.43	3.80
27	VIC	Calder Hwy	Macedon Ranges	100	N	2	1	96	220	2837	11.2	69787	2.54	38.7	17.6	0.6	25.4	17.7	0.0	0.642	1.02	1.15	2.34

Note: ESA/HV values can be obtained by multiplying the ESA/HVAG values by the Axle Groups per HV values

Table A7.2.1: Characteristics of traffic at selected WIM sites (cont.)

Details of the WIM site										Characteristics of the traffic at the WIM site														
														Percentage distribution of axle group types						Traffic parameters for flexible pavements				
																					Standard Axle Repetitions (SAR) per ESA		
ID of WIM site	State	Road	Location	Speed limit	Direction	Lanes in this direction	Lane surveyed	Survey year	Days in the survey	Lane ADT	%HVs	HVs surveyed	Axle groups per HV, N_{HVAG}	SAST	SADT	TAST	TADT	TRDT	QADT	ESA per HVAG	Asphalt fatigue SAR_a/ESA	Rutting and shape loss SAR_s/ESA	Cemented material fatigue SAR_c/ESA
28	VIC	Calder Hwy	Macedon Ranges	100	S	2	1	96	218	2943	13.2	85041	2.42	39.0	18.6	0.6	24.4	17.4	0.0	0.664	1.06	1.27	2.70
29	VIC	Goulburn Valley Hwy	Moira	100	N	2	1	96	281	1294	10.1	36901	2.58	36.6	15.6	1.5	27.3	18.8	0.2	0.762	1.09	1.41	3.90
30	VIC	Hume Fwy	Milawa	100	S	2	1	96	314	2802	27.7	244145	2.86	34.1	5.9	0.3	33.8	25.8	0.0	0.928	1.11	1.47	4.25
31	VIC	Hume Fwy	Mitchell	100	N	2	1	96	351	4900	25.7	441362	2.93	33.9	4.8	0.3	33.7	27.3	0.0	1.104	1.12	1.50	4.07
32	VIC	Hume Fwy	Mitchell	100	S	2	1	96	337	4614	25.8	400027	2.93	33.8	4.5	0.3	34.0	27.4	0.0	0.986	1.11	1.46	4.20
33	VIC	Princes Hwy East	Baw Baw	100	E	2	1	96	249	4230	12.3	129850	2.57	38.0	17.0	0.7	27.0	17.3	0.0	0.704	1.09	1.44	4.22
34	VIC	Princes Hwy East	Baw Baw	100	W	2	1	96	110	3835	12.2	51653	2.47	38.6	16.7	0.8	27.3	16.6	0.0	0.701	1.09	1.41	4.01
35	VIC	Princes Hwy East	Cardinia	80	E	2	1	96	141	5807	10.6	86191	2.51	38.7	17.9	1.0	26.5	15.9	0.0	0.704	1.09	1.41	4.15
36	VIC	Sturt Hwy	Merbein	100	E	2	1	96	114	762	20.0	17373	2.90	33.8	6.3	0.6	33.5	25.8	0.0	0.893	1.11	1.55	5.92
37	VIC	Sturt Hwy	Merbein	100	W	2	1	96	118	863	13.7	13941	2.80	33.6	8.5	1.1	33.6	23.2	0.0	0.848	1.11	1.55	6.32
38	VIC	Western Hwy	Pyrenees	100	E	2	1	96	322	1743	21.7	121428	2.81	34.6	7.5	0.1	32.4	25.4	0.0	0.758	1.06	1.27	2.88
39	VIC	Western Hwy	Pyrenees	100	W	2	1	96	310	1899	21.2	124789	2.81	34.6	7.4	0.1	32.5	25.3	0.0	0.801	1.05	1.26	2.82
40	VIC	Goulburn Valley Hwy	Moira	80	N	1	1	96	160	1183	16.3	30843	2.86	34.3	6.0	0.3	33.4	26.0	0.1	0.864	1.11	1.48	4.27
41	VIC	Goulburn Valley Hwy	Moira	80	S	1	1	96	360	1169	20.8	87359	2.92	33.8	4.6	0.2	33.9	27.4	0.0	0.912	1.10	1.42	3.82
42	VIC	Goulburn Valley Hwy	Moira	100	S	2	1	96	356	1558	12.3	68020	2.57	36.0	10.2	0.4	31.1	22.3	0.0	0.866	1.09	1.37	3.28
43	VIC	Melba Hwy	Yarra Ranges	80	E	1	1	96	148	1550	10.3	23529	2.57	38.2	14.7	0.8	29.1	17.3	0.0	0.904	1.15	1.70	6.80
44	VIC	Melba Hwy	Yarra Ranges	80	W	1	1	96	142	1364	10.5	20323	2.50	39.1	18.4	0.8	26.8	14.9	0.0	0.913	1.17	1.75	7.15
45	VIC	Princes Hwy West	Glenelg	100	S	2	1	96	278	977	11.9	32136	2.62	37.8	15.8	0.3	27.5	18.5	0.0	0.740	1.06	1.30	3.32
46	VIC	Monash Freeway	Greater Dandenong	100	W	3	2	96	266	10495	12.4	347375	2.53	38.7	18.3	0.9	26.7	15.5	0.0	0.600	1.03	1.18	2.43
47	VIC	Monash Freeway	Greater Dandenong	100	E	3	2	96	120	11084	12.0	160515	2.63	37.3	13.7	0.8	30.1	18.2	0.0	0.551	1.00	1.07	1.69
48	VIC	Stud Rd	Knox	80	S	3	2	96	299	6348	10.7	202418	2.25	43.2	28.9	1.2	19.7	7.0	0.0	0.457	1.04	1.31	4.85
49	VIC	Western Ring Rd	Moreland	100	E	4	1	96	130	11204	19.2	279723	2.61	37.3	13.9	1.0	29.1	18.7	0.0	0.908	1.12	1.52	5.16
50	VIC	Western Ring Rd	Moreland	100	E	4	2	96	77	2781	19.0	40718	2.59	37.8	15.0	0.8	28.3	18.2	0.0	0.860	1.17	1.98	37.8
51	VIC	Western Ring Rd	Moreland	100	W	4	2	96	80	1997	23.4	37286	2.62	37.2	13.6	0.8	30.1	18.3	0.0	0.868	1.19	2.04	37.9
52	VIC	Western Ring Rd	Moreland	100	W	4	1	96	168	9820	21.0	346119	2.63	37.0	13.5	0.9	29.4	19.1	0.0	0.809	1.09	1.40	4.91
53	VIC	Cumberland Rd	Pascoe Vale	60	S	1	1	96	6	2949	4.9	845	1.98	48.7	45.0	0.4	5.2	0.7	0.0	0.406	1.05	1.39	9.03
54	VIC	Fairbank Rd	Clayton Sth	60	W	2	1	97	4	4062	15.4	2470	1.92	41.2	28.1	4.6	20.6	5.6	0.0	2.031	1.54	4.19	86.4

Note: ESA/HV values can be obtained by multiplying the ESA/HVAG values by the Axle Groups per HV values

Table A7.2.1: Characteristics of traffic at selected WIM sites (cont.)

Details of the WIM site										Characteristics of the traffic at the WIM site														
														Percentage distribution of axle group types						Traffic parameters for flexible pavements				
																					Standard Axle Repetitions (SAR) per ESA		
ID of WIM site	State	Road	Location	Speed limit	Direction	Lanes in this direction	Lane surveyed	Survey year	Days in the survey	Lane ADT	%HVs	HVs surveyed	Axle groups per HV, N_{HVAG}	SAST	SADT	TAST	TADT	TRDT	QADT	ESA per HVAG	Asphalt fatigue SAR_a/ESA	Rutting and shape loss SAR_s/ESA	Cemented material fatigue SAR_c/ESA
55	VIC	Hammond Rd	Dandenong Sth	60	N	2	1	97	4	2668	16.6	1751	1.95	44.4	30.0	1.3	18.4	5.9	0.0	2.136	1.54	4.13	77.7
56	VIC	Mahoneys Rd	Forest Hill	60	N	2	1	97	1	2988	2.3	76	1.76	50.0	48.5	0.0	1.5	0.0	0.0	1.038	1.22	1.96	9.19
57	VIC	Monterey Rd	Dandenong Sth	60	W	2	1	97	4	1782	17.8	1270	1.97	44.2	30.4	1.6	17.7	6.1	0.0	1.685	1.46	3.56	57.7
58	VIC	Stevens Rd	Forest Hill	60	N	2	1	97	3	2013	2.1	123	1.89	50.0	50.0	0.0	0.0	0.0	0.0	2.253	1.43	3.20	32.7
59	VIC	Bayfield Rd	Bayswater Nth	60	N	2	1	97	5	1779	12.4	996	1.90	47.5	35.2	0.8	15.3	1.1	0.0	1.087	1.33	2.80	34.8
60	VIC	Fulton Rd	Blackburn Sth	60	E	1	1	97	2	1705	3.2	95	1.84	49.7	48.6	0.0	1.7	0.0	0.0	0.838	1.22	1.98	8.77
61	VIC	Hawthorn Rd	Vermont Sth	60	W	2	1	97	4	1713	2.7	168	1.85	50.0	50.0	0.0	0.0	0.0	0.0	0.270	1.22	2.03	9.84
62	VIC	Husband Rd	Forest Hill	60	E	1	1	97	3	1115	2.0	71	1.86	49.2	50.0	0.0	0.8	0.0	0.0	0.238	1.11	1.80	13.0
63	QLD	Barkly Hwy	Cloncurry	100	W	1	1	96	17	409	14.4	1000	3.25	29.3	10.4	0.5	35.5	24.2	0.2	1.410	1.32	2.64	24.3
64	QLD	Cunningham Hwy	Cunninghams Gap	100	E	2	1	96	13	1501	30.1	5981	2.89	34.0	5.9	0.3	32.7	27.0	0.0	1.346	1.16	1.65	4.85
65	QLD	Cunningham Hwy	Cunninghams Gap	100	W	2	1	96	30	1957	21.5	12469	2.92	33.9	6.5	0.3	33.0	26.3	0.0	1.042	1.10	1.40	3.29
66	QLD	Dalby-Kingaroy Hwy	Bell	100	S	2	1	96	82	287	20.8	4910	2.70	34.4	10.0	1.8	29.8	24.0	0.0	1.010	1.17	1.80	9.60
67	QLD	Gregory Hwy	Springsure	100	N	2	1	96	80	358	12.4	3564	2.82	34.1	16.4	0.5	29.6	19.3	0.0	0.801	1.16	1.80	12.0
68	QLD	Gregory Hwy	Springsure	100	S	2	1	96	81	366	10.5	3116	2.41	35.1	18.2	0.8	26.8	19.1	0.1	0.642	1.13	1.62	6.30
69	QLD	Kilcoy-Yarraman Rd	D’aguilar	100	E	2	1	96	39	1123	12.2	5412	2.47	39.2	15.9	1.0	28.7	15.1	0.1	0.981	1.17	1.76	7.80
70	QLD	Toowoomba-Millmerran Rd	Southbrook	100	E	2	1	96	18	1289	17.1	4038	2.81	33.6	7.3	1.3	33.1	24.7	0.1	1.166	1.15	1.60	5.44
71	QLD	Toowoomba-Millmerran Rd	Southbrook	100	W	2	1	96	13	1150	14.9	2180	2.60	34.8	9.6	1.9	32.6	21.1	0.0	0.704	1.08	1.37	3.66
72	QLD	Toowoomba-Warwick Rd	Missens Flat	100	N	2	1	96	39	1046	10.9	4458	2.34	39.1	20.9	1.1	24.2	14.7	0.0	1.248	1.30	2.44	18.3
73	QLD	Warwick-Inglewood Hwy	Durakai Forest	100	E	2	1	96	48	707	31.7	10826	2.98	33.1	4.6	0.2	33.5	28.5	0.0	1.308	1.17	1.68	5.60
74	QLD	Pacific Hwy	Ormeau	100	N	2	1	96	54	11458	12.4	76780	2.25	41.6	25.0	0.9	21.7	10.8	0.0	0.477	1.00	1.11	2.31
75	QLD	Pacific Hwy	Ormeau	100	N	2	2	96	57	6675	6.9	26278	2.08	40.4	22.3	0.8	23.8	12.7	0.0	0.500	1.04	1.27	3.22
76	QLD	Pacific Hwy	Ormeau	100	S	2	1	96	75	12607	12.6	118357	2.21	40.7	23.1	1.1	23.2	12.0	0.0	0.807	1.14	1.62	5.62
77	QLD	Pacific Hwy	Ormeau	100	S	2	2	96	65	6236	7.3	29696	1.87	41.1	22.3	0.9	22.8	13.0	0.0	0.909	1.20	1.93	10.0
78	WA	Eyre Hwy	Norseman	100	W	1	1	96	48	202	42.9	4128	3.83	25.8	2.7	0.3	37.0	34.1	0.0	1.344	1.25	2.26	20.8
79	WA	Great Eastern Hwy	Northam	100	W	1	1	96	202	870	23.2	40843	3.27	28.2	7.7	2.3	36.5	25.2	0.0	0.644	1.06	1.32	3.90
80	WA	Albany Hwy	Bannister	100	N	1	1	96	285	1148	13.4	44027	3.12	28.1	10.4	3.9	35.6	21.9	0.0	0.996	1.20	1.94	11.4
81	WA	Armadale-Ravensthorpe Rd	Brookton	100	W	1	1	96	219	204	14.9	6671	3.38	25.7	7.2	3.8	40.1	23.1	0.0	1.657	1.29	2.29	12.5

Note: ESA/HV values can be obtained by multiplying the ESA/HVAG values by the Axle Groups per HV values

Table A7.2.1: Characteristics of traffic at selected WIM sites (cont.)

Details of the WIM site										Characteristics of the traffic at the WIM site													
														Percentage distribution of axle group types						Traffic parameters for flexible pavements			
																					Standard Axle Repetitions (SAR) per ESA		
ID of WIM site	State	Road	Location	Speed limit	Direction	Lanes in this direction	Lane surveyed	Survey year	Days in the survey	Lane ADT	%HVs	HVs surveyed	Axle groups per HV, N_{HVAG}	SAST	SADT	TAST	TADT	TRDT	QADT	ESA per HVAG	Asphalt fatigue SAR_a/ESA	Rutting and shape loss SAR_s/ESA	Cemented material fatigue SAR_c/ESA
82	WA	North West Coastal Hwy	Geraldton	100	N	1	1	96	224	708	10.9	17257	3.44	24.2	10.5	4.9	39.1	21.3	0.1	1.830	1.48	3.72	59.3
83	WA	South Coast Hwy	Esperance	100	E	1	1	96	263	486	14.2	18163	3.77	22.3	6.4	4.3	36.2	30.8	0.0	1.841	1.26	2.10	9.22
84	WA	South West Hwy	Donnybrook	100	N	2	1	96	178	1151	11.5	23522	3.06	24.9	9.7	7.8	39.1	18.5	0.0	2.744	1.49	3.61	47.0
85	WA	Kwinana Fwy	Leeming	100	N	2	1	97	3	11329	11.7	4097	2.46	36.8	24.6	3.7	25.8	8.9	0.0	1.172	1.39	3.13	41.0
86	WA	Kwinana Fwy	Leeming	100	S	2	1	97	3	8485	15.3	4482	2.48	36.2	23.7	4.1	26.4	9.6	0.0	0.969	1.32	2.68	26.4
87	WA	Reid Hwy	West Swan	80	E	2	1	97	3	5279	9.4	1442	2.04	32.7	22.6	7.3	28.6	8.7	0.1	1.775	1.55	4.16	77.4
88	WA	Roe Hwy	Hazelmere	100	N	2	1	97	2	7065	14.1	2372	2.63	34.6	19.7	3.4	28.4	13.8	0.0	1.229	1.38	2.99	36.9
89	WA	Roe Hwy	Hazelmere	80	S	2	1	97	4	5204	22.6	5187	2.65	34.3	19.0	3.3	29.2	14.1	0.1	1.195	1.42	3.46	79.3
90	WA	Karrinyup Rd	Innaloo	70	W	2	1	97	2	4770	7.5	869	2.12	45.7	39.6	1.6	10.5	2.7	0.0	0.791	1.25	2.35	22.7
91	WA	Leach Hwy	Shelley	70	E	2	1	97	4	5036	14.3	2768	2.51	36.0	22.1	3.9	27.0	11.0	0.0	0.799	1.26	2.51	49.9
92	WA	Leach Hwy	Shelley	70	W	2	1	97	4	5537	10.7	2328	2.50	35.2	21.7	4.8	27.7	10.6	0.0	0.995	1.34	2.83	36.8
93	WA	Port Beach Rd	North Fremantle	70	N	2	2	97	2	270	93.8	602	2.54	33.9	18.2	3.3	32.5	12.0	0.0	0.859	1.19	2.02	20.0
94	WA	Port Beach Rd	North Fremantle	70	N	2	1	97	3	794	89.5	1925	2.84	29.5	9.4	4.5	39.6	17.0	0.0	0.748	1.15	1.73	8.50
95	WA	Port Beach Rd	North Fremantle	70	S	2	2	97	3	302	91.3	713	2.70	29.8	12.6	5.1	38.3	14.3	0.0	0.892	1.20	1.93	10.9
96	WA	Port Beach Rd	North Fremantle	70	S	2	1	97	3	777	91.7	1900	2.71	31.3	11.0	3.7	37.9	16.1	0.0	0.891	1.18	1.81	8.75
97	WA	Stirling Hwy	North Fremantle	60	N	2	1	97	3	9814	4.1	1341	2.14	45.9	42.5	0.9	9.4	1.4	0.0	0.653	1.22	2.21	23.0
98	WA	Stirling Hwy	North Fremantle	60	S	2	1	97	4	8316	4.2	1270	2.12	46.6	44.4	0.6	7.6	0.7	0.0	0.640	1.21	2.27	32.7
99	WA	(Weighbridge)	Fremantle	60	N	2	1	97	2	270	93.8	602	2.54	33.9	18.2	3.3	32.5	12.0	0.0	0.859	1.19	2.02	20.0
100	SA	Barrier Hwy	Oodla Wirra	110	N	1	1	02	301	273	28.4	23333	3.58	27.9	4.9	0.1	33.9	33.2	0.0	1.31	1.18	1.81	10.5
101	SA	Barrier Hwy	Oodla Wirra	110	S	1	1	02	303	273	28.7	23757	3.56	28.0	4.7	0.1	33.5	33.6	0.1	0.89	1.11	1.56	30.1
102	SA	Dukes Hwy	Bordertown	110	E	1	1	02	225	1334	29.2	87668	3.23	30.9	4.3	0.0	32.8	32.0	0.0	0.86	1.10	1.43	4.49
103	SA	Dukes Hwy	Bordertown	110	W	1	1	02	273	1420	30.6	118487	3.23	30.9	4.2	0.1	32.4	32.4	0.0	0.92	1.12	1.50	4.86
104	SA	Eyre Hwy	Iron Knob	110	E	1	1	02	357	280	24.9	24913	3.77	26.4	5.8	0.2	34.2	33.3	0.1	0.98	1.11	1.53	19.5
105	SA	Eyre Hwy	Iron Knob	110	W	1	1	02	323	281	26.2	23812	3.81	26.0	5.1	0.2	34.6	34.0	0.1	0.99	1.14	1.73	43.9
106	SA	South East Fwy	Monarto	110	E	2	1	02	285	4186	17.0	203247	3.06	32.5	6.9	0.2	32.0	28.4	0.0	0.92	1.19	1.86	12.3
107	SA	South East Fwy	Monarto	110	W	2	1	02	269	4250	16.9	193531	3.06	32.4	6.9	0.2	31.9	28.6	0.0	1.08	1.22	2.07	17.8
108	SA	Stuart Hwy	Pimba	110	N	1	1	02	242	375	18.0	16323	4.62	21.4	4.6	0.2	40.3	33.4	0.1	1.10	1.18	1.88	38.3

Note: ESA/HV values can be obtained by multiplying the ESA/HVAG values by the Axle Groups per HV values

Table A7.2.1: Characteristics of traffic at selected WIM sites (cont.)

Details of the WIM site										Characteristics of the traffic at the WIM site													
														Percentage distribution of axle group types						Traffic parameters for flexible pavements			
																					Standard Axle Repetitions (SAR) per ESA		
ID of WIM site	State	Road	Location	Speed limit	Direction	Lanes in this direction	Lane surveyed	Survey year	Days in the survey	Lane ADT	%HVs	HVs surveyed	Axle groups per HV, N_{HVAG}	SAST	SADT	TAST	TADT	TRDT	QADT	ESA per HVAG	Asphalt fatigue SAR_a/ESA	Rutting and shape loss SAR_s/ESA	Cemented material fatigue SAR_c/ESA
109	SA	Stuart Hwy	Pimba	110	S	1	1	02	277	352	19.5	19041	4.44	22.4	5.8	0.2	40.4	31.2	0.0	1.05	1.23	2.11	26.3
110	SA	Sturt Hwy	Truro	110	E	2	1	02	318	1426	23.2	105192	3.18	31.2	5.8	0.3	33.3	29.4	0.0	1.13	1.15	1.68	8.10
111	SA	Sturt Hwy	Truro	110	E	2	2	02	344	311	7.2	7722	3.14	31.5	8.0	0.3	32.8	27.4	0.0	0.87	1.23	2.14	15.4
112	SA	Sturt Hwy	Truro	110	W	1	1	02	185	1762	18.9	61495	3.19	31.0	5.8	0.3	34.1	28.8	0.0	1.15	1.15	1.63	8.11
113	SA	Riddoch Hwy	Naracoorte	110	N	1	1	02	358	864	14.6	45235	2.96	33.7	9.5	0.1	32.2	24.5	0.0	0.97	1.14	1.57	7.12
114	SA	Riddoch Hwy	Naracoorte	110	S	1	1	02	300	874	17.0	44518	3.02	33.0	7.0	0.1	32.4	27.5	0.0	0.75	1.12	1.55	6.54
115	SA	Salisbury Hwy	Barker Inlet	100	E	2	1	02	247	10583	14.4	375584	2.76	35.2	15.5	1.0	30.2	18.1	0.0	0.72	1.13	1.61	14.3
116	SA	Salisbury Hwy	Barker Inlet	100	E	2	2	02	258	8869	5.0	113763	2.92	33.7	33.4	0.5	20.4	12.0	0.0	0.47	1.19	2.03	27.3
117	SA	Salisbury Hwy	Barker Inlet	100	W	2	1	02	48	10452	12.9	64837	2.65	36.9	18.6	0.8	27.4	16.3	0.0	0.65	1.10	1.50	7.67
118	SA	Salisbury Hwy	Barker Inlet	100	W	2	2	02	119	10839	7.4	95267	2.82	34.3	15.7	1.1	31.1	17.8	0.0	0.55	1.05	1.25	3.94
119	TAS	Midland Hwy	Epping	100	N	1	1	96	147	1916	15.0	42255	2.60	37.9	13.8	0.5	30.0	17.8	0.0	0.803	1.10	1.45	4.37
120	TAS	Huon Hwy	(Unknown)	100	E	2	1	96	9	1529	10.5	1387	2.48	39.5	13.1	0.8	30.5	16.1	0.0	1.263	1.19	1.78	6.51
121	TAS	Huon Hwy	(Unknown)	100	W	2	1	96	13	1315	9.8	1645	2.14	46.2	25.2	0.6	23.8	4.2	0.0	0.531	1.01	1.16	2.75
122	NT	Stuart Hwy	Three Ways	–	N	1	1	01	309	198	35.0	21321	3.84	20.5	8.4	0.0	38.9	32.2	0.0	0.68	1.12	1.65	6.44
123	NT	Stuart Hwy	Three Ways	–	S	1	1	01	306	198	30.2	18120	4.31	20.8	6.0	0.0	40.0	33.3	0.0	0.53	1.12	1.63	6.14
124	NT	Barkly Hwy	Avon Downs	–	W	1	1	01	301	121	28.1	10234	3.10	23.2	8.2	0.0	38.7	29.8	0.0	1.00	1.30	2.71	28.5
125	NT	Barkly Hwy	Avon Downs	–	E	1	1	01	292	121	28.1	9928	2.65	23.2	8.2	0.0	39.7	28.8	0.0	0.74	1.13	1.67	7.37
126	NT	Stuart Hwy	Alice Springs	110	N	1	1	01	363	500	20.8	37752	2.44	26.4	12.7	0.0	34.8	26.1	0.0	1.00	1.26	2.39	17.6
127	NT	Stuart Hwy	Alice Springs	110	S	1	1	01	327	500	22.0	35970	1.62	30.3	17.5	0.0	30.1	22.2	0.0	0.56	1.09	1.46	4.59
128	NT	Stuart Hwy	Erldunda	–	N	1	1	01	248	161	21.3	8432	3.92	23.1	10.0	0.0	38.6	28.2	0.0	1.05	1.16	1.81	8.21

Note: ESA/HV values can be obtained by multiplying the ESA/HVAG values by the Axle Groups per HV values

APPENDIX 7.3 ADJUSTMENT OF DESIGN TRAFFIC FOR ANTICIPATED INCREASES IN LOAD MAGNITUDE

A7.3.1 Flexible pavements

Estimation of future increases in the magnitudes of axle group loads, and of when these increases are likely to occur, is an integral part of the determination of design traffic for a project.

While a general algorithm for the incorporation of this information can be readily formulated, which is applicable to any specific estimation scenario, it is considered that the apparent complexity of such an algorithm could deter designers from using it.

As an alternative, two simple estimation scenarios, and the manner by which they are incorporated, are considered below, as a means of illustrating the underlying principles.

Estimation scenario 1

If the designer anticipates that all axle group loads will increase by H% after Q years and remain at this higher level for the duration of the design period, then it is necessary to multiply the values of $DSAR_k$ determined in Section 7.6.3 by the following Load Increase Factor:

$$\text{Load Increase Factor} = X + (1 - X)\,(1 + 0.01H)^m \qquad \text{(A7.3.1)}$$

where

$$X = \frac{\text{Cumulative number of axle groups in the first Q years}}{\text{Cumulative number of axle groups over the design period}}$$

$$= \frac{\text{Cumulative number of axle groups in the first Q years}}{N_{DT}}$$

and damage exponent *m* for each damage type *k* is defined in *Table 7.7*.

For the case where there is uniform growth in the **volume** of heavy vehicle traffic throughout the design period (P) of R% per annum, but there is still only one increase in load magnitude of H% after Q years, then:

$$X = \frac{(1 + 0.01R)^Q - 1}{(1 + 0.01R)^P - 1} \qquad \text{(A7.3.2)}$$

Estimation scenario 2

If the designer anticipates that only specific axle group types will have load increases of H% after Q years and remain at this higher level for the duration of the design period, then it is necessary to first determine the values of $DSAR_k$ for each of these axle group types when no load increases are anticipated. These values are determined as follows:

$$DSAR_{ik} = N_{DT} \times P_i \times \sum_{\text{all } j} P_{ij} \times \left(\frac{L_{ij}}{SL_i}\right)^m \qquad \text{(A7.3.3)}$$

where

$DSAR_{ik}$	=	design number of Standard Axle Repetitions for damage type *k* caused by axle group i;
N_{DT}	=	the cumulative number of Heavy Vehicle Axle Groups traversing the design lane during the design period;
P_i	=	the proportion of all axle groups which are of type i;
P_{ij}	=	the proportion of all type i axle groups which have loads of magnitude L_{ij};
L_{ij}	=	the j^{th} load magnitude in the distribution of loads being carried by the type i Axle Group;
SL_i	=	Standard Load for Axle Group type i (from *Table 7.6*);
k	=	damage type designation (from *Table 7.7*); and
m	=	damage exponent for damage type *k* (from *Table 7.7*).

Each relevant value of $DSAR_{ik}$ is then multiplied by the above Load Increase Factor (evaluated for the specific axle group type i – see eqn A7.3.1) and – for each relevant value of *m* (damage exponent) – the resulting values of $DSAR_{ik}$ are summed for all values of i to give the adjusted value of $DSAR_k$.

A7.3.2 Rigid pavements

Estimation of future increases in the magnitudes of axle group loads and of when these increases are likely to occur, together with incorporation of these estimates, is an integral part of the determination of design traffic for a project.

While a general algorithm can be readily formulated, which is applicable to any specific estimation scenario, it is considered that the apparent complexity of such an algorithm could deter designers from using it.

As an alternative, two simple estimation scenarios – and the manner by which they are incorporated – are considered below, as a means of illustrating the underlying principles.

Estimation scenario 1

If the designer anticipates that all axle group loads will increase by H% after Q years and remain at this higher level for the duration of the design period, then the design traffic consists of two listings of pairs of quantities. The first listing is of the following pairs of quantities:

$$X \times N_{DT} \times P_i \times P_{ij} \quad \textit{and} \quad L_{ij}$$

where X is that fraction of the cumulative number of axle groups (N_{DT}) which traverse the pavement during the first Q years.

The second listing is of the following pairs of quantities:

$$(1 - X) \times N_{DT} \times P_i \times P_{ij} \quad \textit{and} \quad (1 + 0.01H) \times L_{ij}$$

For the case where there is uniform growth in the volume of traffic throughout the design period of R% per annum, then:

$$X = \frac{(1 + 0.01R)^Q - 1}{(1 + 0.01R)^P - 1} \qquad \text{(A7.3.4)}$$

Estimation scenario 2

If the designer anticipates that only specific axle group types will have load increases after Q years and remain at these higher levels for the duration of the design period, then the design traffic consists of a single listing for those axle groups whose loads are not anticipated to increase, plus a pair of listings for each axle group type whose loads are anticipated to increase.

The single listing is of the pairs of quantities:

$$N_{DT} \times P_i \times P_{ij} \quad \textit{and} \quad L_{ij}$$

where i takes the values for those axle group types whose loads are not anticipated to increase.

The pairs of listings for each axle group type whose load is anticipated to increase have the form:

$$X \times N_{DT} \times P_i \times P_{ij} \quad \textit{and} \quad L_{ij}$$

and

$$(1 - X) \times N_{DT} \times P_i \times P_{ij} \quad \textit{and} \quad (1 + 0.01H) \times L_{ij}$$

where

i takes the values for those axle group types whose loads are anticipated to increase;

X_i is that fraction of the cumulative number of type i axle groups which traverse the pavement during the first Q years; and

H_i is the anticipated % increase in the loads on axle group type i after the Qth year.

APPENDIX 7.4 PRESUMPTIVE TRAFFIC LOAD DISTRIBUTIONS FOR URBAN AND RURAL ROADS

Using the traffic load distributions (TLD) measured at sites throughout Australia (see *Appendix 7.2*), representative TLD were estimated for urban and rural roads (*Table A7.4.1*).

Table A7.4.1 Presumptive traffic load distributions

Application	**Table**
Rural roads	Table A7.4.2
Urban roads	Table A7.4.3

Table A7.4.2 Presumptive traffic load distribution for rural roads

Axle group load (kN)	Axle group type SAST %	SADT %	TAST %	TADT %	TRDT %
10	0.5034	3.031	0.0082	0.3815	0.0127
20	6.738	4.968	0.0844	0.5656	0.1246
30	9.237	6.858	1.285	2.364	1.878
40	15.30	8.484	1.986	4.009	3.859
50	28.74	10.23	5.115	4.255	5.213
60	32.19	14.16	11.48	5.173	4.359
70	5.653	13.58	15.24	4.866	4.078
80	1.211	13.29	16.19	4.846	3.843
90	0.2496	11.17	14.59	5.297	3.790
100	0.0768	7.837	15.37	6.244	3.842
110	0.0561	4.130	11.13	9.314	4.116
120	0.0312	2.002	6.396	9.173	3.574
130	0.0139	0.1787	0.6323	10.55	6.690
140		0.0701	0.2245	12.92	7.893
150		0.0117	0.1028	10.35	8.357
160			0.0911	3.767	8.126
170			0.0533	2.566	7.379
180			0.0214	1.598	6.076
190				0.8682	4.981
200				0.4341	4.424
210				0.2348	2.830
220				0.1346	1.928
230				0.0613	1.138
240				0.0279	0.6311
250					0.3486
260					0.2209
270					0.1293
280					0.0815
290					0.0441
300					0.0253
310					0.0079
320					
330					
340					
350					
360					
370					
380					
390					
400					
Total	100.00	100.00	100.00	100.00	100.00
Proportion of each axle group	0.344	0.098	0.007	0.320	0.231

Damage Index	Value
N_{HVAG}	2.8
ESA/HVAG	0.9
ESA/HV	2.5
SAR_a/ESA	1.1
SAR_s/ESA	1.6
SAR_c/ESA	12

Table A7.4.3 Presumptive traffic load distribution for urban roads

Axle group load (kN)	Axle group type				
	SAST %	SADT %	TAST %	TADT %	TRDT %
10	0.2804	3.473	0.0354	0.1444	0.0050
20	7.827	8.696	0.2377	0.5755	0.1568
30	15.46	23.46	0.2763	0.6242	0.3290
40	15.71	21.93	0.5755	1.977	1.317
50	29.94	16.80	2.889	6.496	4.167
60	23.29	9.606	10.27	9.511	7.419
70	6.502	6.500	16.81	10.94	9.777
80	0.7943	4.623	16.61	9.769	8.338
90	0.1087	2.969	15.95	7.611	6.150
100	0.0354	1.393	14.42	7.242	5.029
110	0.0174	0.4098	9.774	6.267	3.701
120	0.0174	0.1158	5.903	5.952	3.298
130	0.0174	0.0244	2.943	5.878	3.147
140			1.539	6.534	3.361
150			0.8439	8.030	4.008
160			0.4279	5.717	4.115
170			0.2308	3.554	4.819
180			0.1367	1.863	6.097
190			0.0723	0.8535	7.733
200			0.0555	0.3331	8.433
210				0.0801	5.136
220				0.0322	2.339
230				0.0160	0.7764
240					0.2503
250					0.0905
260					0.0080
270					
280					
290					
300					
310					
320					
330					
340					
350					
360					
370					
380					
390					
400					
Total	100.00	100.00	100.00	100.00	100.00
Proportion of each axle group	0.393	0.191	0.009	0.259	0.148

Damage Index	Value
N_{HVAG}	2.5
ESA/HVAG	0.7
ESA/HV	1.8
SAR_a/ESA	1.1
SAR_s/ESA	1.6
SAR_c/ESA	12

APPENDIX 7.5 PAVEMENT DAMAGE IN TERMS OF STANDARD AXLE REPETITIONS

A7.5.1 Evaluation of Standard Axle Repetitions (SAR) per axle group for each damage type

With the adoption of the Standard Axle as a reference axle group, the damage caused to any flexible pavement by a specific axle group-load combination can be expressed as follows:

$$\begin{aligned} \text{Damage}_{ijk} &= \frac{\text{Damage}_{ijk} \times \text{Damage}_{SAk}}{\text{Damage}_{SAm}} \\ &= \text{SAR}_{ijk} \times \text{Damage}_{SAk} \end{aligned} \qquad (A7.5.1)$$

where

$DAMAGE_{ijk}$ = damage type k (*Table A7.5.2*) caused by axle group type i with load L_{ij};

$DAMAGE_{SAk}$ = damage type k (*Table A7.5.2*) caused by the Standard Axle; and

SAR_{ijk} = the number of Standard Axle Repetitions (or passages of the Standard Axle) which cause the same amount of damage (type k) as a single passage of axle group type i with load L_{ij}.

The term $DAMAGE_{SAk}$ is directly evaluated – for each relevant damage type (exponent m) – within the pavement design procedure.

SAR_{ijk} is evaluated – for each relevant damage type and each relevant combination of axle group type i with load L_{ij} – in the manner now described.

Because the damage caused to a pavement by a load on an axle group depends on the specific type of axle group, the next step is to assign to each axle group type the load which is considered to cause the same damage as the Standard Axle. These loads are listed in *Table A7.5.1*.

Table A7.5.1 Axle group loads which cause same damage as standard axle*

Axle group type	Load (kN)
Single axle with single tyres (SAST)	53
Single axle with dual tyres (SADT)	80
Tandem axle with single tyres (TAST)	90
Tandem axle with dual tyres (TADT)	135
Triaxle with dual tyres (TRDT)	181
Quad-axle with dual tyres (QADT)	221

* The axle group loads which cause equal damage are taken to be those loads which produce equal maximum deflection of the pavement surface.

Denoting this axle group load (which causes the same damage as a Standard Axle) as the axle group's Standard Load (SL), SAR_{ijk} is evaluated as follows:

$$\text{SAR}_{ijk} = \left(\frac{L_{ij}}{SL_i}\right)^m \qquad (A7.5.2)$$

where

SAR_{ijk} = number of Standard Axle Repetitions (or passages of the Standard Axle) which causes the same amount for each damage type k (damage exponent m) as a single passage of axle group type i with load L_{ij};

SL_i = Standard Load for axle group type i (from *Table A7.5.1*);

L_{ij} = load on the axle group;

k = damage type designation (from *Table A7.5.2*); and

m = an exponent which is specific to the type k damage (from *Table A7.5.2*).

As noted above, for granular pavements with a thin bituminous surfacing, the Standard Axle Repetitions (SAR) – calculated with a damage exponent m of 4 – are commonly referred to as Equivalent Standard Axles (ESA).

Table A7.5.2 Damage exponents for each specific type of damage

Design method	Pavement type	Damage type	Damage designation (k)	Damage exponent (m)
Mechanistic	Pavement containing one or more bound layers	Fatigue of asphalt	a	5
		Fatigue of cemented material	c	7
		Rutting and shape loss	s	12
Empirical	Granular pavement with thin bituminous surfacing	Overall pavement damage	e	4

The Damage Exponents 5 and 12 (for fatigue of asphalt and cemented material respectively) are derived from the fatigue relationships for these materials presented in Chapter 6. The Damage Exponent 7 (for rutting and loss of surface shape of bound pavements) is derived from the subgrade strain criterion presented in Chapter 5. The Damage Exponent 4 (for overall damage to granular pavements with thin bituminous surfacing) is derived from field studies of pavement performance.

A7.5.2 Specification of design traffic and its calculation

The design traffic requirements for flexible pavement design can now be formally stated as follows:

> The Design Traffic for flexible pavement design is – for each relevant damage type – the cumulative traffic over the design period expressed in terms of the number of Standard Axle Repetitions (SAR) which cause the same damage as the cumulative traffic.

If the designer has access to project-specific WIM data, then the relevant design traffic values ($DSAR_k$ values) are determined as follows:

$$DSAR_k = N_{DT} \times \sum_{\text{all } i,j} P_i \times P_{ij} \times \left(\frac{L_{ij}}{SL_i}\right)^m \qquad \text{(A7.5.3)}$$

where

$DSAR_k$ = design number of Standard Axle Repetitions for damage type;

N_{DT} = the cumulative number of axle groups traversing the design lane during the design period;

P_i = the proportion of all axle groups which are of type i;

P_{ij} = the proportion of all type i axle groups which have loads of magnitude L_{ij};

L_{ij} = the j^{th} load magnitude in the distribution of loads being carried by the type i axle group;

SL_i = Standard Load for axle group type i (from *Table A7.5.1*);

k = damage type designation (from *Table A7.5.2*); and

m = damage exponent for damage type (from *Table A7.5.2*).

APPENDIX 7.6 EXAMPLE OF DESIGN TRAFFIC CALCULATIONS

Design parameters:

Design Period:	20 years
Annual Average Daily Traffic (AADT):	5350
Direction Factor	0.5
Percentage Heavy Vehicles:	8%
Lane Distribution Factor:	1.0
Heavy Vehicle Growth Rate (compound):	4%
Traffic Axle Load Distribution:	*Table A7.6.1*

A7.6.1 Total number of heavy vehicle axle groups

The cumulative number of heavy vehicle axle groups (N_{DT}) anticipated in the design lane over the design period is:

$$N_{DT} = 365 \times (AADT * DF) \times \%HV/100 \times LDF \times CGF \times N_{HVAG} \qquad (A7.6.1)$$

where

AADT = Annual Average Daily Traffic in vehicles per day in the first year (Section 7.4.4);

DF = Direction Factor (Section 7.4.4);

%HV = average percentage of all traffic comprising Heavy Vehicles (Section 7.4.4);

LDF = Lane Distribution Factor (Section 7.4.3); and

CGF = Cumulative Growth Factor (Section 7.4.5).

N_{HVAG} = average number of Axle Groups per Heavy Vehicle (Section 7.4.6)

From the above-mentioned design parameters:

- AADT = 5350
- % HV = 8%
- DF = 0.5
- LDF = 1.0

The Cumulative Growth Factor (CGF) is calculated as follows using the R the annual growth rate (4%) and the Design Period P (20 years):

Cumulative Growth Factor (CGF)

$$= \frac{(1+0.01R)^P - 1}{0.01R} \quad \text{for } R > 0 \qquad (A7.6.2)$$

$$= P \quad \text{for } R = 0$$

Hence $CGF = \{(1+0.01 \times 4)^{20} - 1\} / (0.01 \times 4) = 29.8$

Using equation A7.6.1, the cumulative traffic loading is:

$$N_{DT} = 365 \times (5350 \times 0.5) \times 8/100 \times 1.0 \times 29.8 \times N_{HVAG} \qquad (A7.6.3)$$

$$N_{DT} = 1.16 \times 10^6 \text{ heavy vehicles} \times N_{HVAG}$$

As stated in Section 7.4.6, the average number of axle groups per heavy vehicle (N_{HVAG}) may be obtained either from:

- weigh-in-motion survey data
- vehicle classification counts
- presumptive values (e.g. *Table 7.5*)

In this example weigh-in-motion data given in *Table A7.6.1* is available to calculate N_{HVAG}.

From *Table A7.6.1*, it is noted that the sum of the proportions of axles which are single axle single tyre (SAST) and tandem axle with single tyres (TAST) is 0.3799 + 0.0216 = 0.40. To estimate the number of average number of axle groups per heavy vehicle, it is assumed that there is one SAST or one TAST (steer axles) per heavy vehicle. Therefore on average 100 axle groups of loading result from the passage of 40 heavy vehicles. Thus, average number of axle groups per heavy vehicle is 100/40 = 2.5

Using equation A7.6.3, the cumulative heavy vehicle axle groups is:

$$N_{DT} = 1.16 \times 10^6 \times 2.5 = 2.9 \times 10^6 \qquad (A7.6.4)$$

Table A7.6.1 Representative distribution of loads on axle groups for use on urban roads

Axle group load (kN)	Axle group type SAST %	SADT %	TAST %	TADT %	TRDT %
10	0.2569	2.1791	0.1033	0.0971	0.0043
20	13.5274	10.2319	0.9558	0.6798	0.1057
30	18.0167	20.6747	1.2562	1.4088	0.2529
40	19.9923	17.9923	1.3315	3.7622	1.0424
50	25.7379	13.4201	4.5162	7.7252	4.9203
60	17.1140	8.2995	13.6576	10.3152	9.4372
70	4.3708	6.2664	17.9501	10.2244	9.7940
80	0.7690	7.6773	17.3598	8.5571	8.6152
90	0.1182	6.3741	13.2328	6.7596	6.5257
100	0.0573	3.5792	9.9221	5.3419	4.3467
110	0.0128	1.6833	9.7695	4.3809	3.1213
120	0.0128	0.9164	4.6565	4.1481	2.7006
130	0.0086	0.4354	2.3255	4.2917	2.4734
140	0.0053	0.1888	1.1946	4.7138	2.6452
150		0.0486	0.8719	6.1501	3.0875
160		0.0250	0.3289	5.7139	3.4186
170		0.0000	0.3108	4.9741	3.8058
180		0.0079	0.1268	3.3997	4.9435
190			0.1025	2.6397	6.2365
200			0.0278	1.7043	7.2185
210				1.1941	5.2375
220				0.8293	3.7047
230				0.4222	2.0195
240				0.2111	1.4500
250				0.1620	0.8953
260				0.0753	0.6025
270				0.0752	0.6229
280				0.0137	0.3055
290				0.0094	0.1953
300				0.0000	0.1616
310				0.0110	0.0409
320				0.0045	0.0257
330					0.0254
340					0.0181
350					
360					
370					
380					
390					
400					
Total	100.00	100.00	100.00	100.00	100.00
Proportion of each axle group	0.3799	0.2171	0.0216	0.2591	0.1223

A7.6.2 Design traffic for flexible pavements

A7.6.2.1 Estimating standard axle repetitions per heavy vehicle axle group

In the flexible pavement design process, the next step in assessing the ability of a pavement configuration to withstand the design traffic is to determine the average number of Standard Axle Repetitions (SAR) per heavy vehicle axle group (SAR/HVAG). These SAR/HVAG values are determined for each of the damage types that are relevant to the pavement configuration.

To calculate the SAR/HVAG of damage, a procedure is required to calculate the damage associated with each axle group type and load in the traffic load distribution relative to the damage caused by a Standard Axle. As discussed in Section 7.6.2, the average SAR/HVAG for each damage type are calculated as follows:

- The Traffic Load Distribution (TLD) for the project (*Table A7.6.1*) is used to calculate the proportion of axles of each axle group type and load. These proportions are given in *Table A7.6.2*.
- The damage associated with each axle group load of each axle group type. As discussed in Section 7.6.2, it is necessary to calculate the SAR of damage for each distress type. In this example, damage has been calculated for damage exponents (m) of 4, 5, 7 and 12. These damage values are given in *Tables A7.6.3*.
- The average SAR/HVAG for each distress type, is then calculated by multiplying the SAR values for each axle group load of each axle group type (*Table A7.6.3*) by its frequency of occurrence (*Table A7.6.2*) in the TLD. These values are given in *Table A7.6.4*. The average SAR/HVAG for each distress type is the sum of the weighted damage values in these tables. These values are summarised in *Table A7.6.5*.

A7.6.2.2 Estimating Standard Axle Repetitions per ESA

The next step in assessing the ability of a pavement configuration to withstand the design traffic is to determine the average number of Standard Axle Repetitions (SAR) per Equivalent Standard Axle (SAR/ESA) for each distress mode. These SAR/ESA values are calculated by dividing the number of SAR/HVAG by the ESA/HVAG (*Table A7.6.5*).

The SAR/ESA for the three distress modes are given in *Table A7.6.6*.

A7.6.2.3 Design traffic calculation

The design traffic for flexible pavement design is – for each relevant damage type – the total number of Standard Axle Repetitions (SAR) during the design period which cause the same damage as the cumulative traffic.

Firstly, the design number of ESA of loading (DESA) is calculated using equation 7.4:

$$\begin{aligned} \text{DESA} &= \text{ESA/HVAG} \times N_{DT} \\ &= 0.80 \times 2.9 \times 10^6 \\ &= 2.3 \times 10^6 \end{aligned}$$

The DESA is the design traffic for unbound granular pavements with thin bituminous surfacings (*Figure. 8.4*) and for the mechanistic design of pavements incorporating a cemented layer in which the post-cracking phase of life is considered (Section 8.2.4).

For the mechanistic design of all other flexible pavements, the design number of Standard Axle Repetitions (DSAR) for each distress type needs to be estimated. The DSAR values are estimated by multiplying the DESA (2.3×10^6 ESA) by the SAR/ESA given in *Table A7.6.6*. The values are given in *Table A7.6.7*.

Table A7.6.2 Project traffic axle load distribution by proportion of axle groups of each type and load

Axle group load (kN)	Axle group type				
	SAST	SADT	TAST	TADT	TRDT
10	0.000976	0.00473	2.23E-05	0.000252	5.22E-06
20	0.051395	0.022208	0.000206	0.001761	0.000129
30	0.068452	0.044875	0.000271	0.00365	0.000309
40	0.075958	0.039052	0.000288	0.009747	0.001275
50	0.097787	0.029129	0.000975	0.020013	0.00602
60	0.065022	0.018014	0.00295	0.026723	0.011546
70	0.016606	0.013601	0.003877	0.026488	0.011983
80	0.002922	0.016664	0.003749	0.022169	0.01054
90	0.000449	0.013835	0.002858	0.017512	0.007984
100	0.000218	0.007769	0.002143	0.013839	0.005318
110	4.86E-05	0.003654	0.00211	0.011349	0.003819
120	4.86E-05	0.001989	0.001006	0.010746	0.003304
130	3.26E-05	0.000945	0.000502	0.011118	0.003026
140	2E-05	0.00041	0.000258	0.012212	0.003236
150	0	0.000105	0.000188	0.015933	0.003777
160	0	5.43E-05	7.1E-05	0.014803	0.004183
170	0	0	6.71E-05	0.012886	0.004656
180	0	1.71E-05	2.74E-05	0.008807	0.006048
190	0	0	2.21E-05	0.006838	0.00763
200	0	0	0.000006	0.004415	0.008832
210	0	0	0	0.003094	0.006408
220	0	0	0	0.002148	0.004533
230	0	0	0	0.001094	0.002471
240	0	0	0	0.000547	0.001774
250	0	0	0	0.00042	0.001095
260	0	0	0	0.000195	0.000737
270	0	0	0	0.000195	0.000762
280	0	0	0	3.56E-05	0.000374
290	0	0	0	2.44E-05	0.000239
300	0	0	0	0	0.000198
310	0	0	0	2.86E-05	0.00005
320	0	0	0	1.17E-05	3.14E-05
330	0	0	0	0	3.11E-05
340	0	0	0	1.17E-05	2.22E-05
350	0	0	0	0	0
360	0	0	0	0	0
370	0	0	0	0	0
380	0	0	0	0	0
390	0	0	0	0	0
400	0	0	0	0	0
Proportion of each axle group	0.3799	0.2171	0.0216	0.2591	0.1223

Table A7.6.3 SAR for each axle group type

Damage exponent of 4

Axle group load (kN)	Axle group type				
	SAST	SADT	TAST	TADT	TRDT
10	0.001	0.000	0.000	0.000	0.000
20	0.020	0.004	0.002	0.000	0.000
30	0.103	0.020	0.012	0.002	0.001
40	0.324	0.063	0.039	0.008	0.002
50	0.792	0.153	0.095	0.019	0.006
60	1.642	0.316	0.198	0.039	0.012
70	3.043	0.586	0.366	0.072	0.022
80	5.191	1.000	0.624	0.123	0.038
90	8.315	1.602	1.000	0.198	0.061
100	12.673	2.441	1.524	0.301	0.093
110	18.555	3.574	2.232	0.441	0.136
120	26.280	5.063	3.160	0.624	0.193
130	36.197	6.973	4.353	0.860	0.266
140	48.687	9.379	5.855	1.157	0.358
150	64.160	12.360	7.716	1.524	0.472
160	83.057	16.000	9.989	1.973	0.611
170	105.85	20.391	12.730	2.515	0.778
180	133.04	25.629	16.000	3.160	0.978
190	165.16	31.817	19.863	3.924	1.214
200	202.78	39.063	24.387	4.817	1.491
210	246.48	47.481	29.642	5.855	1.812
220	296.88	57.191	35.704	7.053	2.183
230	354.66	68.321	42.652	8.425	2.607
240	420.48	81.000	50.568	9.989	3.091
250	495.06	95.367	59.537	11.760	3.640
260	579.15	111.57	69.650	13.758	4.258
270	673.52	129.75	81.000	16.000	4.952
280	778.98	150.06	93.683	18.505	5.727
290	896.37	172.68	107.80	21.294	6.590
300	1026.6	197.75	123.46	24.387	7.547
310	1170.4	225.47	140.76	27.804	8.605
320	1328.9	256.00	159.82	31.569	9.770
330	1503.0	289.53	180.75	35.704	11.049
340	1693.6	326.25	203.68	40.233	12.451
350	1901.8	366.36	228.72	45.179	13.982
360	2128.7	410.06	256.00	50.568	15.649
370	2375.2	457.56	285.65	56.425	17.462
380	2642.6	509.07	317.81	62.777	19.428
390	2931.9	564.80	352.60	69.650	21.555
400	3244.4	625.00	390.18	77.073	23.852

Damage exponent of 5 – Asphalt fatigue

Axle group load (kN)	Axle group type				
	SAST	SADT	TAST	TADT	TRDT
10	0.000	0.000	0.000	0.000	0.000
20	0.008	0.001	0.001	0.000	0.000
30	0.058	0.007	0.004	0.001	0.000
40	0.245	0.031	0.017	0.002	0.001
50	0.747	0.095	0.053	0.007	0.002
60	1.859	0.237	0.132	0.017	0.004
70	4.019	0.513	0.285	0.037	0.009
80	7.836	1.000	0.555	0.073	0.017
90	14.120	1.802	1.000	0.132	0.030
100	23.912	3.052	1.694	0.223	0.051
110	38.511	4.915	2.727	0.359	0.083
120	59.501	7.594	4.214	0.555	0.128
130	88.785	11.331	6.288	0.828	0.191
140	128.61	16.413	9.108	1.199	0.277
150	181.58	23.174	12.860	1.694	0.391
160	250.74	32.000	17.758	2.338	0.540
170	339.52	43.331	24.045	3.166	0.731
180	451.84	57.665	32.000	4.214	0.973
190	592.09	75.565	41.933	5.522	1.275
200	765.19	97.656	54.192	7.136	1.647
210	976.60	124.64	69.165	9.108	2.102
220	1232.3	157.28	87.277	11.493	2.653
230	1539.1	196.42	109.00	14.354	3.313
240	1904.0	243.00	134.85	17.758	4.099
250	2335.2	298.02	165.38	21.779	5.027
260	2841.1	362.59	201.21	26.497	6.116
270	3431.1	437.89	243.00	32.000	7.386
280	4115.4	525.22	291.46	38.381	8.859
290	4904.7	625.95	347.36	45.743	10.558
300	5810.7	741.58	411.52	54.192	12.509
310	6845.9	873.69	484.84	63.847	14.737
320	8023.6	1024.0	568.25	74.831	17.273
330	9358.2	1194.3	662.76	87.277	20.145
340	1.086E+04	1386.6	769.45	101.33	23.388
350	1.256E+04	1602.8	889.46	117.13	27.036
360	1.446E+04	1845.3	1024.0	134.85	31.126
370	1.658E+04	2116.2	1174.3	154.65	35.696
380	1.895E+04	2418.1	1341.9	176.71	40.787
390	2.157E+04	2753.4	1528.0	201.21	46.444
400	2.449E+04	3125.0	1734.2	228.37	52.712

Damage exponent of 7 – Rutting and loss of surface shape

Axle group load (kN)	Axle group type				
	SAST	SADT	TAST	TADT	TRDT
10	0.000	0.000	0.000	0.000	0.000
20	0.001	0.000	0.000	0.000	0.000
30	0.019	0.001	0.000	0.000	0.000
40	0.139	0.008	0.003	0.000	0.000
50	0.665	0.037	0.016	0.001	0.000
60	2.383	0.133	0.059	0.003	0.000
70	7.011	0.393	0.172	0.010	0.001
80	17.852	1.000	0.438	0.026	0.003
90	40.716	2.281	1.000	0.059	0.008
100	85.127	4.768	2.091	0.122	0.016
110	165.89	9.292	4.074	0.238	0.031
120	305.03	17.086	7.492	0.438	0.056
130	534.16	29.921	13.119	0.768	0.099
140	897.36	50.265	22.039	1.290	0.166
150	1454.5	81.472	35.722	2.091	0.268
160	2285.1	128.00	56.123	3.285	0.422
170	3493.1	195.66	85.792	5.021	0.645
180	5211.7	291.93	128.000	7.492	0.962
190	7609.3	426.23	186.886	10.938	1.405
200	1.090E+04	610.35	267.616	15.663	2.011
210	1.533E+04	858.83	376.563	22.039	2.830
220	2.123E+04	1189.4	521.508	30.523	3.919
230	2.898E+04	1623.5	711.864	41.664	5.350
240	3.904E+04	2187.0	958.917	56.123	7.207
250	5.196E+04	2910.4	1276.093	74.687	9.590
260	6.837E+04	3829.9	1679.252	98.283	12.620
270	8.905E+04	4987.9	2187.000	128.00	16.436
280	1.149E+05	6433.9	2821.036	165.11	21.201
290	1.468E+05	8225.4	3606.521	211.08	27.104
300	1.862E+05	1.043E+04	4572.474	267.62	34.364
310	2.342E+05	1.312E+04	5752.204	336.66	43.230
320	2.925E+05	1.638E+04	7183.768	420.45	53.988
330	3.628E+05	2.032E+04	8910.458	521.51	66.965
340	4.471E+05	2.505E+04	1.098E+04	642.71	82.528
350	5.477E+05	3.068E+04	1.345E+04	787.30	101.09
360	6.671E+05	3.737E+04	1.638E+04	958.92	123.13
370	8.081E+05	4.527E+04	1.985E+04	1161.7	149.16
380	9.740E+05	5.456E+04	2.392E+04	1400.1	179.78
390	1.168E+06	6.544E+04	2.869E+04	1679.3	215.63
400	1.395E+06	7.813E+04	3.425E+04	2004.9	257.44

Damage exponent of 12 – Cemented materials fatigue

Axle group load (kN)	Axle group type				
	SAST	SADT	TAST	TADT	TRDT
10	0.000	0.000	0.000	0.000	0.000
20	0.000	0.000	0.000	0.000	0.000
30	0.001	0.000	0.000	0.000	0.000
40	0.034	0.000	0.000	0.000	0.000
50	0.497	0.004	0.001	0.000	0.000
60	4.431	0.032	0.008	0.000	0.000
70	28.175	0.201	0.049	0.000	0.000
80	139.884	1.000	0.243	0.002	0.000
90	574.910	4.110	1.000	0.008	0.000
100	2035.6	14.552	3.541	0.027	0.001
110	6388.5	45.670	11.112	0.086	0.003
120	18149.5	129.75	31.569	0.243	0.007
130	4.743E+04	339.03	82.492	0.636	0.019
140	1.154E+05	825.01	200.736	1.547	0.046
150	2.641E+05	1888.1	459.394	3.541	0.105
160	5.730E+05	4096.0	996.620	7.681	0.228
170	1.186E+06	8478.3	2062.9	15.899	0.471
180	2.355E+06	1.683E+04	4096.0	31.569	0.936
190	4.505E+06	3.221E+04	7836.7	60.400	1.790
200	8.338E+06	5.960E+04	1.450E+04	111.78	3.313
210	1.497E+07	1.070E+05	2.604E+04	200.74	5.950
220	2.617E+07	1.871E+05	4.552E+04	350.81	10.398
230	4.461E+07	3.189E+05	7.759E+04	598.04	17.725
240	7.434E+07	5.314E+05	1.293E+05	996.62	29.539
250	1.213E+08	8.674E+05	2.110E+05	1626.6	48.210
260	1.943E+08	1.389E+06	3.379E+05	2604.2	77.186
270	3.055E+08	2.184E+06	5.314E+05	4096.0	121.401
280	4.727E+08	3.379E+06	8.222E+05	6337.1	187.825
290	7.202E+08	5.149E+06	1.253E+06	9655.4	286.176
300	1.082E+09	7.733E+06	1.882E+06	1.450E+04	429.845
310	1.603E+09	1.146E+07	2.789E+06	2.149E+04	637.085
320	2.347E+09	1.678E+07	4.082E+06	3.146E+04	932.517
330	3.395E+09	2.427E+07	5.906E+06	4.552E+04	1349.0
340	4.858E+09	3.473E+07	8.450E+06	6.512E+04	1930.2
350	6.879E+09	4.917E+07	1.196E+07	9.222E+04	2733.2
360	9.645E+09	6.895E+07	1.678E+07	1.293E+05	3832.5
370	1.340E+10	9.579E+07	2.331E+07	1.796E+05	5324.5
380	1.845E+10	1.319E+08	3.210E+07	2.474E+05	7332.6
390	2.520E+10	1.802E+08	4.384E+07	3.379E+05	1.001E+04
400	3.415E+10	2.441E+08	5.940E+07	4.578E+05	1.357E+04

Table A7.6.4 SAR for each axle group type weighted by proportion of axle loads of each type and each load in the project TLD (*Table A7.6.2*)

Damage exponent of 4

Axle group load (kN)	Axle group type SAST	SADT	TAST	TADT	TRDT
10	0.00000	0.00000	0.00000	0.00000	0.00000
20	0.00104	0.00009	0.00000	0.00000	0.00000
30	0.00703	0.00089	0.00000	0.00001	0.00000
40	0.02464	0.00244	0.00001	0.00008	0.00000
50	0.07746	0.00444	0.00009	0.00038	0.00004
60	0.10680	0.00570	0.00058	0.00104	0.00014
70	0.05053	0.00797	0.00142	0.00191	0.00027
80	0.01517	0.01666	0.00234	0.00273	0.00040
90	0.00373	0.02216	0.00286	0.00346	0.00049
100	0.00276	0.01897	0.00327	0.00417	0.00050
110	0.00090	0.01306	0.00471	0.00500	0.00052
120	0.00128	0.01007	0.00318	0.00671	0.00064
130	0.00118	0.00659	0.00219	0.00956	0.00081
140	0.00097	0.00384	0.00151	0.01412	0.00116
150	0	0.00130	0.00145	0.02428	0.00178
160	0	0.00087	0.00071	0.02921	0.00255
170	0	0.00000	0.00085	0.03240	0.00362
180	0	0.00044	0.00044	0.02784	0.00592
190	0	0	0.00044	0.02683	0.00926
200	0	0	0.00015	0.02127	0.01317
210	0	0	0	0.01811	0.01161
220	0	0	0	0.01515	0.00989
230	0	0	0	0.00922	0.00644
240	0	0	0	0.00546	0.00548
250	0	0	0	0.00493	0.00399
260	0	0	0	0.00269	0.00314
270	0	0	0	0.00312	0.00377
280	0	0	0	0.00066	0.00214
290	0	0	0	0.00052	0.00157
300	0	0	0	0.00000	0.00149
310	0	0	0	0.00079	0.00043
320	0	0	0	0.00037	0.00031
330	0	0	0	0.00000	0.00034
340	0	0	0	0.00047	0.00028
350	0	0	0	0	0
360	0	0	0	0	0
370	0	0	0	0	0
380	0	0	0	0	0
390	0	0	0	0	0
400	0	0	0	0	0

Average SAR/HVAG: 0.80

Damage exponent of 5 – Asphalt fatigue

Axle group load (kN)	Axle group type SAST	SADT	TAST	TADT	TRDT
10	0.00000	0.00000	0.00000	0.00000	0.00000
20	0.00039	0.00002	0.00000	0.00000	0.00000
30	0.00398	0.00033	0.00000	0.00000	0.00000
40	0.01860	0.00122	0.00000	0.00002	0.00000
50	0.07307	0.00278	0.00005	0.00014	0.00001
60	0.12090	0.00427	0.00039	0.00046	0.00005
70	0.06674	0.00698	0.00110	0.00099	0.00010
80	0.02289	0.01666	0.00208	0.00162	0.00018
90	0.00634	0.02493	0.00286	0.00231	0.00024
100	0.00521	0.02371	0.00363	0.00309	0.00027
110	0.00187	0.01796	0.00575	0.00408	0.00032
120	0.00289	0.01510	0.00424	0.00596	0.00042
130	0.00290	0.01071	0.00316	0.00921	0.00058
140	0.00257	0.00673	0.00235	0.01465	0.00090
150	0	0.00244	0.00242	0.02698	0.00148
160	0	0.00174	0.00126	0.03462	0.00226
170	0	0.00000	0.00161	0.04080	0.00340
180	0	0.00099	0.00088	0.03711	0.00588
190	0	0	0.00093	0.03776	0.00973
200	0	0	0.00033	0.03151	0.01455
210	0	0	0	0.02818	0.01347
220	0	0	0	0.02469	0.01202
230	0	0	0	0.01570	0.00819
240	0	0	0	0.00971	0.00727
250	0	0	0	0.00914	0.00551
260	0	0	0	0.00517	0.00451
270	0	0	0	0.00623	0.00563
280	0	0	0	0.00137	0.00331
290	0	0	0	0.00112	0.00252
300	0	0	0	0.00000	0.00247
310	0	0	0	0.00182	0.00074
320	0	0	0	0.00088	0.00054
330	0	0	0	0.00000	0.00063
340	0	0	0	0.00119	0.00052
350	0	0	0	0	0
360	0	0	0	0	0
370	0	0	0	0	0
380	0	0	0	0	0
390	0	0	0	0	0
400	0	0	0	0	0

Average SAR/HVAG: 0.96

Damage exponent of 7 – Rutting and loss of surface shape

Axle group load (kN)	Axle group type SAST	SADT	TAST	TADT	TRDT
10	0.00000	0.00000	0.00000	0.00000	0.00000
20	0.00006	0.00000	0.00000	0.00000	0.00000
30	0.00127	0.00005	0.00000	0.00000	0.00000
40	0.01059	0.00031	0.00000	0.00000	0.00000
50	0.06503	0.00109	0.00002	0.00002	0.00000
60	0.15495	0.00240	0.00017	0.00009	0.00001
70	0.11642	0.00534	0.00067	0.00027	0.00002
80	0.05216	0.01666	0.00164	0.00057	0.00003
90	0.01829	0.03155	0.00286	0.00102	0.00006
100	0.01853	0.03704	0.00448	0.00169	0.00008
110	0.00807	0.03395	0.00860	0.00271	0.00012
120	0.01483	0.03398	0.00753	0.00471	0.00019
130	0.01742	0.02828	0.00659	0.00854	0.00030
140	0.01791	0.02060	0.00569	0.01575	0.00054
150	0	0.00859	0.00673	0.03331	0.00101
160	0	0.00695	0.00399	0.04862	0.00176
170	0	0.00000	0.00576	0.06470	0.00300
180	0	0.00500	0.00351	0.06598	0.00582
190	0	0	0.00414	0.07480	0.01072
200	0	0	0.00161	0.06916	0.01776
210	0	0	0	0.06818	0.01813
220	0	0	0	0.06557	0.01776
230	0	0	0	0.04557	0.01322
240	0	0	0	0.03070	0.01278
250	0	0	0	0.03134	0.01051
260	0	0	0	0.01918	0.00930
270	0	0	0	0.02493	0.01252
280	0	0	0	0.00588	0.00792
290	0	0	0	0.00515	0.00648
300	0	0	0	0.00000	0.00680
310	0	0	0	0.00962	0.00216
320	0	0	0	0.00494	0.00169
330	0	0	0	0.00000	0.00208
340	0	0	0	0.00754	0.00183
350	0	0	0	0	0
360	0	0	0	0	0
370	0	0	0	0	0
380	0	0	0	0	0
390	0	0	0	0	0
400	0	0	0	0	0

Average SAR/HVAG: 1.67

Damage exponent of 12 – Cemented materials fatigue

Axle group load (kN)	Axle group type SAST	SADT	TAST	TADT	TRDT
10	0.00000	0.00000	0.00000	0.00000	0.00000
20	0.00000	0.00000	0.00000	0.00000	0.00000
30	0.00007	0.00000	0.00000	0.00000	0.00000
40	0.00259	0.00001	0.00000	0.00000	0.00000
50	0.04860	0.00010	0.00000	0.00000	0.00000
60	0.28811	0.00057	0.00002	0.00000	0.00000
70	0.46789	0.00274	0.00019	0.00001	0.00000
80	0.40870	0.01666	0.00091	0.00004	0.00000
90	0.25823	0.05686	0.00286	0.00013	0.00000
100	0.44312	0.11305	0.00759	0.00038	0.00000
110	0.31065	0.16686	0.02345	0.00097	0.00001
120	0.88255	0.25807	0.03175	0.00261	0.00002
130	1.54705	0.32041	0.04143	0.00707	0.00006
140	2.30394	0.33807	0.05179	0.01889	0.00015
150	0	0.19915	0.08651	0.05641	0.00040
160	0	0.22225	0.07080	0.11370	0.00095
170	0	0.00000	0.13848	0.20488	0.00219
180	0	0.28838	0.11219	0.27805	0.00566
190	0	0	0.17343	0.41304	0.01366
200	0	0	0.08702	0.49353	0.02926
210	0	0	0	0.62100	0.03812
220	0	0	0	0.75367	0.04713
230	0	0	0	0.65415	0.04380
240	0	0	0	0.54515	0.05240
250	0	0	0	0.68246	0.05281
260	0	0	0	0.50827	0.05690
270	0	0	0	0.79765	0.09251
280	0	0	0	0.22566	0.07021
290	0	0	0	0.23551	0.06840
300	0	0	0	0.00000	0.08500
310	0	0	0	0.61401	0.03185
320	0	0	0	0.36934	0.02927
330	0	0	0	0.00000	0.04194
340	0	0	0	0.76450	0.04280
350	0	0	0	0	0
360	0	0	0	0	0
370	0	0	0	0	0
380	0	0	0	0	0
390	0	0	0	0	0
400	0	0	0	0	0

Average SAR/HVAG: 18.9

Table A7.6.5 Average SAR per axle group for project traffic load distribution

Pavement	Damage type	Damage exponent (m)	Damage Index	Value
Granular pavements using *Figure 8.4*	Overall damage	4	ESA/HVAG	0.80
Mechanistic design of flexible pavements	Fatigue of asphalt	5	SAR_a/HVAG	0.96
	Rutting and shape loss	7	SAR_s/HVAG	1.67
	Fatigue of cemented materials	12	SAR_c/HVAG	18.9

Table A7.6.6 Average SAR per ESA for project traffic load distribution

Damage type	Damage exponent (m)	Damage Index	Value
Fatigue of asphalt	5	SAR_a/ESA	$0.96/0.80 = 1.2$
Rutting and shape loss	7	SAR_s/ESA	$1.67/0.80 = 2.1$
Fatigue of cemented materials	12	SAR_c/ESA	$18.9/0.80 = 23.6$

Table A7.6.7 Design Standard Axle Repetitions (DSAR)

Damage type	Damage unit	Value
Fatigue of asphalt	$DSAR_a$	$1.2 \times 2.3 \times 10^6 = 2.8 \times 10^6$
Rutting and shape loss	$DSAR_s$	$2.1 \times 2.3 \times 10^6 = 4.8 \times 10^6$
Fatigue of cemented materials	$DSAR_c$	$23.6 \times 2.3 \times 10^6 = 5.4 \times 10^7$

A7.6.3 Design traffic for rigid pavements

For the design of rigid pavements, the cumulative heavy vehicle axle groups (HVAG), together with the load safety factor and the traffic load distribution (TLD), are required to characterise the Design Traffic for all rigid pavement types. Unlike flexible pavements, the Design Traffic is not increased due to considerations of project reliability, but a Load Safety Factor is used to increase the axle group loads of all six axle group configurations.

The Design Traffic (N_{DT}) for rigid pavement design is the cumulative number of axle groups over the Design Period and referred to as HVAG, classified according to the type of axle group and the load on the specific axle group type, i.e. the cumulative number of HVAG applied at each axle group load for each axle group type.

Consequently, the design traffic for rigid pavements comprises:

- cumulative HVAG (2.9×10^6, Section A7.6.1); and
- the project Traffic Load Distribution (*Table A7.6.1*), with axle groups loads increase by the selected Load Safety Factor for the desired project reliability (Section 9.3.6).

APPENDIX 8.1 EXAMPLES OF USE OF DESIGN CHARTS

Design parameters:

To illustrate the use of the example design charts given in Section 8.4, consider a project with the following design parameters:

- Design Subgrade CBR = 5%
- Design traffic for 20 year design life = 1×10^7 ESA. The traffic distribution is in accordance with *Table A7.4.3* Urban roads.

1. Granular pavement with sprayed seal

Design traffic = 1×10^7 ESA

Subgrade CBR = 5%

Select the pavement composition from *Figure 8.4*.

200 mm Granular base
290 mm Granular subbase
Subgrade CBR 5

2. Granular pavement with asphalt wearing surface

Design traffic = 1×10^7 ESA

Subgrade CBR = 5%

From Chart EC02 select design. Pavement cannot be designed with less than 215 mm of asphalt for Design traffic = 1×10^7 ESA. Alternative designs are given below.

215 mm Asphalt		240 mm Asphalt
500 mm Granular	or	100 mm Granular
Subgrade CBR 5		Subgrade CBR 5

3. Asphalt surfaced pavement on cemented base

Design traffic = 1×10^7 ESA

Subgrade CBR = 5%

Design from Chart EC09 – Cemented Material Modulus = 2000 MPa

100 mm Asphalt		220 mm Asphalt
320 mm Cemented material	or	200 mm Cemented material
Subgrade CBR 5		Subgrade CBR 5

Design from Chart EC10 – Cemented Material Modulus = 2000 MPa

As the cover over the cemented material is 175 mm or more, the second phase of life of the pavement cracking of the cemented material has been considered.

175 mm Asphalt		205 mm Asphalt
150 mm Cemented material	or	100 mm Cemented material
Subgrade CBR 5		Subgrade CBR 5

Design from Chart EC15 – Cemented Material Modulus = 5000 MPa

100 mm Asphalt		180 mm Asphalt
265 mm Cemented material	or	200 mm Cemented material
Subgrade CBR 5		Subgrade CBR 5

Design from Chart EC16 – Cemented Material Modulus = 5000 MPa

As the cover over the cemented material is 175 mm or more, the second phase of life of the pavement cracking of the cemented material has been considered.

175 mm Asphalt		205 mm Asphalt
150 mm Cemented material	or	100 mm Cemented material
Subgrade CBR 5		Subgrade CBR 5

4. Full depth asphalt

Design traffic = 1×10^7 ESA

Subgrade CBR = 5%

Design from Chart EC05 Asphalt Modulus = 3000 MPa

240 mm Asphalt
Subgrade CBR 5

5. Asphalt wearing surface, granular base and cemented subbase

Design traffic = 1×10^7 ESA

Subgrade CBR = 5%

Design from Chart EC21 Cemented Subbase Modulus = 5000 MPa

125 mm Asphalt		200 mm Asphalt
100 mm Granular base	or	100 mm Granular base
300 mm Cemented subbase		100 mm Cemented subbase
Subgrade CBR 5		Subgrade CBR 5

Design from Chart EC22 Cemented Subbase Modulus = 5000 MPa

As the cover over the cemented material is 175 mm or more, the second phase of life of the pavement cracking of the cemented material has been considered.

140 mm Asphalt		190 mm Asphalt
100 mm Granular base	or	100 mm Granular base
200 mm Cemented subbase		100 mm Cemented subbase
Subgrade CBR 5		Subgrade CBR 5

APPENDIX 8.2: EFFECT OF ASPHALT THICKNESS ON FATIGUE LIFE OF ASPHALT-SURFACED PAVEMENTS

Some of the example design charts which are contained in this Guide indicate that, for a given design traffic, asphalt stiffness, thickness of granular or cemented base or subbase, and subgrade stiffness, two thicknesses of asphalt provide the same theoretical fatigue performance. An example of this is Chart EC01, where a fatigue life of 2×10^5 ESA can be obtained by using either 40 mm or 70 mm of asphalt on 440 mm of granular material.

The reason for this can be explained by examining the relationship between asphalt thickness and horizontal strain at the bottom of the asphalt, induced by a Standard Axle load for a given composition of underlying material. A typical relationship is illustrated in *Figure A8.2.1*.

As asphalt thickness is increased, horizontal strain increases from a negative value (i.e. compressive) – at zero asphalt thickness – to low positive values (tension). Asphalt layers which are relatively thin, and represent only a small proportion of the overall pavement stiffness, offer little resistance to the flexure of the underlying structure. In this range of thickness, the greater the depth of asphalt, the greater themagnitude of tensile strain induced at its underside.

With further increases in thickness, the asphalt layer begins to exert an influence on the total pavement structure. A peak strain level is reached usually in the range of 40–80 mm for highway traffic loading. Further increases in asphalt thickness reduce the flexure of the structure and the resulting strain in the asphalt.

Therefore, there are generally two asphalt thicknesses which give the same magnitude of strain, one to the left of the maximum point and one to the right.

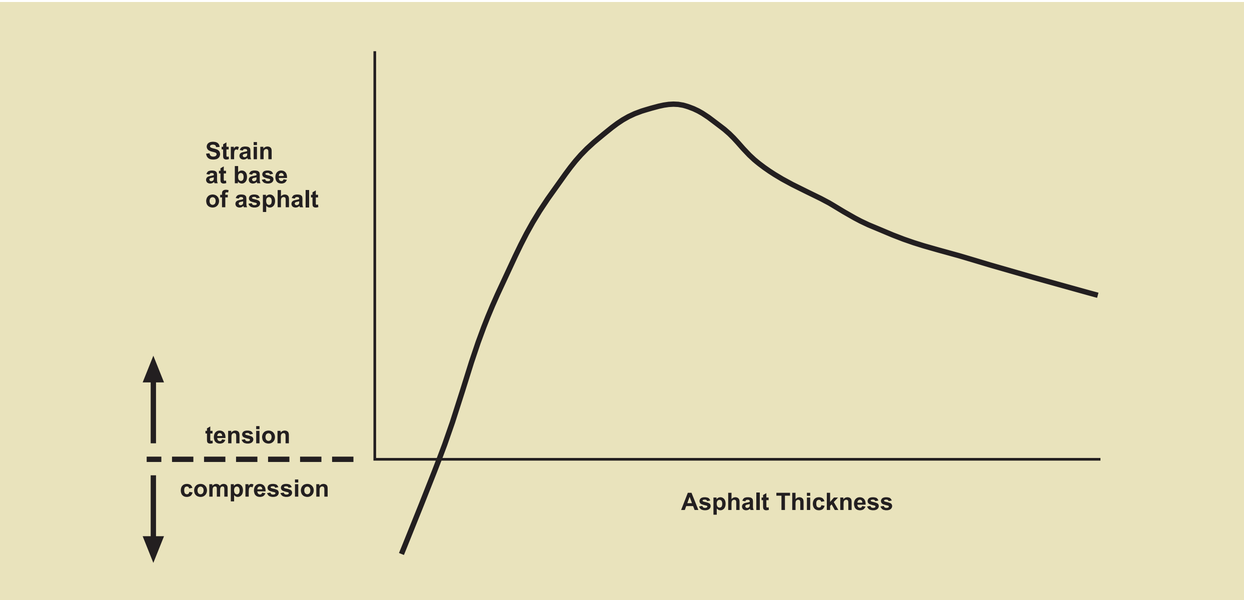

Figure A8.2.1 General relationship between asphalt thickness and horizontal strain at the base of an asphalt layer

APPENDIX 8.3 EXAMPLES OF USE OF MECHANISTIC PROCEDURE FOR FLEXIBLE PAVEMENTS

This Appendix gives examples of the use of mechanistic procedures for the design of the following three flexible pavement types:

- sprayed seal surfaced unbound granular pavement,
- asphalt pavement containing a cemented material subbase, and
- full depth asphalt pavement,

using the following Design Parameters:

- Design Subgrade CBR = 5%
- Design traffic for 30 year design life = 10^7 HVAG. The traffic distribution is in accordance with the presumptive Urban distribution *Appendix 7.4 Table A7.4.3*.
- Using a Weighted Mean Annual Pavement Temperature = 28°C and design traffic speed 60km/h, the following asphalt design moduli were calculated:

  Asphalt modulus size 14 mm mix = 2200 MPa

  Asphalt modulus size 20 mm mix = 2500 MPa
- Desired project reliability = 97.5%.

A8.3.1 Unbound granular pavement

Following the steps in *Table 8.1*

Step

1. Try pavement composition of:

Material Type	Thickness (mm)
Sprayed seal surface	–
Unbound granular material	475
Subgrade, CBR = 5%	Semi-infinite

2. Subgrade

   CBR = 5%

   E_V = 50 MPa – Section 5.6

   E_H = 25 MPa

   $\nu_V = \nu_H = 0.45$ – Section 5.6

   f = 34.5

3. Top granular sublayer

   Minimum of:

   - $E_{V\text{ top of base}}$

     $= E_{V\text{ subgrade}} \times 2^{(\text{total granular thickness}/125)}$

     = 696 MPa – Section 8.2.3
   - $E_{V\text{ top of base}}$ = 500 MPa – *Table 6.3* assuming High Standard crushed rock

   E_V = Minimum (696 MPa, 500 MPa) = 500 MPa

   E_H = 250 MPa

   $\nu_V = \nu_H = 0.35$ – *Table 6.3*

   f = 370.4

4. Other granular sublayers

   Divide the total granular layer thickness into 5 equi-thick sublayers (Section 8.2.3), each of thickness 475/5 = 95 mm.

   Calculate the ratio of moduli of adjacent layers:

$$R = \left[\frac{E_{\text{top of granular sublayer}}}{E_{\text{subgrade}}}\right]^{\frac{1}{5}},$$

   i.e. $R = (500/50)^{1/5} = 1.585$

   Therefore, elastic parameters of the 1st granular sublayer on top of the subgrade are:

   $E_{V1} = R \times E_{V\text{ subgrade}} = 1.585 \times 50 = 79$ MPa

   $E_{H1} = 0.5 \times E_{V1} = 39.5$ MPa

   $\nu_V = \nu_H = 0.35$

   $f = E_{V1}/(1+\nu_V) = 79/(1.35) = 58.5$

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f
		E_V	E_H	ν_V	ν_H	Value
Granular	95	500	250	0.35	0.35	370.4
Granular	95	315	157.5	0.35	0.35	233.3
Granular	95	199	99.5	0.35	0.35	147.4
Granular	95	126	63	0.35	0.35	93.3
Granular	95	79	39.5	0.35	0.35	58.5
Subgrade	Semi-infinite	50	25	0.45	0.45	34.5

Elastic properties of other sublayers are calculated similarly using the elastic properties of the underlying sublayer and are listed in the table above.

Steps 5 and 6 not relevant

7. Permanent deformation allowable loading (eqn 5.3)

$$N = \left[\frac{9300}{\mu\varepsilon}\right]^7$$

Steps 8 and 9 not relevant

10. $N_{DT} = 10^7$ HVAG

Permanent deformation is the controlling distress mode.

Using the presumptive Urban Traffic Load Distribution in *Appendix 7.4 Table A7.4.3*, ESA/HVAG = 0.7.

Therefore using eqn 7.4:

DESA = 0.7×10^7 ESA

Using Traffic Load Distribution in *Table A7.4.3*, $SAR_s/ESA = 1.6$.

$DSAR_s = 1.6 \times 0.7 \times 10^7 = 1.1 \times 10^7$

11. Standard Axle load represented as:

Contact stress = 750 kPa

Load radius = 92.1 mm

Four circular areas separated centre-to-centre 330 mm, 1470 mm and 330 mm (*Figure 8.2*).

12. Critical locations to calculate strains are:
   - top of subgrade directly beneath one wheel load
   - top of subgrade midway between the two loaded wheels.

13. Maximum vertical compressive strain from CIRCLY = 906 $\mu\varepsilon$ midway between the two loaded wheels.

14. Permanent deformation allowable loading – from step 7

$$N = \left[\frac{9300}{906}\right]^7 = 1.2 \times 10^7 \text{ SAR}$$

15. The allowable loading is 1.2×10^7 SAR compared to the design traffic = 1.1×10^7 SAR from Step 10.

16. As allowable loading exceeds the design traffic, the trial pavement composition is acceptable.

A8.3.2 Asphalt pavement containing cemented material subbase

Following the steps in *Table 8.1*:

Step

1. Try pavement composition of:

Material type	Thickness (mm)
Size 14 mm Asphalt, E = 2200 MPa	50
Size 20 mm Asphalt, E = 2500 MPa	125
Cemented material, E = 2000 MPa	150
Granular material	200
Subgrade, CBR = 5%	Semi-infinite

As the cover over the cemented material is greater than or equal to 175 mm, the post-cracking phase of the cemented material life may be considered (Section 8.2.4).

2. Subgrade

   CBR = 5%

   E_V = 50 MPa – Section 5.6

   E_H = 25 MPa

   ν_V = $\nu_H = 0.45$ – Section 5.6

   f = 34.5

3. Top granular sublayer

   Minimum of:

   - $E_{V\ \text{top of base}}$

     $= E_{V\ \text{subgrade}} \times 2^{(\text{total granular thickness}/125)}$

     = 152 MPa – Section 8.2.3

   - $E_V = 210$ MPa – *Table 6.4(b)* assuming High Standard crushed rock

   E_V = Minimum (152 MPa, 210 MPa) = 152 MPa

   E_H = 76 MPa

   ν_V = $\nu_H = 0.35$ – *Table 6.3*

   f = 112.6

4. Other granular sublayers

   Divide the total granular layer thickness into 5 equi-thick sublayers (Section 8.2.3), each of thickness 200/5 = 40 mm.

   Calculate the ratio of adjacent layers:

   $R = (152/50)^{1/5} = 1.249$

   Sublayer elastic properties calculation procedure is shown in the previous example (Section A8.3.1).

5. Cemented Materials

   Pre-cracking phase

   E = 2000 MPa (*Table 6.6*)

   $\nu_V = \nu_H = 0.2$

   Post-cracking phase

   Ev = 500 MPa – Section 6.4.3.6

   $E_H = 250$ MPa

   $\nu_V = \nu_H = 0.35$

   f = 370

   No sublayering.

6. Asphalt

   Size 14 mm asphalt : $E_V = E_H = 2200$ MPa

   Size 20 mm asphalt : $E_V = E_H = 2500$ MPa

   $\nu_V = \nu_H = 0.4$

   Elastic properties of all materials, including granular sublayers, are listed in the table below.

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f
		E_V	E_H	ν_V	ν_H	Value
size 14 mm Asphalt	50	2200	2200	0.4	0.4	1571
size 20 mm Asphalt	125	2500	2500	0.4	0.4	1786
Cemented material Pre-cracked/post-cracked	150	2000/500	2000/250	0.2/0.35	0.2/0.35	1667/370
Granular	40	152	76	0.35	0.35	112.6
Granular	40	122	61	0.35	0.35	90.4
Granular	40	97	48.5	0.35	0.35	71.9
Granular	40	78	39	0.35	0.35	57.8
Granular	40	62	31	0.35	0.35	45.9
Subgrade	Semi-infinite	50	25	0.45	0.45	34.5

7. Permanent deformation allowable loading

$$N = \left[\frac{9300}{\mu\varepsilon}\right]^7$$

8. Cemented material fatigue allowable loading (eqn 6.4)

$$N = RF\left[\frac{(113\,000/E^{0.804} + 191)}{\mu\varepsilon}\right]^{12}$$

Reliability Factor, RF = 0.5 (*Table 6.7*)

9. Asphalt fatigue allowable loading

Assuming volume of bitumen (V_B) = 11%

Size 20 mm asphalt :

$$N = RF\left[\frac{6918(0.856 \times 11 + 1.08)}{2500^{0.36}\ \mu\varepsilon}\right]^5$$

Reliability Factor, RF = 0.67 (*Table 6.13*)

10 $N_{DT} = 10^7$ HVAG

Using the presumptive Urban Traffic Load Distribution in *Appendix 7.4 Table A7.4.3*,

ESA/HVAG = 0.70

SAR_a/ESA = 1.1

SAR_s/ESA = 1.6

SAR_c/ESA = 12

Design traffic loading (ESA)

$= N_{DT} \times ESA/HVAG = 10^7 \times 0.70$

$= 7 \times 10^6$ ESA

11. Standard Axle load as in Section A8.3.1

12. Critical locations to calculate strains are:

- top of subgrade
- bottom of asphalt layer
- bottom of cemented layer

All the above strains are calculated directly beneath one of the loaded wheels and midway between the loaded wheels.

13. Critical strains from CIRCLY output

Pre-cracking Cemented Material Phase

Asphalt – maximum tensile strain is 47 με beneath a loaded wheel

Cemented material – maximum tensile strain is 134 με between the loaded wheels

Subgrade – maximum vertical compressive strain is 290 με between the loaded wheels.

Post-cracking Cemented Material Phase

Asphalt – maximum tensile strain is 168 με beneath a loaded wheel

Subgrade – maximum vertical compressive strain is 439 με between the loaded wheels.

14. Calculation of allowable loading. As the cover over the cemented material is greater than or equal to 175 mm, the post-cracking phase of the cemented material life may be considered (Section 8.2.4). That is, the allowable loading is the sum of the loading to fatigue cracking of cemented material plus the loading for each distress mode (i.e. asphalt fatigue permanent deformation) post-cracking (refer Section 8.2.4). This calculation requires the allowable loadings to be converted from Standard Axle Repetitions to ESA.

Pre-cracking phase

Cemented materials fatigue:

$$N = 0.5\left[\frac{113{,}000/2000^{0.804} + 191}{134}\right]^{12}$$

$$= 8.21 \times 10^5 \text{ SAR}$$

Convert from Standard Axle Repetitions of allowable loading to ESA of allowable loading using SAR_c/ESA = 12

$$N = 8.21 \times 10^5/12 = 6.84 \times 10^4 \text{ ESA}$$

Permanent deformation:

$$N = \left[\frac{9300}{290}\right]^7 = 3.49 \times 10^{10} \text{ SAR}$$

Convert from Standard Axle Repetitions of allowable loading to ESA of allowable loading using $SAR_s/ESA = 1.6$:

$$N = 3.49 \times 10^{10}/1.6 = 2.18 \times 10^{10} \text{ ESA}$$

Asphalt fatigue:

$$N = 0.67\left[\frac{6918(0.856 \times 11 + 1.08)}{2500^{0.36} \times 47}\right]^5$$

$$= 4.51 \times 10^9 \text{ SAR}$$

Convert from Standard Axle Repetitions of allowable loading to ESA of allowable loading using $SAR_a/ESA = 1.1$

$$N = 4.51 \times 10^9/1.1 = 4.06 \times 10^9 \text{ ESA}$$

Post-cracking phase

Permanent deformation:

$$N = \left[\frac{9300}{439}\right]^7 = 1.91 \times 10^9 \text{ SAR}$$

Convert from Standard Axle Repetitions of allowable loading to ESA of allowable loading using $SAR_s/ESA = 1.6$

$$N = 1.91 \times 10^9/1.6 = 1.20 \times 10^9 \text{ ESA}$$

Asphalt fatigue:

$$N = 0.67\left[\frac{6918(0.856 \times 11 + 1.08)}{2500^{0.36} \times 168}\right]^5$$

$$= 7.76 \times 10^6 \text{ SAR}$$

Convert from Standard Axle Repetitions of allowable loading to ESA of allowable loading using $SAR_a/ESA = 1.1$

$$N = 7.76 \times 10^6/1.1 = 7.03 \times 10^6 \text{ ESA}$$

As discussed in Section 8.2.4, the total allowable loading of pre-cracking and post-cracking phases are:

Permanent deformation allowable loading using equation 8.6:

$$N_S = 6.84 \times 10^4 + \left[1 - \frac{6.84 \times 10^4}{2.10 \times 10^{10}}\right] \times 1.20 \times 10^9$$

$$= 1.2 \times 10^9 \text{ ESA}$$

Asphalt fatigue allowable loading using equation 8.5:

$$N_A = 6.84 \times 10^4 + \left[1 - \frac{6.84 \times 10^4}{4.06 \times 10^9}\right] \times 7.76 \times 10^6$$

$$= 7.1 \times 10^6 \text{ ESA}$$

15. From Step 14, the following allowable loading in ESA were calculated:
    - permanent deformation = 1.2×10^9 ESA,
    - asphalt fatigue = 7.1×10^6 ESA,

    compared to the design traffic (Step 10) of 7.0×10^6 ESA

16. As allowable loading for both distress modes exceeds the design traffic, the trial pavement composition is acceptable.

A8.3.3 Full depth asphalt pavement

Following the steps in Table 8.1

Step

1. Try pavement composition of:

Material type	Thickness (mm)
Size 14 mm Asphalt, E = 2200 MPa	50
Size 20 mm Asphalt, E = 2500 MPa	210
Subgrade, CBR = 5%	Semi-infinite

2. Subgrade as in Section A8.3.2

   Steps 3, 4 and 5: not relevant

6. Asphalt

   Size 14 mm asphalt: $E_V = E_H = 2200$ MPa

   Size 20 mm asphalt: $E_V = E_H = 2500$ MPa

7. Subgrade strain criterion as in Section A8.3.2

8. Not relevant

9. Fatigue criterion for asphalt as per previous example (Section A8.3.2)

10. $N_{DT} = 10^7$ HVAG

    Using the presumptive Urban Traffic Load Distribution in *Appendix 7.4 Table A7.4.3*,

    $ESA/HVAG = 0.7$

    $SAR_a/ESA = 1.1$

    $SAR_s/ESA = 1.6$

    Therefore using eqn 7.4:

    $DESA = 10^7 \times 0.7 = 7.0 \times 10^6$

    $DSAR_a = 1.1 \times 7.0 \times 10^6 = 7.7 \times 10^6$

    $DSAR_s = 1.6 \times 7.0 \times 10^6 = 1.1 \times 10^7$

11. Standard Axle load as in Section A8.3.1

12. Critical locations are:

    - top of subgrade
    - bottom of asphalt layer

    Both should be checked directly beneath one of the loaded wheels and midway between the loaded wheels

13 Critical strains from CIRCLY output:

   Asphalt 166 $\mu\varepsilon$ midway between the loaded wheels

   Subgrade 412 $\mu\varepsilon$ beneath one of the loaded wheels

14. Asphalt fatigue allowable loading

    Volume of binder in mixes $(V_B) = 11\%$

    Desired Project Reliability is 97.5%.

    Project Reliability Factor, RF = 0.67 (*Table 6.13*)

    $N = 0.67\{6918\,(0.856 \times 11 + 1.08)]/(2500^{0.36} \times 166)\}^5$

    $= 8.2 \times 10^6$ SAR

    Permanent deformation allowable loading

    $N = (9300/412)^7 = 3.0 \times 10^9$ SAR

15. Compare allowable loadings (Step 14) to design traffic (Step 10):

Distress mode	Allowable loading (SAR)	Design Traffic (SAR)
Asphalt fatigue	8.2×10^6	7.7×10^6
Permanent deformation	3.0×10^9	1.1×10^7

16. As allowable loading for each distress mode exceeds the design traffic, the trial pavement composition is acceptable.

APPENDIX 9.1 EXAMPLES OF USE OF DESIGN PROCEDURE FOR RIGID PAVEMENTS

This Appendix gives examples of the use of design procedure for a plain concrete pavement (PCP) using the following design parameters:

- Desired project reliability = 95%
- Subgrade design CBR = 5%
- Design traffic for 40 years design life = 4×10^7 HVAG. The traffic distribution is in accordance with the Urban distribution *Table A7.4.3*.

Following the steps in *Table 9.3*.

Design Steps:

1. The pavement selected for this project is a plain concrete pavement (PCP).
2. A concrete shoulder is to be provided.
3. From *Table 9.1*, a 150 mm lean-mix concrete (LMC) subbase is required for the design traffic (4×10^7 HVAG).
4. From *Figure 9.1*, for a 150 mm LMC subbase and a subgrade CBR of 5%, effective subgrade CBR = 75%.
5. The characteristic 28-day flexural strength of the concrete is 4.5 MPa.
6. As the desired project reliability is 95%, *Table 9.2* indicates a Load Safety Factor of 1.3 should be used.
7. *Figure 9.2* suggests a base thickness of approximately 210 mm for this case. This value is adopted as a trial base thickness.
8. The data from *Table A7.4.3* is entered in the first 3 columns of the "CALCULATION OF EXPECTED REPETITIONS" sheet (*Figure A.9.1.1*), together with the design traffic in HVAG in column 4. The expected repetitions of each load on each axle group type (column 5) is then calculated as the product of the entries in column 2, 3 and 4. The axle load value and the corresponding expected repetitions are then transferred to columns 1 and 3 of the "THICKNESS CALCULATION" sheet. Using Equations 9.2 and 9.3 allowable repetitions in fatigue distress mode are calculated for SAST axle group and are entered into column 4 of the "THICKNESS CALCULATION" Sheet.
9. The ratio of the expected fatigue repetitions and the allowable repetitions is calculated and multiplied by 100 before entering into column 5 of the "THICKNESS CALCULATION" Sheet.
10. Allowable repetitions for erosion are calculated using equation 9.5 and are entered into column 6 of the "THICKNESS CALCULATION" Sheet.
11. The ratio of the expected erosion repetitions and the allowable repetitions is calculated and multiplied by 100 before entering into column 7 of "THICKNESS CALCULATION" Sheet.
12. Steps 8 to 11 are repeated for each load on this axle group up to a load level where the allowable load repetitions exceed 10^8.
13. Total percentage fatigue and erosion are calculated for all relevant loads in this axle group.
14. Steps 8 to 13 are repeated for all other axle group type (i.e. TAST, SADT, TADT, TRDT and QADT).
15. Total percent fatigue and erosion for all axle group types are calculated.

    Total percent fatigue = 144 %

    Total percent erosion = 120 %.
16. As in Step 15 both fatigue and erosion were not less than or equal to 100%, it is necessary to repeat steps 7 to 15 until both the total percent fatigue and erosion are less than 100%.

    Selected final base thickness = 215 mm

    Total percent fatigue = 76 %

    Total percent erosion = 94 %
17. Selected final base thickness = 215 mm meets the minimum base thickness requirement (200 mm from *Table 9.7*).

RIGID PAVEMENT DESIGN
CALCULATION OF EXPECTED REPETITIONS

Project: Rigid Pavement Design Example for 4 x 10^7 HVAG

SINGLE AXLES/SINGLE WHEELS (SAST)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
130	0.0002	0.39	40,000,000	3,144
120	0.0002	0.39	40,000,000	3,144
110	0.0002	0.39	40,000,000	3,144
100	0.0004	0.39	40,000,000	6,288
90	0.0011	0.39	40,000,000	17,292
80	0.0079	0.39	40,000,000	124,188
70	0.0650	0.39	40,000,000	1,021,800
60	0.2329	0.39	40,000,000	3,661,188
50	0.2994	0.39	40,000,000	4,706,568
40	0.1571	0.39	40,000,000	2,469,612
30	0.1546	0.39	40,000,000	2,430,312
20	0.0783	0.39	40,000,000	1,230,876
10	0.0027	0.39	40,000,000	42,444

TANDEM AXLES/SINGLE WHEELS (TAST)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
200	0.0006	0.01	40,000,000	216
190	0.0007	0.01	40,000,000	252
180	0.0014	0.01	40,000,000	504
170	0.0023	0.01	40,000,000	828
160	0.0043	0.01	40,000,000	1,548
150	0.0084	0.01	40,000,000	3,024
140	0.0154	0.01	40,000,000	5,544
130	0.0294	0.01	40,000,000	10,584
120	0.0590	0.01	40,000,000	21,240
110	0.0977	0.01	40,000,000	35,172
100	0.1442	0.01	40,000,000	51,912
90	0.1595	0.01	40,000,000	57,420
80	0.1661	0.01	40,000,000	59,796
70	0.1681	0.01	40,000,000	60,516
60	0.1027	0.01	40,000,000	36,972

Figure A.9.1.1 Calculation of expected repetitions (Page 1 of 3)

RIGID PAVEMENT DESIGN
CALCULATION OF EXPECTED REPETITIONS

Project: Rigid Pavement Design Example for 4 x 10^7 HVAG

SINGLE AXLES/DUAL WHEELS (SADT)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
130	0.0002	0.19	40,000,000	1,528
120	0.0012	0.19	40,000,000	9,168
110	0.0041	0.19	40,000,000	31,324
100	0.0139	0.19	40,000,000	106,196
90	0.0297	0.19	40,000,000	226,908
80	0.0462	0.19	40,000,000	352,968
70	0.0650	0.19	40,000,000	496,600
60	0.0961	0.19	40,000,000	734,204
50	0.1680	0.19	40,000,000	1,283,520
40	0.2193	0.19	40,000,000	1,675,452
30	0.2346	0.19	40,000,000	1,792,344
20	0.0870	0.19	40,000,000	664,680
10	0.0347	0.19	40,000,000	265,108

TANDEM AXLES/DUAL WHEELS (TADT)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
230	0.0002	0.26	40,000,000	2,072
220	0.0003	0.26	40,000,000	3,108
210	0.0008	0.26	40,000,000	8,288
200	0.0033	0.26	40,000,000	34,188
190	0.0085	0.26	40,000,000	88,060
180	0.0186	0.26	40,000,000	192,696
170	0.0355	0.26	40,000,000	367,780
160	0.0572	0.26	40,000,000	592,592
150	0.0803	0.26	40,000,000	831,908
140	0.0653	0.26	40,000,000	676,508
130	0.0588	0.26	40,000,000	609,168
120	0.0595	0.26	40,000,000	616,420
110	0.0627	0.26	40,000,000	649,572
100	0.0724	0.26	40,000,000	750,064
90	0.0761	0.26	40,000,000	788,396
80	0.0977	0.26	40,000,000	1,012,172
70	0.1094	0.26	40,000,000	1,133,384
60	0.0951	0.26	40,000,000	985,236

Figure A.9.1.1 Calculation of expected repetitions (Page 2 of 3)

RIGID PAVEMENT DESIGN
CALCULATION OF EXPECTED REPETITIONS

Project: Rigid Pavement Design Example for 4 x 10^7 HVAG

TRIAXLES/DUAL WHEELS (TRDT)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
260	0.0001	0.15	40,000,000	592
250	0.0009	0.15	40,000,000	5,328
240	0.0025	0.15	40,000,000	14,800
230	0.0078	0.15	40,000,000	46,176
220	0.0234	0.15	40,000,000	138,528
210	0.0514	0.15	40,000,000	304,288
200	0.0843	0.15	40,000,000	499,056
190	0.0773	0.15	40,000,000	457,616
180	0.0610	0.15	40,000,000	361,120
170	0.0482	0.15	40,000,000	285,344
160	0.0411	0.15	40,000,000	243,312
150	0.0401	0.15	40,000,000	237,392
140	0.0336	0.15	40,000,000	198,912
130	0.0315	0.15	40,000,000	186,480
120	0.0330	0.15	40,000,000	195,360
110	0.0370	0.15	40,000,000	219,040

QUAD-AXLES/DUAL WHEELS (QADT)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
260	0.0000	0.00	40,000,000	0
250	0.0000	0.00	40,000,000	0
240	0.0000	0.00	40,000,000	0
230	0.0000	0.00	40,000,000	0
220	0.0000	0.00	40,000,000	0
210	0.0000	0.00	40,000,000	0
200	0.0000	0.00	40,000,000	0
190	0.0000	0.00	40,000,000	0
180	0.0000	0.00	40,000,000	0
170	0.0000	0.00	40,000,000	0
160	0.0000	0.00	40,000,000	0
150	0.0000	0.00	40,000,000	0
140	0.0000	0.00	40,000,000	0
130	0.0000	0.00	40,000,000	0
120	0.0000	0.00	40,000,000	0

Figure A.9.1.1 Calculation of expected repetitions (Page 3 of 3)

RIGID PAVEMENT DESIGN
THICKNESS CALCULATION

Project: Rigid Pavement Design Example for 4 x 10^7 HVAG **Date:** 25/06/2003

Source of load data	Table A.7.4.3 Urban		**Characteristic (28 day) flexural strength f'_{cf}**	4.5 MPa
CRC/Dowelled joints	FALSE		**Subgrade CBR**	5%
Concrete shoulder	TRUE		**Subbase thickness**	150 mm
Design period	40	years	**Subbase type**	LMC
Design traffic	4.0E + 07	HV axle groups	**Effective subgrade CBR**	75%
Load Safety Factor LSF	1.30		**Trial base thickness**	210 mm

SINGLE AXLES/SINGLE WHEELS (SAST)

Equivalent Stress: 0.75 Stress Ratio Factor: 0.177 Erosion Factor: 1.86

Axle group load (kN)	Design load (kN)	Expected repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue (%)	Allowable repetitions	Damage (%)
130	169.0	3,144	2,974	105.72	162,052	1.94
120	156.0	3,144	11,812	26.62	269,239	1.17
110	143.0	3,144	47,245	6.65	483,605	0.65
100	130.0	6,288	196,228	3.20	977,194	0.64
90	117.0	17,292	1,808,199	0.96	2,419,503	0.71
80	104.0	124,188	UNLIMITED	0.00	9,462,726	1.31
70	91.0	1,021,800	UNLIMITED	0.00	978,333,565	0.10

TANDEM AXLES/SINGLE WHEELS (TAST)

Equivalent Stress: 0.75 Stress Ratio Factor: 0.177 Erosion Factor: 2.46

Axle group load (kN)	Design load (kN)	Expected repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue (%)	Allowable repetitions	Damage (%)
200	260.0	216	196,228	0.11	14,262	1.51
190	247.0	252	494,164	0.05	19,043	1.32
180	234.0	504	1,808,199	0.03	25,860	1.95
170	221.0	828	16,212,762	0.01	35,814	2.31
160	208.0	1,548	UNLIMITED	0.00	50,762	3.05
150	195.0	3,024	UNLIMITED	0.00	73,978	4.09
140	182.0	5,544	UNLIMITED	0.00	111,567	4.97
130	169.0	10,584	UNLIMITED	0.00	175,728	6.02
120	156.0	21,240	UNLIMITED	0.00	293,156	7.25
110	143.0	35,172	UNLIMITED	0.00	530,027	6.64
100	130.0	51,912	UNLIMITED	0.00	1,083,356	4.79
90	117.0	57,420	UNLIMITED	0.00	2,746,341	2.09
80	104.0	59,796	UNLIMITED	0.00	11,519,479	0.52
70	91.0	60,516	UNLIMITED	0.00	UNLIMITED	0.00

Figure A.9.1.2 Pavement thickness calculation (Page 1 of 3)

RIGID PAVEMENT DESIGN
THICKNESS CALCULATION

Project: Rigid Pavement Design Example for 4 x 10^7 HVAG **Date:** 25/06/2003

SINGLE AXLES/DUAL WHEELS (SADT)

Equivalent Stress: 1.06 Stress Ratio Factor: 0.250 Erosion Factor: 2.47

Axle group load (kN)	Design load (kN)	Expected repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue (%)	Allowable repetitions	Damage (%)
150	195.0	0	57,500	0.00	69,271	0.00
140	182.0	0	159,117	0.00	104,306	0.00
130	169.0	1,528	601,365	0.25	163,912	0.93
120	156.0	9,168	5,962,495	0.15	272,483	3.36
110	143.0	31,324	UNLIMITED	0.00	489,873	6.39
100	130.0	106,196	UNLIMITED	0.00	991,416	10.71
90	117.0	226,908	UNLIMITED	0.00	2,462,593	9.21
80	104.0	352,968	UNLIMITED	0.00	9,721,111	3.63
70	91.0	496,600	UNLIMITED	0.00	1,373,296,172	0.04
60	78.0	734,204	UNLIMITED	0.00	UNLIMITED	0.00

TANDEM AXLES/DUAL WHEELS (TADT)

Equivalent Stress: 0.89 Stress Ratio Factor: 0.209 Erosion Factor: 2.46

Axle group load (kN)	Design load (kN)	Expected repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue (%)	Allowable repetitions	Damage (%)
240	312.0	0	UNLIMITED	0.00	293,156	0.00
230	299.0	2,072	UNLIMITED	0.00	389,462	0.53
220	286.0	3,108	UNLIMITED	0.00	530,027	0.59
210	273.0	8,288	UNLIMITED	0.00	743,295	1.12
200	260.0	34,188	UNLIMITED	0.00	1,083,356	3.16
190	247.0	88,060	UNLIMITED	0.00	1,662,739	5.30
180	234.0	192,696	UNLIMITED	0.00	2,746,341	7.02
170	221.0	367,780	UNLIMITED	0.00	5,082,000	7.24
160	208.0	592,592	UNLIMITED	0.00	11,519,479	5.14
150	195.0	831,908	UNLIMITED	0.00	42,456,964	1.96
140	182.0	676,508	UNLIMITED	0.00	UNLIMITED	0.00

Figure A.9.1.2 Pavement thickness calculation (Page 2 of 3)

RIGID PAVEMENT DESIGN
THICKNESS CALCULATION

Project: Rigid Pavement Design Example for 4 x 10^7 HVAG **Date:** 25/06/2003

TRIAXLES/DUAL WHEELS (TRDT)

Equivalent Stress: 0.73 Stress Ratio Factor: 0.173 Erosion Factor: 2.45

Axle group load (kN)	Design load (kN)	Expected repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue (%)	Allowable repetitions	Damage (%)
270	351.0	0	UNLIMITED	0.00	2,845,859	0.00
260	338.0	592	UNLIMITED	0.00	4,236,980	0.01
250	325.0	5,328	UNLIMITED	0.00	6,786,246	0.08
240	312.0	14,800	UNLIMITED	0.00	12,191,794	0.12
230	299.0	46,176	UNLIMITED	0.00	27,021,679	0.17
220	286.0	138,528	UNLIMITED	0.00	101,961,571	0.14
210	273.0	304,288	UNLIMITED	0.00	UNLIMITED	0.00
200	260.0	499,056	UNLIMITED	0.00	UNLIMITED	0.00

QUAD-AXLES/DUAL WHEELS (QADT)

Equivalent Stress: 0.73 Stress Ratio Factor: 0.173 Erosion Factor: 2.45

Axle group load (kN)	Design load (kN)	Expected repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue (%)	Allowable repetitions	Damage (%)
300	390.0	0	UNLIMITED	0.00	1,114,927	0.00
290	377.0	0	UNLIMITED	0.00	1,475,874	0.00
280	364.0	0	UNLIMITED	0.00	2,011,238	0.00
270	351.0	0	UNLIMITED	0.00	2,845,859	0.00
260	338.0	0	UNLIMITED	0.00	4,236,980	0.00
250	325.0	0	UNLIMITED	0.00	6,786,246	0.00
240	312.0	0	UNLIMITED	0.00	12,191,794	0.00
230	299.0	0	UNLIMITED	0.00	27,021,679	0.00
220	286.0	0	UNLIMITED	0.00	101,961,571	0.00
210	273.0	0	UNLIMITED	0.00	UNLIMITED	0.00
200	260.0	0	UNLIMITED	0.00	UNLIMITED	0.00
190	247.0	0	UNLIMITED	0.00	UNLIMITED	0.00
180	234.0	0	UNLIMITED	0.00	UNLIMITED	0.00
170	221.0	0	UNLIMITED	0.00	UNLIMITED	0.00
160	208.0	0	UNLIMITED	0.00	UNLIMITED	0.00
150	195.0	0	UNLIMITED	0.00	UNLIMITED	0.00

TOTAL Fatigue %	143.75	TOTAL Erosion %	119.89

Figure A.9.1.2 Pavement thickness calculation (Page 3 of 3)

INFORMATION RETRIEVAL

AUSTROADS (2004) Pavement Design - A Guide to the Structural Design of Road Pavements,
Sydney, A4, loose-leaf (various pagings), AP-G17/04 (formerly AP-17/92).

KEYWORDS: Pavement design, axle load, thickness, flexible pavement, rigid pavement, reinforced concrete, strengthening (pavement), subgrade.

ABSTRACT: This Guide is intended to assist those required to plan and design new pavements. The methods described have been generally developed from the approaches followed in the Member Authorities. However, as a guide which encompasses the wide range of materials and conditions found in Australia and New Zealand, some parts are broadly based.

Pavement composition is related to availability of materials and knowledge of their performance in any particular locality.

The Guide covers the assessment of input parameters needed for design, design methods for flexible and rigid pavements, and gives guidance to the economic comparisons of alternative pavement designs.

AUSTROADS PUBLICATIONS

AUSTROADS

Austroads publishes a large number of guides and reports. Some of its publications are:

AP-G1/03 Rural Road Design: A Guide to the Geometric Design of Rural Roads
AP-8/87 Visual Assessment of Pavement Condition
Guide to Traffic Engineering Practice
- AP-11.1/88 Traffic Flow
- AP-11.2/88 Roadway Capacity
- AP-G11.3/04 Traffic Studies
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- AP-11.5/88 Intersections at Grade
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(Medical standards for licensing and clinical management guidelines) [September 2003]
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