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Guide to Pavement Technology Part 2

Pavement Structural Design

Austroads

Guide to Pavement Technology Part 2: Pavement Structural Design

Austroads

Sydney 2025

Guide to Pavement Technology Part 2: Pavement Structural Design

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Abstract

The purpose of the *Guide to Pavement Technology Part 2: Pavement Structural Design* is to provide advice for the structural design of new sealed road pavements.

The target audience for the Austroads *Guide to Pavement Technology* includes all those involved with the management of roads, including industry and students seeking to learn more about the fundamental concepts, principles, issues and procedures associated with pavement technology.

The advice contained in this Guide has been generally developed from the approaches followed by the Austroads member agencies. However, as it encompasses the wide range of materials and conditions found in Australia and New Zealand, some parts are broadly based.

This Part covers the assessment of input parameters needed for design, outlines design methods for flexible and rigid pavements, and also provides guidance on the economic comparisons of alternative pavement designs.

Keywords

Pavement design, pavement evaluation, pavement materials, subgrade, design traffic.

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Edition 5.0 includes:

- distinction made between lightly bound and heavily bound cemented materials
- design procedure introduced for lightly bound cemented materials. New characterisation of post-cracking phase of heavily bound cemented materials
- revisions to WIM data in Appendix E
- numerous clarifications and revisions.

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About Austroads

Austroads is the association of Australasian transport agencies.

Austroads' purpose is to support our member organisations to deliver an improved Australasian road transport network. To succeed in this task, we undertake leading-edge road and transport research which underpins our input to policy development and published guidance on the design, construction and management of the road network and its associated infrastructure.

Austroads provides a collective approach that delivers value for money, encourages shared knowledge and drives consistency for road users.

Austroads is governed by a Board consisting of senior executive representatives from each of its eleven member organisations:

- Transport for NSW
- Department of Transport and Planning (Transport Victoria)
- Queensland Department of Transport and Main Roads
- Main Roads Western Australia
- Department for Infrastructure and Transport South Australia
- Department of State Growth Tasmania
- Department of Logistics and Infrastructure Northern Territory
- City and Environment Directorate, Australian Capital Territory
- Department of Infrastructure, Transport, Regional Development, Communications, Sport and the Arts
- Australian Local Government Association
- NZ Transport Agency Waka Kotahi.

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1. Introduction

1.1 Scope of the Guide and this Part

Australian state/territory and New Zealand road agencies, local government, and industry have amassed a great deal of knowledge on pavement technologies, techniques, and considerations. The purpose of the Austroads *Guide to Pavement Technology* is to assemble this knowledge into a single authoritative and comprehensive resource for practitioners in Australia and New Zealand.

The target audience for the Austroads *Guide to Pavement Technology* includes all those involved with the management of roads, including industry, and students seeking to learn more about the fundamental concepts, principles, issues and procedures associated with pavement technology.

Part 2: Pavement Structural Design – provides advice on the structural design of new sealed road pavements. The advice has been generally developed from the approaches followed in the Austroads member agencies. However, as it encompasses the wide range of materials and conditions found in Australia and New Zealand, some parts are broadly based.

This Part covers the assessment of input parameters needed for design, design methods for new flexible and rigid pavements and gives guidance to the economic comparisons of alternative pavement designs. Specifically, this Part contains procedures for the design of the following forms of new sealed road pavement construction subjected to conventional road traffic:

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

The design of structural overlays is included in *Part 5: Pavement Evaluation and Treatment Design* of the Guide (Austroads 2025a).

Terminology used in this Part is defined in *Austroads Glossary of Terms* (Austroads 2015a) 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 this Part. This Part 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.

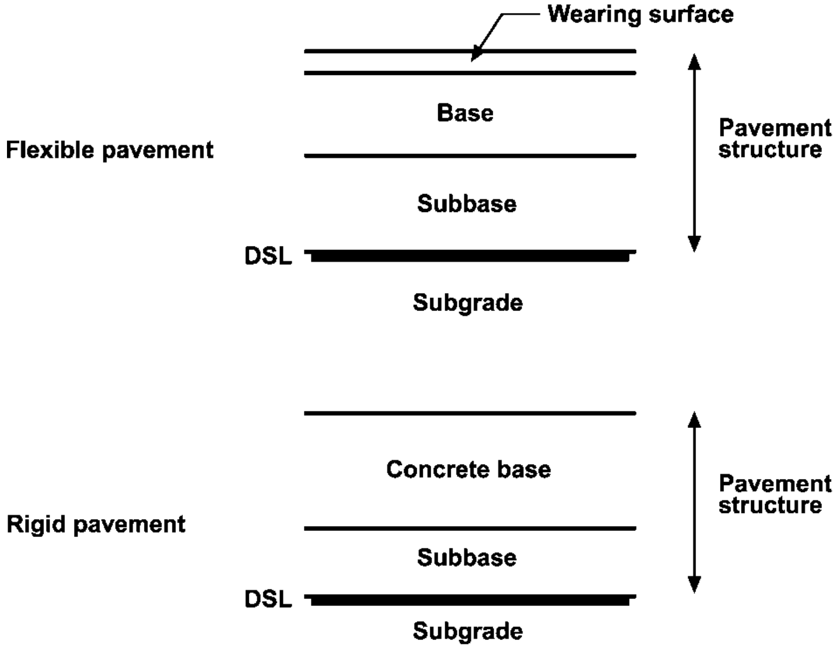
An integral part of the pavement design process is an assessment of how well the outcome of the design – the constructed pavement – will perform. Because of the many factors which must be evaluated to design pavements, there is no absolute certainty that the desired performance will be achieved. Chapter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

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

- Selection of surfacings is not covered in this Part. Surfacing selection is addressed in *Part 3: Pavement Surfacings* of the Guide (Austroads 2025b).
- The design procedures described in Chapters 2 to 11 of this Part apply for pavements subjected to a minimum design traffic loading of 10^5 Equivalent Standard Axles (ESA), that is, for moderate-to-heavily trafficked roads. For lightly trafficked roads environmental distress has a more significant effect on pavement performance. Chapter 12 provides procedures for lightly trafficked pavements.

- Whilst the mechanistic-empirical procedures presented in this Part can be applied to any pavement type and traffic load, the pavement types addressed in this Part 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 to 350 mm has 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



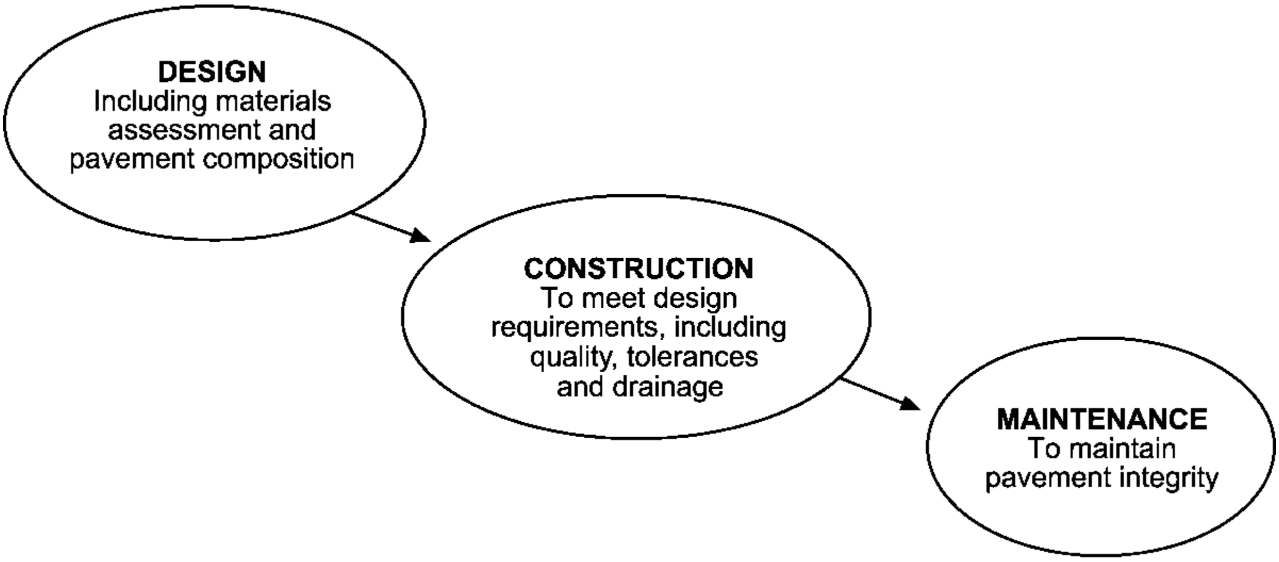
Notes:
1. **DSL = Design Subgrade Level**
2. **Base and subbase may contain more than one layer**
3. **Wearing course of a flexible pavement may be asphalt or bituminous seal**
4. **In a rigid pavement the concrete base may be surfaced with an asphalt wearing course**
5. **An imported subgrade or selected subgrade material may be placed over the natural subgrade.**

- Pavements are assumed to be constructed to the usual quality standards specified by Austroads member agencies.
- Unsurfaced pavements (i.e. roads that are not surfaced with sprayed seals, asphalt or concrete) are not considered in this Part of the Guide because the performance of these pavements is heavily dependent on the performance of local materials, local environmental conditions and maintenance policies. *Part 6*: *Unsealed Pavements* of the Guide (Austroads 2009a) and the *Unsealed Roads: Best Practice Guide 2* (ARRB Group 2020), provide useful guidance in this regard.
- The design and selection of pavement rehabilitation treatments is not considered in this Part. Such treatments, including structural overlays, are provided in *Part 5: Pavement Evaluation and Treatment Design* of the Guide (Austroads 2025a).
- This Part 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. Engineering judgement must be exercised by the designer in arriving at decisions regarding the parameters that are incorporated into particular designs. In addition, users of this Part need to apply suitable risk management practices when determining design configurations from the calculation procedures, to avoid the construction of inappropriate or counter-intuitive pavement structures.

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

Figure 1.2: Primary factors influencing pavement performance



Australian and New Zealand road agencies have used this Part to develop design manuals or supplements as detailed in Appendix A.

This Part should be read in conjunction with the other parts of the *Guide to Pavement Technology*:

- Part 1: Introduction to Pavement Technology
- Part 2: Pavement Structural Design
- Part 3: Pavement Surfacings
- Part 4: Pavement Materials
  - Part 4A: Granular Base and Subbase Materials
  - Part 4B: Asphalt
  - Part 4C: Materials for Concrete Road Pavements
  - Part 4D: Stabilised Materials
  - Part 4E: Recycled Materials
  - Part 4F: Bituminous Binders
  - Part 4G: Geotextiles and Geogrids
  - Part 4H: Test Methods (withdrawn) – Austroads publishes test methods primarily developed as outcomes of various funded projects or to support the harmonisation of testing practices, supplementing Austroads Technical Specifications. Additional test methods can be obtained from Standards Australia and Standards New Zealand.
  - Part 4I: Earthworks Materials

  - Part 4J: Aggregates and Source Rock
  - Part 4K: Selection and Design of Sprayed Seals
  - Part 4L: Stabilising Binders
- Part 5: Pavement Evaluation and Treatment Design
- Part 6: Unsealed Pavements
- Part 7: Pavement Maintenance
- Part 8: Pavement Construction
- Part 9: Pavement Work Practices (withdrawn) – Note, Pavement Work Tips (Austroads 2024a), jointly published by Austroads and Australian Flexible Pavement Association, provide technical notes and similar publications related to pavement work practices
- Part 10: Sub-Surface Drainage.

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 a 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. 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.

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 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.

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, construction expediency, innovation, sustainability, infrastructure resilience, public relations?
Pavement options	• Road agency policy or preferences • Alternative 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, traffic capacity • Management of users during investigation and construction • Significance of project in terms of the network • Other uses: flood levee, floodway, 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 • Impacts: 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

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 quality, tolerances and drainage
- maintenance – to maintain pavement integrity.

2.2 Common pavement types

2.2.1 General

In designing a new pavement one of the first tasks is to select one or more pavement types for detailed design. This section describes the common pavement types and their uses. Part 3 of the Guide (Austroads 2025b) provides additional information on common pavement surfacings.

The choice of pavement type varies markedly with the function of the road, traffic loading, availability of materials, and the environment.

Lightly trafficked roads usually comprise unbound granular pavements with thin bituminous surfacings. Where an asphalt surfacing is provided it is common for the thickness of asphalt to be 25–50 mm.

More heavily trafficked roads may require the asphalt to extend to more than the surface layer, with the asphalt commonly supported by a granular subbase.

Some heavily trafficked roads (e.g. freeways) have high level performance requirements and need to be designed to minimise traffic delays due to road maintenance during their service lives. Such pavements commonly have a design traffic loading exceeding 10^7 ESA and are sometimes referred to as ‘heavy-duty’ pavements. Pavement types commonly used for heavy duty pavements in conjunction with high materials standards include:

- deep strength asphalt, with thick asphalt on cement stabilised granular subbase
- flexible composite, comprising a thick asphalt on lean-mix concrete subbase
- full depth asphalt
- unbound granular with sprayed seal surfacing
- jointed plain (unreinforced) concrete pavements
- jointed reinforced concrete pavements
- continuously reinforced concrete pavements.

Such heavy-duty pavements are commonly supported by higher strength, stable materials consisting of granular subbases and/or selected subgrade and earthworks materials.

Part 1 (Austroads 2025c) also discusses pavement types and components.

2.2.2 Granular pavements with sprayed seal surfacings

Unbound granular pavements with sprayed seal surfacings are the major pavement type in rural Australia, comprising some 90% of the length of all surfaced roads. They comprise the majority of light and moderately trafficked rural roads and have also been successfully used on heavily trafficked roads, subject to suitable materials, environments and construction and maintenance standards. This pavement type is extensively used due to its low initial cost.

2.2.3 Cemented granular bases with sprayed seal surfacings

The use of heavily bound cemented bases with sprayed seal surfacings is more commonly associated with the rehabilitation treatments of granular pavements than new construction works. With the exception of temporary pavements, this pavement type is seldom used for new works due to significant performance issues associated with shrinkage cracking. Surfacing performance requirements are generally the same as for unbound granular pavements other than the need to consider the possibility of shrinkage cracking.

The use of slow-setting cementitious binders in these types of pavements has the benefit of producing more closely spaced, finer cracks than traditional faster setting binders. This reduces the likelihood of cracking reflecting through the surfacing and also allows more time for placement, compaction and trimming. For moderately and heavily trafficked roads a geotextile reinforced sprayed seal may be provided to delay reflection cracking.

2.2.4 Granular pavements with thin asphalt surfacings

Unbound granular pavements with single thin asphalt surfacings are structurally similar to sprayed seal pavements except that asphalt surfacing may fatigue crack. For this pavement type the asphalt surface makes little contribution to the overall strength of the pavement but provides greater resistance to minor traffic damage as well as a smoother and more durable surface. These attributes make it particularly suited to residential streets and other light traffic urban applications where risk of fatigue cracking is lower.

With suitable quality of materials and construction standards, these pavements are sometimes used for urban collector and occasionally main roads, although they are unlikely to provide the same serviceability as more heavily bound pavements. Some agencies restrict the use of single layer asphalt surfacings on new pavements because of the performance risks associated with them. They are not commonly used for urban freeway applications due to the risk of premature distress (refer to Section 8.2.8).

Thin asphalt surfacing can also be used on light to moderately trafficked rural road pavements, where sprayed seals do not provide adequate serviceability, and where the risk of fatigue cracking is acceptable, e.g. intersections and other areas of turning traffic, or to provide improved ride quality.

It is important that these types of pavements have a primed or initial sealed (formerly known as primersealed) surface beneath the asphalt surface to aid bonding of the asphalt layer to the granular material. Application of a sprayed seal (following application of a prime) or an initial seal, prior to application of an asphalt surfacing, will also improve waterproofing of the pavement.

The most common surfacing types are dense graded asphalt 7 or 10 mm in size for lightly trafficked pavements or lower speed environments, and 10 or 14 mm aggregate for more heavily trafficked applications. Detailed asphalt selection criteria are provided in Part 3 (Austroads 2025b).

2.2.5 Asphalt over granular pavements

These pavements comprise multiple asphalt layers over a granular base and/or subbase. In these pavements the purpose of the asphalt layers is to provide a wearing surface and to make a significant contribution to the structural capacity of the pavement. Where the asphalt thickness is less than 150 mm, the granular base layer(s) provides a substantial proportion of the load carrying capacity and both deformation and fatigue distress mechanisms are possible. Therefore, the asphalt and granular base materials must be of appropriate quality to ensure the intended service life results.

The main application for asphalt on granular pavement is on medium traffic urban roads. It may also be suitable for rural highways and main roads depending on climate and traffic loads.

The most common surfacing type is 14 mm dense graded asphalt except where open graded asphalt or stone mastic asphalt is needed due to functional requirements. Binder type and mix design requirements will vary according to traffic loading as detailed in Part 3 (Austroads 2025b) and Part 4B (Austroads 2025d).

Moisture retained in asphalt surfacings can increase the risk of moisture damage to the underlying asphalt. Thick asphalt pavements usually incorporate 10 or 14 mm dense graded asphalt over the base asphalt/intermediate layers. This dense graded asphalt can inhibit moisture entry if its properties and construction are carefully managed. Trafficking of the dense graded asphalt can also assist in decreasing surface permeability, although it also increases the risk of moisture ingress in the short term due to increased exposure. A heavy tack coat or sprayed seal (depending on local practice) may be required before the surface asphalt is placed to aid bonding and waterproofing.

2.2.6 Flexible composite, deep strength and full depth asphalt pavements

In these pavement systems, asphalt is used in both the surface and bound base layers to provide a significant portion of the overall load-carrying capacity. Flexible composite, deep strength and full depth asphalt pavements are typically constructed with multiple layers of asphalt of substantial thickness. The key distinction among these pavement types generally lies in the subbase material used, as outlined below:

- Deep strength asphalt pavement is typically constructed over a cement-treated subbase.
- Flexible composite pavement is constructed over a lean-mix concrete subbase.
- Full depth asphalt pavement is generally constructed over a granular subbase; however, under certain conditions, it may be placed directly on a subgrade with suitable engineering properties.

Additional granular subbases and/or improved subgrade materials may be incorporated beneath the bound layers to enhance structural performance. Refer to Section 3.14.2 for further details.

These pavement types are suitable for moderately to heavily trafficked roads, particularly in urban environments.

Surfacing requirements for these pavements align with those specified for asphalt pavements constructed over granular bases in this section.

2.2.7 Rigid pavements

Rigid pavements contain a cementitious concrete base. The principal types of rigid pavements are:

- plain (jointed unreinforced) concrete pavements (PCP)
- jointed reinforced concrete pavements (JRCP)
- continuously reinforced concrete pavements (CRCP)
- steel fibre reinforced concrete pavements (SFCP).

Further details are provided in Section 9.2.1.

Another type of rigid pavement is one with a roller compacted concrete (RCC) base. Design procedures for this pavement type are not included in this Part.

Rigid (concrete) pavements may be used in all urban classes of road pavement and moderate to heavily trafficked rural road pavements. For heavy duty pavements, rigid pavements are particularly resistant to the effects of slow-moving and heavily loaded vehicles, as well as fuel spillages. Rigid pavements may also be used to achieve specific traffic calming, landscape and architectural effects through their ability to display a variety of colours, textures and forms.

For moderately and heavily trafficked roads it is common practice to support the concrete slab with a lean-mix concrete subbase. Cement stabilised crushed rock may sometimes be considered for moderately trafficked roads. For lightly trafficked roads, unbound granular subbases may be used.

Surface finish forms part of the construction process. The selection of the concrete finish types requires the consideration of a number of factors including surface texture, noise properties and aesthetics. Typical surface finishes used for rigid pavements are described in Part 3.

Rigid pavements may be surfaced with asphalt to meet particular functional requirements. Typical applications include surfacing of concrete bridge decks or use of open graded asphalt for reduced noise levels. Careful consideration of the effect of concrete joints on the performance of the overlying asphalt is required. Cracking of the asphalt surfacing over most rigid pavement types, excluding CRCP, due to joints can be expected.

Concrete surfaces should be primed with a very light bituminous primer before placing asphalt. The primer should be capable of penetrating fine shrinkage cracks and pores in concrete to a depth of at least 2 mm.

Bridge designers may require the application of a sprayed seal to the concrete bridge deck prior to the placement of asphalt to enhance the deck's waterproofing. In most cases, a regulation layer of asphalt will also be placed before the surfacing course to correct the deck's shape as certain types of bridge decks (e.g. those with prestressed concrete beams) may have an inherent shape due to hogging or sagging etc., necessitating level adjustments in the concrete deck via asphalt regulation.

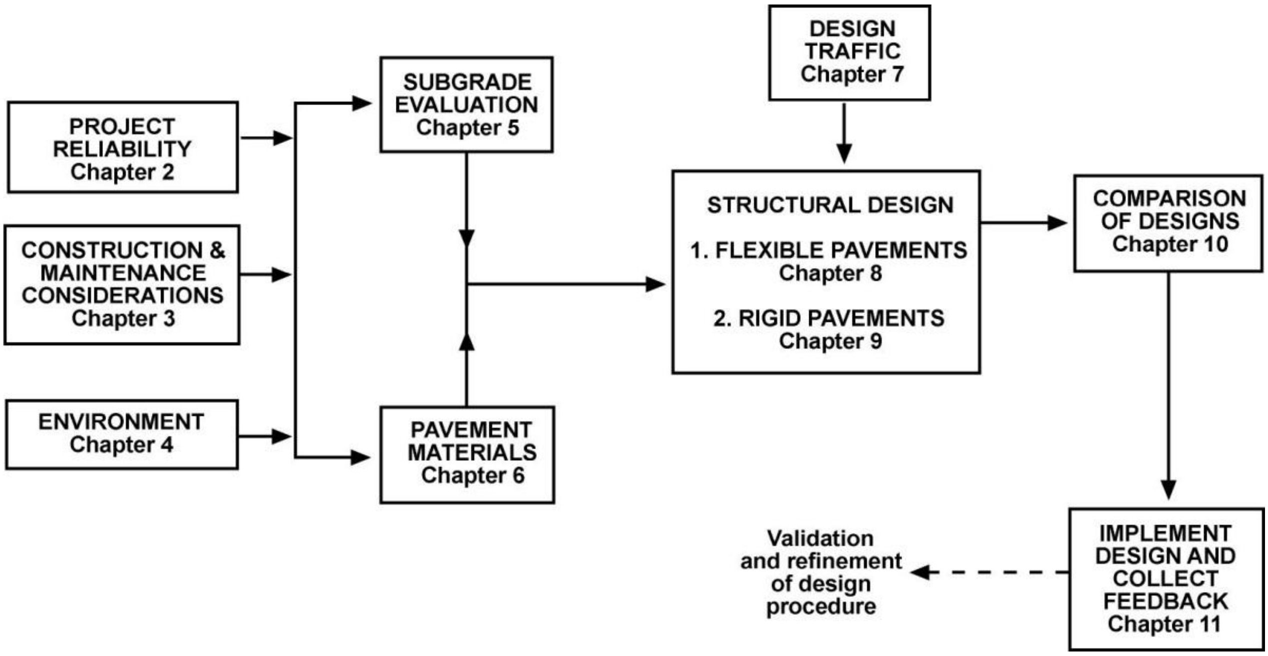
Furthermore, the regulation layer offers additional protection to the deck when the surfacing course is removed and replaced, particularly when using open graded asphalt surfacing.

2.3 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.

The method of structural analysis presented in this Part is consistent with the extent of knowledge of pavement materials and their performance which exists within the Austroads member agencies 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.

Figure 2.1: Pavement design system



2.3.1 Input variables

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. It is a fundamental parameter in the entire pavement management process.

Pavement structural designs are undertaken using the estimated design traffic that will occur over the design period.

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, axle loads and composition must be estimated during the design period. Detailed consideration of traffic is presented in Chapter 7 of this Part for moderate-to-heavily trafficked roads, while Section 12.7 describes additional requirements for lightly trafficked pavements.

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.

Project reliability

An integral part of the pavement design process is an assessment 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 Part
- the pavement is constructed in accordance with standard specifications
- 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 range (%)	Typical project reliability (%)
Freeway	95–97.5	95
Highway: lane AADT > 2000	90–97.5	95
Highway: lane AADT ≤ 2000	85–95	90
Main road: lane AADT > 500	85–95	85
Other roads: lane AADT ≤ 500	80–90	80

To achieve the desired project reliability in the mechanistic-empirical design of flexible pavements it is necessary to use an appropriate performance relationship to estimate allowable loading from the calculated strains induced by the design traffic axles 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 (Section 5.8) 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 3), is expected to result in appropriate levels of project reliability across the range of design traffic levels covered in this Part. Consequently, RF values are not explicitly provided within this performance relationship.

In the development of reliability guidelines (Austroads 2008), it was assumed, based on the best available information, that when pavements are:

- designed using the procedures presented in this Part
- constructed to Austroads member agency standard specification requirements
- maintained to Austroads member agency 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 agency 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 this Part. Hence there is no need to adjust granular thicknesses derived from this chart for project reliability. Construction quality and pavement material quality, particularly granular base, are critical factors affecting the reliability of these pavement configurations.

In the design of rigid pavements, the axle loads are multiplied by a Load Safety Factor (L_{SF}), which enables the design to be conducted to a selected reliability level. As well as depending on the desired reliability for the project under consideration, the L_{SF} also depends on the specific pavement type and design method being considered. The procedures are detailed in Chapter 3 of this Part.

The above-mentioned procedures relate to the reliability of pavements outlasting the Design Traffic. No detailed guidance is given in this Part 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).

Subgrade and pavement materials

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

- the strength/modulus 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 (Table 2.2)
- the limiting value(s) of stresses or strains at which a given degree of distress will occur, commonly known as the performance criteria.

Table 2.2: Distress modes for flexible and rigid pavements

Pavement type	Distress mode	Likely causes
Flexible	Rutting	Traffic associated: • densification, shoving.
	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.
	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.

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

Performance criteria (e.g. 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 modulus and other material characteristics. 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 Part.

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 subsurface and surface drainage incorporated in the design.

Construction and maintenance considerations are discussed more fully in Chapter 3 of this Part and in Parts 7 and 8 of the Guide (Austroads 2009b and 2025e respectively).

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 this Part.

2.3.2 Selecting a trial pavement configuration

The design process consists of selecting a trial pavement configuration, determining the design input parameters (Section 2.3.1), analysing the pavement's response to applied load (Section 2.3.3) and the allowable traffic loading to pavement distress (Section 2.3.4).

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 an empirical procedure 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.

The example design charts in Chapters 3 and 12 of this Part may also be used to select a trial configuration. These charts have been derived from the design system for rigid and flexible pavements 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 configuration. It can then be analysed as the next step to obtaining a more appropriate pavement configuration.

2.3.3 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/moduli. 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 of pavement layers 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.

Details of the methods of structural analysis which have been adopted for this Part are given in Chapters 8 and 9 for moderately-to-heavily trafficked flexible and rigid pavements respectively, whilst Chapter 12 details methods for lightly trafficked pavements.

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

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 traffic 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 'not bound') material (Section 8.2.7).

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 structural 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 structural 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 consideration of rutting and shape loss 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 this Part to design rigid pavements and for the consideration of fatigue in asphalt and cemented materials in flexible 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 distress 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 modulus may have to be made. The new pavement configuration is then re-analysed and a new distress prediction made.

2.3.5 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 including infrastructure resilience and sustainability of materials and processes used for construction.

Comparison of alternative designs is discussed in Chapter 10 of this Part.

3. Construction and Maintenance Considerations

3.1 General

The design procedures presented in this Part assume that appropriate standards of construction and maintenance practice will be adopted. For moderately-to-heavily trafficked roads, such standards are well-documented in specifications of individual Austroads member agencies and other Austroads publications.

Section 12.2 provides guidance on construction and maintenance considerations for the design of lightly trafficked pavements.

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
- acceptable risk.

Part 7: Pavement Maintenance, Part 8: Pavement Construction, and *Part 10: Subsurface Drainage* (Austroads 2009b, 2025e and 2009c respectively) of the Guide provide further guidance on construction and maintenance issues.

3.2 Extent and type of drainage

Special drainage provisions may be provided, including subsurface 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 binders (refer Parts 4D and 4L of the Guide).

In high rainfall regions, areas subject to high ground-water levels, tunnels and underpasses, 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 or no fines concrete). 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. Geotextiles have been used in lieu of fine filter material.

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-bound 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 *Part 10*: *Subsurface Drainage* of the Guide (Austroads 2009c).

3.2.1 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
- 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.
- 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 in pavement performance.

3.2.2 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
- 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 3×10^{-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). Section 5.4 of Part 10 of the Guide 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 material in which the subsurface drain is placed is an expansive soil
- it is difficult to connect the required layer to a drain below subgrade level (e.g. rock cuttings)
- 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. refer to 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 subsurface 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 subsurface drainage system to drain the less permeable material.

3.2.3 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 to improve its strength when wet.

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.

Part 10 of the Guide (Austroads 2009c) provides further details.

3.2.4 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. *Part 4I: Earthworks Materials* of the Guide (Austroads 2009d) provides more detailed advice.

3.2.5 Full depth asphalt pavements on moisture-sensitive subgrades

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 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 subgrade design CBR of less than 5%. An in situ subgrade CBR in excess of 10% is required at the time of construction to achieve adequate compaction of the asphalt layers.

3.2.6 Treatment of stormwater run-off

Stormwater run-off from roads has the potential to impact aquatic and terrestrial ecosystems through changes to water quality, water quantity and water flow path. It is important to manage stormwater to reduce such pressures on sensitive receiving environments. In addition, fill materials used in embankments should have characteristics that minimise erosion, dispersion of material, etc.

Austroads *Guide to Road Design Part 5: Drainage* (Austroads 2025x) includes guidance on the selection and design of road run-off treatment measures, hydrologic design standards and design computations.

3.3 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 subsurface 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 subsurface 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.

3.4 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.

3.5 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 is 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
- difficulties associated with achieving a high standard of construction with some paving materials if construction under traffic is necessary (Section 3.11).

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 may be 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-empirical design process allows the likely life of the pavement configuration of each stage 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.

3.6 Use of stabilisation

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
- reduce layer thicknesses compared to unbound materials.

Part 4D: Stabilised Materials of the Guide (Austroads 2019a) contains comprehensive information on most forms of soil and pavement material improvement.

For the purposes of pavement design, 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 inhibit 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 for cemented materials are discussed in Chapter 8. For lightly trafficked roads it is common to use a thin bituminous surfacing and surface cracking may appear several years after construction. Section 6.5.4 lists alternative measures to reduce reflection cracking.

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.

3.7 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. In some situations it may be necessary to provide a sprayed seal immediately under open-graded asphalt wearing surfaces to inhibit the ingress of moisture into the lower layers.

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

For some heavily trafficked asphalt pavements a high bitumen content asphalt fatigue layer at the bottom of full depth asphalt pavements has been used. 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 Part 10 of the Guide (Austroads 2009c) provides more details.

Construction specifications commonly include minimum and maximum layer thicknesses for compaction according to material size. Such limits need to be considered in selecting trial pavement configurations.

3.8 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 possibly a geotextile. SAMIs have proved to be effective in the reduction of reflective cracking. Refer to Part 3 of the Guide (Austroads 2025b) for more details.

A SAMI placed beneath an asphalt layer is normally applied at a rate of 1.8 to 2.3 L/m^2 and covered with a light coating of size 10 mm or size 14 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 asphalt incorporating a polymer modified binder (PMB). The effectiveness of a SAMI diminishes significantly when 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.

SAMIs are typically applied over existing bituminous treatments (such as a sprayed seal or asphalt). Where an existing bituminous treatment is not present, a sprayed bituminous treatment, such as a prime or an initial seal, must be applied prior to the application of the SAMI.

3.9 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 Part 3 of the Guide (Austroads 2025b).

3.10 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. Refer to Part 3 of the Guide (Austroads 2025b) for more details.

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

3.11 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 microcracking 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 two days, compared to a seven-day delay in opening. A delay of three to four 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 seven 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 in situ stabilisation is undertaken for pavement strengthening and rehabilitation, acceptable pavement performance has been obtained after allowing immediate access to traffic.

3.12 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-surfaced 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.

3.13 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) 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 (Section 8.2.8).

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 varies with the function of the road for which the pavement is being designed.

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

3.14 Improved subgrades

3.14.1 Soft subgrades

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

Subgrades with an in situ California Bearing Ratio (CBR) less than about 5% at the time of construction may require treatment to avoid delays in construction and assist in compaction of subsequent pavement layers.

Treatment 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 using lime and/or cement as appropriate
- provision of a working platform of cemented material
- provision of a lean-mix concrete working platform
- use of geotextiles
- a combination of the above.

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

3.14.2 Improved layers under bound layers

The provision of an improved subgrade (i.e. selected subgrade materials or stabilised subgrade) – or granular subbase layer of at least 150 mm thick – under asphalt or cemented materials layers provides the following benefits:

- reduces the likelihood of damage after construction to the bound layer due to heavy construction traffic
- protects subgrade from rainfall and trafficking during construction
- improves the compaction of bound layers
- provides greater resistance to erosion and pumping of fines through shrinkage cracks in cementitiously bound pavements
- reduces local stresses around shrinkage cracks in the cemented materials layers, and the effects of local subgrade softening and swelling due to the ingress of moisture
- for some pavement types with cemented materials, improves pavement life by providing a longer post-cracking phase life after the cemented layer has fatigued (Section 8.2.7).

3.15 Surfacing type

The desired characteristics of the wearing surface will impact on the selection and design of the pavement. Pavements incorporating 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 seal. 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 *Part 3: Pavement Surfacings* of the Guide (Austroads 2025b).

3.15.1 Sprayed seals

For many rural roads, a prime and seal or initial seal (formerly called a primerseal) is used. An initial seal 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 better suited to circumstances such as:

- to apply a robust sprayed seal 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
- for work using unmodified bitumen emulsion at total binder rates of application exceeding 1.5 L/m^2 (to avoid runoff of excess emulsion)
- for pedestrian and parking areas, town streets and other areas where a reasonably smooth, quieter fine textured surface is required
- to attain a longer life.

Part 4K: Selection and Design of Sprayed Seals of the Guide (Austroads 2019b) provides guidance on the design of sprayed seals.

3.15.2 Asphalt or concrete surfaces

These surfaces are used:

- on urban freeways/arterials 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 sprayed seals
- where the use of an asphalt surface is required for vehicle tyre noise reduction.

3.15.3 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
- better high speed skid resistance in wet conditions.

OGA is being increasingly used as the surfacing on major high speed 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, on the low side of the pavement, to facilitate its drainage.

3.15.4 Surfacings in tunnels

The service life of asphalt surfacings within tunnels can be longer than on the more exposed open roads. Nevertheless, high performance surfacings may be warranted considering the disruption, dust and fumes of asphalt removal and replacement operations within tunnels. Open graded asphalt and other porous surfacings are unsuitable surfacings as they may retain flammable or toxic spillages arising from an incident which makes clean up almost impossible and firefighting more difficult (Highways Agency 1999).

The surface of concrete bases in tunnels are long lasting and can provide the following benefits:

- reduced lighting requirements due to better reflectivity
- longer maintenance cycles
- improved long-term skid resistance.

Diamond grinding of concrete surfaces can provide:

- improved ride quality
- long-term macrotexture and skid resistance.

3.16 Pavement widenings

When designing pavement widening consideration needs to be given to the following to lessen the likelihood of premature distress along the longitudinal joint and in adjoining pavement areas:

- If appropriate, and where possible, widenings should have similar pavement stiffness to the existing pavement as this will reduce the likelihood of a crack developing along the longitudinal joint.
- If appropriate, and where possible, the permeabilities and layer thicknesses of the widening and existing pavement should be similar to assist the drainage of moisture to the outer edge. It is also desirable for the subgrade level of the widening to be at or below the subgrade level of the existing pavement.
- As a crack may develop along the longitudinal joint, consideration needs to be given to providing a longitudinal pavement drain below subgrade level to drain any moisture entering the pavement. Such drainage may also be required where the permeabilities and layer thicknesses of the widening and existing pavement differ, as such differences may lead to moisture accumulating at the interface as it drains towards the outer edge on the low side of the pavement. Subsurface drains in trafficked pavement areas may require a no-fines concrete backfill to reduce the likelihood of further compaction and therefore pavement deformation under traffic.
- The structural competency of the pavement at longitudinal joints is generally not as sound as in other areas. This may be due to reduced compaction, lack of aggregate interlock, or material segregation and may lead to cracking and deformation near the joint. Where appropriate and practical, longitudinal joints may be offset from one layer to the next by not less than 150 mm.
- It is preferable to locate longitudinal joints away from the wheel paths. Consideration should be given to locating longitudinal joints within 300 mm of the planned position of traffic lane lines or within 300 mm of the centre of a traffic lane.
- A geotextile or SAMI may be useful in delaying the onset of longitudinal cracking along the joint.

4. Environment

4.1 General

This Part 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 the major distress mode is environmentally induced are not specifically discussed in the Part except for lightly trafficked pavements (Chapter 12).

The environmental factors which significantly affect pavement performance are moisture, and temperature. Freeze/thaw conditions are not discussed in this Part 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 modulus/strength of unbound materials and subgrades is heavily dependent on the moisture content of the materials. The design, construction and maintenance of subsurface drainage systems for roads are described in Parts 8 and 10 of the Guide (Austroads 2025e and 2009c respectively). Austroads *Guide to Road Design Part 5: Drainage* (Austroads 2025f) includes guidance on good design process and practice, and focuses on the hydraulic design of the drainage systems and facilities, including the required design charts and formulae.

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

- the rainfall/evaporation pattern
- reactivity of subgrade to variation in the moisture regime
- permeability of the wearing surface
- depth of the watertable or to water-bearing strata
- relative permeability of pavement layers and their location within the pavement structure
- whether or not to seal shoulders
- type of vegetation to be used in medians or on verges, and their proximity to the pavement
- geometric design including the form of pavement construction (boxed or full width)
- pavement drainage, e.g. availability of table-drains, subsurface drainage, etc.
- provision made for surface run-off, e.g. the number of stormwater pits, the effectiveness and proximity of table drains, 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
- 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
- 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 equilibrium moisture contents above optimum moisture content yet, because they are still only partially saturated, they cannot be drained. A detailed discussion of subsurface drainage, including the design of filter systems, is contained in Part 10 of the Guide (Austroads 2009c).

Potential problems with shallow water tables may be identified by any of the following observations on adjacent roads:

- failures along the edge of a channel or at the junction of two dissimilar pavement types
- free water (seepage) apparent on the surfacing particularly from a cracked surface, and fines staining adjacent to a crack
- failures predominantly in the sag of a road
- unusual growth of grass at specific locations on the verge or nature strip
- geology of the area – specifically the presence of open jointed or fractured rock material in cuttings. These formations frequently have permeable layers which may allow in high seepage flows.

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/modulus.

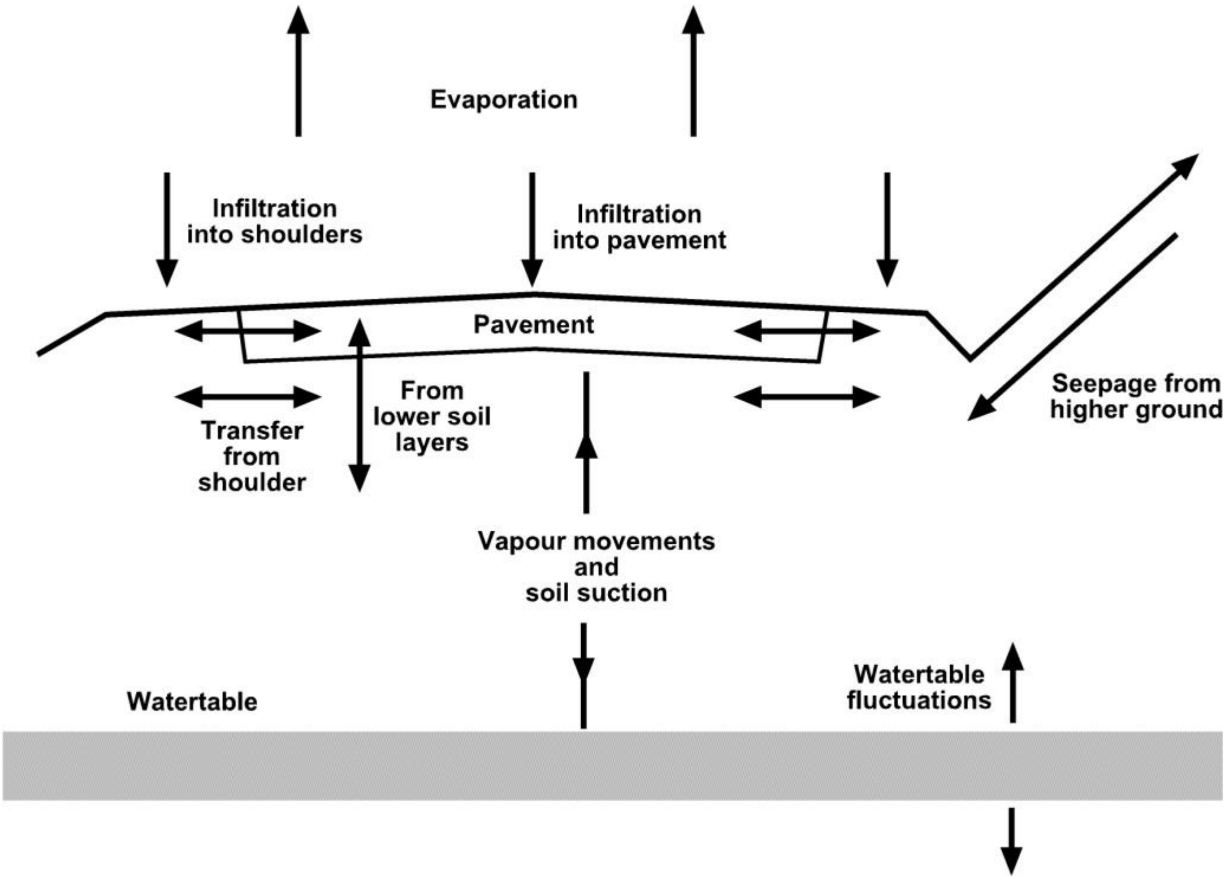
Most of the above sources of moisture infiltration are illustrated in Figure 4.1.

An alteration of the subgrade moisture content 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/modulus 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/modulus.
- For silty soils, small fluctuations in moisture content produce little change in volume but may produce large changes in strength/modulus.
- For clay, small fluctuations in moisture may produce large variations in volume and, if the moisture content is near optimum moisture content, then large changes in strength/modulus may also occur.

Figure 4.1: Moisture movements in road pavements



Particular problems associated with expansive soils are discussed in Section 5.3.5.

In salt-affected areas and high water tables, salinity may adversely affect pavement performance. The *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/modulus of the subgrade is taken into consideration by evaluating the strength/stiffness 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 (Section 5.3.5).

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

4.2.1 Equilibrium moisture content

The term equilibrium moisture content describes a condition 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:

- drainage
- climate
- soil type
- depth to watertable
- vegetation
- 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)
- 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 National Association of Australian State Road Authorities (NAASRA) (1974), Organisation for Economic Co-operation and Development (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 one to two 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 wheel path 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 this Part.

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 distress 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 distress mode directly considered in this Part 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 modulus increases with time and, in the case of thick asphalt layers, increases of the order of four from the initial modulus over the design life have been recorded without performance being affected (Butcher 1997; Chaddock and Pledge 1994; 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 WMAPTs for selected sites throughout Australia and New Zealand, are presented in Appendix B.

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. 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 wheel path) of the slab.

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 modulus is dependent on the soil type, density and moisture conditions at construction and during service.

One of the principal objectives of subgrade evaluation is to determine, for design, a subgrade CBR value. A subgrade design CBR is determined for each identifiable unit defined on the basis of topography, drainage and soil type.

The guidance provided in this chapter is of a broad nature and covers the principles of subgrade evaluation for moderate-to-heavily trafficked roads. *Part 4I: Earthworks Materials* of the Guide (Austroads 2009d) provides more detailed advice. In addition, many of the Austroads member agencies have developed more detailed procedures, which are based on local conditions of climate, traffic, topography and materials. These are included in the references. Section 12.4 provides guidance for evaluation of subgrade for lightly trafficked roads.

This document defines in situ subgrade as the existing material at or below subgrade level prior to the addition of any additional earthworks materials.

For design purposes, the subgrade is the in situ subgrade but may also include earthworks, selected materials or lime-stabilised materials (Section 5.3.8) that are placed above the in situ subgrade.

5.2 Measures of subgrade support

The measures of subgrade support used in this Part 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	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
- subsurface drainage and the depth to the watertable
- 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.

5.3.2 Performance risk

The investigation methodology and the strength (or modulus) 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 can consider the use of selected or improved subgrade materials that result in significant construction savings. The pavement design can be based on the CBR of the selected or improved 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.

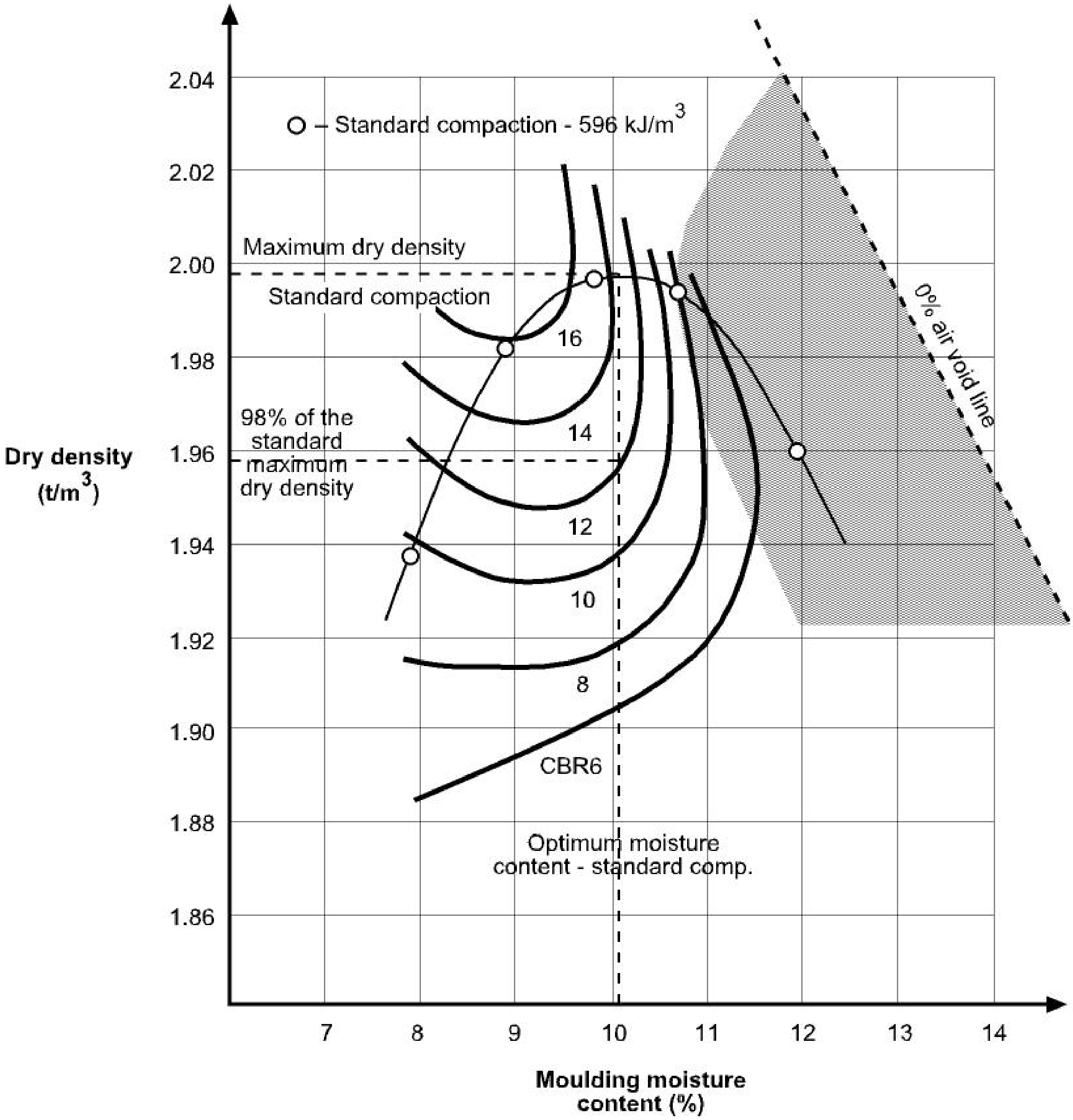
Even where the design cannot be based on selected or improved subgrade material, good management of the earthworks during construction will ensure that the best available material is used. The careful selection of subgrade materials during pre-construction will result in a more controlled subgrade.

Careful selection of subgrade material beginning at the pre-construction stage also has the significant advantage that it provides a more uniform subgrade. This leads to more uniform pavement configurations, less subgrade testing and more uniform pavement performance.

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 is shown in Figure 5.1.

Figure 5.1: Example of variation of CBR with density and moisture content for clayey sand



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.

Note that the OMC for field compaction can differ significantly from the long-term in-service moisture content as described in Section 5.3.5. This needs to be considered when selecting the moisture conditions for determination of subgrade design CBR.

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 one to two 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 – can have a major influence on the subgrade moisture conditions. Specific action should therefore be taken to guard against this influence.

The proximity of the ground water table or local perched water table to the pavement wearing surface may also play a significant role in influencing the subgrade moisture conditions. In circumstances where the height of the water table 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 water table 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.

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 swell test is preferred to the plasticity index test if facilities are available.

Table 5.2: Guide to classification of expansive soils

Expansive nature	Liquid limit (%)	Plasticity Index	PI x % < 0.425 mm	Swell (%) (1)
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

1 Swell at OMC and 98% MDD using standard compactive effort; four-day soak. Based on 4.5 kg surcharge.

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

- Construct the subgrade or fill material at a time when its soil suction (the ability of a soil to attract moisture) is likely to be near the long-term equilibrium value.
- Compact the soil at its EMC. This value occurs when a soil is at its equilibrium soil suction value (Section 4.2.1).
- Provide a low-permeability lower subbase or a 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. This capping layer should extend at least 500 mm past the edge of pavement, and if provided, past the kerb and channel, to reduce edge movement.
- 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 1.5% for the top 300 mm and less than 2.5% for the remaining thickness 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/select fill capping layer 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 one to 1.5 m is required outside the edge of the traffic lanes to minimise subgrade moisture changes under the outer wheel path.
- 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.

5.3.6 Pavement cross-section and subsurface 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 moisture 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 subgrade design CBR.

Cross-section types with relatively high permeability pavement materials either ‘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 homogeneous 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.

Where strength decreases with depth, the subgrade may be sublayered for the purposes of the mechanistic-empirical pavement design of flexible pavements and when calculating the effective subgrade strength for rigid pavement designs (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.

5.3.8 Lime-stabilised subgrades

The potential uses of lime-stabilised subgrades are described in Section 3.6.

For subgrades stabilised with sufficient lime to ensure design properties achieve long-term strength (refer to *Part 4D Stabilised Materials*, Austroads 2019a), provision is made in the thickness design calculations (Chapters 8 and 12) for the structural contribution of the stabilised subgrade.

The mix design procedures described in Part 4D provide two approaches to determine the required lime content after the minimum value to ensure long-term strength properties have been established:

- Method A: requires the lime content such that the unconfined compressive strength (UCS) is within the range 1.0 – 1.5 MPa tested unsoaked after 28 days curing. Generally, this strength requirement increases the lime content above that to satisfy the lime demand test.
- Method B: requires the CBR of the material to be tested and the lime content adjusted to satisfy the required design CBR.

Due to their higher lime contents, the strength and modulus of materials designed using Method A are generally higher than Method B.

Structural thickness design procedures (Chapters 8, 9 and 12) are based on Method B, the design CBR and the design modulus assigned to the stabilised subgrade. For the purposes of pavement design, the stabilised subgrade should generally be assigned a design CBR value not exceeding 15% irrespective of measured CBR values. A maximum design modulus of 150 MPa is normally adopted for lime-stabilised subgrade materials.

If the amount of lime is insufficient to achieve enhanced properties long-term, no allowance should be made for the change in design CBR due to stabilisation.

5.4 Methods for determining subgrade design CBR value

One of the principal objectives of subgrade evaluation is to determine, for design, a subgrade CBR value at the density and moisture conditions which are expected to prevail in-service for the long-term. A subgrade design CBR is determined for each identifiable unit defined on the basis of topography, drainage and soil type.

As the moisture conditions at the time of construction usually differ from those used in the determination of the design CBR (refer to Section 5.6.2), it is seldom appropriate for in situ field testing during construction to increase the design CBR previously assessed unless a comprehensive investigation concludes the materials encountered are significantly better. Nevertheless, during construction the subgrade is exposed and this often provides an opportunity to confirm that the design CBR has not been overestimated by sampling the full range of subgrade materials and undertaking laboratory testing at appropriate field densities and moisture conditions (Section 5.6.2). In situations where in situ CBR testing is considered appropriate and reliable, this can also be used to confirm or reduce the design CBR value.

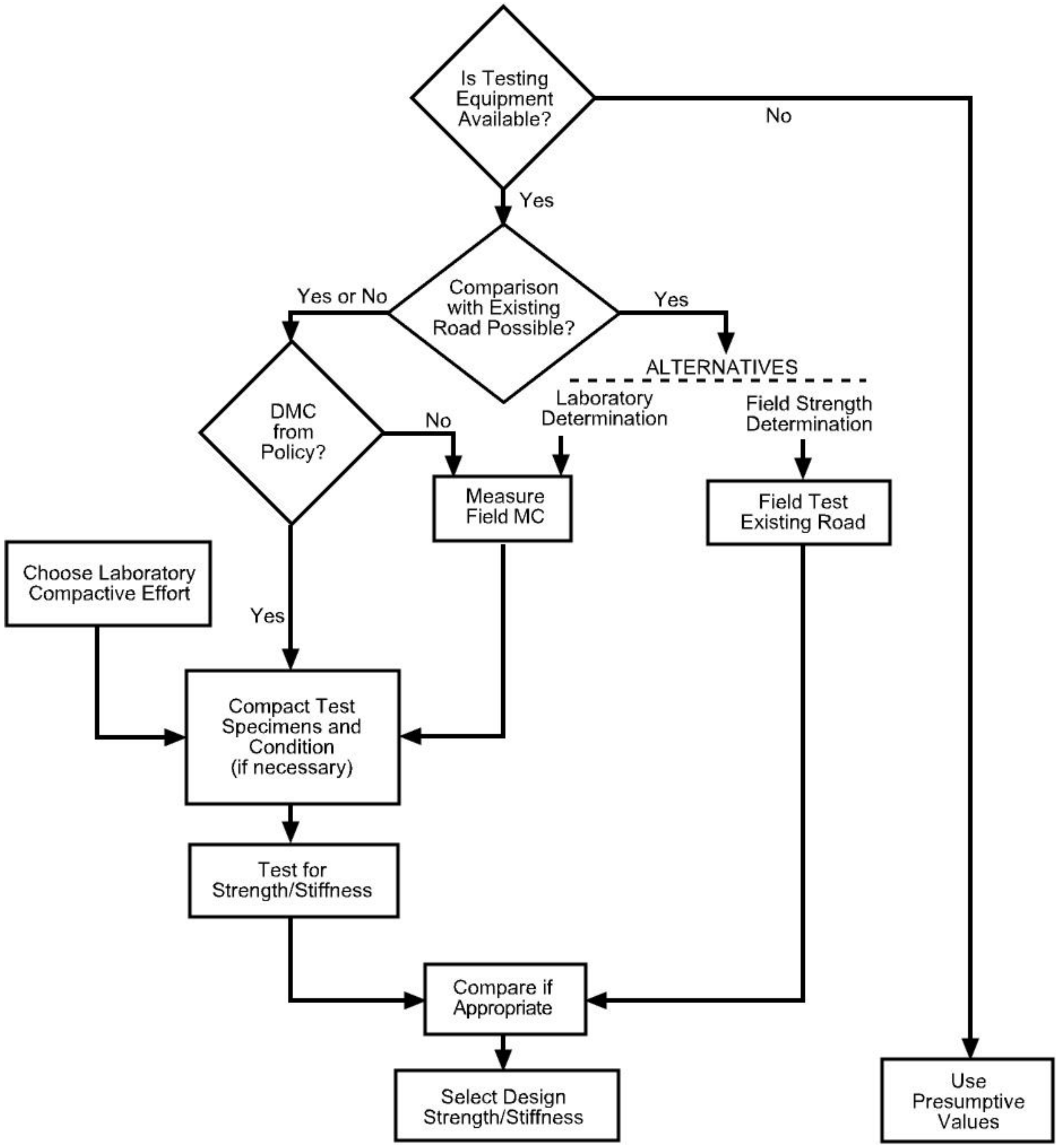
There are primarily two modes of testing available for estimating subgrade support values: field testing and laboratory testing.

Field testing is applicable to situations where the support values from the in situ 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.

Figure 5.2: Methods for estimating subgrade support values



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 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. in situ CBR test or cone penetrometer (Sections 5.5.1 and 5.5.2).

If the testing interval and data are 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. 90% of results exceed this level) 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 2018, 2019). VicRoads (1995) provides guidance on field testing of subgrades.

5.5.1 In situ CBR test

The in situ CBR test should be carried out in accordance with AS 1289.6.1.3. This test is time-consuming and expensive and its best application is usually as a supplement to other forms of testing.

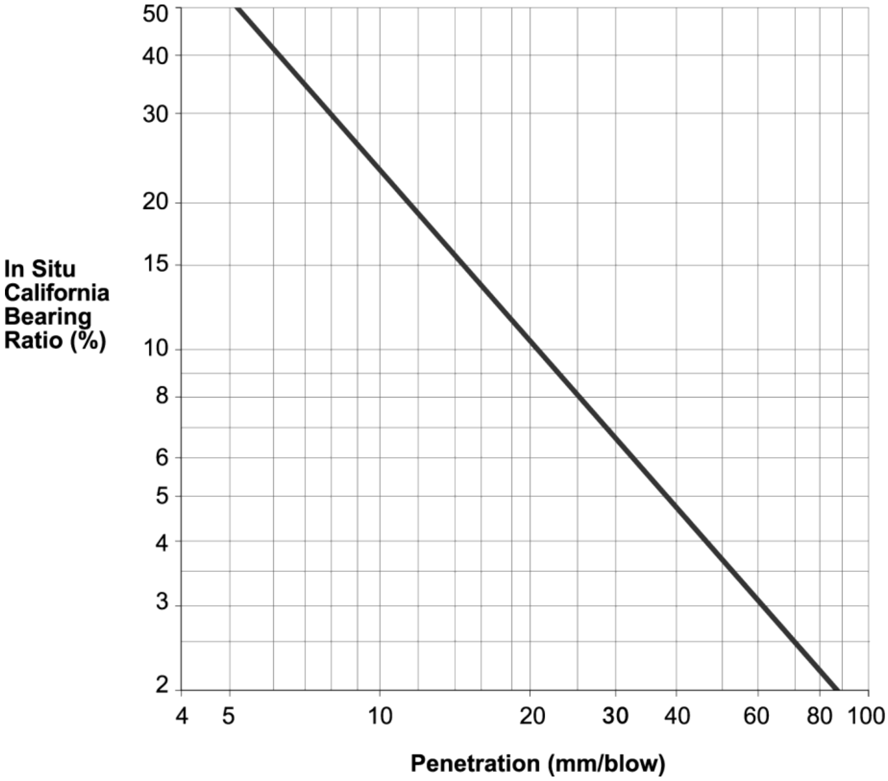
5.5.2 Cone penetrometers

Cone penetrometer tests should also be carried out in accordance with AS 1289.6.3.2. 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. Mulholland (1984), Schofield (1986) 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.

Figure 5.3: Correlation between dynamic cone penetration and CBR for fine-grained cohesive soils



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 2017). 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 in situ subgrades by a theoretical model that requires defined sublayer thicknesses.

The back-calculated modulus values are generally within a factor of 2 of the true value, and this may be unacceptable for many situations. The EFROMD2 user manual (Vuong 1991) 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.

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

5.6 Laboratory determination of subgrade CBR and elastic parameters

Laboratory procedures 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 in situ testing.

Laboratory tests may be undertaken on specimens tested at a density which corresponds 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 subgrade CBR of the material can then be determined from interpolation of the results for these specimens.

The test procedures for the laboratory CBR test are given in AS 1289.6.1.1 (laboratory moulded specimens) and AS 1289.6.1.2 (undisturbed specimens) and materials testing manuals of Austroads member agencies.

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.

If the testing interval and data are 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. 90% of results exceed this level) is commonly adopted as the design CBR of highway pavements. For roads in arid climates, or roads of lesser importance, higher percentile values may be appropriate (VicRoads 2018, 2019).

For thickness design purposes using mechanistic-empirical 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). In this Part, 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 (Equation 1).

$$f = \frac{\text{Vertical modulus}}{(1 + \text{Poisson's ratio})} \tag{1}$$

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

The vertical modulus of subgrade can be determined from laboratory testing of conditioned specimens (Thompson and Robnett 1976) or by using the empirical relationship (Equation 2).

$$\text{Modulus (MPa)} = 10 \times \text{CBR} \tag{2}$$

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). This equation may overestimate the subgrade moduli for soils with relatively high CBRs and hence should only be used for CBR ≤ 15. 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:

- in situ density of undisturbed or reworked subgrade as appropriate
- minimum standard of compaction achieved in construction (embankments)
- 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.

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

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.

Table 5.4: Typical presumptive subgrade design CBR values

Description of subgrade		Typical CBR values (%)	
Material	Unified Soil Classification	Excellent to good drainage	Fair to poor drainage
Highly plastic clay Silt	CH ML	5 4	2–3 2
Silty-clay Sandy-clay	CL CL	5–6	3–4
Sand	SW, SP	10–18	10–18

5.8 Limiting subgrade strain criterion

In the mechanistic-empirical design of flexible pavements (Sections 8.2 and 12.8), 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 wheel paths, 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.

Note that the vertical compressive strain at the top of the subgrade is taken as a determinant for surface rutting in the unbound portions of the pavement structure. Monismith, Sousa and Lysmer (1988) express the logic for the use of the subgrade strain as a measure of surface rutting:

> *In pavement materials the magnitude of the plastic strain is directly proportional to the magnitude of the (vertical) elastic strain. In a pavement system the elastic strain increases from the subgrade to the surface. Accordingly, by setting the elastic strain at the subgrade surface at a specific value, the elastic strain in the components above this plane are controlled as are the values for the associated plastic strains. Integration of the plastic strains over the pavement section provides a measure of the permanent deformation (rut depth) which will occur at the pavement surface.*

The limiting strain criterion for the subgrade is given in Equation 3. The criterion is applicable to in situ and selected and improved subgrades. It was derived by applying the mechanistic-empirical procedure presented in Chapter 10 to a range of pavements selected using Figure 8.4 and the 90% confidence design chart for lightly trafficked pavements in Austroads Pavement Research Group (APRG) (1998), which provides similar thicknesses to Figure 12.2. It represents a 'best fit' relationship. Pavements designed in accordance with Figure 8.4 and Figure 12.2 have been found to provide satisfactory service under Australian conditions.

$$N = \left[\frac{9150}{\mu\varepsilon}\right]^7 \qquad 3$$

where

N	=	the allowable number of repetitions of a Standard Axle at this strain before an unacceptable level of pavement surface deformation develops (units of ESAs)
$\mu\varepsilon$	=	the vertical compressive strain (in terms of microstrain), developed under a Standard Axle, at the top of the subgrade

The use of this relationship in the mechanistic-empirical procedure presented in Sections 8.2 and 12.8 will therefore produce designs that are generally consistent with observed performance of road pavements throughout Australia in terms of the development of pavement surface deformation. However, its use for the design of pavements that carry significantly different loads to road pavements should be treated with considerable caution.

6. Pavement Materials

6.1 General

The design procedures in this Part 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 six categories according to their fundamental behaviour under the effects of applied loadings:

1. unbound granular materials
2. modified granular materials
3. lightly bound cemented materials
4. heavily bound cemented materials
5. asphalt
6. concrete (in rigid pavements).

Part 4: Pavement Materials (Austroads 2025e) provides general advice on the selection of pavement materials, whilst various sub-parts provide detailed advice on each material type.

Whilst foamed bitumen-stabilised material rehabilitation treatments are considered in *Part 5: Pavement Evaluation and Treatment Design* of the Guide (Austroads 2025a), design procedures for new pavements are still under development.

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

The manner of characterising the categories of material, other than base concrete for rigid pavements, is determined by the available design procedures (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 flexible pavements, utilises a computerised analytical technique 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’s ratio. The manner of characterising these materials is discussed in Sections 6.2 to 6.6. Section 12.6 provides additional guidance for lightly trafficked roads.

6.2 Unbound granular materials

6.2.1 Introduction

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 the Guide: *Part 4A*: *Granular Base and Subbase Materials* (Austroads 2024b) and *Part 4J: Aggregate and Source Rock* (Austroads 2025g). Their performance is largely governed by their shear strength, modulus 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.

Table 6.1: Pavement material categories and characteristics

Characteristics	Pavement material category					
	Unbound granular (Section 6.2)	Modified granular (Section 6.3)	Lightly bound cemented (Section 6.6)	Heavily bound cemented (Section 6.5)	Asphalt (Section 6.7)	Concrete (Section 6.8)
Material types	• Crushed rock • Gravel • Soil aggregate • Granular-stabilised materials	• Bitumen-stabilised materials • Chemically modified materials • Various cementitiously modified materials, including blends with lime	• Various cementitiously stabilised materials, including blends with lime	• Various cementitiously stabilised materials, including blends with lime • Lean-mix concrete for flexible pavements (Section 6.8.3)	• Asphalt	• Lean-mix concrete subbases for flexible pavements (Section 6.8.3) • Lean-mix concrete for rigid pavement • Concrete bases for rigid pavements (Section 6.8.4)
Behaviour characteristics	• Development of shear strength through particle interlock • No significant tensile strength • Properties are moisture sensitive	• Development of shear strength through particle interlock • No significant tensile strength	• Development of shear strength through particle interlock and weak chemical bonding • Low 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	• 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 • Degree of anisotropy	• Modulus • Poisson’s ratio	• Modulus • Poisson’s ratio	• 28-day flexural strength or 28-day compressive strength

Characteristics	Pavement material category					
	Unbound granular (Section 6.2)	**Modified granular (Section 6.3)**	**Lightly bound cemented (Section 6.6)**	**Heavily bound cemented (Section 6.5)**	**Asphalt (Section 6.7)**	**Concrete (Section 6.8)**
Performance criteria	Current materials specifications (not covered in this Part)	Current materials specifications (not covered in this Part)	Current materials specifications (not covered in this Part)	Fatigue relationships (Sections 6.5.5 and 6.8.3)	Fatigue relationships (Section 6.7.10)	• Flexible pavement lean-mix concrete subbases: Fatigue relationship (Sections 6.5.5 and 6.8.3) • Rigid pavement bases: Fatigue and erosion relationships (Section 9.4.2)

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 modulus of the granular material in-service.

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 (Figure 8.4 and Figure 12.2).

When the mechanistic-empirical 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-empirical 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 pavement model 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 (Figure 6.3). It has been found that the stress dependency of vertical modulus can be modelled using the linear elastic model by dividing the granular layers into several sublayers, with each assigned a vertical modulus to reflect the stress level at which it operates (Section 8.2.3). However, the horizontal component of stress dependency cannot be directly modelled using a linear elastic model. The use of finite element models, which allow for stress dependent and anisotropic behaviour in both the vertical and horizontal directions, would enable a more exact fit between calculated and measured deflections.

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.

Dry density and moisture content

The modulus depends on both density and moisture content. Figure 6.1 and Figure 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.

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 where seepage is likely
- other situations where the water table is within 1 m of the subgrade level.

Table 6.2: Factors affecting modulus of granular 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, then decrease
Stress level – mean normal stress	Increase
Stress level – shear stress	Decrease
In-service moisture content	Decrease
Age	No change
Temperature	No change
Rate of loading	No change

Figure 6.1: Example relationships between modulus and dry density

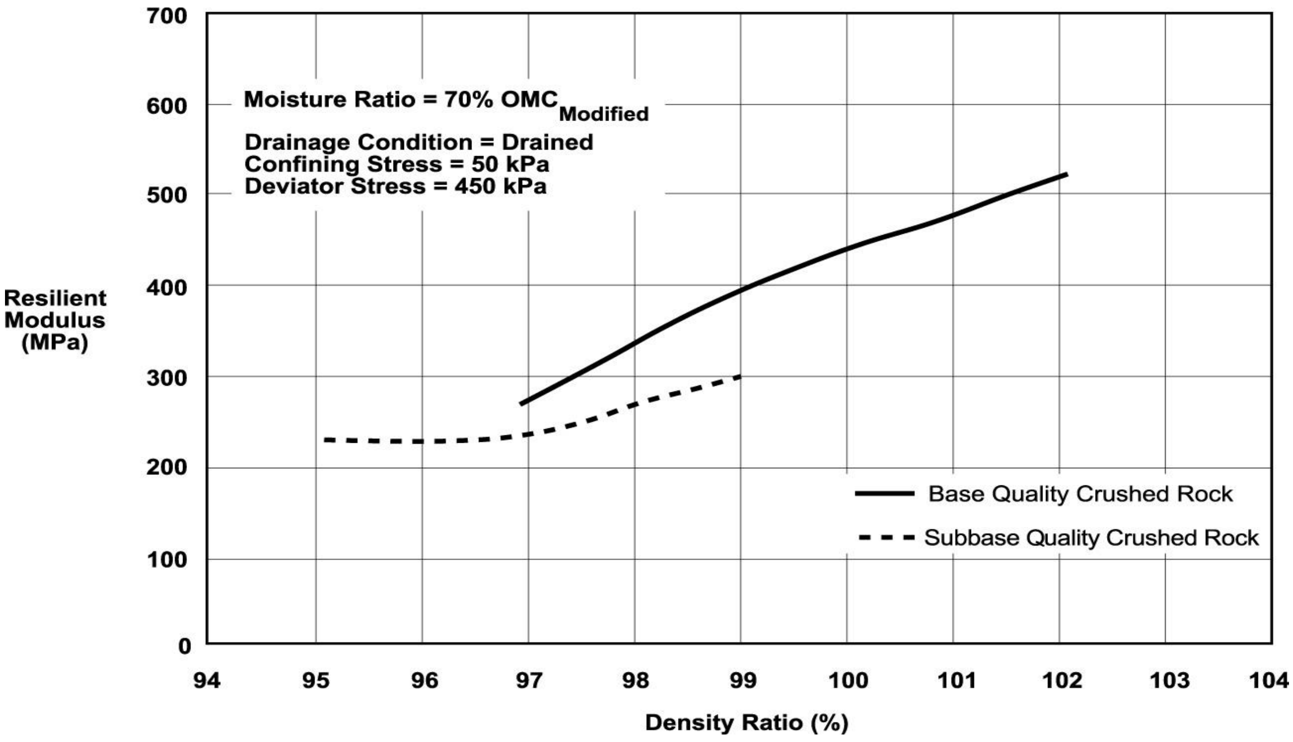


Figure 6.2: Example relationships between modulus and moisture content

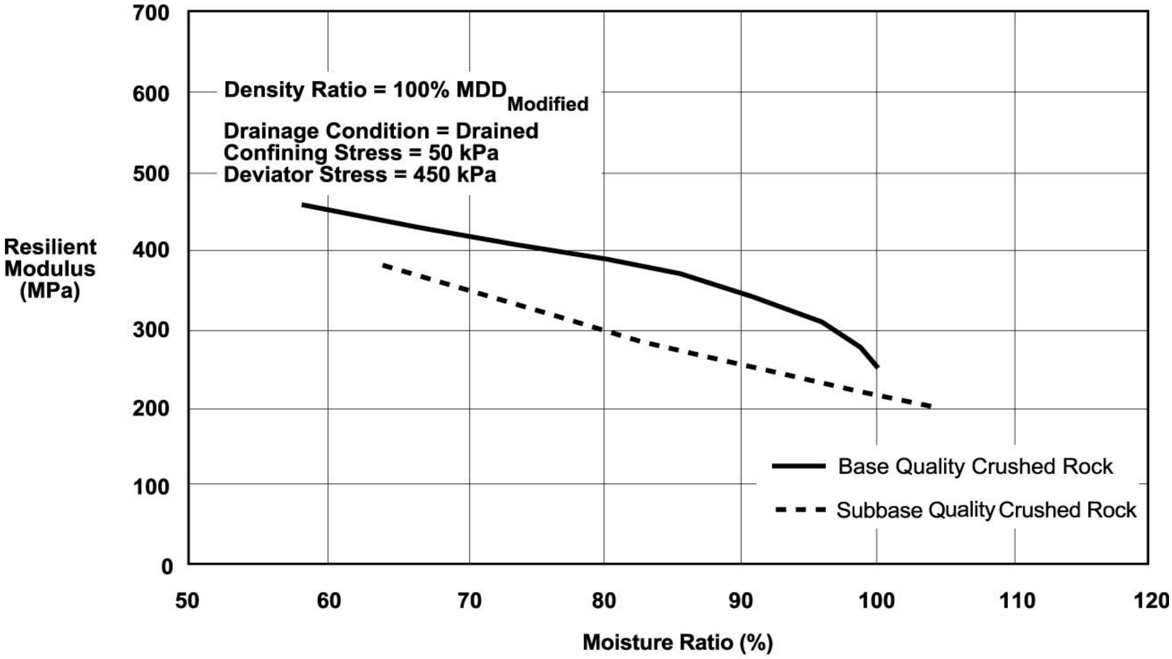
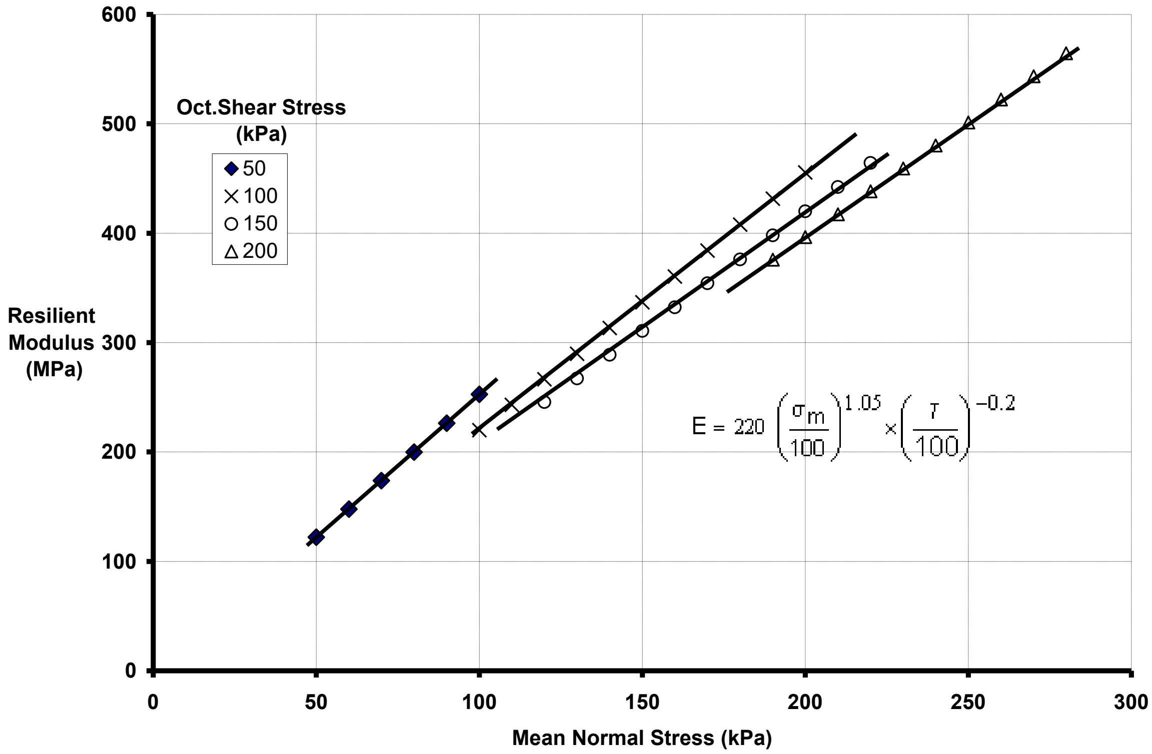


Figure 6.3: Example relationships between modulus and stress level



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 (Witczak and Uzan 1988) (Equation 4).

$$E = K_1 \times \left(\frac{\sigma_m}{\sigma_{ref}}\right)^{K2} \times \left(\frac{\tau}{\sigma_{ref}}\right)^{K_3} \qquad 4$$

where in the repeated load triaxial test:

E = resilient modulus (MPa)

σ_m = mean normal stress, $(\sigma_1 + 2\sigma_3)/3$, (kPa)

τ = octahedral shear stress, $(\sigma_1 - \sigma_3) \times \sqrt{2}/3$, (kPa)

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

K_1, K_2, K_3 = are experimental test constants

σ_1, σ_3 = axial and radial stresses respectively (kPa)

However, fine-grained soils commonly display a different response as modulus does not increase 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 an example of a relationship between modulus and stress level for a crushed rock base obtained from laboratory repeated load triaxial testing. It is apparent that the design elastic 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

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 in situ density, moisture content and stress levels under a Standard Axle.

As discussed in Section 8.2.3, it is necessary to assign a modulus to the top granular sublayer to determine the moduli of the underlying four granular sublayers. The modulus of the top granular sublayer is dependent not only on the intrinsic characteristics of the material, but also on the stress levels at which they operate, the total thickness of granular materials and the modulus of the underlying subgrade or selected or improved subgrade materials.

Guidance is provided below on procedures for assigning sublayer moduli based on the intrinsic characteristics of the material and also on the stress levels at which they operate. However, as discussed in Section 8.2.3, in determining sublayer moduli, consideration also needs to be given to the support provided by the underlying materials.

Determination of modulus of top granular sublayer

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

- direct measurement
- assigning presumptive values, e.g. Table 6.3, Table 6.4 and Table 6.5.

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 (Austroads Test Method AGPT/T053). In this simplified test, 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. (However, there is increasing interest in testing materials up to a maximum size of 37.5 mm, which requires samples 150 mm in diameter and 300 mm high). 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 used (Austroads Test Method AGPT/T053), 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.

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 agencies. 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 are 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. e.g. a minimum in situ density 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
  - the surface course.

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 vertical modulus (MPa)	300–700	200–500	150–400	150–400
Typical vertical modulus (MPa)	500	350	300	250 (1)
Degree of anisotropy (2)	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	Given by formula $f = \frac{\text{Vertical modulus}}{(1 + \text{Poisson's ratio})}$			

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

2 Degree of anisotropy = vertical modulus/horizontal modulus.

Table 6.4 and Table 6.5 may be used as a guide when assigning maximum values to base quality crushed rock 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 vary 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 the lesser of the value from Table 6.4 and 210 MPa may be used, otherwise a value of 150 MPa may be assigned. Note that the maximum moduli used for subbase should not exceed that for normal standard base as indicated in Table 6.4.

Some pavements comprise layers of different moduli of bound material (asphalt, lightly bound cemented material, cracked/uncracked heavily bound cemented material, lean-mix concrete or a combination of these) overlying granular layers. For these pavements an equivalent modulus (E_e) of the bound material may be calculated using the following Equation 5.

$$E_e = \left[\frac{\sum_i h_i E_i^{1/3}}{T}\right]^3 \qquad 5$$

where

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

E_i = modulus of bound layer i (MPa)

h_i = thickness of bound layer i (mm)

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 and Table 6.5.

Table 6.4: Suggested vertical modulus of top sublayer of normal standard base material

Thickness of overlying bound material	Modulus of overlying[(1)] material (MPa)						
	400	600	1000	2000	3000	4000	5000
40 mm	350	350	350	350	350	350	350
75 mm	350	350	350	350	340	320	310
100 mm	350	350	350	310	290	270	250
125 mm	350	340	320	270	240	220	200
150 mm	330	310	280	230	190	160	150
175 mm	310	290	250	190	150	150	150
200 mm	290	260	220	150	150	150	150
225 mm	260	230	180	150	150	150	150
250 mm	240	200	150	150	150	150	150
275 mm	210	170	150	150	150	150	150
≥ 300 mm	190	150	150	150	150	150	150

1 *Overlying material is either asphalt, lightly bound cemented material, cracked/uncracked heavily bound cemented material, lean-mix concrete or a combination of these materials.*

Table 6.5: Suggested vertical modulus of top sublayer of high standard base material

Thickness of overlying bound material	Modulus of overlying(1) material (MPa)						
	400	600	1000	2000	3000	4000	5000
40 mm	500	500	500	500	500	500	500
75 mm	500	500	500	500	480	460	440
100 mm	500	500	500	450	410	390	360
125 mm	500	490	450	390	350	310	280
150 mm	480	450	400	330	280	240	210
175 mm	440	410	360	270	210	210	210
200 mm	410	370	310	210	210	210	210
225 mm	370	330	260	210	210	210	210
250 mm	340	290	210	210	210	210	210
275 mm	300	250	210	210	210	210	210
≥ 300 mm	270	210	210	210	210	210	210

1 *Overlying material is either asphalt, lightly bound cemented material, cracked/uncracked heavily bound cemented material, lean-mix concrete or a combination of these materials.*

6.2.4 Permanent deformation

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

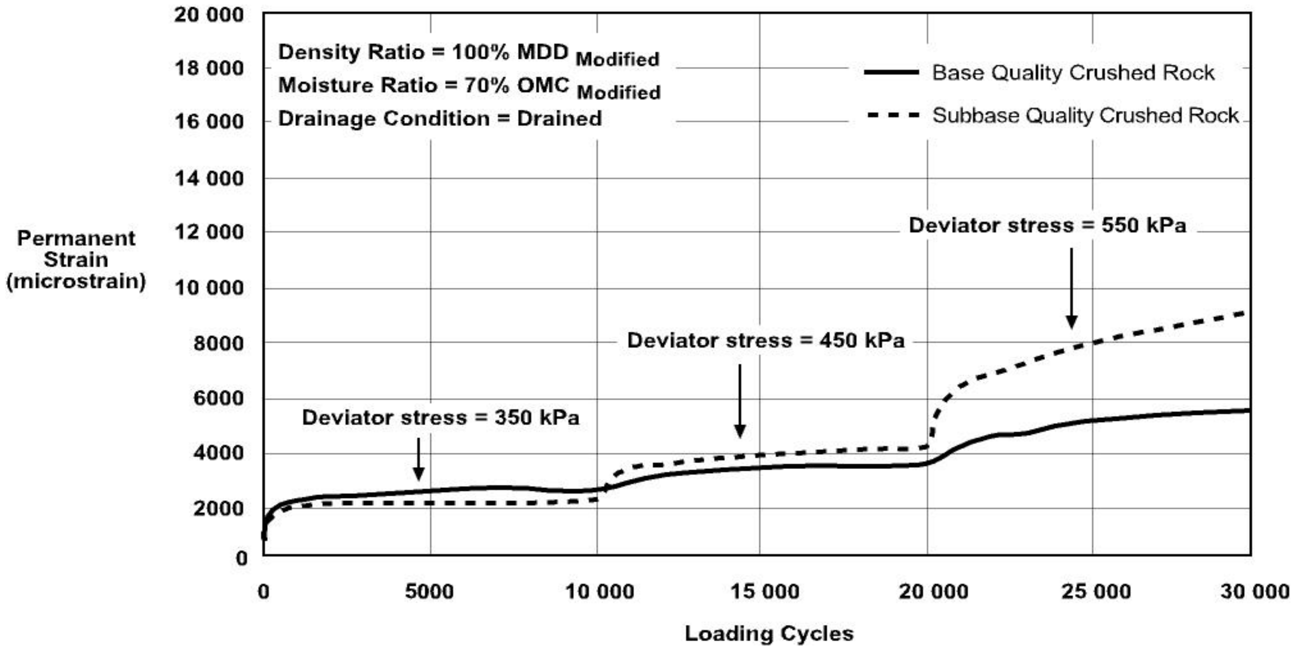
While it is acknowledged that permanent deformation is a primary distress mode for granular layers, it is not included in the procedures detailed in Chapter 8 of this Part. At this stage, there is not a suitable model to reliably predict the development of rutting in a granular material under traffic.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

To estimate rutting potential using models such as VESYS, the simplified test procedure developed can be used (Austroads Test Method AGPT/T053). 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 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. Specific procedures for translating the laboratory-based permanent deformation into design are not presently available. It should be noted, however, that recent research (Austroads 2010, 2017a) has cast doubts about the usefulness of the test in assessing the rut resistance of shear susceptible granular bases under thin bituminous surfacings. An extra-large wheel tracker test (AGPT/T054) has been demonstrated to prove a more useful means of assessing rut resistance.

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.

Figure 6.4: Example relationships between permanent strain and loading cycles



6.3 Modified granular materials

Modified granular materials are granular materials to which small amounts of stabilising binder have been added to improve modulus 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 materials have a maximum 28-day unconfined compressive strength (UCS) of 1.0 MPa, tested after moist curing but without soaking at 100% standard maximum dry density and optimum moisture content.

Modified granular materials are considered to behave as unbound granular materials, i.e. they do not develop tensile strain under load. For binders that can produce bound materials, UCS testing over a range of binder contents which reflects the variability of construction processes should be undertaken to assess the resultant field properties and performance risks.

Modified materials are characterised as unbound materials 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
- require sublayering for modelling purposes, as for unbound granular materials.

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

- direct measurement as described in Section 6.2.3
- assigning presumptive values used for unbound granular materials.

Within the design model, the lesser of the characteristic modulus determined by direct measurement, a presumptive value, and 1000 MPa, is normally adopted for modified materials. As discussed in Section 8.2.3, consideration also needs to be given to the support provided by the underlying materials in determining top sublayer modulus.

6.4 Cemented materials

6.4.1 Introduction

Cemented materials are described as a combination of a cementitious binder, 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 chemical binder and may include one or more supplementary cementitious materials such as fly ash or ground granulated blast furnace slag.

A distinction is made in this Part between lightly bound and heavily bound cemented materials.

Lightly bound materials are granular materials to which moderate amounts of stabilising binder have been added to improve modulus, and where an increase in tensile capacity may occur. Lightly bound granular materials may exhibit behaviour between modified granular materials (Section 6.3) and more heavily bound cemented materials. It is currently common practice to categorise materials with a 28-day UCS of 1.0 to 2.0 MPa as lightly bound. The binder should be added in sufficient quantity to produce a weakly bound layer with low tensile strength. For the purposes of design, lightly bound materials are considered to not have significant tensile strength, and therefore, flexural fatigue of the materials is not considered.

General categories and characteristics of cemented materials are given in Table 6.1. Further guidance can be gained in *Part 4D: Stabilised Materials* of the Guide (Austroads 2019a).

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 generally has 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.

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 6.6.

Table 6.6: 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

Mix composition

The mix composition is dependent on the pavement layer to be stabilised, traffic volume and environmental conditions, and the mix design procedures for cemented material is covered in *Part 4D: Stabilised Materials* of the Guide (Austroads 2019a). 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. Whilst the modulus increases with increased binder content, as the binder content increases so does the potential for drying/shrinkage cracking.

Density and moisture

These factors are interrelated: varying the moisture content (from optimum) will generally result in a decrease in density for a given compactive effort. Adequate compaction greatly improves the modulus 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 within the working time of the material. Slower setting binders and retarders can extend the working time of cemented materials. Where retarders are used, their application has to be carried out by a separate water tanker to avoid excess application.

Heavily bound cemented materials are usually constructed 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. 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, the stabilised layer may be sublayered in the mechanistic-empirical model with the sublayer moduli reflecting the in-service densities.

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

Ageing and curing

The modulus and strength of cemented materials stabilised with GP cement increase rapidly (compared to other cementitious binders, e.g. GB cement, slag/lime, etc.) in the first one or two days, after which they increase slowly, providing curing is sustained. The variability of properties over time is dependent on both the granular material and binder type (e.g. 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.

6.5 Heavily bound cemented materials

6.5.1 Introduction

Heavily bound cemented materials are described as a combination of a cementitious binder, water and granular material that 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 another chemical binder 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.

Characterisation for pavement design

Heavily bound 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 generally has 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.

6.5.2 Determination of Design Modulus

Definition of design modulus

For pavement design purposes the appropriate value of the modulus of heavily bound cemented materials is an estimate of the in situ flexural modulus after 90 days curing in the road-bed. It is expected that most cemented materials, even those with slow setting binders, will be substantially cured at this time with little appreciable change in properties expected beyond this time.

Alternative methods

Design moduli may be estimated from:

- flexural moduli measurements of laboratory compacted and cured beams, then adjusted to representative in situ values
- UCS tests
- presumptive values.

Issues to be considered with the laboratory determination of the design modulus of heavily bound 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
- relationships between the results of the laboratory characterisation and the properties of the material cured in situ.

A standardised test for the determination of flexural modulus for design purposes is available (Austroads Test Method AGPT/T600). Laboratory characterisation can be used to verify assumed presumptive design values or to optimise binder/host material combinations in terms of modulus and/or working time.

Forward planning of laboratory testing is essential due to the long curing times required. 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.

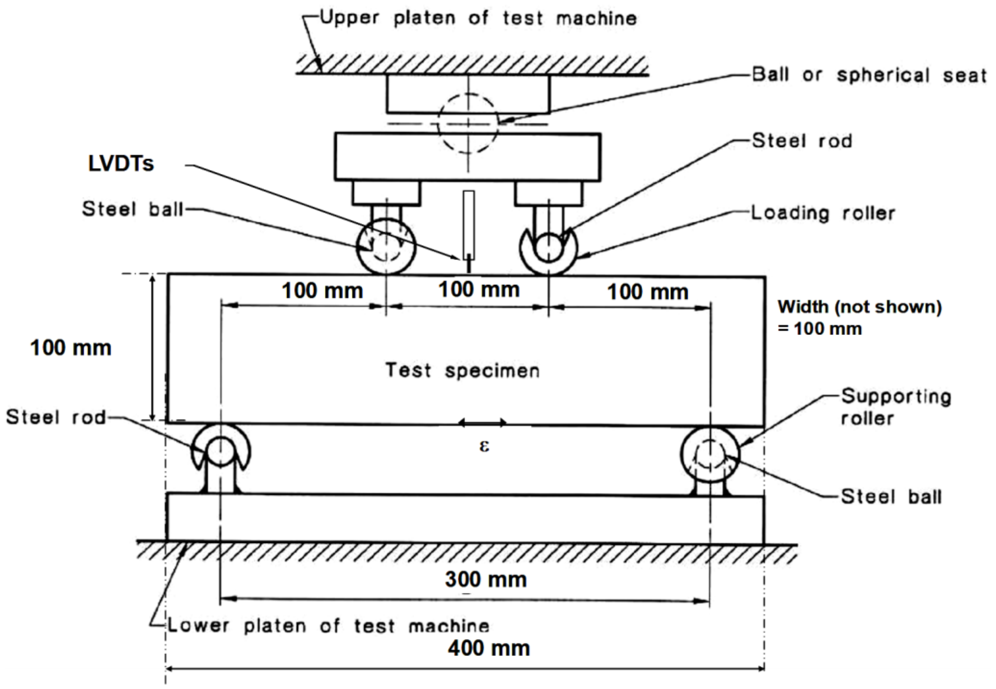
Flexural modulus measurement

The Austroads laboratory flexural modulus test (Austroads Test Method AGPT/T600) is the preferred method to determine the design modulus of cemented materials as the test conditions are considered to simulate the stress/strain gradients generated within a pavement layer.

Test slabs should be compacted at the design cement content and grading and compacted as close as possible to the selected density ratio. The Austroads test method for preparing asphalt slabs using a segmental roller may be adapted for this purpose. Test beams to the dimensions specified in the test method are saw cut from the test slabs. The beams are then moist-cured for 90 days prior to modulus testing.

In the four-point bending test (Figure 6.5), 100 haversine loading pulses (25 millisecond load duration, 75 millisecond rest) are applied to a test beam and the resulting beam deflection measured. If the test beams are to be subsequently tested for fatigue, the load level applied to the beam is adjusted such that it does not induce micro-damage during the modulus measurement. As a guide, load levels up to about 40% of the ultimate breaking load have been observed to be suitable. Otherwise, the modulus is measured at a load level that results as close as possible to the standard strain of 50 microstrain.

Figure 6.5: Cross-sectional view of flexural beam testing apparatus for cementeded materials



Source: AS 1012.11.

The flexural modulus is calculated from the applied peak load, the resulting mid-span elastic displacement, the distance between the support rollers and the beam width and height. As the modulus varies with the applied strain, the following equation is used to adjust the measured modulus to a standard strain level of 50 microstrain (Equation 6).

$$E_{50} = E_M - 40 \times (50 - \varepsilon_M) \qquad 6$$

where

E_{50} = flexural modulus standardised to a strain of 50 $\mu\varepsilon$ (MPa)

E_M = measured flexural modulus at strain ε_M (MPa)

ε_M = tensile strain during flexural modulus testing (microstrain)

The flexural modulus reported is the average of these adjusted moduli calculated between cycles 50 and 100.

The Austroads test method is given in AGPT/T600.

In the event that the laboratory flexural modulus tests were undertaken at a different dry density ratio (DR_{test}) to the in-service value ($DR_{\text{in-service}}$), the following relationship may be used to adjust the standardised measured moduli (E_{50}) (Equation 7).

It is recommended that the test specimens be prepared as close as possible to the in-service density ratio. Equation 7 is limited to up to a 3% difference in density ratio.

$$E_{\text{in-service}} = E_{50}\left(1 + 0.05 \times (DR_{\text{in-service}} - DR_{\text{test}})\right) \qquad 7$$

where

$E_{\text{in-service}}$ = flexural modulus at in-service density ratio (MPa)

E_{50} = measured flexural modulus standardised to 50 microstrain and tested at a density ratio DR_{test} (MPa)

$DR_{\text{in-service}}$ = density ratio in-service (%)

DR_{test} = density ratio of test beams (%)

Moduli of heavily bound cemented materials in-service vary markedly within a road project, with the areas low in modulus having low fatigue life and hence limiting the structural life of the project. To provide a structural design method that reflects the performance of the fatigue susceptible areas, the design modulus is estimated to be 1/3rd of the laboratory measured flexural modulus after 90 days moist curing at the in-service dry density.

For heavily bound cement treated crushed rocks and natural gravels design moduli determined using this procedure are limited to a maximum of 5000 MPa.

Estimation from UCS

Relationships have been derived between flexural modulus and other parameters such as UCS. A typical relationship for heavily bound cemented crushed rock and cemented natural gravel is (Equation 8).

$$E_{flex} = k_{UCS} \times UCS \qquad 8$$

where

E_{flex} = flexural modulus of laboratory-manufactured beams at 90 days (MPa), unsoaked

UCS = unconfined compressive strength of laboratory specimens at 28 days (MPa), unsoaked

k_{UCS} = a constant. Values of 1150 to 1400 are typically used for GP 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 (Austroads 2008). The equation should be used as a guide only as there was significant scatter in the data. Design moduli calculated using Equation 8 are limited to a maximum of 5000 MPa.

Presumptive values

The moduli of cemented materials are dependent on a number of factors such as material quality, 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 6.7 are considered appropriate for 100% standard compactive effort and may be used as a guide if no other more reliable information is available.

Table 6.7: Presumptive values for elastic characterisation of heavily bound cemented materials

Property	Base 4–5% cement(1)	Subbase quality crushed rock 3–4% cement(1)	Subbase quality natural gravel 4–5% cement(1)
Range of modulus (MPa)	3000–8000	3000–6000	3000–6000
Typical modulus (MPa)	5000	4000	3000
Degree of anisotropy(2)	1	1	1
Range of Poisson's ratio (vertical, horizontal and cross)	0.1–0.3	0.1–0.3	0.1–0.3
Typical value of Poisson's ratio	0.2	0.2	0.2

1 *Although figures are only quoted for cement, other cementing binders such as lime, lime fly ash, cement fly ash and granulated slag may be used. The moduli of such materials should be determined by testing (refer to Part 4D of the Guide, Austroads 2019a).*

2 *Degree of anisotropy = vertical modulus/horizontal modulus.*

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 modulus 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 thickness greater than or equal to 175 mm, the post-fatigue cracking life may be estimated as detailed in Section 8.2.7.

Procedures have yet to be developed to prepare laboratory test specimens in a cracked state that reflects the state in situ. Consequently, the characterisation of heavily bound cemented materials in a post-fatigue cracking phase of life is undertaken using presumptive characteristics.

For the purpose of mechanistic-empirical modelling in the post-fatigue phase, cemented materials may be assumed to have a stress-dependent modulus, with the modulus affected by both the underlying support provided to the material, and the thickness and modulus of overlying material.

For subbases and bases manufactured from granular materials with a laboratory soaked CBR less than 30%, the post-fatigue phase is modelled as an unbound granular material (Section 6.2.3).

For bases and subbases and subbases manufactured from granular materials with a laboratory-soaked CBR of 30% or more, the layer is not sublayered and is considered to be cross-anisotropic, with a degree of anisotropy of 2. A Poisson's ratio of 0.35 should be used. The vertical design modulus of the post-fatigue phase heavily bound cemented material is the minimum of:

- the value obtained from Figure 6.6 reflecting the effect of the supporting layer (the modulus is limited to four times the vertical design modulus of the underlying support layer, with a minimum of 240 MPa, and a maximum of 600 MPa)

for materials covered by bound materials (asphalt and lightly and heavily bound cemented materials), the value determined from Table 6.8, which reflects the modulus stress-dependency of these materials.

Figure 6.6: Limits on design modulus for post-fatigue phase cemented materials

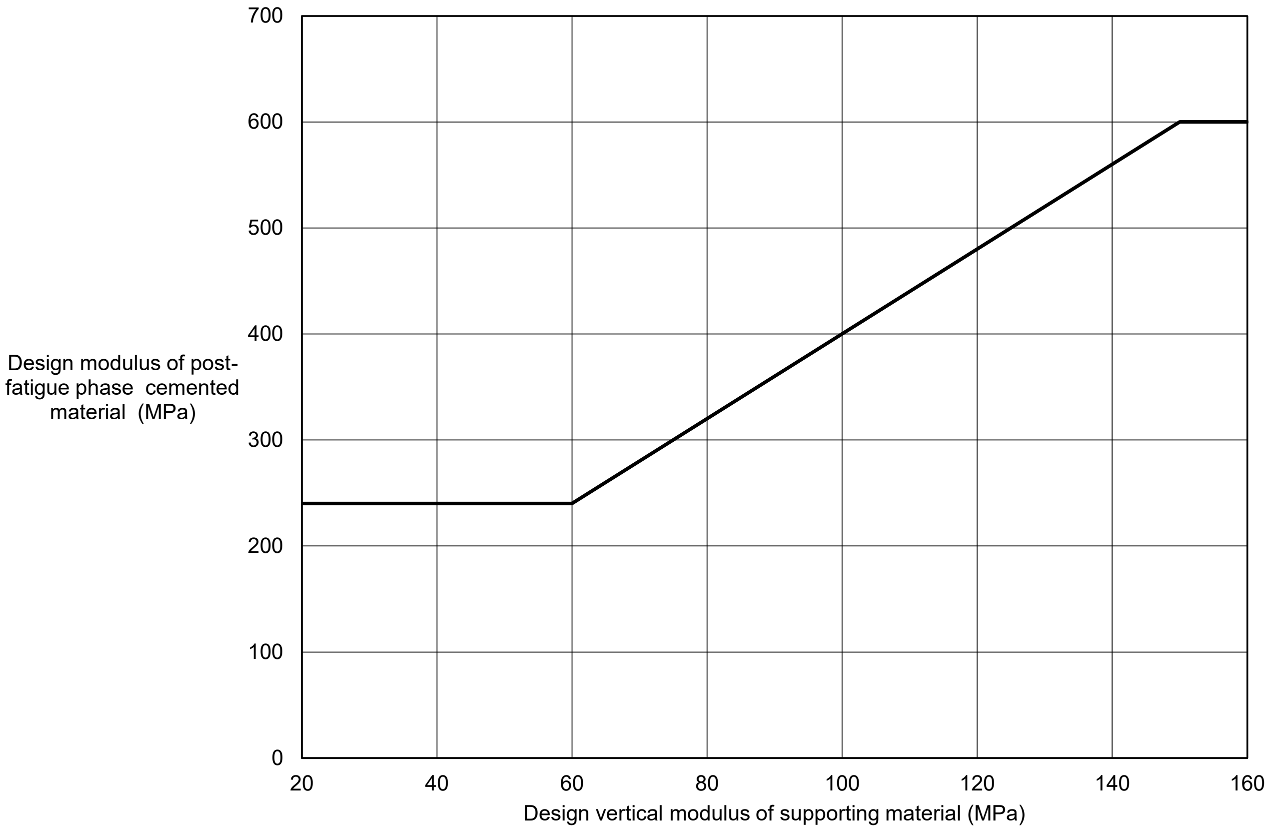


Table 6.8: Suggested maximum vertical modulus of post-fatigue phase cemented material bases and subbases

Thickness of overlying bound material	Modulus of overlying[1] bound material (MPa)					
	600	1000	2000	3000	4000	5000
40 mm	600	600	600	600	600	600
75 mm	600	600	600	590	580	570
100 mm	600	600	570	550	530	520
125 mm	590	570	530	510	490	470
150 mm	570	540	500	470	440	430
175 mm	550	510	460	430	430	430
200 mm	520	480	430	430	430	430
225 mm	500	460	430	430	430	430
250 mm	470	430	430	430	430	430
275 mm	450	430	430	430	430	430
≥ 300 mm	430	430	430	430	430	430

1 *These maxima apply to cemented bases and subbases manufactured using unbound granular material with a laboratory soaked CBR of 30% or greater.*

Note: Overlying material is either asphalt or cemented material (heavily or lightly bound) or a combination of these materials. Equation 5 can be used to determine an equivalent modulus of multiple overlying bound materials.

6.5.3 Determination of design flexural strength

Definition of design flexural strength

For use in characterising fatigue performance the appropriate value of the flexural strength of heavily bound cemented materials is determined from testing laboratory-manufactured beams compacted to representative field densities, then moist cured in the laboratory for 90 days. It is expected that materials stabilised with cement will be substantially cured at this time with little appreciable change in properties expected beyond this time.

Alternative methods

Design flexural strengths may be estimated from:

- flexural strength measurements of laboratory compacted and cured beams, then adjusted to representative in situ values
- presumptive values.

Issues to be considered with the laboratory determination of the design flexural strength of cemented materials are similar to those described in Section 6.5.1 for modulus.

Flexural strength measurement

The Austroads laboratory flexural strength test (Austroads Test Method AGPT/T600) is the preferred method to determine the design strength of cemented materials. The testing is carried out using the same flexure beam testing (Figure 6.5) as used for modulus.

The process to manufacture the test beams is the same as described in Section 6.5.1 for modulus. Commonly, the test beams are those previously tested for modulus.

In the test (Austroads Test Method AGPT/T600), the applied load is increased at 3.3 kN/minute until the beam ruptures. The flexural strength is calculated from the applied peak load, the resulting peak mid-span elastic displacement, the distance between the support rollers and the beam width and height.

In the event that the laboratory flexural strength tests were undertaken at a different dry density ratio to the in-service value, the following relationship may be used to adjust the measured strength provided the density of the test beams is within 5% of the in-service value (Equation 9).

$$F_{\text{in-service}} = F_m\left(1 + 0.05 \times (DR_{\text{in-service}} - DR_{\text{test}})\right) \tag{9}$$

where

$F_{\text{in-service}}$	=	flexural strength at in-service density ratio (MPa)
F_m	=	measured flexural strength at density ratio of test beams (MPa)
$DR_{\text{in-service}}$	=	density ratio in-service (%)
DR_{test}	=	density ratio of test beams (%)

The design flexural strength is the mean of the flexural strengths at the in-service density ratio.

6.5.4 Factors affecting the fatigue life of heavily bound cemented materials

The principal factors affecting the fatigue life of cemented materials include: particle size distribution, particle shape, density, moisture content, mixing efficiency, and cracking pattern. Some of these factors are in turn dependent on binder type and content, etc.

Dry density and moisture content

As a general rule, an increase in density results in an increase in the fatigue life of heavily bound 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 and an increase in the amount of shrinkage cracking.

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 in situ stabilisation equipment for binder application and mixing.

Cracking

Cracking in cemented materials is normally 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, the binder type and content and the moisture content at the time of compaction.

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)
- 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. The following measures may be considered to reduce shrinkage cracking in cemented layers:

- 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 less shrinkage than 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
  - pre-treating with lime or lime and cement, followed by stabilisation with fly ash blend cement
  - 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
  - applying both of the above treatments
  - 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 sprayed seal to inhibit potential shrinkage cracking of the surfacing.
- Use an appropriate polymer modified binder asphalt surfacing in preference to conventional asphalt. More details are provided in *Part 3: Pavement Surfacings* of the Guide (Austroads 2025b).

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 styrene-butadiene-styrene (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 *Part 4F: Bituminous Binders* of the Guide (Austroads 2021a).

6.5.5 Determining the in-service fatigue characteristics from laboratory fatigue measurements

Introduction

In this, the preferred procedure, the in-service fatigue relationship is determined from fatigue testing of laboratory manufactured beams. Note that the flexural beam test is not the only test that can be used for fatigue characterisation. It is known that this test does not simulate field conditions accurately. The results are affected by specimen size, support conditions and differences in test beam condition (e.g. micro-cracking) from the material in the road bed. Further research is required to develop an improved yet practical laboratory fatigue test.

Due to differences between the laboratory fatigue life and the conditions applying to the in-service pavement, in-service fatigue lives differ from laboratory fatigue lives. A laboratory-to-field tolerable strain shift factor (SF) is used to adjust fatigue characteristics determined in the laboratory to an appropriate predicted in-service fatigue life.

The in-service fatigue relationship is of the following general form (Equation 10).

$$N = RF\left(\frac{K}{\mu\varepsilon}\right)^{12} \qquad 10$$

where

N	=	allowable number of repetitions of the induced load-induced tensile strain
$\mu\varepsilon$	=	load-induced tensile strain at the base of the cemented material (microstrain)
K	=	a constant, calculated by multiplying the laboratory fatigue constant k (Equation 11) by the laboratory-to-field tolerable strain shift factor (SF), 1.55 is the presumptive SF value
RF	=	the reliability factor for cemented materials fatigue (Table 6.9)

Table 6.9: Suggested reliability factors (RF) for heavily bound cemented materials fatigue

Desired project reliability					
50%	**80%**	**85%**	**90%**	**95%**	**97.5%**
25	4.7	3.3	2.0	1.0	0.5

Note that the procedure assumes fatigue life is related to the 12th power of strain, as a very large number of fatigue test beams may be required to determine the strain damage exponent from the laboratory fatigue measurements. Austroads (2014) provides the background to the selection of the 12th power.

The steps involved to determine the in-service fatigue relationship from laboratory fatigue testing are as follows:

1. Select the appropriate density ratio at which to test the fatigue beams.
2. Manufacture the test beams and moist-cure in the laboratory for 90 days and then undertake laboratory fatigue testing.
3. Determine the fatigue constant of the laboratory fatigue relationship (k).
4. Select an appropriate laboratory-to-field tolerable strain shift factor (SF).
5. Determine the in-service fatigue relationship.

The steps are described in detail below.

Test beam density ratio

The fatigue life of cemented materials varies with density to which the material is compacted: fatigue life increases as the density ratio increases. Consequently it is important that the fatigue beams be tested at a density ratio representing the in-service level. In selecting this value, consideration may be given to the minimum field density ratio specified for pavement construction.

Laboratory fatigue testing

Laboratory fatigue testing of cemented materials can be carried out using flexure beam testing (Figure 6.5). The Austroads test method is given in AGPT/T600.

Test slabs should be compacted at the design cement content and grading and compacted as close as possible to the selected density ratio. The Austroads test method for preparing asphalt slabs using a segmental roller (AGPT/T220) may be adapted for this purpose. Test beams to the dimensions specified in the test method are saw-cut from the test slabs.

In the beam fatigue testing, repeated application of a haversine load is applied to the upper surface of a rectangular test beam, while recording the resulting vertical displacement of the centre of the beam. The loading continues until the flexural modulus of the beam reduces to half the initial value.

The results of fatigue testing vary appreciably between specimens of essentially the same composition tested on the same apparatus. Due to this variability it is recommended that the fatigue testing be limited to determining the mean laboratory strain with a fatigue life of 10^5 load repetitions and assume a strain-damage exponent of 12. To accurately determine the entire fatigue characteristics over a range of fatigue lives involves testing a very large number of beams. Accordingly, the applied load is adjusted to target a fatigue life of 10^5 load repetitions.

Consideration needs to be given to the number of fatigue results required to achieve a representative and statistically significant value for the mean strain with a fatigue life of 10^5 load repetitions. It is anticipated that 10 to 20 fatigue results will be required with fatigue lives in the range 3×10^4 to 3×10^5.

Calculation of the laboratory fatigue constant k

Using the fatigue results a laboratory fatigue relationship of the following form is determined (Equation 11).

$$N = \left(\frac{k}{\mu\varepsilon}\right)^{12} \tag{11}$$

where

N = allowable number of load repetitions from laboratory fatigue testing

$\mu\varepsilon$ = load-induced tensile strain at the base of the test beam (microstrain)

k = a constant

The fatigue constant k is calculated as follows:

- for each test beam, using the measured fatigue life and tolerable strain calculate the fatigue constant K for the beam using Equation 11 (k_i)
- calculate the mean and standard deviation of the k_i values of all test beams
- delete any outliers amongst the k_i values
- recalculate the mean of the k_i values.

The laboratory fatigue constant k is the mean of the individual test beam k_i values.

Laboratory-to-field shift factor

Due to differences between the laboratory fatigue life and the conditions applying to the in-service pavement, in-service fatigue lives differ from laboratory fatigue lives. A laboratory-to-field tolerable strain shift factor, SF, is used to adjust the laboratory fatigue constant k to an appropriate value to predict in-service fatigue life. In determining this shift factor, consideration must be given to the strain levels resulting from the combination of axle-group and load levels within the design traffic load distribution.

Where appropriate pairs of laboratory and field performance data are not available to determine the shift factor, a value of 1.55 is used in this Part.

In-service fatigue relationship

Equation 10 is the form of the in-service fatigue relationship. The in-service fatigue constant K is determined using Equation 12.

$$K = k \times SF \qquad 12$$

where

K = fatigue constant of the in-service fatigue relationship (Equation 10), subject to maximum values determined using Equation 13

k = fatigue constant determined from the laboratory fatigue data

SF = laboratory-to-field shift factor, presumptive value of 1.55

To ensure design thicknesses for cement treated crushed rocks and cement treated natural gravels are not less than lean-mix concrete, upper limits are placed (Austroads 2014) on the fatigue constant K as follows (Equation 13).

$$K_{max} = \frac{18880}{\sqrt{E}} \qquad 13$$

where

K_{max} = maximum value of fatigue constant of the in-service fatigue relationship

E = heavily bound cemented materials design modulus (MPa)

6.5.6 Determining the in-service fatigue characteristics from laboratory measured flexural strength and modulus

An alternative procedure is to determine the in-service fatigue relationship from the design modulus and design strength determined from laboratory testing (Austroads Test Method AGPT/T600).

The fatigue constant K for use in the in-service fatigue relationship (Equation 10) is the minimum of the value determined from Equation 14 and the maximum K value obtained from Equation 13.

$$K = 240FS + \frac{919300}{E} - 285 \qquad 14$$

where

K = in-service fatigue constant

FS = design flexural strength (MPa)

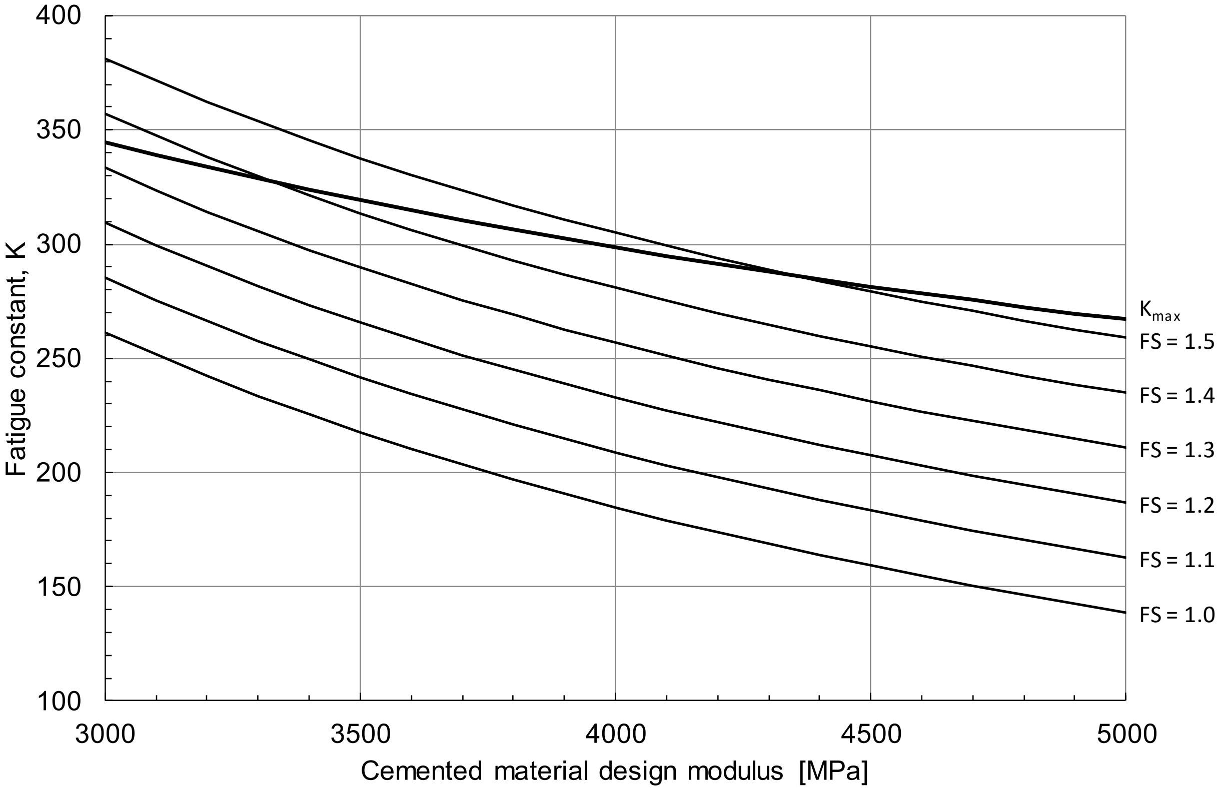
E = heavily bound cemented material design modulus (MPa)

This method is limited to cement treated crushed rock and cement treated natural gravels with:

- cementitious binder contents in the range 3–5%
- design flexural strengths in the range 1.0–1.5 MPa
- design moduli in the range 3000–5000 MPa.

Figure 6.7 plots Equation 14 for a range of design flexural strengths.

Figure 6.7: In-service fatigue constants K determined from heavily bound cemented materials design modulus and flexural strength (FS)



6.5.7 Determining the in-service fatigue characteristics from presumptive flexural strength and modulus

In the event that measured flexural strengths and moduli are not available, in-service fatigue relationships may be estimated from presumptive strengths and moduli. Table 6.10 lists the presumptive fatigue constants for use in the following in-service fatigue relationship (Equation 15).

$$N = \left(\frac{K}{\mu\varepsilon}\right)^{12} \qquad (15)$$

where

N = allowable number of repetitions of the load-induced tensile strain

$\mu\varepsilon$ = load-induced tensile strain at the base of the cemented material (microstrain)

K = presumptive constant, as given in Table 6.10

Note that reliability factors have yet to be developed for this method to enable design to a selected project reliability.

Table 6.10: Presumptive fatigue constants

Property	Base quality granular material 4–5% cement	Subbase quality crushed rock 3–4% cement	Subbase quality natural gravel 4–5% cement
Typical modulus (MPa)	5000	4000	3000
Typical flexural strength (MPa)	1.4	1.2	1.0
In-service fatigue constant K	235	233	261

6.6 Lightly bound cemented materials

6.6.1 Introduction

Lightly bound cemented materials are described as a combination of a cementitious binder, water and granular material that are mixed and compacted in the early stages of the hydration process to form a pavement layer. The cementitious binder may consist of Portland cement, blended cement and lime 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 weakly bound layer with low tensile strength. For the purposes of design, lightly bound materials are considered to not have significant tensile strength, and therefore flexural fatigue of the materials is not considered.

At this stage, this Part does not provide specific guidance for the design of these lightly bound materials in lightly trafficked environments or in rehabilitation design. The characterisations of lightly bound materials discussed in this section, and the design procedure discussed in Section 8.2.4, are limited to moderate-to-high levels of design traffic.

Where lightly bound cemented bases are designed to inhibit block cracking and crocodile cracking, a maximum 28-day UCS of 2.0 MPa applies, and a minimum above 1.0 MPa determined after moist curing but without soaking at 100% standard maximum dry density and optimum moisture content applies.

The addition of binder provides several benefits when compared to the unbound granular material, which include:

- reduced moisture sensitivity
- higher strength and stiffness
- reduced permeability
- reduced erodibility
- reduced sensitivity to variations in grading and plasticity.

Lightly bound cemented materials are not designed to inhibit fatigue cracking because uneconomic thickness would be required due to the low strength of the materials. Bases and subbases are designed and constructed with limited strength such that the allowable traffic loading before fatigue cracking is insignificant compared to the allowable traffic loading post-cracking. As such, there is no need to consider the fatigue life of lightly bound cemented materials in structural design.

As the structural design only considers pavement performance in the post-cracking phase, bases must be designed and constructed to limit the development of block cracking and crocodile cracking from a fatigue-induced microcracking. That is, they are designed to inhibit the development of macrocracking.

South African experience suggests that weakly cemented materials are susceptible to crushing. Jameson et al. (1996) confirmed this during accelerated pavement testing of cement-stabilised fly ash in Australia. To avoid this type of distress and to maintain load transfer across microcracks, limits are placed on the quality of granular materials used in lightly bound cemented materials. Austroads (2025a) provides guidance on materials suitable for cementitious stabilisation.

Lightly bound cemented materials may be used as subbase, in which case it is usually not necessary to inhibit macrocracking.

Characterisation for pavement design

Lightly bound cemented materials' 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.

Procedures have yet to be developed to prepare laboratory test specimens that represent the post-cracking state in situ. Therefore, the characterisation of lightly bound cemented materials is undertaken using presumptive characteristics. The same presumptive characteristics used for the post-fatigue cracking phase of heavily bound cemented materials (Section 6.5.2) applies.

For the purpose of mechanistic-empirical modelling, lightly bound cemented materials may be assumed to have a stress-dependent modulus, with the modulus affected by both the underlying support provided to the material, and the thickness and modulus of overlying material.

For subbases and bases manufactured from granular materials with a laboratory-soaked CBR less than 30%, the lightly bound cemented material is modelled as an unbound granular material (Section 6.2.3).

For subbases and bases manufactured from granular materials with a laboratory-soaked CBR of 30% or more, the layer is not sublayered and is considered to be cross-anisotropic, with a degree of anisotropy of 2. A Poisson's ratio of 0.35 should be used.

6.6.2 Determination of design modulus

For bases and subbases manufactured from granular materials with a laboratory-soaked CBR of 30% or more, the vertical design modulus is the minimum of:

- the value obtained from Figure 6.6 reflecting the effect of the supporting layer (the modulus is limited to four times the vertical design modulus of the underlying support layer, with a minimum of 240 MPa, and a maximum of 600 MPa)
- the value determined from Table 6.8, which reflects the stress-dependency of these materials, for materials covered by bound materials (asphalt and lightly and heavily bound materials).

6.7 Asphalt

6.7.1 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 conventional bitumen there is an increasing usage of polymer modified binders (PMB) and multigrade binders, particularly for surfacing layers. In addition harder bitumen is used for the production of EME2 asphalt.

The strength/modulus 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 on moderate-to-heavily trafficked pavements are:

- rutting and shoving due to insufficient resistance to permanent deformation
- cracking due to fatigue
- durability (oxidation) leading to top-down cracking.

Section 12.6.2 describes the required asphalt properties for lightly trafficked pavements.

General categories and characteristics of asphalt are given in Table 6.1. A guide to the selection of nominal size of asphalt is provided in Table 6.11. The nominal size of an asphalt mix is an indication of the maximum particle size present and is usually expressed as a convenient whole number above the largest sieve size to retain more than 0% and less than 10% of the aggregate material (AS 2150). Further details on the selection and design of asphalt mixes are contained in *Part 4B: Asphalt* of the Guide (Austroads 2025d).

The use of PMB 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. Further details on bituminous binders are contained in *Part 4F: Bituminous Binders* of the Guide (Austroads 2021a).

Table 6.11: Selection of nominal size of dense graded asphalt mix

Nominal mix size (mm)	Compacted layer thickness (mm)
7	25 to 35
10	35 to 50
14	45 to 70
20	60 to 100

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 modulus of which depends on temperature and loading rate (traffic speed). Asphalt is considered to be isotropic.

The modulus 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.

6.7.2 Factors affecting modulus of asphalt

Factors affecting the modulus of asphalt – and the effects of increasing these factors – are shown in Table 6.12. Some of these factors are briefly discussed below.

Table 6.12: Factors affecting modulus 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

Mix composition

- Aggregate angularity and grading

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

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

- 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 properties of asphalt under any given set of conditions, the binder characteristics also affect the sensitivity of properties such as asphalt modulus to changes in temperature and rate of loading. AS 2008 classifies different types of conventional road-making bitumen and multigrade bitumen on the basis of specific key characteristics. *Part 4F* of the Guide (Austroads 2021a) also describes types of multigrade and modified bitumens.

  If the Shell nomographs (Shell 1978) are used to determine a value for asphalt modulus as described in Section 6.7.3, then bitumen is characterised by its Penetration Index (PI) and $T_{800\ pen}$ as defined in Section 6.7.3. These nomographs have been developed only for conventional bitumens and are not applicable to mixes containing multigrade bitumens or modified binders. At high (> 15%) reclaimed asphalt pavement (RAP) contents, care also needs to be taken that the influence of the RAP binder on the viscosity of the binder blend is taken into account. Guidelines for the characterisation of RAP binder blends can be found in Austroads Test Methods AGPT/T191, ATM 192 and AGPT/T193.

- 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 modulus 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 modulus. 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 modulus.

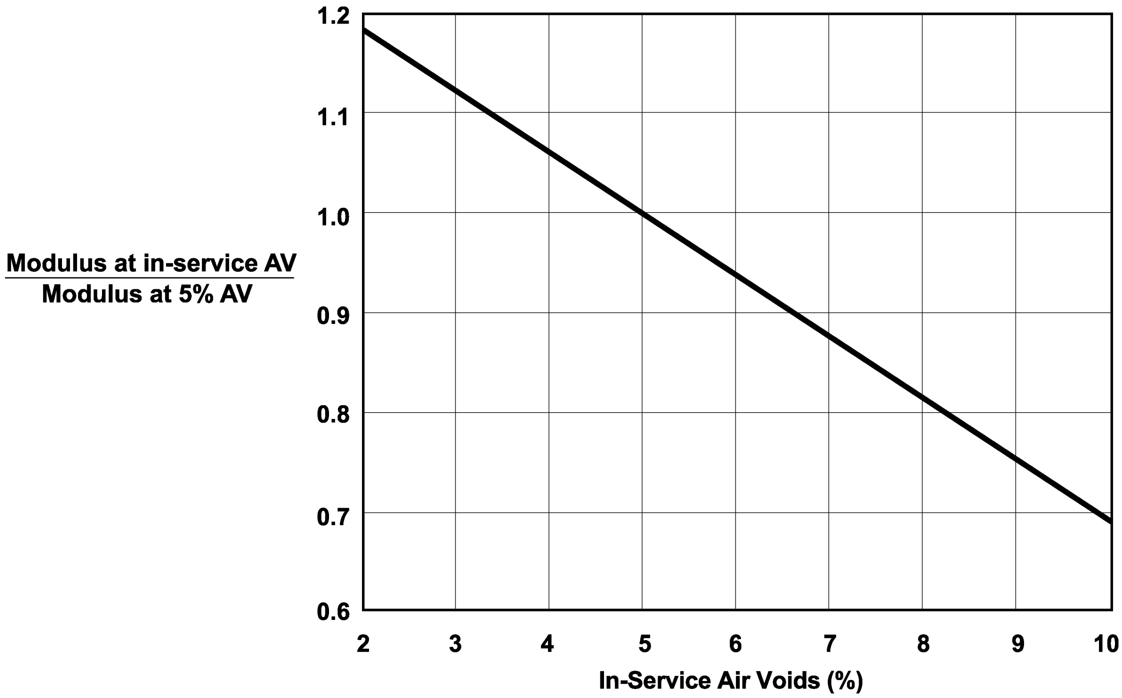
Construction considerations

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

A typical relationship between modulus and air voids is shown in Figure 6.8 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.

Figure 6.8: Variation of ratio of modulus at in-service air voids to modulus at 5% air voids with air voids content



Temperature and rate of loading (traffic speed)

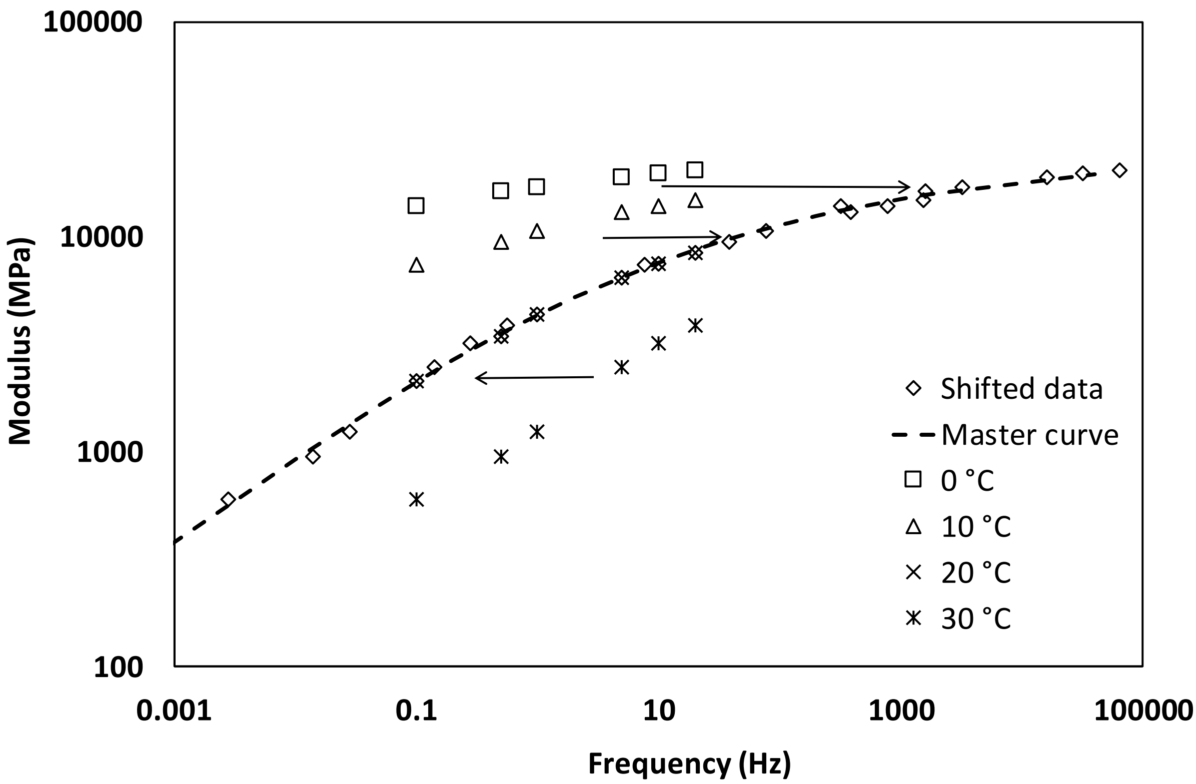
Temperature and rate of loading are important factors determining the modulus of asphalt. Figure 6.9 shows the results of flexural modulus tests of an AC14 asphalt mix with Class 320 bitumen run at different load frequencies and temperatures. It can be seen that the modulus of asphalt can vary by up 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 layer moduli at the Weighted Mean Annual Pavement Temperature (WMAPT). WMAPT values for Australian and New Zealand cities are presented in Appendix B, together with the method for calculating the WMAPT.

It can further be seen that because of the viscoelastic nature of the bituminous binder, the modulus of asphalt is highly dependent on the rate at which it is loaded – the slower the rate, the lower the modulus. This effect can be significant, especially in pavement areas such as intersection approaches, bus stops and parking areas.

The results of temperature and frequency sweep tests as shown in Figure 6.9 can be used to develop modulus relationships that are valid for any combination of loading speed and temperature. The results are shifted, using the time-temperature superposition principle, to fit a continuous function, the so-called modulus master curve. The shift is shown schematically in Figure 6.9.

Figure 6.9: Example of the variation of laboratory measured flexural modulus with temperature and load frequency



Age

The modulus 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, 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 (Nunn et al 1997, Nunn 1998). Butcher (1997) also reported the increase in asphalt moduli of South Australian pavements. The best methods for determining modulus at a given age are to sample and test the asphalt layer, or to derive the modulus by back-calculation from loading tests on the pavement.

6.7.3 Definition of asphalt design modulus

For pavement design purposes, the asphalt modulus is determined from either:

- direct measurement of the flexural modulus obtained from four-point bending tests conducted at the in-service temperature (WMAPT) and for the rate of loading (frequency) in the road-bed
- interpolation of flexural modulus at the WMAPT and rate of loading in the road-bed from a range of four-point bending tests that span the WMAPT and rate of loading in the road-bed conditions
- estimation of flexural modulus from the resilient modulus measured using the standard indirect tensile test (ITT) adjusted to the WMAPT and for the rate of loading in the road-bed
- estimation from the flexural modulus from bitumen properties and mix volumetrics using Shell nomographs and the WMAPT and rate of loading in the road-bed.

If all 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.

6.7.4 Determination of design modulus from direct measurement of flexural modulus

Austroads Test Method ATM 274 is the standard Austroads procedure for characterising the asphalt modulus through flexural testing of beam specimens in four point bending configuration.

It can be used to directly measure the flexural modulus of an asphalt mixture at the temperature and rate of loading test conditions representing the WMAPT and design heavy vehicle traffic. The cyclic displacement controlled test is configured such that it results in a sinusoidal strain response in the sample with an amplitude of 50 microstrain.

The relationships between loading time, heavy vehicle speed and cyclic load frequency in the Austroads Test Method ATM 274 flexural modulus test method (f_{T274}) is given by Equations 16, 17 and 18.

$$t = \frac{1}{2\pi f_{T274}} \tag{16}$$

$$t = \frac{1}{V} \tag{17}$$

$$f_{T274} = \frac{V}{2\pi} \tag{18}$$

where

t = loading time (seconds)

V = velocity of heavy vehicles (km/h)

f_{T274} = cyclic frequency in ATM 274 flexural modulus testing (Hz)

Alternatively testing can be conducted over ranges of load frequencies or temperatures, allowing the interpolation of flexural modulus at specific combinations of design WMAPT and rates of loading. The achievable range of temperatures and frequencies used in the test is equipment dependent, but a typical set of frequencies could be 0.1 Hz, 0.2 Hz, 0.5 Hz, 1 Hz, 2 Hz, 5 Hz, 10 Hz, 20 Hz, 30 Hz. A typical set of temperatures could be 0 °C, 10 °C, 20 °C and 30 °C.

Austroads Test Method ATM 274 describes how the results of a range of temperature and frequency tests can be used to construct a *master curve* to represent the relationship between modulus, temperature and loading frequency. The sigmoidal model form shown in Equation 19 is applied in the definition of the modulus master curves.

$$\log_{10}|E^*| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log_{10} f_r}} \tag{19}$$

where

f_r = reduced frequency

$\delta, \alpha, \beta, \gamma$ = fitting parameters

The results of the complex modulus tests at different temperatures are shifted with respect to time of loading until a single smooth curve emerges, by means of the reduced frequency parameter (f_r). The reduced frequency is defined in Equation 20 as the actual loading frequency in the test multiplied by the time-temperature shift factor, a_T.

$$\boldsymbol{f_r = a_T \times f} \tag{20}$$

where

f_r = reduced frequency (Hz)

a_T = shift factor as a function of temperature (°C)

f = test frequency (Hz)

The temperature shift function is fitted using a second-order polynomial equation (Equation 21).

$$\log_{10}(a_T) = a(\mathrm{T} - \mathrm{T}_{ref})^2 + \mathrm{b}(\mathrm{T} - \mathrm{T}_{ref}) \qquad 21$$

where

a, b = fitting parameters

T = temperature (°C)

T_{ref} = reference temperature

Detailed instructions on fitting the master curve to flexural modulus data obtained in laboratory testing is provided in ATM 274.

The design modulus is determined from the flexural modulus master curve using the following steps:

1. Determine (from project information) a representative value for heavy vehicle traffic speed (V km/h) and calculate the load frequency using Equation 18.
2. Select the WMAPT for the project location.
3. Determine the design modulus from the master curve using the WMAPT and load frequency as input.
4. The design modulus from the master curve needs to be corrected for any difference in air voids content between the laboratory tests and in-service air voids. Using the following relationship, calculate the ratio of the modulus at the in-service air voids to the modulus of the laboratory test specimen (Equation 22, this relationship is shown as Figure 6.8). Correct the modulus obtained from the master curve for the influence of air voids content by multiplying the measured modulus by this ratio.

$$\frac{\text{Modulus at in-service air voids}}{\text{Modulus at test air voids}} = \frac{21 - AV_{in\text{-}service}}{21 - AV_{test}} \qquad 22$$

where

$AV_{in-service}$ = air voids in-service

AV_{test} = air voids in tests

For dense graded asphalt mixes which include conventional bitumen binders, the design modulus calculated using the above process should not be less than 1000 MPa. This minimum value reflects in situ asphalt moduli lower limits, back-calculated from surface deflection measurements.

6.7.5 Determination of design modulus from measurement of ITT modulus

The indirect tensile test (AS/NZS 2891.13.1) is commonly used in Australia for the determination of asphalt modulus.

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 as close as possible to the air voids in-service (refer Appendix A of *Part 4B* of the Guide, Austroads 2025d) 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.

Comparative testing at different temperatures and of a variety of mixes has shown that the indirect tensile modulus determined as described above is approximately equivalent to the flexural modulus at 15 Hz. This equivalency is used to convert the measured ITT values using a rise time of 40 ms to flexural modulus results consistent with the other methods as part of the loading rate adjustment (Equation 24).

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

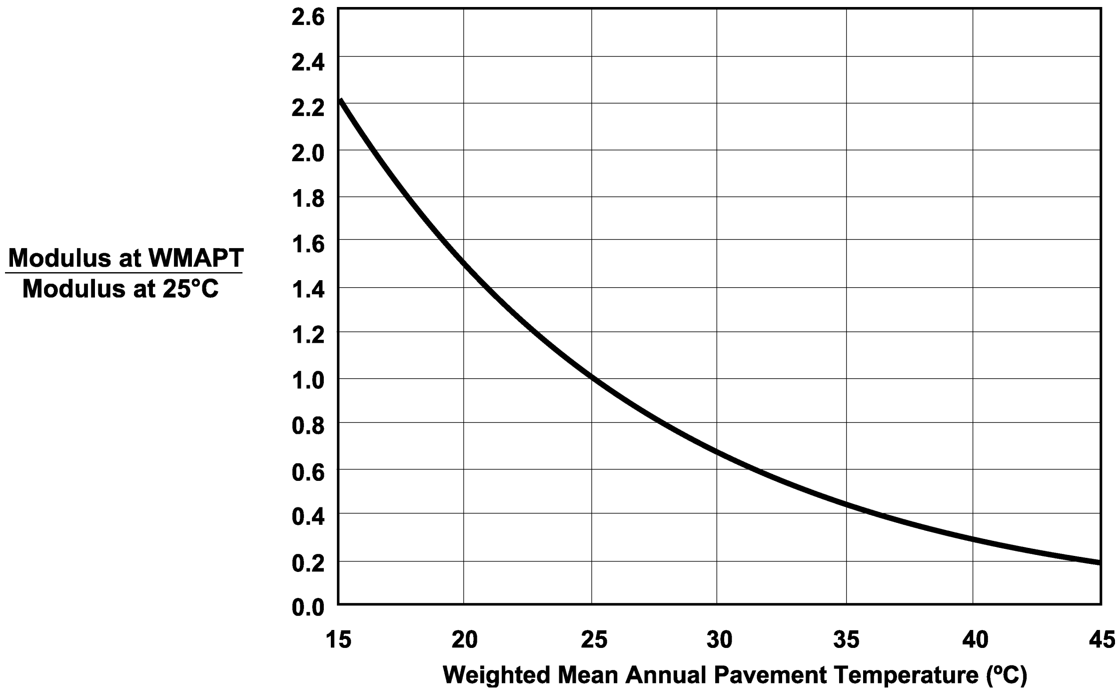
1. Select the appropriate in-service air voids (AV) level representing the air voids level in-service and consistent with the volume of binder used in the fatigue life calculations (Section 6.7.10).
2. Determine (from project information) a representative value for heavy vehicle traffic speed (V km/h).
3. Select the WMAPT for the project location (from Appendix B).
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 relationship in Equation 22 (shown as Figure 6.8), calculate the ratio of the modulus at the in-service air voids to the modulus of the laboratory test specimen.
6. Correct the measured modulus for air voids by multiplying the measured modulus by this modulus ratio.
7. Using the following relationship, calculate the ratio of the field modulus at the in-service temperature (WMAPT) to the modulus at the laboratory test temperature (25 °C) (Equation 23).

$$\frac{\text{Field modulus at WMAPT}}{\text{Laboratory modulus at test temperature (T)}} = e^{(-0.08(\text{WMAPT}-\text{T}))} \qquad 23$$

(This relationship is shown in Figure 6.10).

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

Figure 6.10: Variation of modulus at WMAPT to modulus from standard indirect tensile test with WMAPT (mixes with conventional binders only)



8. Using the following relationship, calculate the ratio of the modulus at the rate of loading in-service to the modulus at the laboratory loading rate (40 ms rise time) (Equation 24).

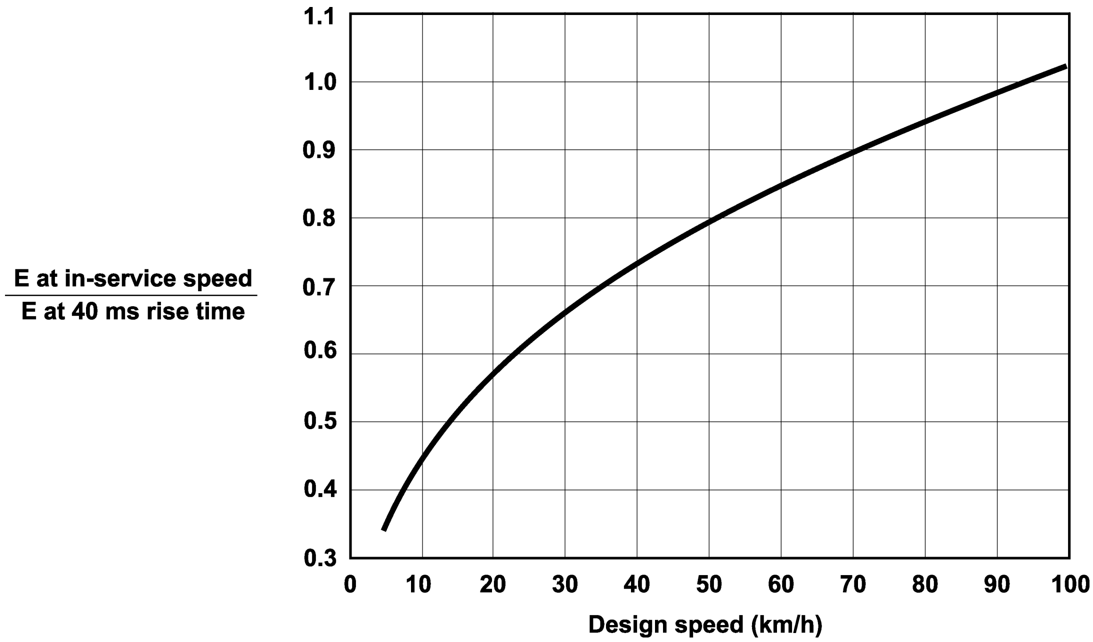
$$\frac{\text{Modulus at speed V}}{\text{Modulus at test loading rate}} = 0.19V^{0.365} \tag{24}$$

(This relationship is shown in Figure 6.11).

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

For dense graded asphalts which include conventional bitumen binders, the design modulus calculated using the above process should not be less than 1000 MPa. This minimum value reflects in situ asphalt moduli lower limits, back-calculated from surface deflection measurements.

Figure 6.11: Variation of ratio of modulus at vehicle speed V to modulus from standard indirect tensile test (40 ms rise time) with design speed (conventional mixes only)



6.7.6 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).
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 Figure 6.12 and Figure 6.13. A software package – BitProps – is also available (from Abatech) which undertakes these two steps. (Note that Figure 6.12 and Figure 6.13 use the term 'stiffness modulus' as the modulus is time and temperature dependent).

In this method, the nomograph developed by van der Poel (1954) and reproduced in Figure 6.12 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 6.13.

During the mixing and laying of asphalt, the bitumen undergoes considerable hardening and decrease in temperature susceptibility. The Rolling Thin Film Oven (RTFO) test (AS/NZS 2341.10) has been developed to simulate these effects. Hence, it is important to note that the penetration and viscosity data used in Table 6.13 refer to values obtained on bitumen which has been subject to the RTFO test.

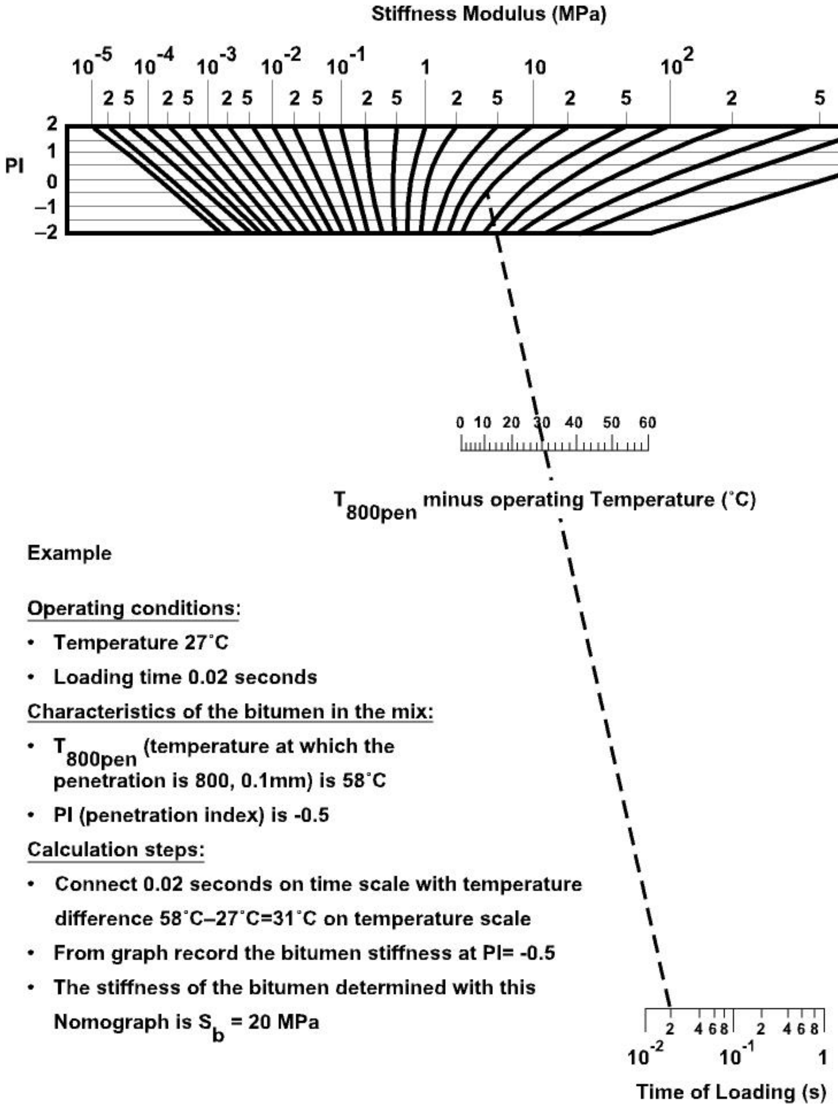
Table 6.13: Relationships for determining PI and $T_{800\ pen}$ from bitumen penetration and viscosity data

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	$\frac{1}{T_2 - T_1} \log \frac{\text{Pen 2}}{\text{Pen 1}}$	$\frac{1}{T_2 - T_1}\left[\log \frac{800}{\text{Pen 1}} + \frac{5.42 \log \frac{1300}{\text{Vis 2}}}{8.5 - \log \frac{1300}{\text{Vis 2}}}\right]$	$\frac{1}{T_2 - T_1}\left[\frac{46.07 \log \frac{\text{Vis 1}}{\text{Vis 2}}}{\left[8.5 - \log \frac{1300}{\text{Vis 1}}\right]\left[8.5 - \log \frac{1300}{\text{Vis 2}}\right]}\right]$
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:

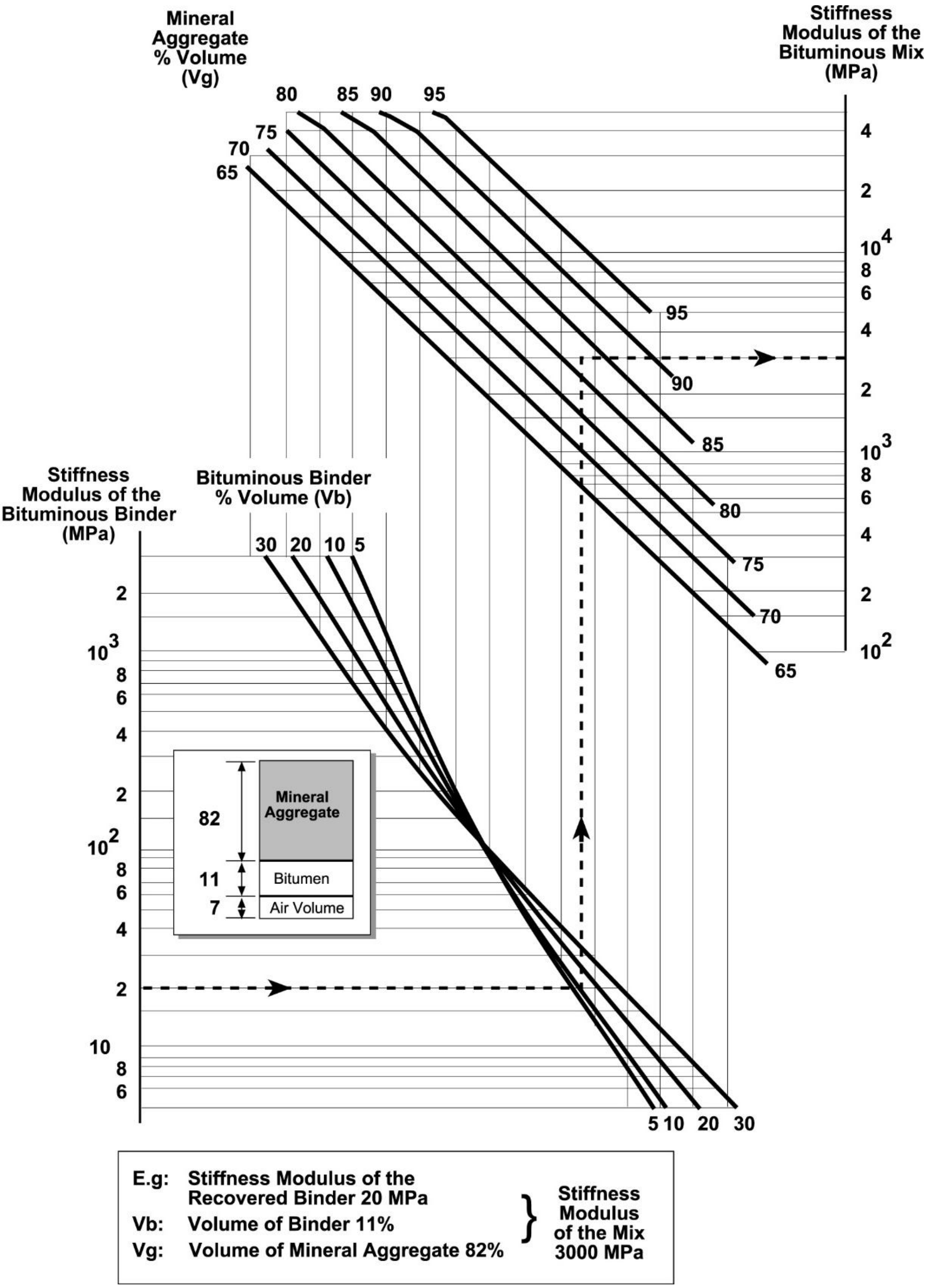
- *Pen 1, Pen 2 are penetrations (0.1mm) determined using 100g mass and loading time of 5 seconds.*
- *Vis 1, Vis 2 are dynamic viscosities (Pa.s).*
- *A is the change in log (penetration) per °C change in temperature.*

Figure 6.12: Nomograph for determining modulus of conventional bituminous binders



Source: Adapted from Shell (1978).

Figure 6.13: Nomograph for predicting the modulus of asphalt



Source: Adapted from Shell (1978).

After the van der Poel nomograph has been used to estimate the modulus of the bitumen, the nomograph developed by Bonnaure et al. (1977) and reproduced in Figure 6.13 is used to estimate the modulus of the asphalt mix.

The information required as input to this nomograph is:

Sb modulus of the bitumen at the assumed temperature and loading rate as derived from the van der Poel nomograph (Figure 6.12).

Vb the percentage by volume of bitumen in the asphalt. For a typical mix containing 5% bitumen by mass, Vb may be taken as 11%.

Vg 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, Vg will be approximately 83%.

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 6.14. The factors were derived from limited testing of laboratory-prepared mixes, primarily dense graded types without RAP. Therefore, these factors should be considered indicative only. Additionally, they may not be representative of asphalt mixes subjected to high temperatures, used at intersections, or other asphalt types e.g. SMA.

Table 6.14: Factors to estimate the modulus of polymer modified binder asphalts for modulus estimated from the Shell nomographs using a Class 320 bitumen

Austroads binder grade(1)	Modulus adjustment factor
A5E	1.3
A10E	0.5
A15E	0.6
A20E	0.7
A35P	1.2

1 *Part 4F of the Guide (Austroads 2017b) and the Austroads Technical Specification ATS 3110 (Austroads 2023c) include the Austroads binder grades of PMBs for use in asphalt applications.*

6.7.7 Design modulus from published data

The use of typical modulus values to assign a design modulus should only be adopted when testing facilities are unavailable; and either the binder is not conventional bitumen (i.e. it is a multigrade or a polymer-modified binder) or the required data on bitumen and mix volumetric properties 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 6.15 presents modulus values obtained using the indirect tensile test procedure under standard test conditions for typical dense-graded asphalt mixes used by Austroads member agencies. 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 Table 6.15 to determine a design modulus value involves the following steps:

1. Select from Table 6.15 a modulus value obtained from the standard indirect tensile test which is appropriate for the proposed mix.
2. Calculate the design modulus at the in-service air voids, temperature (WMAPT) and loading rate using Equation 23 and Equation 24.

The values given in Table 6.15 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.

Table 6.15: 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

Binder	Mix size (maximum particle size) (mm)					
	10		14		20	
	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 1000	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 (AS/NZS 2891.13.1-2013).

6.7.8 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.

6.7.9 Factors affecting asphalt fatigue life

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

- the support (modulus) 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
- the temperature environment in which the asphalt is operating.

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

Effect of support provided by the 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 modulus of granular material depends on the stress level it is subject to – the lower the stress, the lower its modulus.

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

Consider, firstly, the extreme situations: the negligible contribution to overall pavement stiffness provided by thin, very flexible, asphalt surfacings 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 modulus. In the latter case, when the asphalt is loaded, the tensile stress at the bottom of the asphalt is independent of asphalt modulus, while the strain is inversely proportional to the modulus. 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 modulus increases, for the 'controlled stress' situation the fatigue life is increased by increasing the asphalt modulus.

Consider, now, how the situation changes when progressing 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 modulus. 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 modulus.

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 modulus.

It should be noted that, in the above discussion, the asphalt is considered to be of one modulus 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.

Effects of asphalt modulus 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 modulus will result in a reduction in fatigue life.

The factors which, in turn, affect mix modulus are discussed in Section 6.7.2. The direct effects that these factors have on fatigue life are indicated – for both the controlled stress and the controlled strain situations – in Table 6.16.

Table 6.16: Effect of increasing mixture variables on fatigue life and flexural stiffness

Increasing mixture variable	Effect on fatigue life	Effect on flexural stiffness
Binder content	Increase	Decrease
Binder viscosity	Decrease	Increase
Compaction level	Increase	Increase
Grading coarse to fine	Increase	Decrease
Temperature	Increase	Decrease

Source: Austroads (2025d).

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

Effect of binder type

For mixes using conventional binders, there is some increase in mix modulus 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.

Effect of temperature environment

Because asphalt fatigue is dependent on mix modulus 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 B.

Note that these WMAPTs may not be applicable to asphalt in tunnels and a first principles approach to mechanistic-empirical thickness design, considering the proportion of loads applied at various pavement temperatures, may be required.

6.7.10 Fatigue criteria

The asphalt fatigue relationship used in this Part 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 2008, 2021b). (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 used in asphalt placed on moderate-to-heavily trafficked pavements, 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 (Equation 25):

$$N = \frac{SF}{RF}\left[\frac{6918(0.856V_b + 1.08)}{E^{0.36}\mu\varepsilon}\right]^5 \qquad 25$$

where

N = allowable number of repetitions of the load-induced tensile strain

$\mu\varepsilon$ = load-induced tensile strain at the base of the asphalt (microstrain)

V_b = percentage by volume of bitumen in the asphalt (%)

E = asphalt modulus (MPa)

SF = shift factor between laboratory and in-service fatigue lives (presumptive value = 6)

RF = reliability factor for asphalt fatigue (Table 6.17)

Table 6.17: Suggested reliability factors (RF) for asphalt fatigue

Desired project reliability					
50%	**80%**	**85%**	**90%**	**95%**	**97.5%**
1.0	2.4	3.0	3.9	6.0	9.0

Note that the SF/RF term in Equation 25 is a transfer function that relates a mean laboratory fatigue life (Shell 1978) to the in-service fatigue life predicted using this Part at a desired project reliability. It comprises two components:

- a shift factor, SF, 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
- a reliability factor, RF, relating mean in-service fatigue life to the in-service life predicted using this Part at a desired project reliability (Austroads 2008, 2021b), taking into account those factors (e.g. construction variability, environment, traffic loading) discussed in Chapter 2.

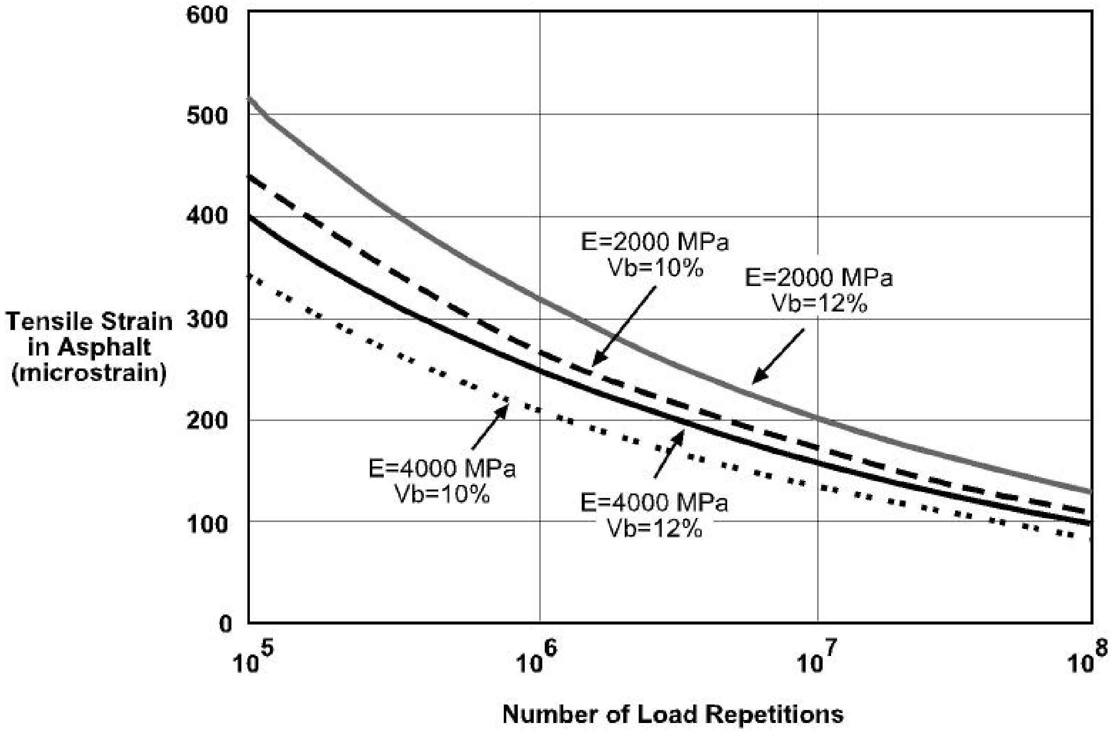
Guidance on the selection of desired project reliability levels is provided in Section 2.3.1. Where the in-service fatigue life is governed by a wearing course that includes a modified binder, the project reliability is likely to be improved significantly. Note that for lightly trafficked roads load-induced fatigue cracking is uncommon (Section 12.6.3).

Plots of tensile strain against allowable loading to asphalt fatigue are presented in Figure 6.14, indicating the effects of asphalt modulus and bitumen content for a desired project reliability of 95%.

There is increasing recognition of the notion that asphalt mixes have endurance strain limits for asphalt fatigue, such that below a given applied strain repeated cycles of loading no longer result in fatigue damage. For instance, as a result of the work by Nunn et al. (1997) the UK procedure for design of asphalt pavements was revised to include a minimum asphalt thickness, corresponding to minimum threshold pavement strength, for the most common asphalt mixes beyond which the pavement should have a very long but indeterminate structural life, so-called long-life pavement structures.

As an interim measure and pending further research to quantify the increase in crack healing with increasing temperature, limits may be placed on the design traffic used in the asphalt fatigue damage calculations (Section 7.6.3).

Figure 6.14: Example plots of tensile strain against load repetitions to asphalt fatigue for a project reliability of 95%



6.7.11 Means of determining asphalt fatigue characteristics

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

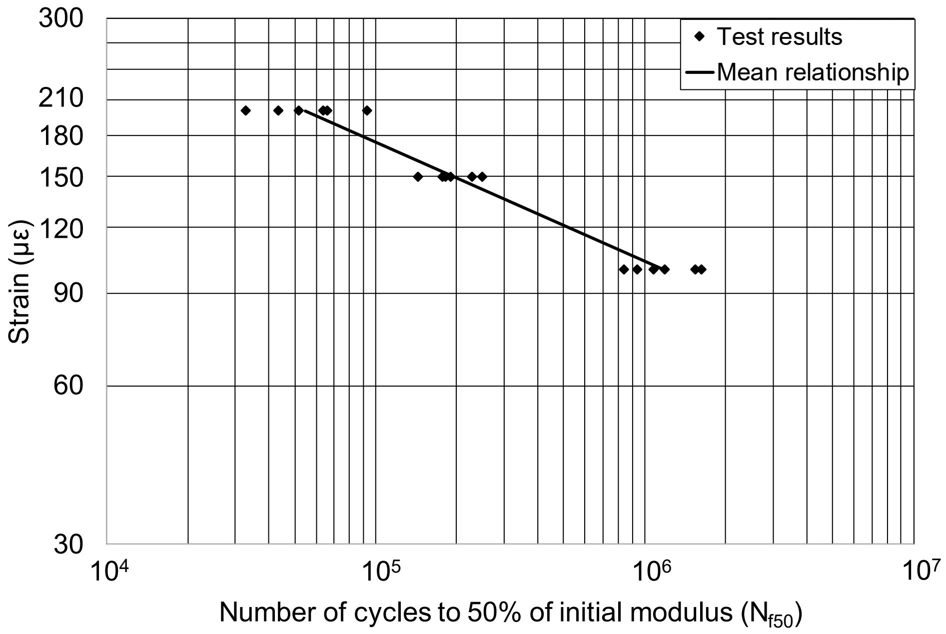
Laboratory fatigue testing

The method adopted in Australia for characterising the fatigue properties of asphalt is flexural fatigue testing of asphalt beams (Austroads Test Method ATM 274). A simply supported beam of asphalt of rectangular cross-section (390 mm long, 50 mm high, 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 method requires a minimum of 18 specimens to be tested. The specimens should be tested divided over at least three different strain levels. At the highest strain level, the number of load cycles to 50% stiffness reduction should be at least 10^4, at the lowest strain level, the number of load cycles to failure should exceed 10^6.

An example of a full set of fatigue results for an asphalt mix is shown in Figure 6.15 and the regression function in Equation 26. In reporting fatigue results, it is important to include constants a and b, the number of specimens (n) and the standard deviation of the residuals (σ_y) as these parameters combined fully define the fatigue curve and confidence intervals.

Figure 6.15: Example of mix specific fatigue curve



$$\ln(N_{f50}) = a + b\ln(\mu\varepsilon) \qquad 26$$

where

N_{f50} = number of cycles to 50% of initial modulus

a, b = constants determined from a set of fatigue test results

$\mu\varepsilon$ = strain (microstrain)

It is possible to develop mix specific in-service fatigue relationships based on laboratory testing to be used instead of the model in Equation 25. To develop a laboratory model for a single mix design, would involve sets of ATM 274 tests at relevant temperature(s). The development of a mix specific in-service fatigue relationship requires of an appropriate SF/RF term to relate laboratory to field performance and reliability.

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 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.

6.7.12 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 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 6.16. The effects of varying the above factors are summarised in Table 6.18.

Figure 6.16:Factors affecting mix stability

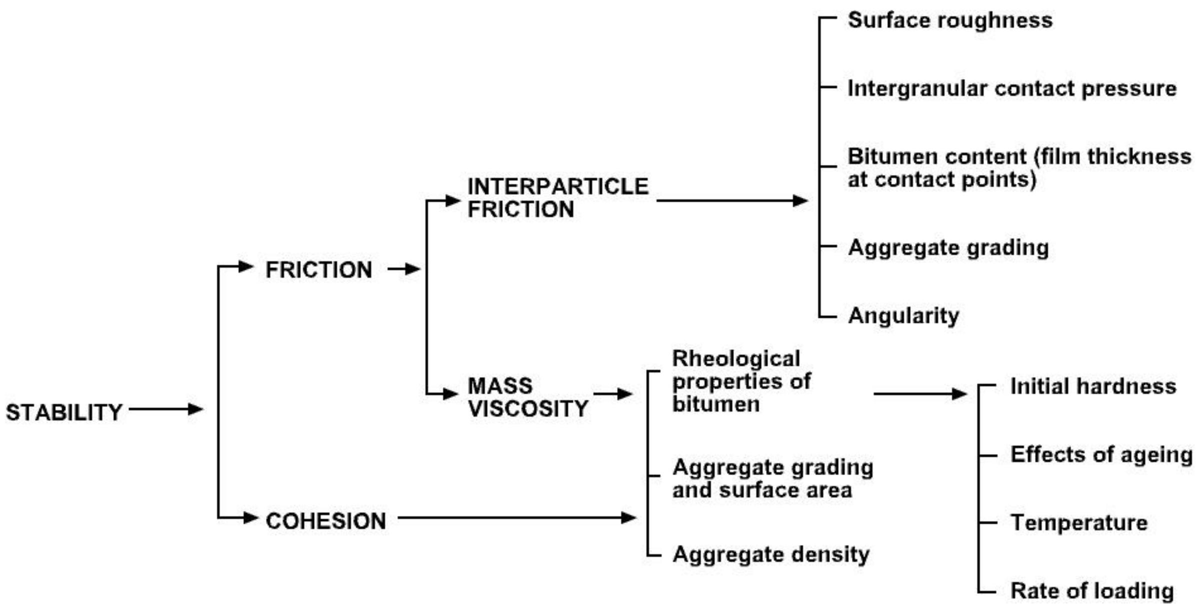


Table 6.18: Factors affecting the stability of asphalt

Variable	Resistance to permanent deformation
Increased binder content	Decrease
Increased binder viscosity	Increase
Grading (coarse to fine)	Decrease
Increased load duration	Decrease
Increased compaction level	Increase
Increased temperature	Decrease

Wheel tracking is used to indicate the resistance of the candidate mix to plastic deformation under traffic. The wheel tracking test procedure is available as Austroads Test Method ATM 231.

Samples are conditioned to a test temperature of 60 °C. The test is usually conducted for 10 000 load passes or until the rut depth reaches 15 mm. Data may be reported as:

- central and average steady state tracking rate in mm/kPasses
- maximum rut depth (at 10 000 passes) or number of load passes to attain a 15 mm rut depth if deformation exceeds 15 mm before 10 000 passes have elapsed.

6.8 Concrete

6.8.1 Introduction

Concrete refers to a homogeneous mixture of hydraulic cement, fine and coarse aggregate, water, and chemical admixtures. Further details on concrete are contained in *Part 4C: Materials for Concrete Pavements* of the Guide (Austroads 2017a).

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 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.

6.8.2 Subbase concrete

Lean-mix concrete which is used for subbase construction may contain 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 construction of both rigid and flexible bases over poor subgrades is facilitated by the adoption of a lean-mix 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.

6.8.3 Subbase concrete for flexible pavements

Where lean-mix concrete subbase is used in the design of a flexible pavement, the characteristics which must be known and evaluated for structural design purposes are moduli, Poisson’s ratio and performance under repeated loading (Section 6.4).

Many of the factors that affect the modulus and strength of cemented materials (Section 6.4) are relevant to lean-mix concrete.

Presumptive values

Presumptive values for elastic characterisation of lean-mix concrete given in Table 6.19 may be used as a guide if no other more reliable information is available.

Table 6.19: Presumptive values for elastic characterisation of lean-mix concrete

Property	Lean-mix concrete
Range of modulus (MPa)	5 000–15 000
Typical modulus (MPa)	7 000 (rolled) 10 000 (screeded)
Degree of anisotropy(1)	1
Range of Poisson's ratio (vertical, horizontal and cross)	0.1–0.3
Typical value of Poisson's ratio	0.2

1 Degree of anisotropy = vertical modulus/horizontal modulus.

Determining the in-service fatigue characteristics of lean-mix concrete using presumptive values

A presumptive in-service fatigue relationship may be used for lean-mix concrete. Table 6.20 lists the presumptive fatigue constants for use in the in-service fatigue relationship (Equation 27).

$$N = RF\left(\frac{K}{\mu\varepsilon}\right)^{12} \qquad 27$$

where

N = allowable number of repetitions of the load-induced tensile strain

$\mu\varepsilon$ = load-induced tensile strain at the base of the lean-mix concrete (microstrain)

K = presumptive constant, as given in Table 6.20

RF = the reliability factor for cemented materials fatigue (Table 6.9)

Table 6.20: Presumptive fatigue constants for lean-mix concrete

Property	Lean rolled concrete	Lean screeded concrete
Typical modulus (MPa)	7000	10 000
In-service fatigue constant K	242	223

6.8.4 Base concrete for rigid pavements

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 5.5 to 6.5 MPa.

Since at the time of undertaking the thickness design the concrete will only have 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 (AS 3600).

A typical relationship for converting 28-day compressive strength to 28-day flexural strength for concrete with crushed aggregate is (Equation 28).

$$f_{fc} = 0.75 \times \sqrt{f_c} \tag{28}$$

where

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

f_c = 28-day concrete compressive 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 (Equation 29).

$$f_{cf} = 1.37 f_{cs} \tag{29}$$

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.

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 Section 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. Section 7.6 and 7.7 provide more information on the design traffic for moderate-to-heavily trafficked flexible pavements and rigid pavements, respectively. Section 12.7 provides guidance on the design traffic for lightly trafficked pavements.

The procedures in this Section may be adapted to assess the pavement impacts of different heavy vehicle types in carrying a given amount of freight. In such cases, consideration needs to be given not only to the damage due to a single vehicle pass (e.g. ESA), but also the number of trips required to transport the freight. As the number of trips is inversely related to the payload, one index that has been used in such investigations is the ESA/Payload. Alternatively, as the payload of vehicles in the fleet generally increases with vehicle gross mass, the ESA/Gross mass has also been utilised. Vehicles with lower ESA/Payload or ESA/Gross mass cause less pavement wear per unit of freight carried.

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 Table 7.1 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 used 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
- the loading applied to the pavement through each of these axle groups – the axle group load.

1 The designer is cautioned that this Part 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-empirical 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.

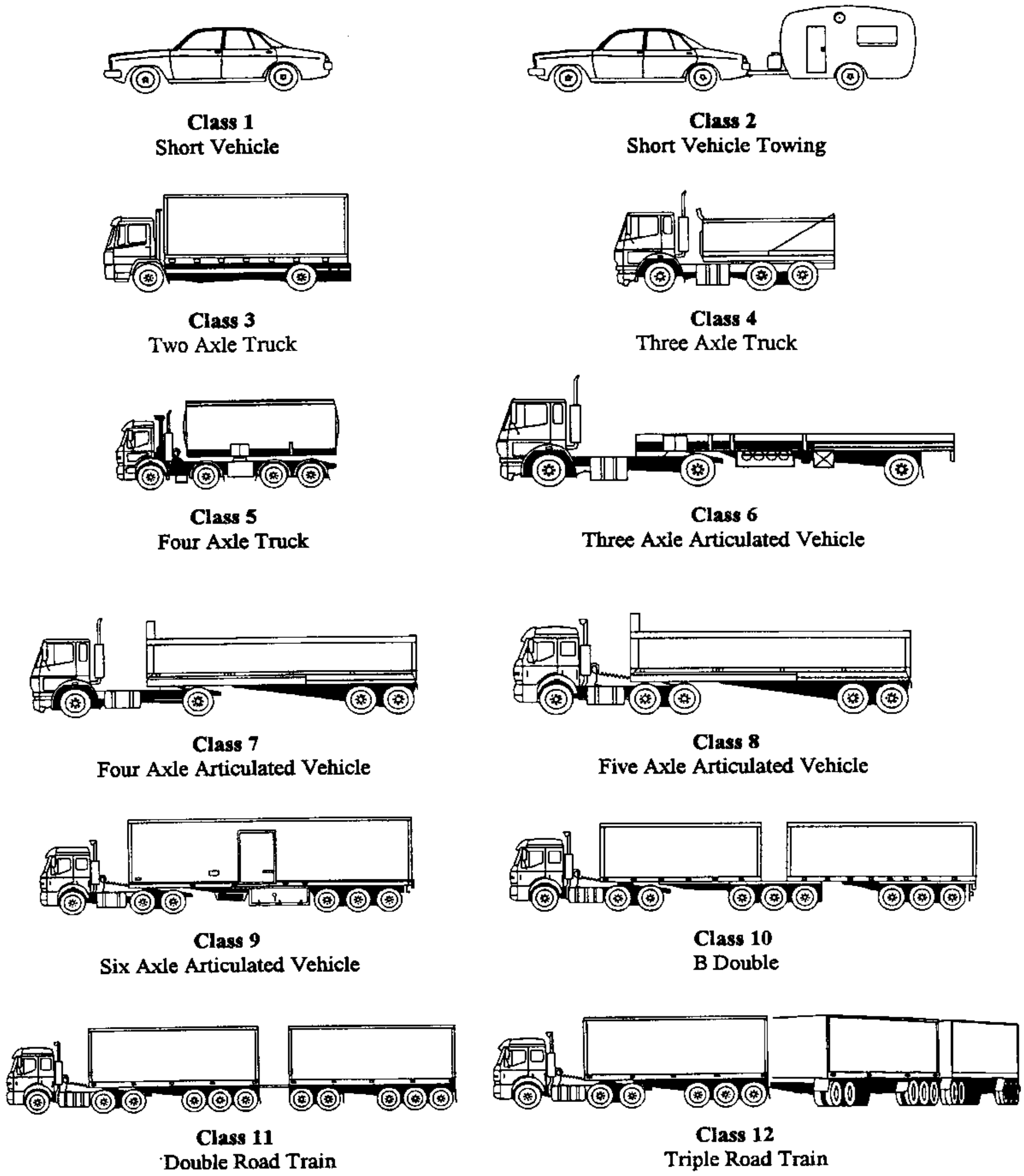
Table 7.1: Austroads vehicle classification system relevant to pavement thickness design

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 5.5 m	2	1 or 2	Short Sedan, wagon, 4WD, utility, light van, bicycle, motorcycle, etc.	1	d1 ≤ 3.2 m and axles = 2
	> 2	3	Short – towing Trailer, caravan, boat, etc.	2	Groups = 3 2.1 m ≤ d1 ≤ 3.25 m, d2 > 2.1 and axles > 2
Heavy vehicles					
Medium 5.5 m to 14.5 m	2	2	Two axle rigid truck or bus	3	d1 > 3.25 m and axles = 2
	3	2	Three axle rigid truck or bus	4	Axles = 3 and groups = 2
	> 3	2	More than 3 axle rigid truck, bus or crane	5	Axles > 3 and groups = 2
Long 11.5 m to 19.0 m	3	3	Three axle articulated or rigid vehicle and trailer	6	d1 > 3.25 m, axles = 3 and groups = 3
	4	> 2	Four axle articulated or rigid vehicle and trailer	7	d2 < 2.1 m 2.1 m ≤ g1 ≤ 3.25 m, axles = 4 and groups > 2.
	5	> 2	Five axle articulated or rigid vehicle and trailer	8	G1 > 3.25 m, axles = 5 and groups > 2
	6 > 6	> 2 3	Six (or more) axle articulated or rigid vehicle and trailer	9	Axles = 6 and groups > 2; or axles > 6 and groups = 3, axles in a group up to 2.5 m apart
Medium combination 17.5 m to 36.5 m	> 6	4	B-double, or heavy truck and trailer	10	Groups = 4 and axles > 6, axles in a group up to 2.5 m apart
	> 6	5 or 6	Double road train or heavy truck and two trailers	11	Groups = 5 or 6, and axles > 6, axles in a group up to 2.5 m apart
Long combination Over 33.0 m	> 6	> 6	Triple road train or heavy truck and three trailers	12	Groups > 6 and axles > 6, axles in a group up to 2.5 m apart

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 m)
d1: distance between first and second axle
d2: distance between second and third axle
g1: distance between first and second group.

Source: Austroads (2023).

Figure 7.1: Dominant vehicles in each Austroads class



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)
- quad-axle with dual tyres (QADT).

All tyres referred to are conventional tyres. In order to consider axle groups fitted with wide super single tyres in the design of flexible pavements.Table 7.8 provides axle loads which cause the same pavement damage as a Standard Axle.

Appendix I provides guidance on the damage to road pavements caused by the passage of specialised vehicles with unusual configurations of axle wheel loads, tyre loads and types (e.g. mobile cranes).

The design tyre-pavement contact stresses for pavement analysis are taken as 750 and 800 kPa (Chapter 8). However, data collected in Tasmania by Chowdhury and Rallings (1994) indicates that tyre inflation 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
- 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
- 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.

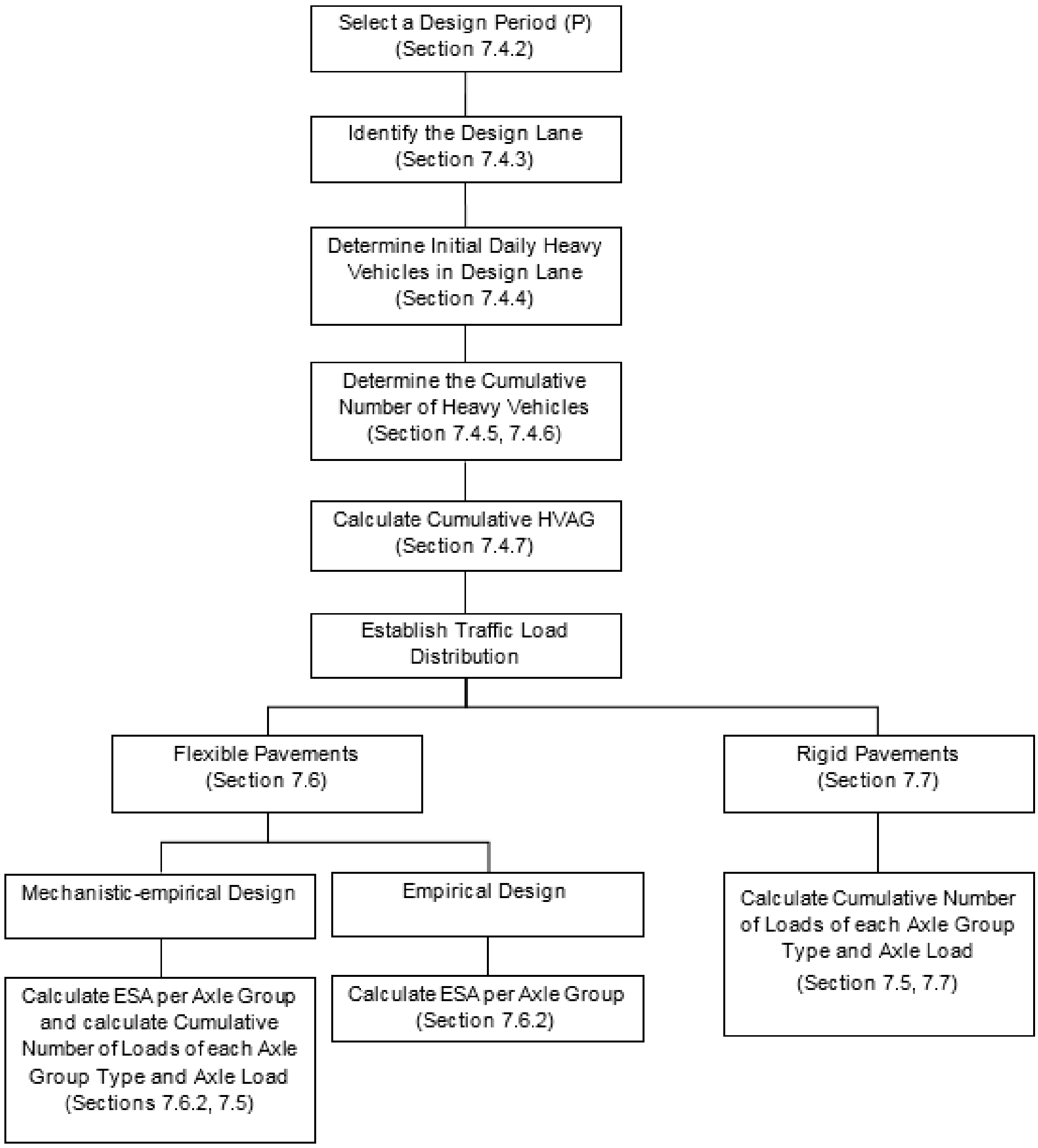
The sequence of steps leading to the specification of the design traffic for a project is as follows:

1. Select a design period (Section 7.4.2).
2. Identify the most heavily trafficked lane in the carriageway – designated the design lane (Section 7.4.3).
3. Estimate the average daily number of heavy vehicles in the design lane during the first year of the project's life (Section 7.4.4).
4. Estimate heavy traffic growth throughout the design period (Sections 7.4.5 and 7.4.6).
5. Estimate the average number of axle groups per heavy vehicle (Section 7.4.7).
6. Combine the above three estimates to calculate the cumulative heavy vehicle axle groups over the design period (Section 7.4.7).
7. Estimate the proportion of axle group types and the distribution of axle group loads (Section 7.5).
8. Express the cumulative traffic loading in a form suitable for the 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 reconstruction, 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.

Figure 7.2: Procedure for determining design traffic



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, designed N_{DT}. Calculation of the cumulative HVAG is commonly broken down into the following elements:

1. Select a design period (Section 7.4.2).
2. Identify the most heavily trafficked lane in the carriageway – designated the design lane (Section 7.4.3).
3. Estimate the average daily number of heavy vehicles in the design lane during the first year of the project's life (Section 7.4.4).
4. Estimate the cumulative heavy vehicles over the design period (Section 7.4.5 and 7.4.6).
5. Estimate the cumulative heavy vehicle axle groups over the design period (Section 7.4.7).

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 (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. 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. expansive 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 channel, 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

Flexible pavements	20–40 years
Rigid pavements	30–40 years

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, on/off ramps
- the primary and secondary functions of the road.

Weigh-in-motion (WIM) systems are available which provide lane-specific heavy vehicle traffic loading data across a carriageway. If project-specific information is not available, then Table 7.3 provides guidance on the proportion of heavy vehicle traffic loading assigned to the design lane. The proportion assigned to a specific lane is termed the lane distribution factor (LDF) for the lane.

Table 7.3: Typical lane distribution factors

Location	Lanes each direction	Lane distribution factor (LDF)		
		Left lane	Centre lane	Right lane
Rural	2 lane	1.00[1]	N/A	0.50
	3 lane	0.95	0.65	0.30
Urban	2 lane	1.00[1]	N/A	0.50
	3 lane	0.65	0.65	0.50

1 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.

7.4.4 Initial daily heavy vehicles in the design lane

To calculate the cumulative HVAG in the design lane (Equation 30), 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.

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. WIM survey data either collected specifically for the project or recently collected for other purposes. WIM data also provides the number of axle groups per heavy vehicle required to estimate the cumulative number of HVAG (Section 7.4.7) and the distribution of axle group types and loads required to calculate the design traffic for flexible and rigid pavements (Section 7.5).
2. Use of data obtained from vehicle classification counters. Such data will furnish the number of vehicles of various types (Table 7.1) as well as data on the number of axle groups per heavy vehicle required to estimate the cumulative number of HVAG (Section 7.4.7). 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 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.

For the latter method, the equation to derive the initial daily heavy vehicles (N_i) traversing the design lane is (Equation 30).

$$N_i = AADT \times DF \times \%HV/100 \times LDF \qquad 30$$

where

N_i	=	initial daily heavy vehicles in the design lane
$AADT$	=	Annual Average Daily Traffic[2] in vehicles per day in the first year (Section 7.4.4)
DF	=	direction factor is the proportion of the two-way AADT travelling in the direction of the design lane
$\%HV$	=	average percentage of heavy vehicles (Section 7.4.4)
LDF	=	lane distribution factor, proportion of heavy vehicles in the design lane (Section 7.4.3)

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 Part may not be appropriate.

7.4.5 Cumulative number of heavy vehicles when below capacity

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 7.4.8.

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 greenfield project, then the estimation should be based on the growth 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 (CGF), when constant, over the design period is readily calculated as follows (Equation 31).

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

$$CGF = \frac{(1+0.01R)^P - 1}{0.01R} \text{ for } R > 0 \qquad 31$$

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

where

CGF = cumulative growth factor

R = annual growth rate (%)

P = design period (years)

For this case of below-capacity traffic volumes throughout the design period, values of the CGF for a range of annual growth rates and design periods are presented in Table 7.4.

Table 7.4: 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 case of below-capacity traffic volumes throughout the design period, the equation to derive the Design Traffic (N_{HV}) – in cumulative heavy vehicles – traversing the design lane during the specified period is (Equation 32).

$$N_{HV} = 365 \times CGF \times N_i \qquad 32$$

where

N_{HV} = design traffic in cumulative heavy vehicles

CGF = cumulative growth factor (Section 7.4.5 and Section 7.4.6)

N_i = average daily number of heavy vehicles in the first year of opening to traffic (Section 7.4.4)

The cumulative number of heavy vehicles calculated using the above procedures assumes the design lane is operating below capacity for the entire design period. For projects with cumulative number of heavy vehicles estimated with the above procedures exceeding 10^7 it is recommended that a check be made, in accordance with Section 7.4.6, as to whether the volume of heavy vehicles has reached capacity before the end of the design period. In the event that capacity has been exceeded, Section 7.4.6 provides a procedure to estimate the cumulative number of heavy vehicles by limiting the annual heavy vehicle loading.

7.4.6 Cumulative number of heavy vehicles considering capacity

As described in Section 7.4.5, for heavily trafficked roads a check should be made as to whether the annual number of heavy vehicles estimated in the design lane exceeds the lane capacity. If it does, the cumulative number of heavy vehicles calculated using the method provided in Section 7.4.5 is incorrect and needs to be revised as described below.

Detailed traffic modelling can be used to predict when capacity is reached (Austroads 2013). This approach requires the use of complex models and specialist software. In traffic engineering, capacity is often defined by saturation flow in the peak hour, while for pavement design purposes, capacity is typically defined as saturation flow over a longer period of time (in this Part a 24 hour period is considered).

This section presents a simplified method for estimating capacity based flow rates expressed as passenger car equivalent (PCE) volumes per hour. This simplified procedure can be adopted for pavement design purposes, but is not suitable to be used in place of detailed traffic modelling for other purposes.

For pavement design, capacity is defined as when vehicle flows are such that the road must operate at saturation capacity 24 hours per day. Beyond this point in time the heavy vehicle growth rate is likely to be significantly reduced (the number of heavy vehicles can only increase if the number of light vehicles decreases).

Using information in the *Guide to Traffic Management Part 3: Traffic Studies and Analysis* (Austroads 2013), Table 7.5 lists presumptive maximum hourly numbers of passenger cars at capacity.

Table 7.5: Capacity flow rates

Road conditions	Capacity flow rate (passenger cars per hour per lane)
Motorway	
Through lanes	2300
Ramps	2000
Multilane road (two or more lanes in each direction, other than motorway)	
posted speed 100 km/h	2200
posted speed 90 km/h	2100
posted speed 80 km/h	2000
posted speed 70 km/h	1900
posted speed <70 km/h	1850
Two-lane, two-way (one lane in each direction)	1700
Signalised intersections	1850[(1)]

1 For signalised intersections, this flow rate is multiplied by green time proportion (see Austroads 2013).

Source: Austroads (2013).

The capacity is affected by the presence of heavy vehicles and is addressed using an average passenger car equivalent factor (E_{HV}). The average number of passenger cars equivalent to the hourly volume of heavy vehicles is calculated by multiplying the heavy vehicle volume by E_{HV} values. E_{HV} values of 2.0 (urban traffic) and 2.5 (rural traffic) will generally be sufficiently accurate for pavement design purposes. Alternatively, the following presumptive factors (Austroads 2013) may be used to derive a more appropriate E_{HV} value if vehicle classification counts are available:

- Austroads vehicle classes 2 to 5: $E_{HV,Class\ 2\ to\ 5}$ = 2
- Austroads vehicle classes 6 to 9: $E_{HV,Class\ 6\ to\ 9}$ = 3
- Austroads vehicle classes 10 to 11: $E_{HV,Class\ 10,11}$ = 4
- Austroads vehicle class 12: $E_{HV\ Class\ 12}$ = 5

Using Equation 33, the maximum daily volume of vehicles travelling in the direction of the design lane can be determined.

$$C_{veh} = \frac{24\ x\ C_{pc}\ x\ N_{lanes}}{\left(1 + \left(\frac{\%HV}{100}\right)(E_{HV} - 1)\right)} \qquad 33$$

where

C_{veh}	=	maximum daily volume of vehicles traveling in the direction of the design lane (vehicles/day)
C_{pc}	=	maximum hourly volume of passenger cars per lane (see Table 7.5) (pc/h/lane)
N_{lanes}	=	number of traffic lanes in the direction of the design lane
%HV	=	average percentage of heavy vehicles in the direction of the design lane (Section 7.4.4)
E_{HV}	=	average number of passenger cars equivalent to a heavy vehicle (pc/HV)

The associated maximum annual heavy vehicle volume in the design lane is calculated using Equation 34 assuming all lanes in the direction of the design lane operate at capacity each hour of each day of the year.

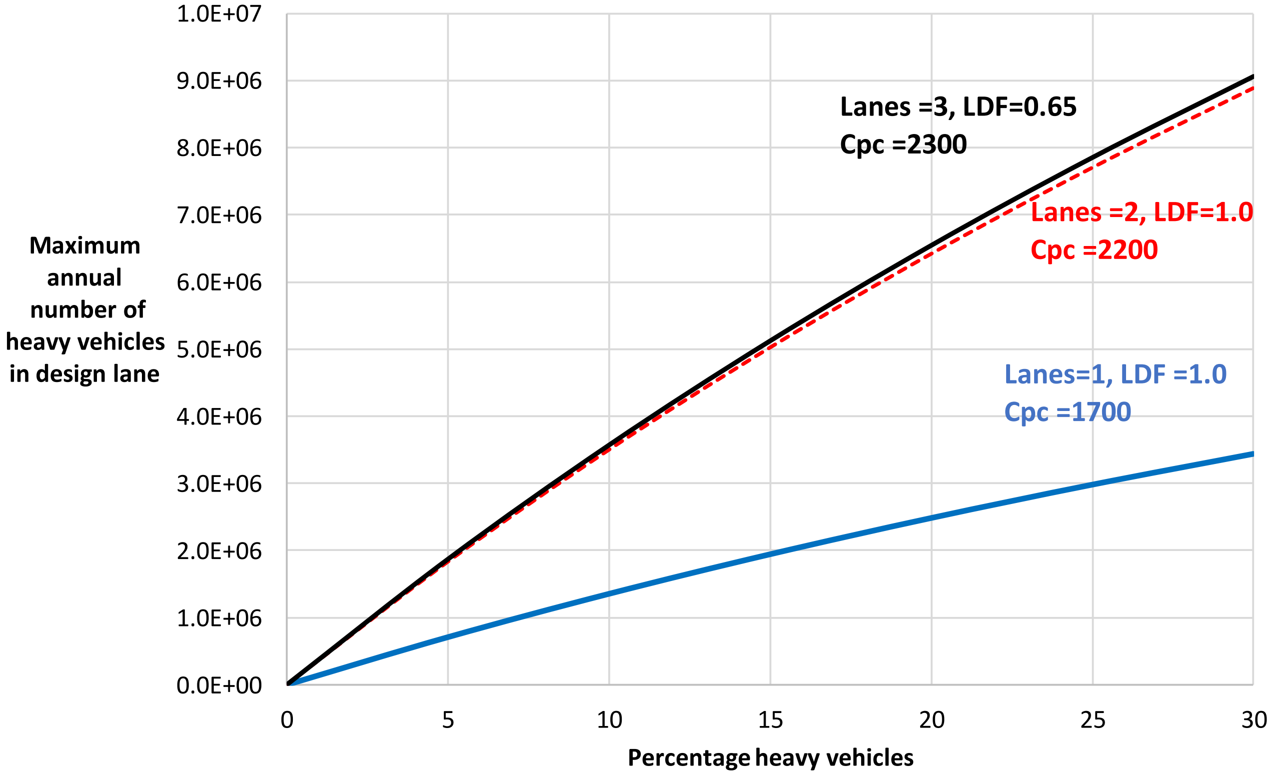
$$HV_{max} = \left(\frac{\%HV}{100}\right)\ x\ LDF\ x\ 365\ x\ C_{veh} \qquad 34$$

where

HV_{max}	=	maximum annual volume of heavy vehicles in the design lane (HV/year)
C_{veh}	=	maximum daily volume of vehicles traveling in the direction of the design lane (vehicles/day)
%HV	=	average percentage of heavy vehicles in the direction of the design lane (Section 7.4.4)
LDF	=	lane distribution factor, proportion of heavy vehicles in the design lane (Section 7.4.3)

Figure 7.3 shows examples of the maximum annual number of heavy vehicles in the design lane using E_{HV} value of 2.0 and capacity flow rates of 1700, 2200 and 2300 passenger cars per hour for 1, 2 and 3 lanes in each direction (Table 7.5), respectively.

Figure 7.3: Maximum annual heavy vehicles in design lane for E_{HV} = 2



The cumulative number of heavy vehicles is calculated as follows:

1. From the initially daily HV (Section 7.4.4), calculate the annual number of HV in the design lane in the first year.
2. Using the annual HV growth rate (R%, Section 7.4.5), calculate the annual number of HV in the design lane for each year of the design period:
3. $HV_{year\ n} = HV_{year\ 1} \times (1 + 0.01R)^{n-1}$
4. Determine an appropriate value for maximum hourly number of cars per lane (C_{pc}) (Table 7.5).
5. Using equation 33, determine the maximum daily number of vehicles travelling summed over all lanes in the direction of the design lanes.
6. Using equation 34, determine the maximum annual number of HV in the design lane.
7. Calculate the annual number of HV each year as the minimum value calculated in step 2 and the maximum value calculated in step 5.
8. The cumulative number of HV in the design lane over the design period is the sum of the adjusted annual HV vehicle volumes (step 6).

Appendix C shows an example of the calculation of the cumulative HV (N_{HV}) considering capacity.

7.4.7 Cumulative heavy vehicle axle groups

To calculate the cumulative heavy vehicle axle groups (in the design lane) over the design period (N_{DT}), the average number of axle groups per heavy vehicle (N_{HVAG}) is required (Equation 35).

$$N_{DT} = N_{HV} \times N_{HVAG} \tag{35}$$

where

N_{DT}	=	the cumulative heavy vehicle axle groups in the design lane over the design period
N_{HV}	=	cumulative number of heavy vehicles (Section 7.4.5, Section 7.4.6)
N_{HVAG}	=	average number of axle groups per heavy vehicle

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.

If WIM data does not provide a measured value for N_{HVAG}, an estimate can be made by assuming that all vehicles in the traffic load distribution have either a single or tandem axle with single tyres as the steer axle groups, and that the only single and tandem axles with single tyres in the distribution are steer axle groups (Equation 36).

$$N_{HVAG} = \frac{1}{0.01 \times \%SAST + 0.01 \times \%TAST} \tag{36}$$

where

N_{HVAG}	=	average number of axle groups per heavy vehicle
$\%SAST$	=	percentage proportion of single axle single tyre axle groups in traffic load distribution
$\%TAST$	=	percentage proportion of tandem axle single tyre axle groups in traffic load distribution

In the absence of WIM or classification counter data, a presumptive value for N_{HVAG} needs to be selected. Appendix D 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 and urban moderate-to-heavily trafficked roads have been calculated using WIM data. These presumptive values are given in Table 7.6. Section 12.7 provides guidance on procedures for lightly trafficked pavements.

Table 7.6: Presumptive numbers of heavy vehicle axle groups per heavy vehicle (N_{HVAG})

Location	N_{HVAG}
Rural roads	2.8
Urban roads	2.5

7.4.8 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 E.

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
- for each axle group type, the proportion of axles applied at each load magnitude.

The uses of the TLD in calculating design traffic loadings are 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 D are representative of the results from WIM surveys undertaken 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.

Section 12.7 provides guidance on procedures for lightly trafficked pavements.

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 Part provides two flexible pavement design procedures as follows:

1. Empirical design applicable to new flexible pavements consisting of a thin bituminous surfacing (sprayed seal or asphalt less than 40 mm thick) over granular material, using Figure 8.4.
2. Mechanistic-empirical design applicable to new flexible pavements which contain one or more layers of bound material (asphalt, cemented material or lean-mix concrete).

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
- fatigue damage to cemented material or lean-mix concrete.

Design traffic loading is commonly described in terms of the number of Equivalent Standard Axles (ESA). The design traffic used in the empirical design of unbound granular pavements with thin bituminous surfacings (Figure 8.4) is expressed in terms of ESAs. Additionally, when considering rutting and loss of surface shape in the mechanistic-empirical design of pavements the design traffic is considered also in units of ESAs (Chapter 5).

For the mechanistic-empirical design of pavements containing bound materials, the cumulative HVAG (Section 7.4), together with the traffic load distribution (TLD), are required to characterise the Design Traffic when considering fatigue damage to asphalt, cemented materials and lean-mix concrete.

7.6.2 Pavement damage in terms of Equivalent Standard Axle repetitions

The Standard Axle is defined as a single axle with dual tyres (SADT) applying a load of 80 kN to the pavement. To determine Design Traffic in terms of ESAs the loads on axle configurations that are considered to cause the same damage (i.e. overall pavement damage when using the empirical procedure, and rutting and loss of surface shape in the mechanistic-empirical design procedure) are given in Table 7.7 and Table 7.8. Denoting this axle group load (which causes the same damage as a Standard Axle) as the axle group's Standard Load (SL_i), ESAs of damage is evaluated as follows (Equation 37).

$$ESA_{ij} = \left(\frac{L_{ij}}{SL_i}\right)^4 \qquad (37)$$

where

ESA_{ij} = number of repetitions of a Standard Axle which causes the same amount of 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 7.7 and Table 7.8)

L_{ij} = j^{th} load magnitude on the axle group type i

Table 7.7: Loads on axle groups with dual tyres which cause same damage as a Standard Axle

Axle group type	Load (kN)
Single axle with dual tyres (SADT)	80
Tandem axle with dual tyres (TADT)	135
Triaxle with dual tyres (TRDT)	182
Quad-axle with dual tyres (QADT)	226

Table 7.8: Loads on axle groups with single tyres which cause same damage as a Standard Axle

Axle group type	Nominal tyre section width	Load (kN)
Single axle with single tyres (SAST)	Less than 375 mm	53
	At least 375 mm but less than 450 mm	58
	450 mm or more	71
Tandem axle with single tyres (TAST)	Less than 375 mm	89
	At least 375 mm but less than 450 mm	98
	450 mm or more	119
Triaxle with single tyres (TRST)	Less than 375 mm	121
	At least 375 mm but less than 450 mm	132
	450 mm or more	162
Quad-axle with single tyres (QAST)	Less than 375 mm	150
	At least 375 mm but less than 450 mm	164
	450 mm or more	201

The calculation of the ESAs of damage due to the design traffic requires the estimation of the average number of ESAs 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 D.

The ESA/HVAG values for a range of selected WIM sites are provided in Appendix D. Section 12.7 provides guidance on procedures for lightly trafficked pavements.

Details of the procedure to estimate ESA/HVAG are given in Appendix G.

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

The design number of Equivalent Standard Axles of traffic loading (DESA) is calculated as follows (Equation 38).

$$DESA = ESA/HVAG \times N_{DT} \tag{38}$$

where

$ESA/HVAG$ = average number of Equivalent Standard Axles per Heavy Vehicle Axle Group

N_{DT} = cumulative number of Heavy Vehicle Axle Groups over design period (from Equation 35)

7.6.3 Design traffic for mechanistic-empirical design procedure

The Design Traffic used in the mechanistic-empirical design procedure when considering rutting and loss of surface shape is expressed in terms of ESAs (Section 7.6.2).

For the consideration of the performance of bound materials using the mechanistic-empirical design procedure, the Design Traffic is characterised by the cumulative HVAG (Section 7.4.7) and the traffic load distribution (TLD).

As discussed in Section 6.7.10, there is increasing recognition of the notion that asphalt mixes experience increased crack healing with increasing temperature to a greater extent than provided in the asphalt fatigue relationship (Equation 25).

As an interim measure, pending further research to quantify the increase in crack healing with increasing temperature, an upper limit may be used in the design traffic loading used in the asphalt fatigue damage calculations. Table 7.9 provides suggested limiting design traffic loadings in ESA. The limit reduces with increasing temperature due to the increase in crack healing. Based on the ESA limits and using the number of ESAs per heavy vehicle axle groups, the upper limit in terms of cumulative HVAG used in the asphalt fatigue damage calculations is determined using Equation 39.

Table 7.9: Suggested upper limits on design traffic for asphalt fatigue

WMAPT	≤ 25 °C	26–34 °C	≥ 35 °C
Design traffic loading limit (DESA)	4×10^8	2×10^8	10^8

$$N_{DT\ limit} = \frac{DESA_{limit}}{(ESA/HVAG)} \tag{39}$$

where

$N_{DT\ limit}$ = upper limit of cumulative number of Heavy Vehicle Axle Groups over design period for use in the asphalt fatigue damage calculations

$DESA_{limit}$ = upper limit of the design traffic expressed as Equivalent Standard Axles (ESA) for use in the asphalt fatigue damage calculations (Table 7.9)

$ESA/HVAG$ = average number of ESA per HVAG from the project traffic load distribution

Note that such design traffic loading limits are only used in the asphalt fatigue calculations; they are not applicable to the design traffic used for cemented materials and lean-mix concrete fatigue or permanent deformation damage calculations.

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 E for guidance.

The design of rigid pavements is discussed in Chapter 9 of this Part.

7.8 Example of design traffic calculations

An example of design traffic calculations is given in Appendix H.

8. Design of Flexible Pavements

8.1 General

The general design procedure which is contained in this Part is mechanistic-empirical 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 agencies, and has been found to give results consistent with the mechanistic-empirical procedure.

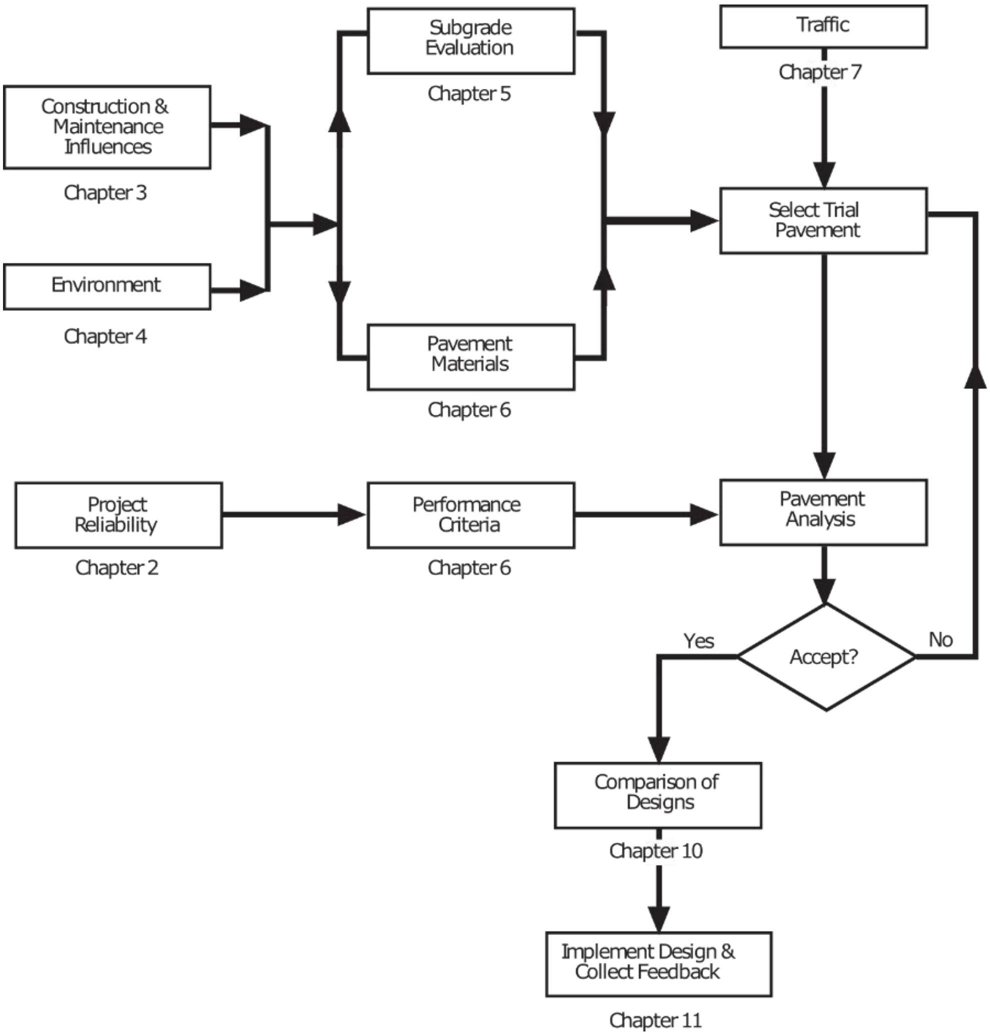
Layer thicknesses derived from any of the procedures contained in this Part should be considered to be minimum requirements and take no account of construction tolerance. This is particularly critical for bound pavement materials.

The procedures described in this chapter are applicable to moderate-to-heavily trafficked pavements. Guidance for the design of lightly trafficked flexible pavements is given in Section 12.8 of this Part.

8.2 Mechanistic-empirical procedure

The mechanistic-empirical procedure 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:

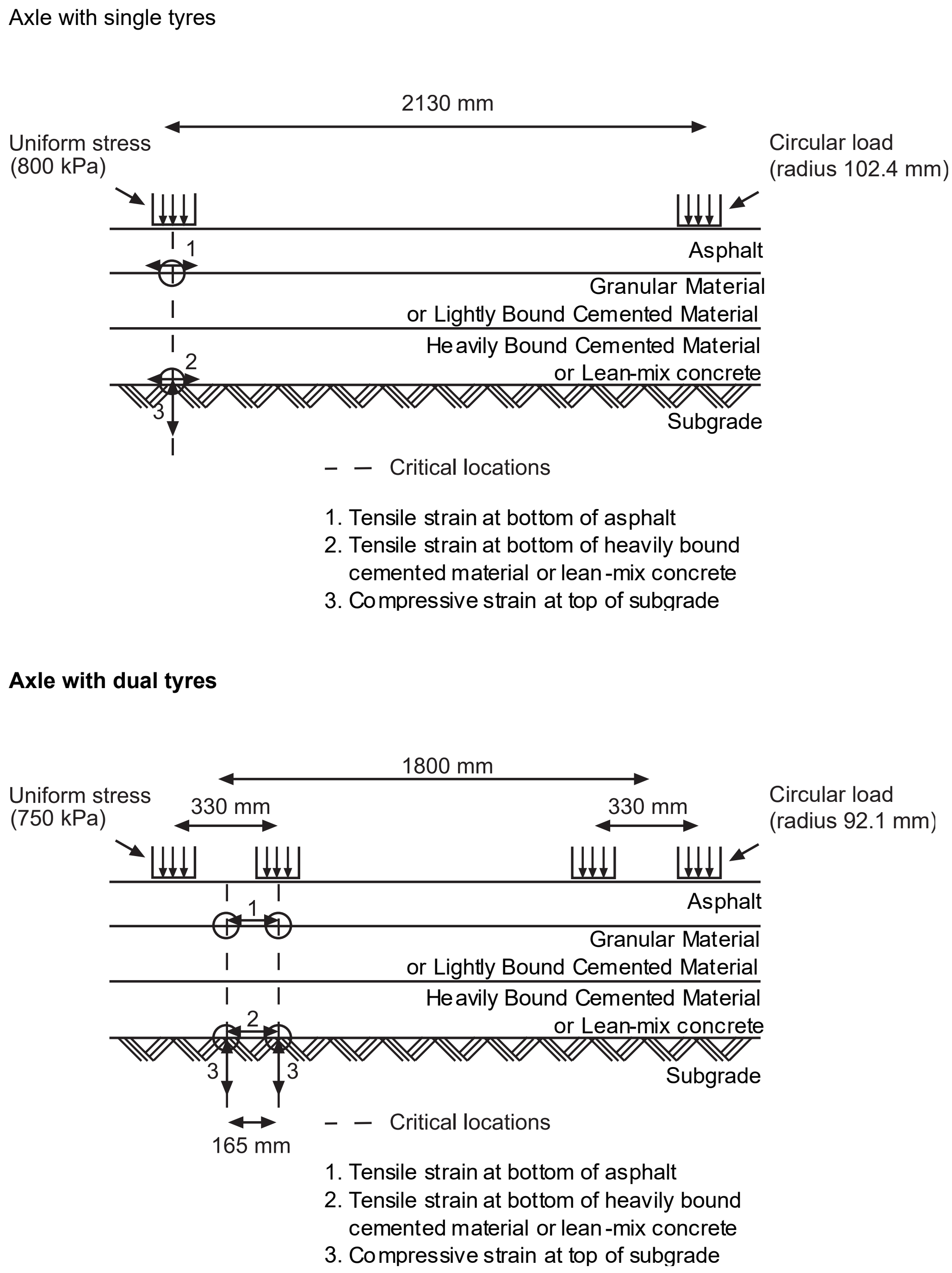
- evaluating the input parameters (materials, traffic, environment etc.)
- selecting a trial pavement
- analysing the trial pavement to determine the allowable traffic
- comparing this with the design traffic
- finally, accepting or rejecting the trial pavement.

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)
- design traffic loading (Chapter 7).

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

Figure 8.2: Pavement model for mechanistic-empirical procedure



Selected or improved subgrades may be considered in the pavement analysis. These materials include selected subgrade, lime stabilised subgrade, select materials, capping layers or earthworks materials. Section 8.2.2 provides the method for determining whether these materials should be considered as layers supporting the pavement structure or be incorporated within the in situ subgrade. Bitumen, cement or other chemical stabilised materials are not considered selected or improved subgrades in this document.

Significant features of the assumed model are as follows:

1. Materials are considered to be homogeneous, elastic and isotropic (except for unbound granular materials, modified materials, lightly bound cemented materials, cracked heavily bound cemented materials and subgrades which, as discussed in Chapters 5 and 6, are considered to be anisotropic).
2. Responses resulting from each axle within each axle group and applied load level within the design traffic load distribution (TLD) are determined.
3. Response to load is calculated using a linear elastic model, such as the computer programs AustPADS (Austroads Pavement Analysis Design Software) and CIRCLY. The program must be able to model anisotropic materials.
4. The critical responses assessed for pavement and subgrade materials are:
5. asphalt – horizontal tensile strain at bottom of layer
6. heavily bound cemented material/lean-mix concrete – horizontal tensile strain at bottom of layer
7. in situ subgrade and selected or improved subgrade materials – vertical compressive strain at the top of the layer.
8. Note, responses in unbound granular materials (including modified materials) and lightly bound cemented materials are not considered by the design model.
9. Responses are determined under a single-tyred single axle applying a load of 53 kN and a dual tyred single axle applying a load of 80 kN. These responses are linearly scaled with load to determine the responses resulting from other axle loads within the design TLD.
10. For flexible pavements, the critical responses within the pavement occur either along the vertical axis directly below the tyre of the single tyre group and the inner-most tyre of the dual tyre group or along the vertical axis located symmetrically between a pair of dual tyres (Figure 8.2).
11. Single-tyred axle loading is represented by two uniformly loaded circular areas of equal area (radius 102.4 mm) separated by a centre-to-centre distance of 2130 mm as illustrated in Figure 8.2. The contact stress is assumed to be uniform over the loaded area and, for the purpose of design, is taken to be 800 kPa. The contact stress is related to the air pressure in the tyre in-service which for highway traffic is assumed to be in the range 500–1000 kPa.
12. Dual tyred axle loading is represented by four uniformly loaded circular areas of equal area (radius 92.1 mm) separated by centre-to-centre distances of 330 mm, 1470 mm, and 330 mm respectively as illustrated in Figure 8.2. 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 air pressure in the tyre in-service which for highway traffic is assumed to be in the range 500–1000 kPa.
13. The dual tyred axle of the geometry shown in Figure 8.2, applying a 80 kN load and with circular tyre contact stress of 750 kPa is termed the Standard Axle.
14. 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. However, this is rarely undertaken for most pavement design situations and there is little case study experience to relate the calculated pavement responses to pavement performance.
15. For some projects, the mechanistic-empirical modelling may indicate that both a thin (< 50 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-empirical modelling of asphalt layers less than 40 mm thick is less certain than for thicker asphalt layers (Section 8.2.8). Appendix J discusses why more than one asphalt thickness is theoretically possible.

Table 8.1, Table 8.2 and Table 8.3 list in detail the steps which are required to carry out the mechanistic-empirical design procedure.

- Table 8.1 deals with design in inputs.
- Table 8.2 deals with the analysis.
- Table 8.3 deals with the interpretation of the results of the analysis.

Table 8.1: Mechanistic-empirical design procedure: input requirements

Step	Activity	Reference
1	Select a trial pavement and a desired project reliability.	Section 8.2.1 Section 2.3.1
2	Determine the following elastic parameters for the in situ subgrade selected or improved 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 and Section 8.2.3
4	Determine the elastic parameters and thickness of the other granular sublayers (if relevant).	Section 8.2.3
5	Determine the elastic parameters for heavily bound cemented materials and lean-mix concrete, pre and post-fatigue cracking (if relevant).	Section 6.4
6	Determine the elastic parameters for asphalt (if relevant).	Section 6.6
7	Determine the elastic parameters of lightly bound material (if relevant), and check minimum base thickness and subbase support (if relevant).	Section 6.6 and Section 8.2.4
8	Adopt the subgrade strain criterion.	Section 5.8
9	Determine fatigue criteria for heavily bound cemented materials and lean-mix concretes (if relevant).	Section 6.4
10	Determine fatigue criteria for asphalt (if relevant).	Section 6.6
11	Select cumulative number of HVAGs and traffic load distribution that comprise the Design Traffic.	Section 7.4 and Section 7.5
12	Determine the design number of ESAs.	Section 7.6.2

Table 8.2: Mechanistic-empirical design procedure: calculation of critical strains

Step	Activity
13	Approximate the Standard Axle tyre 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 = 92.1 mm for highway traffic.
14	Determine critical locations in the pavement for the calculation of strains resulting from the Standard Axle as follows: • bottom of each asphalt, heavily bound cemented and lean-mix concrete layer, and • top of in situ subgrade and the top of selected or improved subgrade, on vertical axes through the centre of an inner tyre load and through the point midway between the two tyre loads at a centre-to-centre spacing of 330 mm.
15	Input the above values (Step 14) into the linear elastic model (e.g. AustPADS, CIRCLY) and determine the maximum vertical compressive strain at the top of the subgrade and the top of the selected or improved subgrade materials and the maximum horizontal tensile strain at the bottom of each asphalt, heavily bound cemented material and lean-mix concrete layer. If the post-fatigue cracking phase of heavily bound cemented materials or lean-mix concrete life is being considered (Section 8.2.7), it is necessary to calculate critical strains for both pre-cracking and post-cracking phases of life. If only the post-cracking phase of heavily bound cemented materials is being considered, i.e. the pre-cracking phase is not being considered, then critical strains need only be calculated for the post-cracking phase of life.

Step	Activity
16	If asphalt, heavily bound cemented material or lean-mix concrete layers are present in the candidate structure, approximate a single axle with single tyres as two uniformly loaded circular areas at centre-to-centre spacings of 2130 mm; a vertical load of 26.5 kN is applied to each circular area at a uniform vertical stress distribution of 800 kPa. Radius of each loaded area, R = 102.4 mm for highway traffic.
17	If asphalt, heavily bound cemented material or lean-mix concrete layers are present in the candidate structure, determine critical locations in the pavement for the calculation of strains resulting from the single axle load (Step 16) at the bottom of each asphalt, heavily bound cemented material and lean-mix concrete layer on a vertical axis through the centre of the tyre.
18	If asphalt, heavily bound cemented material or lean-mix concrete layers are present in the candidate structure, input the above values (Step 17) into the linear elastic model and determine the maximum horizontal tensile strain at the bottom of each asphalt, heavily bound cemented material and lean-mix concrete layer. If the post-fatigue cracking phase of heavily bound cemented material or lean-mix concrete is being considered (Section 8.2.7), it is necessary to calculate critical strains for both pre-cracking and post-cracking phases of life. If only the post-cracking phase of heavily bound cemented materials is being considered, i.e. the pre-cracking phase is not being considered, then critical strains need only be calculated for the post-cracking phase of life.

Table 8.3: Mechanistic-empirical design procedure: interpretation of results

Step	Activity
19	Using the strain criteria selected in Step 7 and the critical strains determined at the top of the in situ subgrade and selected or improved subgrade (Step 15), determine the allowable number of Equivalent Standard Axles.
20	For each in situ subgrade and selected or improved subgrade, compare the allowable number of ESAs (Step 19) with the design number of ESAs (Step 11). If the post-cracking phase of cemented material or lean-mix concrete layers is being considered, the total allowable number of ESAs is determined in Step 29. If, for any of the subgrade materials, the allowable number of ESAs does not exceed the design number of ESAs, the candidate pavement structure is unacceptable. Return to Step 1.
21	If asphalt, heavily bound cemented material and lean-mix concrete layers are not present in the candidate structure skip to Step 31. If asphalt, heavily bound cemented material or lean-mix concrete layers are present, repeat steps 22 to 28 for each asphalt, heavily bound cemented material and lean-mix concrete layer present in the candidate structure.
22	Repeat Steps 23 to 27 for each axle group type (SAST, SADT, TAST, TADT etc.) in the design TLD.
23	For each axle group load of the selected axle group from the design TLD, calculate the number of expected repetitions of this axle group at this load level in the design traffic.
24	Determine the number of allowable design repetitions of the selected axle group at this load level for the selected pavement layer.
25	Calculate the damage resulting from the axle load for the selected axle group as the ratio of expected repetitions (Step 23) to the allowable repetitions (Step 24).
26	Repeat Steps 23 to 25 for all other axle load levels for the selected axle group.
27	Sum the calculated damage for all load levels (Step 26) for the selected axle group type.
28	Calculate the total damage for the pavement layer resulting from the design traffic by summing the damage calculated for each axle group type.
29	If the post-cracking phase of heavily bound cemented material or lean-mix concrete layers 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. If the allowable loading exceeds the design traffic loading, expressed in ESA, for all asphalt, in situ subgrade and selected or improved subgrade, the candidate pavement is acceptable. Skip to Step 31.
30	If the total damage calculated for each asphalt, cemented and lean-mix concrete layer does not exceed 1, the trial pavement is acceptable. Proceed to Step 32.
31	If the total damage for any asphalt, heavily bound cemented material or lean-mix concrete layers exceeds 1 or additional pavement configurations are required for comparison, select a new trial pavement, return to Step 1 and repeat Steps 1 to 30.
32	Compare alternative acceptable designs.

Design examples using the mechanistic-empirical procedure are contained in Appendix K.

8.2.1 Selection of trial pavement

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

Lightly bound cemented material bases with unbound granular subbases

The elastic characterisation of some materials is, in part, influenced by the material thickness and elastic characterisation of adjacent materials. Of particular note is the requirement for minimum thicknesses of lightly bound cemented material bases to inhibit the development of macrocracks (Section 8.2.4). In this instance an iterative approach to developing the trial pavement configuration is necessary prior to calculating strains resulting from traffic loads.

8.2.2 Procedure for elastic characterisation of selected or improved subgrade materials

The selected or improved subgrade materials considered in this section may include selected subgrade, lime-stabilised subgrade, capping layers and earthworks materials. Bitumen, cement or other chemical stabilised materials are not considered improved subgrades in this document.

In the case of lime-stabilised subgrade materials the procedures are only applicable where:

- sufficient lime has been added to satisfy the lime demand test (refer to Section 4.7.2 of Part 4D, Austroads 2019a) such that enhanced properties are maintained in the long-term
- the lime content was determined using the Method B mix design (refer to Section 4.7.2 of Part 4D, Austroads 2019a).

These thickness design procedures may be conservative for materials designed using Method A (refer to Section 4.7.2 of Part 4D, Austroads 2019a) which includes a minimum strength requirement and alternative procedures have been developed (Queensland Department of Transport and Main Roads 2021).

The modulus of selected or improved subgrade materials is dependent not only on the intrinsic characteristics of these materials but also the stiffness of the underlying in situ subgrade. The effect reduces with the thickness of selected or improved subgrade materials and may be neglected if the total thickness of material exceeds 2 m.

If a selected or improved subgrade material is greater than 2 m thickness, the pavement support condition in mechanistic-empirical designs is modelled with this material as the semi-infinite subgrade layer and no sublayering is required – i.e. the top of this material is considered to be the in situ subgrade for design purposes. The process is the same if there are a series of selected or improved subgrade materials and the uppermost material is greater than 2 m thickness.

Otherwise, each selected or improved subgrade material is subdivided into sublayers according to the following guidelines:

1. Divide the thickness of each selected or improved subgrade material into five equi-thick sublayers.
2. The vertical modulus of the top sublayer of selected or improved subgrade material is the minimum of 150 MPa, 10 times the design CBR of the material and that dependent on the support provided by the underlying material (i.e. in situ subgrade, selected or improved subgrade material or lime-stabilised subgrade) determined using Equation 40.

$$E_{V\ \text{top sublayer}} = E_{V\ \text{underlying material}} \times 2^{(\text{thickness of each selected or improved subgrade layer}/150)} \qquad 40$$

3. Where there is more than one type of selected or improved material, the thickness to use in Equation 40 is the thickness of each selected or improved subgrade material type, rather than the total thickness of all materials.
4. The ratio of moduli of adjacent sublayers is given by Equation 41.

$$R = \left[\frac{E_{V\ \text{material top sublayer}}}{E_{V\ \text{underlying material}}}\right]^{\frac{1}{5}} \qquad 41$$

5. The modulus of each sublayer may then be calculated from the modulus of the adjacent underlying sublayer, beginning with the in situ subgrade, the modulus of which is known.
6. For all selected or improved subgrade materials, the other elastic parameters required for each sublayer may be calculated from the following relationships:

   $E_H = 0.5E_V$ (refer Section 5.6)

   $F = E_V/(1 + \nu_V)$

8.2.3 Procedure for elastic characterisation of granular material

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 modulus 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 modulus of the subgrade. Iterative analyses with a finite element model would permit allowance to be taken of the stress-dependent 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 Part utilises a linear elastic layer model, 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 onto a bound cemented material or lean-mix concrete subbase, no sublayering is required. The modulus is determined using the procedures discussed in Chapter 6.
2. For granular materials placed directly on the in situ subgrade or selected subgrade or material, sublayering is required and should be conducted as follows:
   a. Divide the total thickness of unbound granular materials into five equi-thick sublayers.
   b. The vertical modulus of the top sublayer is the minimum of the value indicated in Table 6.4 or Table 6.5 and that determined using Equation 42.

$$E_{V\ \text{top granular sublayer}} = E_{V\ \text{underlying material}} \times 2^{(\text{total granular thickness}/125)} \qquad 42$$

   c. The ratio of moduli of adjacent sublayers is given by Equation 43.

$$R = \left[\frac{E_{V\ \text{top granular sublayer}}}{E_{V\ \text{underlying material}}}\right]^{\frac{1}{5}} \qquad 43$$

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 or improved subgrade material as appropriate, the modulus of which is known. Granular materials need to be selected such that the vertical modulus calculated for each sublayer does not exceed the maximum modulus that the granular material in the sublayer can develop due to its intrinsic characteristics (Section 6.2.2 and 6.2.3). If this condition is not met, a material with a higher modulus needs to be used in this sublayer or an alternative pavement configuration selected.

3. For all granular materials, the other elastic parameters required for each sublayer may be calculated from the following relationships:

$$E_H = 0.5E_V \text{ (refer Section 6.2)}$$
$$F = E_V/(1 + \nu_V)$$

8.2.4 Procedure for elastic characterisation of lightly bound cemented materials

Like other cemented materials, lightly bound cemented materials are susceptible to fatigue cracking, but the severity of fatigue cracking is different from heavily bound cemented materials with binder contents of typically 3% or more (Austroads 2020).

At this stage, this Part does not provide specific guidance for the design of these lightly bound materials in lightly trafficked environments or in rehabilitation design. The characterisations of lightly bound materials discussed in this section and in Section 6.6 are limited to moderate-to-high levels of design traffic.

Pavements containing lightly bound bases and subbases are designed and constructed with limited strength such that the allowable traffic loading before fatigue cracking is insignificant compared to the allowable traffic loading post-fatigue cracking. As a result, elastic characterisation of lightly bound material (Section 6.6) only considers the fatigue cracked phase of material behaviour. The same elastic characterisation method is used for lightly bound as for the post-fatigue phase of heavily bound cemented materials.

Lightly bound cemented materials are susceptible to fine shrinkage cracking and load induced microcracking. To reduce the likelihood that macrocracking will develop, the loss of load transfer across such fine cracks with traffic loading needs to be minimised.

When lightly bound cemented materials are used as subbase it is usually not necessary to inhibit macrocracking.

However, when the materials are used in bases, with significantly less cover, it is necessary to inhibit the development of macrocracking and to ensure that the material stays in a microcracked only state. To maintain load transfer across microcracks and inhibit the development of macrocracking distress, limits are placed on the minimum lightly bound cemented material layer thicknesses and minimum support provided to lightly bound cemented material bases.

Minimum support to lightly bound cemented bases

For the lightly bound cemented material base of new pavements, the base shall be supported on a subbase with a thickness of at least 150 mm and which achieves a vertical design modulus of at least 150 MPa in the top sublayer of the subbase (Section 8.2.3). This may be achieved by increasing the thickness of the subbase and/or including additional improved subgrade or unbound granular materials beneath the subbase.

Minimum thickness of lightly bound cemented bases

The minimum base thickness of lightly bound cemented material pavements is determined from Equation 44.

The minimum base thickness determined using Equation 44 may be reduced by the thickness of any overlying structural asphalt or stone mastic asphalt; however, the base thickness should not be reduced to less than 200 mm.

An iterative approach is needed to select trial pavement configurations to satisfy both the minimum lightly bound cemented material base thickness and the maximum design modulus for the underlying support.

$$t_{min} = \text{maximum}\left[200, 250 + 35\log_{10}\left(\frac{DESA}{5\times 10^6}\right) - 0.25\left(E_{V\ \text{underlying material}} - 150\right)\right] \quad 44$$

where

t_{min}	=	minimum thickness of lightly bound cemented material base (mm)
$DESA$	=	design traffic expressed in ESA
$E_{V\ \text{underlying materiual}}$	=	vertical modulus of the material layer or sublayer immediately underlying the base (MPa)

8.2.5 Procedure for determining critical strains for asphalt, heavily bound cemented material and lean-mix concrete

In the calculation of design ESAs the standard loads on axle configurations that are considered to cause the same damage (i.e. overall pavement damage when using the empirical procedure, and rutting and loss of surface shape in the mechanistic-empirical design procedure), given in Table 7.7 and Table 7.8, do vary with the structure of the pavement being loaded. However research has shown (Austroads 2015b, Moffatt 2015) that similar standard loads used to equate tensile strain associated damage for bound materials are dependent upon pavement structure.

Accordingly the determination of allowable loading for bound materials does not use the concept of standard loads, but rather is based upon the determination of the bound material damage resulting from each axle load and each axle group type within the design traffic load distribution.

The determination of allowable loading (Section 8.2.6) requires the determination of the maximum horizontal tensile strain generated by axles with different load magnitudes, requiring a response-to-load calculation for each load magnitude and axle type. However it has been demonstrated (Austroads 2015b) that, for layered linear-elastic modelling, responses resulting from a given load can be linearly scaled to estimate responses resulting from a different load without significantly affecting design outcomes. Therefore Equation 45 can be used to determine the strains induced by a range of load magnitudes for single axles with single and dual tyres.

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SAST,53}}{53} \quad \text{for single axle with single tyres} \quad \text{(a)} \qquad 45$$

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SADT,80}}{80} \quad \text{for single axle with dual tyres} \quad \text{(b)}$$

where

$\mu\varepsilon_{ij}$ = load-induced strain caused by a single axle, with the same number of tyres as those used by the individual axles within axle group i, applying a load equal to the j^{th} load magnitude divided by n (microstrain)

L_{ij} = magnitude of the j^{th} load applied to axle group i (kN)

n = number of individual axles within axle group type i (e.g. $n = 2$ for a tandem axle group)

$\mu\varepsilon_{SAST,53}$ = strain induced by a single axle with single tyres applying a load of 53 kN (microstrain)

$\mu\varepsilon_{SADT,80}$ = strain induced by a single axle with dual tyres applying a load of 80 kN – i.e. the Standard Axle (microstrain)

8.2.6 Procedure for determining allowable loading for asphalt, heavily bound cemented material and lean-mix concrete

Response-to-load modelling of all grouped axles in the axle group could be used to determine the tensile strain developed under each constituent axle. However the material performance relationships and reliability factors used in this guide are not based upon such calculations, but utilise strain responses determined under single axles.

The allowable repetitions of an axle group with a given load level are calculated using Equation 46 for asphalt and Equation 47 or Equation 48 for heavily bound cemented materials and Equation 49 for lean-mix concretes. These equations relate the allowable repetitions to the maximum horizontal tensile strain developed at the base of these materials under a single constituent axle of the group.

Asphalt

$$N_{ij} = \frac{1}{n} \times \frac{SF}{RF} \times \left[\frac{6918(0.856V_b + 1.08)}{E^{0.36}\mu\varepsilon_{ij}}\right]^5 \qquad 46$$

where

N_{ij} = allowable number of repetitions of axle group type i with total load equal to the j^{th} load magnitude

n = number of individual axles within axle group type i (e.g. $n = 2$ for a tandem axle group)

$\mu\varepsilon_{ij}$ = load-induced tensile strain at the base of the asphalt (microstrain) caused by a single axle, with the same number of tyres as those used by the individual axles within axle group i, applying a load equal to the j^{th} load magnitude divided by n (microstrain)

V_b = percentage by volume of bitumen in the asphalt (%)

E = asphalt modulus (MPa)

SF = shift factor between laboratory and in-service fatigue lives (presumptive value = 6)

RF = reliability factor for asphalt fatigue (Table 6.17)

Heavily bound cemented materials (laboratory measured material characteristics)

$$N_{ij} = \frac{1}{n} \times RF \times \left(\frac{K}{\mu\varepsilon_{ij}}\right)^{12} \quad (47)$$

where

N_{ij}	=	allowable number of repetitions of axle group type i with total load equal to the j^{th} load magnitude
n	=	number of individual axles within axle group type i (e.g. $n = 2$ for a tandem axle group)
$\mu\varepsilon_{ij}$	=	load-induced tensile strain at the base of the heavily bound cemented material (microstrain) caused by a single axle, with the same number of tyres as those used by the individual axles within axle group i, applying a load equal to the j^{th} load magnitude divided by n (microstrain)
K	=	a constant, calculated by multiplying the laboratory fatigue constant k (Equation 11) by the laboratory-to-field shift factor (SF), 1.55 is the presumptive SF value
RF	=	the reliability factor for heavily bound cemented materials fatigue (Table 6.9)

Heavily bound cemented materials (presumptive material characteristics)

$$N_{ij} = \frac{1}{n} \times \left(\frac{K}{\mu\varepsilon_{ij}}\right)^{12} \quad (48)$$

where

N_{ij}, n, $\mu\varepsilon_{ij}$	=	as for Equation 47
K	=	presumptive constant, as given in Table 6.10

Lean-mix concrete

$$N_{ij} = \frac{1}{n} \times RF \times \left(\frac{K}{\mu\varepsilon_{ij}}\right)^{12} \quad (49)$$

where

N_{ij}, n, $\mu\varepsilon_{ij}$	=	as for Equation 47
K	=	presumptive constant, as given in Table 6.20
RF	=	the reliability factor for heavily bound cemented materials fatigue (Table 6.9)

To determine whether a candidate pavement structure is acceptable in terms of asphalt, heavily bound cemented material or lean-mix concrete fatigue life, the asphalt, heavily bound cemented material or lean-mix concrete fatigue damage caused by the design traffic is calculated as follows:

- The damage caused by each combination of axle group type and applied load within the design traffic load distribution is calculated – by dividing the number of expected repetitions of the axle group type/load by the number of allowable repetitions determined from Equation 46 for asphalt, Equation 47 or Equation 48 for heavily bound cemented materials, and Equation 49 for lean-mix concrete – as shown in Equation 50.
- The total asphalt, heavily bound cemented material or lean-mix concrete damage caused by the design traffic is the sum of the damage caused by each axle group/load – Equation 51.
- For the candidate pavement to be acceptable this total damage must not exceed 1.0 for any asphalt or heavily bound cemented material within the pavement.

$$d_{ij} = \frac{e_{ij}}{N_{ij}} \tag{50}$$

where

d_{ij} = damage caused by axle group i with total load equal to the j^{th} magnitude

e_{ij} = expected number of repetitions of axle group i with total load equal to the j^{th} magnitude

N_{ij} = allowable number of repetitions of axle group type i with total load equal to the j^{th} load magnitude

$$D = \sum_{all\ i,j} d_{ij} \tag{51}$$

where

D = total damage of the asphalt, heavily bound cemented material or lean-mix concrete layer resulting from the design traffic

d_{ij} = damage caused by axle group i with total load equal to the j^{th} magnitude

It follows that the allowable HVAG loading for an asphalt, heavily bound cemented material or lean-mix concrete layer is equal to the design number of HVAG, (N_{DT}), divided by the total damage of the material caused by the design traffic, D. This is shown in Equation 52. The allowable loading for the material layer can also be expressed in terms of ESA as shown in Equation 53 – this is necessary when considering the post-cracking phase of pavements containing heavily bound cemented materials or lean-mix concrete (Section 8.2.7).

$$A_{HVAG} = \frac{N_{DT}}{D} \quad 52$$

where

A_{HVAG} = allowable HVAG repetitions for the asphalt, heavily bound cemented material or lean-mix concrete layer

N_{DT} = cumulative heavy vehicle axle groups traversing the design lane during the design period

D = total damage of the asphalt, cemented material or lean-mix concrete layer resulting from the design traffic (Equation 51)

$$A_{ESA} = \frac{N_{DT} \times ESA/HVAG}{D} \quad 53$$

where

A_{ESA} = allowable ESA repetitions for the asphalt, heavily bound cemented material or lean-mix concrete layer

N_{DT} = cumulative heavy vehicle axle groups traversing the design lane during the design period

D = total damage of the asphalt, heavily bound cemented material or lean-mix concrete layer resulting from the design traffic (Equation 51)

$ESA/HVAG$ = average ESA/HVAG for the design traffic load distribution (Section 7.6.2)

8.2.7 Consideration of post-cracking phase in heavily bound cemented material and lean-mix concrete

Where a pavement incorporates a heavily bound cemented material or lean-mix concrete layer which reaches its allowable loading in terms of fatigue, the pavement may then enter a post-cracking phase whereby other layers continue to further the fatigue life of the pavement structure, and the heavily bound cemented material or lean-mix concrete layer is considered to have no remaining tensile capacity.

The requirements for consideration of a post-cracking phase in heavily bound cemented material and lean-mix concrete pavement structures are only required if the secondary phase of pavement life is being considered.

A post-cracking phase of the design life can only be considered in the mechanistic-empirical thickness design calculations if cracking from the fatigued heavily bound cemented material or lean-mix concrete does not reflect through to the surface. To reduce the risk of reflective cracking the pavement should provide a minimum cover equivalent to 175 mm of asphalt over the heavily bound cemented material or lean-mix concrete. 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 × thickness of granular material cover) + (thickness of asphalt cover) ≥ 175 mm

When a SAMI (refer to Section 3.8) is used in conjunction with these cover requirements the amount and/or severity of reflection cracking may be further reduced.

Cracked heavily bound cemented materials should be modelled as a cross-anisotropic material with a vertical modulus determined using the process for determining the post-cracking presumptive design modulus in Section 6.5.2, and a Poisson's ratio of 0.35. No sublayering of the material is required.

Cracked lean-mix concrete materials should be modelled as shown in Table 8.4.

Table 8.4: Elastic characterisation of post-cracking phase of lean-mix concrete

Cracked material	Modulus	E_V/E_H	Poisson's ratio	Sublayered
Lean rolled concrete (cracked by normal traffic)	600 MPa	2.0	0.35	No
Lean rolled concrete (cracked by construction traffic)	350 MPa	2.0	0.35	No
Lean screeded concrete	700 MPa	1.0	0.2	No

To calculate the total allowable loading of the pre-cracking and post-cracking fatigue phases, the allowable loadings of each material need to be expressed in terms of Equivalent Standard Axles (ESA).

$$\text{Asphalt fatigue: } N_A = N_C + \left(1 - \frac{N_C}{N_{1stA}}\right) \times N_{2ndA} \quad (54)$$

where

N_A = total allowable loading to asphalt fatigue (ESA)

N_C = allowable number of load repetitions (ESA) to heavily bound cemented material or lean-mix concrete fatigue (1st phase life)

N_{1stA} = allowable number of load repetitions (ESA) to asphalt fatigue prior to heavily bound cemented material or lean-mix concrete fatigue (1st phase)

N_{2ndA} = allowable number of load repetitions (ESA) to asphalt fatigue after heavily bound cemented material or lean-mix concrete fatigue (2nd phase life)

$$\text{Permanent deformation: } N_S = N_C + \left(1 - \frac{N_C}{N_{1stS}}\right) \times N_{2ndS} \quad (55)$$

where

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

N_C = allowable number of load repetitions (ESA) to heavily bound cemented material or lean-mix concrete fatigue (1st phase life)

N_{1stS} = allowable number of load repetitions (ESA) to unacceptable permanent deformation during the heavily bound cemented material or lean-mix concrete (1st phase)

N_{2ndS} = allowable number of load repetitions (ESA) to unacceptable permanent deformation after heavily bound cemented material or lean-mix concrete fatigue (2nd phase life)

Note that Equation 54 is only applicable if N_C is less than N_{1stA} and Equation 55 is only applicable if N_C is less than N_{1stS}.

For thick asphalt base on a heavily bound cemented material or lean-mix concrete subbase, pavements may also be designed without consideration of the asphalt life in the post-cracking phase, that is, pavements designed such that the heavily bound cemented material or lean-mix concrete layer outlasts the design traffic. It should be noted that for some pavements, particularly thin heavily bound cemented material or lean-mix concrete subbase layers on low strength subgrades, the required asphalt thickness to inhibit heavily bound cemented material and lean-mix concrete 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.

8.2.8 Design of granular pavements with thin bituminous surfacings

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

Designers are cautioned, however, that the mechanistic-empirical design model has not been validated for granular pavements having asphalt surface layers less than about 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
- the omission of environmental effects.

Tyre loading – Accurate measurements of tyre-pavement contact stress patterns (De Beer, Fisher and Jooste 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/modulus of the layer allow 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.

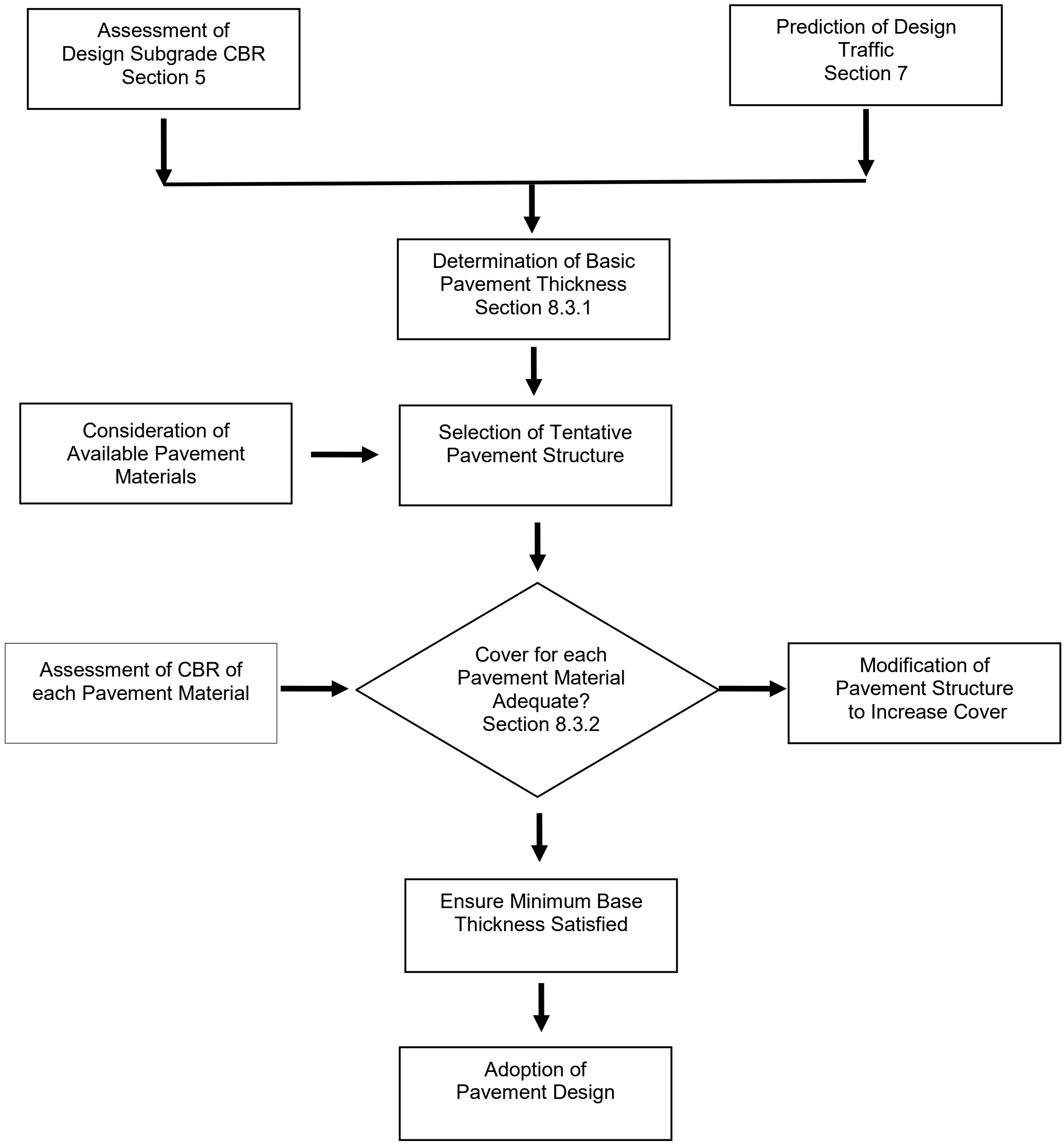
Environmental effects – Asphalt oxidises and hardens over time due to exposure to the atmosphere and becomes more prone to ravelling and surface cracking. The rate of oxidation is very closely related to its exposure to the atmosphere and hence the level of in situ air voids in the asphalt layer, which 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-empirical procedures to assess the fatigue life of such surfacings is discussed in Section 8.2.8.

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



8.3.1 Determination of basic thickness

The thickness of material required over the in situ subgrade is determined using the empirical design chart given in Figure 8.4 and design traffic loading in ESA (Equation 38). Note that Figure 8.4 is applicable to pavements with design traffic loading of 10^5 to 10^8 ESA. Section 12.8 describes procedures for design of lightly trafficked flexible pavements.

Note the mechanistic-empirical design procedures, as described in Section 8.2.8, yield a similar total granular thickness as Figure 8.4 using a top granular vertical modulus of 350 MPa.

8.3.2 Pavement composition

The total thickness of material over the in situ subgrade may be made up of the following materials:

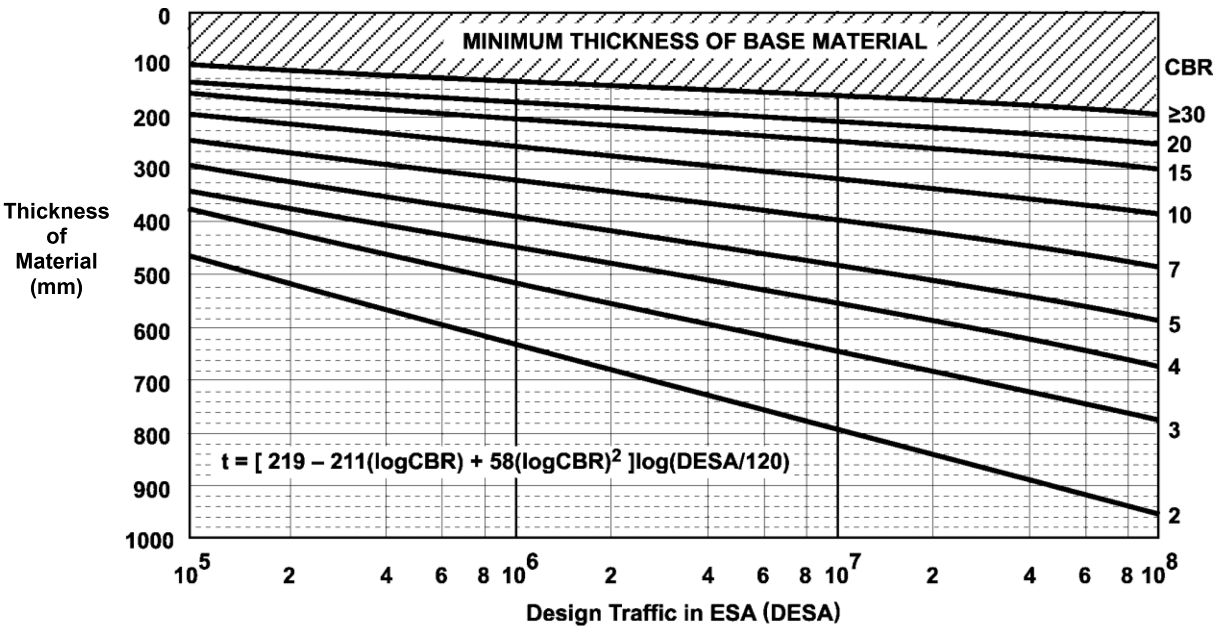
- unbound granular base and subbase courses
- improved subgrade materials including selected subgrade, lime stabilised subgrade, select materials, capping layers or earthworks materials (bitumen, cement or other chemical stabilised materials are not considered improved subgrades in this Part).

It is required that lime stabilised subgrade materials have sufficient lime to ensure design properties are achieved long term (refer to *Part 4D: Stabilised Materials*, Austroads 2019a). As discussed in Section 5.3.8, these thickness design procedures may be conservative for materials designed using Method A, which includes a minimum UCS of 1 MPa. If the amount of lime is insufficient to achieve long-term strength, no allowance should be made for the increase in subgrade CBR due to stabilisation.

As discussed in Section 8.2.2, if the uppermost material of a series of improved subgrade materials placed above the in situ subgrade is greater than 2 m thickness, the properties of this material can be considered to be properties of the in situ subgrade for design purposes.

The composition of the pavement structure is made up by providing sufficient cover over the in situ subgrade and each successive material course. The thickness of cover required over a material is determined from its design CBR. If the design CBR value of a material is less than 30%, then the cover required to inhibit deformation is determined as for an in situ subgrade material, from Figure 8.4. Appendix L provides worked examples.

Figure 8.4: Design chart for granular pavements with thin bituminous surfacing



For a granular 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%) granular 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 CBR test results are not the sole measure used to assess adequacy of unbound granular materials (Section 6.2.1).

Beneath the granular layers, selected or improved subgrade materials, including lime-stabilised subgrade materials, may be used to provide the required cover of materials over the in situ subgrade. The thicknesses of cover required over these materials are determined from their design CBR.

If the thin surfacing is dense graded asphalt or stone mastic asphalt, its thickness (< 40 mm) may be considered to contribute to the required total thickness over the in situ subgrade, but does not affect the required thickness of granular base. Other surfacing types (such as sprayed seals) are considered to make no contribution to the required thickness of granular material.

For pavements without selected or improved subgrade materials, the total thickness of granular material over the in situ subgrade and the minimum design CBR of each granular layer is determined directly from Figure 8.4.

For pavements with selected or improved subgrade materials, an iterative approach to the design is required as follows:

1. Select a trial pavement configuration.
2. The design CBR of each selected or improved subgrade material is the minimum of (1) 15%, (2) the value determined from CBR tests or a presumptive CBR, and (3) the value determined from the support provided by the underlying material (i.e. in situ subgrade, selected subgrade or lime-stabilised subgrade material) using Equation 56 .

$$CBR_{\text{selected or improved subgrade}} = CBR_{\text{underlying material}} \times 2^{(\text{thickness of selected or improved subgrade}/150)} \qquad 56$$

3. Using Figure 8.4, determine the total thickness of cover required to protect each selected or improved subgrade material. Select appropriate thicknesses and qualities of granular materials to provide the required cover.
4. Calculate the total thickness of all materials over the in situ subgrade and compare this to the thickness of cover required (Figure 8.4).
5. If there is insufficient cover over the in situ subgrade, repeat steps 1 to 4.

9. Design of Rigid Pavements

9.1 General

This chapter provides guidance on the thickness design of rigid pavements consisting of cast in situ concrete and is proposed for roads with a design traffic loading of 10^6 HVAG or more. Section 12.9 provides guidance for road pavements with lower traffic loadings. The information in Chapter 3 and Section 12.9 is not intended for use in the design of industrial or airport pavements.

Whilst this chapter 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. Further information on these issues is available from Austroads member agencies. More detailed advice on materials for concrete pavements is provided in *Part 4C: Materials for Concrete Pavements* of the Guide (Austroads 2017a).

This base thickness design method is based on the USA Portland Cement Association (PCA) method (Packard 1984) with revisions to suit Australian conditions (Austroads 2008). The method assumes that the base and subbase layers are not bonded.

The thickness design method is based on assessments of the:

- predicted traffic volume and composition over the design period (Chapter 7)
- strength of the subgrade in terms of its California Bearing Ratio (Chapter 5)
- flexural strength of the base concrete (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 base 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)
- 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. To assist users, Section 9.2.1 provides guidance on typical joint spacing and other details for four rigid (concrete) pavement types used commonly for roads in Australia.

9.2 Pavement types

9.2.1 Base types

The principal types of rigid (concrete) pavements are:

- plain (jointed unreinforced) concrete pavements (PCP)
- jointed reinforced concrete pavements (JRCP)
- continuously reinforced concrete pavements (CRCP)
- steel fibre reinforced concrete pavements (SFCP).

Design procedures for prestressed concrete bases and RCC bases are not considered in this Part.

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:

- slabs 4.2 m long, with undowelled skewed joints
- slabs 4.5 m long, with dowelled square joints.

PCP slabs are typically 4.2 m long with undowelled skewed joints with corner angles not less than 84 degrees (i.e. a skew of 1:10).

PCP 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 on length.

PCP slabs must be reinforced in discrete areas such as odd-shaped and mismatched slabs and in base anchor slabs. Designers must be mindful of joint continuity surrounding odd-shaped and mismatched slabs.

JRCP is typically mesh reinforced, with dowelled joints spaced between 8 and 10 m. The transverse joints are square to the direction of travel.

For CRCP, 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 during service and may be continuous under sawn longitudinal joints as equivalent tiebars.

Anchors are typically installed at the termination of rigid pavements to minimise effects of dynamic movements against adjacent flexible pavements and fixed structures such as bridge decks.

For all rigid pavement types, longitudinal joints should be provided to limit slab widths to about 4.3 m. All longitudinal joints should be tied up to a maximum total tied width as follows:

- for PCP 16 m
- for JRCP and CRCP about 30 m, provided transverse reinforcement is designed accordingly
- for SFCP 20 m.

Induced longitudinal joints must be placed close to lane linemarking with an offset or, alternatively, placed centrally in the middle of the lane to ensure the crack develops in the desired location.

Untied longitudinal joints do not satisfy the ‘with shoulder’ design condition (refer to Section 9.3.5) unless they are located at least 2 m outside a traffic lane.

SFCP 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 rigid 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 75° or 70° for trafficked and untrafficked slabs respectively. However, where possible, these low corner angles should be located away from heavy vehicle wheelpaths.

Transverse contraction joints in SFCP are typically square (or radial in the case of roundabouts) and undowelled, at maximum spacings of 6.0 m.

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 subbase 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
- dense graded asphalt
- lean-mix concrete.

Lean-mix concrete subbase (LCS) has a characteristic 28-day compressive strength of not less than 5 MPa and is designed to have low shrinkage, typically less than 450 microstrain.

A minimum 220 mm no fines concrete subbase under CRCP pavements in tunnels with drained linings has shown acceptable performance in Australia whilst providing a comprehensive drainage system. A minimum 30 mm dense graded asphalt interlayer is commonly used to separate the base and subbase. Any structural contribution of this interlayer is not considered in the thickness design process.

It should be noted that:

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. Material properties are therefore very important but are beyond the scope of this document.

   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 (Table 9.10).
2. 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.

Australasian experience is that lean-mix concrete 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. Experience has shown that the minimum thickness for subbases for heavy duty rigid pavements, including roundabouts, is 150 mm.

Table 9.1: Minimum subbase requirements for rigid pavements

Design traffic (HVAG)	Subbase type
Up to 10^6	125 mm heavily bound cemented crushed rock
Up to 5×10^6	150 mm heavily bound cemented crushed rock or 125 mm LCS
Up to 1×10^7	170 mm heavily bound cemented crushed rock or 125 mm LCS
Greater than 1×10^7	150 mm LCS(1)

1 *Under CRCP only, a heavily bound cemented crushed rock subbase with an asphalt interlayer is an acceptable alternative.*

Note: Minimum subbase requirement for mined tunnels with a CRCP base is 220 mm of no fines concrete. This is considered to provide equivalent support to 150 mm LCS. A minimum 30 mm dense graded asphalt interlayer is commonly used to separate the base and subbase. Any structural contribution of this interlayer is not considered in the thickness design process.

9.2.3 Wearing surface

The wearing surface texture specified for the road should take into consideration the traffic speed, grade, crossfall, carriageway width, and rainfall. For further details refer to *Part 3*: *Pavement Surfacings* of the Guide (Austroads 2025b).

Asphalt wearing surfaces should be avoided for all rigid pavement types except CRCP as reflective cracking from joint movements has been shown to propagate through the wearing surface at the joint locations.

For the purpose of the base design thickness, wearing surface layers of asphalt or concrete segmental pavers 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 subgrade CBR are discussed in Chapter 5.

For thickness design purposes, all materials within 1 m below the subbase must be assessed for determination of the design subgrade CBR. It is not permissible to adopt as Design CBR 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 (Section 5.3.5) that is constant for the service life of the pavement with a minimum soaked CBR of 5%. For pavement designs over highly expansive soils, a suitable strategy to mitigate the impact of volumetric changes should be considered in the development of the design, as discussed in Section 5.3.5.

9.3.2 Effective subgrade strength

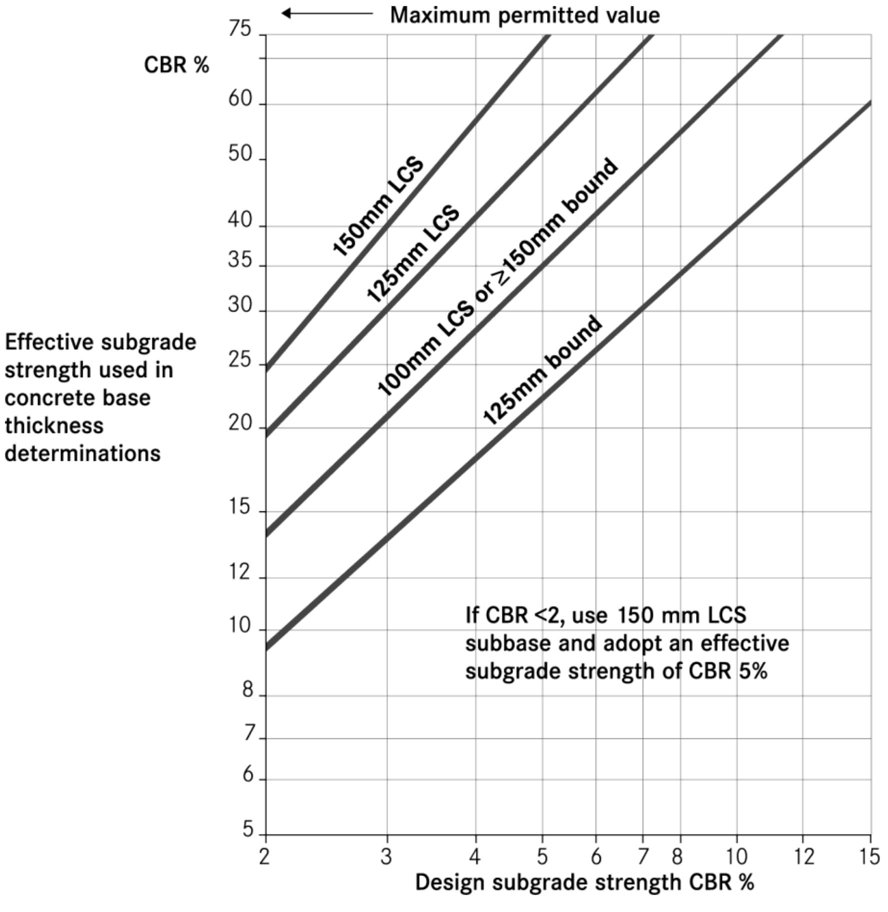
The recommended minimum subbase thickness and type for various levels of traffic loadings are described in Section 9.2.2.

For jointed undowelled bases a lean-mix concrete subbase is recommended with a minimum of 150 mm of improved subgrade material or unbound granular material below the subbase layer.

For guidance regarding the strength of subbases for use in lightly trafficked roads and roundabouts refer to Section 12.6.4 and Roads and Maritime Services (Roads and Maritime) (2019a).

Research work and experience has identified that there is an increase in effective subgrade strength with the use of bound and lean-mix concrete subbase. Figure 9.1 may be used to determine the increase in strength for use in the base thickness design.

Figure 9.1: Effective increase in subgrade strength due to the provision of bound or lean-mix concrete subbase (LCS) course (to be used for rigid pavement thickness design)



Note: For design purposes a 220 mm thick layer of no fines concrete is considered to provide equivalent support to 150 mm lean-mix subbase for applications in mined tunnels. 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 m 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 57 provides the model that must be used to determine this equivalent subgrade design strength (CBR_E) based on the strength of the supporting soil depth (Japan Road Association 1989).

$$CBR_E = \left[\frac{\sum_i (h_i \, CBR_i^{0.333})}{\sum_i h_i}\right]^3 \quad 57$$

where

CBR_E = equivalent subgrade design strength (%)

CBR_i = the CBR value of layer i (%)

h_i = the thickness of layer i (m)

$\sum h_i$ = taken to a depth of 1.0 m

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 15%.

9.3.3 Base concrete strength

The determination of concrete strength is discussed in Section 6.8. 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 rigid pavements with design traffic of 10^6 HVAG or more, is 4.5 MPa at 28 days, except for SFCP where it is 5.5 MPa.

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 (Federal Highway Administration (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. Through the provision of edge support, 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 connected with a tied corrugated 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 an 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 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
- they are effectively tied to the pavement and the joint has a corrugated face for load transfer
- the bottom of the kerb and channel or safety barrier must be at the same level as the bottom of the pavement base.

Slipformed and fixed-form kerbs can satisfy these criteria, but extruded concrete 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 (L_{SF}). The load safety factors used in the equations in Section 9.4.2 are derived from the values in Table 9.2 according to the desired project reliability (Section 2.3.1) for a specified pavement type.

Table 9.2: Load safety factors (L_{SF}) 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 and 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
- 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 that the average stresses are similar for these pavement types (Packard 1984).

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 design base thickness is the greater of the calculated base thickness (including rounding) and the minimum base thickness from Table 9.7 for different base types and traffic volumes.

It is recommended that consideration be given to additional thickness to the design base thickness to account for factors such as:

- the limitations of paving equipment and measuring systems
- possible losses associated with future surface retexturing treatments such as diamond grinding and/or grooving
- future removal and replacement of an asphalt surfacing (when present) which may result in the reduction of base thickness.

The specified base thickness for construction is the sum of the design base thickness and the above additional thickness where required for the project.

9.4.2 Base thickness design procedure

A trial design 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% and a total erosion damage less than or equal to 100%.

The steps in the thickness design are detailed in Table 9.3.

Examples of the use of the base thickness design procedure for PCP are contained in Appendix M.

The allowable axle load repetitions may be determined from the following equations (Austroads 2008):

Fatigue distress mode

Allowable load repetitions (N_f) for a given axle load are (Equation 58 and Equation 59).

$$\log_{10} N_f = \left(\frac{0.9719 - S_r}{0.0828}\right) \quad \text{when } S_r > 0.55 \qquad (58)$$

$$N_f = \left(\frac{4.258}{S_r - 0.4325}\right)^{3.268} \quad \text{when } 0.45 \le S_r \le 0.55 \qquad (59)$$

where

S_r = $\dfrac{S_e}{0.944 f_{cf}}\left(\dfrac{PL_{SF}}{4.45F_1}\right)^{0.94}$

S_e = equivalent concrete stress (MPa)

f_{cf} = design characteristic flexural strength at 28 days (MPa)

P = axle group load (kN)

L_{SF} = load safety factor

F_1 = 9 for single axle with single tyres (referred to as SAST axle group)

= 18 for single axle with dual tyres (referred to as SADT axle group)

= 18 for tandem axle with single tyres (referred to as TAST axle group)

= 36 for tandem axle with dual tyres (referred to as TADT axle group)

= 54 for triaxle with dual tyres (referred to as TRDT axle group)

= 72 for quad axle with dual tyres (referred to as QADT axle group)

N_f is infinite or commonly referred to as unlimited when S_r is less than 0.45.

The equivalent stress (S_e) and erosion factor (F_3) is determined from Equation 60 using the coefficients a to j in Table 9.4 to Table 9.6.

$$S_e \text{ or } F_3 = a + \frac{b}{D} + c.\ln(E_f) + \frac{d}{D^2} + e.[\ln(E_f)]^2 + f.\frac{\ln(E_f)}{D} + \frac{g}{D^3} + h.[\ln(E_f)]^3 + i.\frac{[\ln(E_f)]^2}{D} + j.\frac{\ln(E_f)}{D^2} \qquad (60)$$

where a, b, c, d, e, f, g, h, i, j are coefficients in Table 9.4 to Table 9.6

D = thickness of concrete base (mm)

E_f = effective subgrade design CBR (%)

Erosion distress mode

Allowable load repetitions (N_e) for a given axle load is (Equation 61).

$$\log_{10}(F_2 N_e) = 14.524 - 6.777\left[\max\left(0, \left(\frac{PL_{SF}}{4.45F_4}\right)^2 . \frac{10^{F_3}}{41.35} - 9.0\right)\right]^{0.103} \quad 61$$

where P and LSF as for Equations 58 and 59

F_2 = adjustment for slab edge effects

= 0.06 for base with no concrete shoulder

= 0.94 for base with concrete shoulder

F_3 = erosion factor

F_4 = load adjustment for erosion due to axle group

= 9 for single axle with single tyres (referred to as SAST axle group)

= 18 for single axle with dual tyres (referred to as SADT axle group)

= 18 for tandem axle with single tyres (referred to as TAST axle group)

= 36 for tandem axle with dual tyres (referred to as TADT axle group)

= 54 for triaxle with dual tyres (referred to as TRDT axle group)

= 54 for quad axle with dual tyres (referred to as QADT axle group)

The erosion factor (F_3) is determined from Equation 60 using the coefficients a to j in Table 9.5 and Table 9.6.

There are no limits set for the axle load input and load safety factors used in Equation 58 and Equation 60. However, caution is advised when using allowable loadings calculated with values of $(4.5 \times PL_{SF}/F_1)$ or $(4.5 \times PL_{SF}/F_4)$ exceeding 65 kN.

Table 9.3: Design procedure for base thickness

Step	Activity	Section
1	Select a rigid pavement type, either jointed undowelled, jointed dowelled or continuously reinforced concrete base.	9.2.1
2	Decide whether tied or integrally cast concrete shoulders are to be provided.	9.3.5
3	Using the subgrade design 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 subgrade design CBR limit in Chapter 5.	9.2.2
4	Using the subgrade design 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 fcf.	6.8.4 9.3.3
6	Select the desired project reliability and hence the load safety factor.	2.3.1 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	Calculate the expected load repetitions of each axle group load of each axle group type.	7.7
9	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 terms of fatigue from Equation 58 and Equation 59.	9.4.2
10	Calculate the ratio of the expected fatigue repetitions (Step 8) to the allowable repetitions (Step 9). Multiply by 100 to determine the percentage fatigue.	
11	Determine from Equation 61 the allowable number of repetitions for erosion for the highest axle load for the SAST axle group.	9.4.2
12	Calculate the ratio of the expected erosion repetitions (Step 8) to the allowable repetitions (Step 11). Multiply by 100 to determine the percentage erosion damage.	
13	Repeat steps 9 to 12 for each axle group load up to a load level where the allowable load repetitions exceed 1011, at which point further load repetitions are not deemed to contribute to pavement distress.	
14	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.	
15	Repeat steps 9 to 14 for each axle group type (i.e. SADT, TAST, TADT, TRDT and QADT).	
16	Sum the total fatigue and total erosion damage for all axle group types.	
17	Steps 9 to 16 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 design base thickness.	
18	Obtain the minimum base thickness requirement from Table 9.7.	9.4.3
19	Calculate the design base thickness and consider the application of additional thickness tolerance as described in Section 9.4.1.	9.4.1

Table 9.4: Coefficients for prediction of equivalent stresses

	Without concrete shoulders				Concrete shoulders			
	Axle group type				Axle group type			
Coefficient	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	3 01.5	258.6
c	–0.2396	–0.6663	–0.3742	–0.1340	0.0899	–0.4684	–0.1846	0.0080
d	26 969	44 405	–38	–10 007	35 774	28 661	4 418	1 408
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	–352 174	–942 585	681 381	1 215 750	–861 548	–686 510	280 297	488 079
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	–1 709	–4 082	2 003	4 400	–4 009	–2 717	1 768	2 564

Table 9.5: Coefficients for prediction of erosion factors for undowelled bases

	Without concrete shoulders				Concrete shoulders			
	Axle group type				Axle group type			
Coefficient	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	–42 419	–43 035	–35 827	–36 194	–44 908	–44 900	–32 024	–15 007
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	1 547 570	1 586 100	1 291 870	1 315 330	1 676 710	1 654 590	1 150 280	518 916
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	–1 384	–1 344	50	–625	–913	–765	469	–599

Table 9.6: Coefficients for prediction of erosion factors for dowelled or CRCP bases

	Without concrete shoulders				Concrete shoulders			
	Axle group type				Axle group type			
Coefficient	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	–58 342	–58 371	–39 430	–25 576	–50 996	–52 519	–47 500	–28 901
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	2 139 330	2 131 390	1 437 580	923 081	1 874 370	1 949 350	1 719 950	1 085 800
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	–1 327	–374	–326	1 675	758

9.4.3 Minimum base thickness

Irrespective of the calculated base thicknesses determined in accordance with this procedure, the minimum thickness of concrete base to be trafficked by heavy vehicles is noted in Table 9.7. The minimum values for design traffic loading exceeding or equal to 10^7 HVAG is to account for environmental factors such as warping and curling of slabs, which are not otherwise considered in the thickness design process.

Table 9.7: Minimum base thickness

Pavement type (base)	Design traffic		
	$1 \times 10^6 \leq$ HVAG $< 1 \times 10^7$	$1 \times 10^7 \leq$ HVAG $< 5 \times 10^7$	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

9.4.4 Example of the use of the design procedure

An example thickness design of a rigid pavement is given in Appendix L.

9.4.5 Example design charts

Example design charts for rigid pavements are presented in Figure 9.2 and Figure 9.3.

These charts are based on:

- an example traffic load distribution (Appendix F)
- the provision of concrete shoulders (Section 9.3.5)
- a concrete base design flexural strength of 4.5 MPa
- load safety factors appropriate for desired project reliabilities of:
  80% (LSF = 1.05 for dowel jointed and CRCP, LSF = 1.15 for PCP) and
  95% (LSF = 1.2 for dowel jointed and 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.

Table 9.8: Example design charts for various traffic and pavement configurations

Base type	Effective subgrade strength (CBR)	Load safety factor	Chart
PCP	35% and 75%	1.15 and 1.3	Figure 9.2
Jointed reinforced and dowelled and CRCP	35% and 75%	1.05 and 1.2	Figure 9.3

Figure 9.2: Example design chart for plain concrete pavements without dowelled joints for combinations of two effective subgrade strengths and two load safety factors

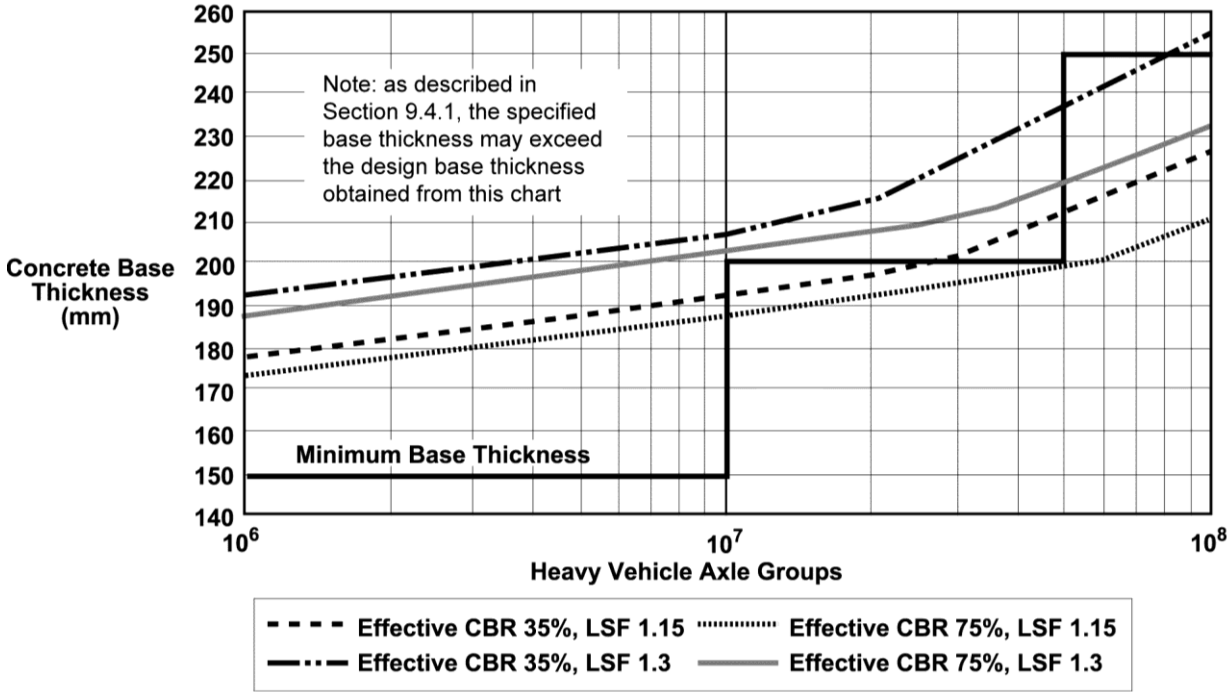
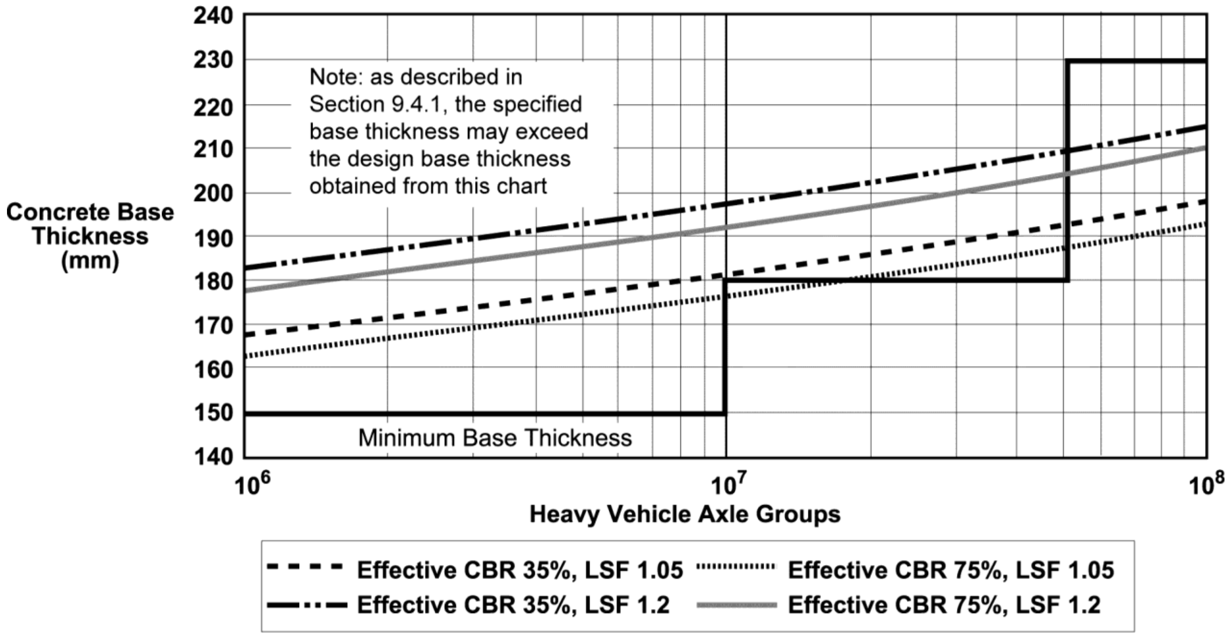


Figure 9.3: Example design chart for CRCP or pavements with dowelled joints for combinations of two effective subgrade strengths and two load safety factors



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 250 N 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, both horizontally and vertically, otherwise joint locking is likely to occur.

Table 9.9: Minimum dowel bar diameters for rigid pavements

Design base thickness (D) (mm)	Dowel diameter (mm)
150 < D ≤ 175	24
175 < D ≤ 200	28
200 < D ≤ 260	32
D > 260	36

Source: Roads and Maritime (2016).

9.4.7 Provision of tiebars

Tiebars 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%.

Tiebars 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 Equation 62) and is influenced by parameters such as base thickness, interlayer friction, and distance to the nearest free edge of pavement (known as the relief edge distance). For constructability, 16 mm bars may be used to increase spacing where the relief edge distance would result in impractical spacing of the standard 12 mm bars. The designer should take care when substituting bar sizes to ensure that using 16 mm diameter bars does not prevent hinging at joints.

In joints between CRCP and jointed bases (such as at shoulders and ramp junctions), it is desirable to cluster tiebars 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 (refer to Table 9.10).

It is beyond the scope of this Part to provide a full design method for tiebars, and designers are referred to Roads and Maritime (2015a, 2015b, 2016, 2019b) pavement standard drawings and *Part 4C* of the Guide (Austroads 2017a) for further information.

9.5 Reinforcement design procedures

9.5.1 General

In JRCP the purpose of reinforcing steel 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 CRCP, the longitudinal steel is designed to initiate the desired transverse cracking pattern and then hold those cracks tight.

In JRCP the amount of steel is governed by the spacing of contraction joints. In CRCP, 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.

Materials for steel reinforcement of rigid pavements are described in *Part 4C* of the Guide (Austroads 2017a).

9.5.2 Reinforcement in plain concrete pavements

In plain (jointed unreinforced) concrete pavements, reinforcement is sometimes necessary to control cracking. Slabs are reinforced if 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
- slabs containing pits or structures.

Designers should be aware that the provision of reinforcement cannot be expected to prevent cracking but only to hold cracks tight if they occur. If suitable reinforcement is provided in a slab of acceptable proportions, cracks should not require any treatment during the pavement's design life.

Guidance on the detailing of reinforcement is provided in Roads and Maritime standard rigid pavement drawings (Roads and Maritime 2014, 2015a, 2015b, 2016).

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 Equation 62.

$$A_s = \frac{\mu L \rho g D}{1000 f_s} \qquad 62$$

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 (mm), including any asphalt surfacing
L	=	relief edge distance to untied joints or edges of the base (m)
ρ	=	mass per unit volume of the base (kg/m^3)
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 of 8 m provides an optimum balance of joint performance, cost, 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. However, limits apply to the ratio of fibre length to the minimum dimension of test specimens (such as test cylinders and beams). Typically, fibres are added to the concrete at a rate of approximately 55 to 75 kg/m^3.

Table 9.10: Estimated values for coefficient of friction

Subbase type	Base type	Recommended treatments		Estimated friction coefficient[2, 3]
		Subbase curing treatment	Debonding treatment	
Lean-mix concrete	PCP and CRCP	Wax emulsion	Bitumen sprayed seal with 5–7 mm aggregate	1.5
	JRCP and CRCP	Wax emulsion	(i) Bitumen sprayed seal with 5–7 mm aggregate, or (ii) bitumen emulsion	(i) 1.5 (ii) 2.0
	CRCP and SFCP	Wax emulsion	Wax emulsion	1.7
RCC and CTCR[1]	All	(i) Hydrocarbon resin (ii) Bitumen emulsion	Bitumen sprayed seal with 5–7 mm aggregate	2.5
Dense graded asphalt	All	Note[4]		2.5–3.0[4]

1 *RCC = roller compacted concrete CTCR = cemented crushed rock.*

2 *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 (i.e. high) friction values have been adopted.*

3 *Where more than one type of curing and debonding treatment is acceptable for a particular base type, designers should consider the durability of the curing/debonding treatment during pavement construction when specifying treatments.*

4 *Friction values for asphalt could vary widely depending on factors such as age, modulus and surface texture. Aged, stiff asphalt with an open-textured surface could yield a high friction level. By contrast, new (and relatively flexible) asphalt is likely to have a lower effective friction level.*

9.5.4 Reinforcement in continuously reinforced concrete pavements

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 Equation 63.

Equation 63 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.

$$p = \frac{(f_t'/f_b')d_b(\varepsilon_s + \varepsilon_t)}{2W} \qquad 63$$

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

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 microstrain for a concrete with a laboratory shrinkage not exceeding 450 microstrain at 21 days when tested in accordance with AS 1012.13 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 microstrain 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

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

For deformed bars, Equation 63 may be simplified as Equation 64.

$$p = \frac{0.25 d_b (\varepsilon_s + \varepsilon_t)}{W} \quad (64)$$

To ensure against yielding of the steel, the steel reinforcement ratio should exceed the critical value given by Equation 65.

$$p_{crit} = \frac{f_{ct}(1.3 - 0.2\mu)}{f_{sy} - m f_{ct}} \quad (65)$$

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 concrete flexural strength (f_{cf}) may be assumed

μ = coefficient of friction between concrete base and 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 65 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 0.67%.

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. Experience with continuously reinforced pavements indicates that the optimum crack spacing is between 0.5 and 2.5 m.

If the spacing of the cracks is too wide, the cracks 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 Equation 66.

$$L_{cr} = \frac{f_{ct}^{\,2}}{mp^2 u f_b[(\varepsilon_s + \varepsilon_t)E_c - f_{ct}]} \quad (66)$$

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 the 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 $2f_{ct}$
ε_s	=	estimated shrinkage strain – the shrinkage strain may be considered to be in the range 200 to 300 microstrain for a concrete with a laboratory shrinkage not exceeding 450 microstrain at 21 days when tested in accordance with AS 1012.13 (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 microstrain 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 microstrain may be assumed
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. Hence the use of deformed bars is preferred.

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 62 from joint or section under design to the nearest relief edge, except that a maximum spacing of 750 mm is typically adopted to prevent sagging in the longitudinal steel.

Glass fibre reinforcement at traffic detection loops

The use of glass fibre reinforcement polymer (GFRP) (in accordance with AS 5204-2023) as reinforcement to create a steel-free window around traffic loop locations has the following benefits:

- When exposed to moisture and air, GFRP does not oxidise and expand in volume causing concrete to crack. Corrosion of steel is usually mitigated in steel concrete by providing sufficient concrete cover; however, traffic loops installed by saw cuts increase the risk of crack initiation in concrete and moisture ingress.
- The accuracy of speed and traffic class counts by the traffic detection loops improves as the induced charge of a passing vehicle is not shared with the steel reinforcement especially with high bed trucks and other vehicles.

Transport for NSW (TfNSW) Technical Guide for *Design of Continuously Reinforced Concrete Pavement using Glass Fibre Reinforced Polymer (GFRP) Bars at Traffic Loops Locations* (2021) provides a design methodology for glass fibre reinforcement in CRCP.

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:

- Plain concrete pavements and 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.
- 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 CRCP.

  Further details on anchors can be found in sources such as the Roads and Maritime *standard rigid pavement drawings* (Roads and Maritime 2015a, 2015b, 2016).

9.7 Joint types and design

9.7.1 Introduction

There are two basic types of stresses which may be present in a rigid (concrete) road pavement:

- stresses induced by externally applied loads resulting from traffic using the pavement
- stresses within the pavement resulting from contraction and expansion movements, and differential moisture and temperature changes within the depth of the pavement.

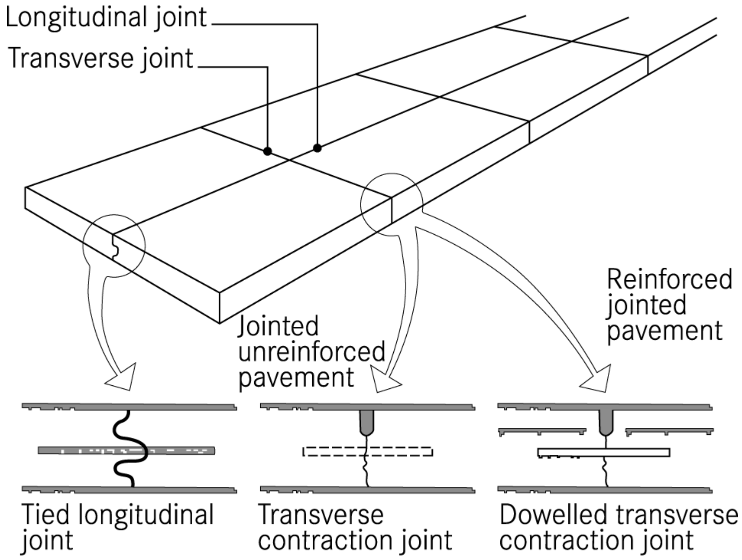
Whereas thickness design is dominated by distress caused by externally applied traffic loading, the location and design of joints are dominated by the need to control stresses and strains induced by environmental changes within the pavement.

The four most common joints are:

- transverse contraction joints
- transverse construction joints
- expansion and isolation joints
- longitudinal (hinge) joints.

Figure 9.4 illustrates various joint types.

Figure 9.4: Typical joints in concrete bases



Source: Adapted from Cement and Concrete Association of Australia (1982).

The Cement and Concrete Association of Australia Technical Note 47 (CCAA 1982) was a valuable source of information for the text in this section. More detailed guidance can be found in Roads and Maritime *standard rigid pavement* drawings (Roads and Maritime 2015a, 2015b, 2016). For concrete roundabout pavements, guidance is available in Roads and Maritime *Concrete Roundabout Pavements* (Roads and Maritime 2019a).

9.7.2 Transverse contraction joints

The major environmental movements in a rigid road pavement occur in the direction of the traffic, rather than across the carriageway. This is simply due to the fact that the length of a road pavement is very many times its width. Consequently, the design and construction of transverse joints is more critical than that of other joints in road pavements.

The principal requirements of contraction joints in a jointed pavement whether unreinforced or reinforced are:

- to ensure that a crack is induced in the pavement at a predetermined location to control transverse cracking (and the crack so formed needs to be effectively sealed against the ingress of solids and water)
- to permit the joint to open and close
- to transfer loads across the joints.

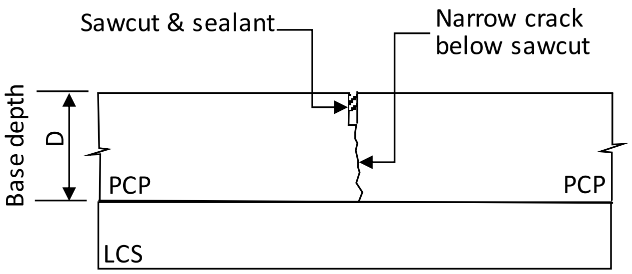
To control cracking, the appropriate spacing between contraction joints in jointed unreinforced pavements depends on shrinkage properties of the concrete, subbase and subgrade friction characteristics and the slab thickness. A joint spacing of 4.2 m is commonly used.

For jointed reinforced concrete pavements with dowelled joints (JRCP), the spacing is typically increased to 8 to 10 m. Intermediate cracks are still expected to form at about 4 to 6 m centres (often within days of paving), but these are held tight by the reinforcing mesh.

The most common method of forming the contraction joint is to sawcut to ¼ slab depth.

Load transfer across the joint is provided by dowels or, in the case of undowelled PCP, by aggregate interlock across the rough crack faces (Figure 9.5). Dowels must not 'lock' the joint, otherwise an uncontrolled crack may occur elsewhere in the slab. Therefore dowels must be coated with a bond breaker on one side and must be aligned parallel to the longitudinal direction of the pavement and the surface of the pavement to within close tolerances.

Figure 9.5: Example of PCP sawn transverse contraction joint



Source: Adapted from Roads and Maritime (2016).

9.7.3 Transverse construction joints

Transverse construction joints are required for planned interruptions such as at the end of each day's operations, at block outs for bridges and intersections or for unexpected delays when the suspension of operations is likely to create a joint.

Plain concrete pavement

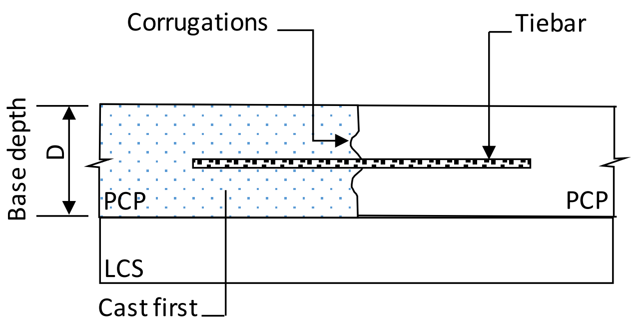
Transverse construction joints can be formed in either of the following ways:

- Formed and tied joint

  In a formed and tied joint the intention is that the joint becomes an integral part of the slab. The corrugation provides load transfer, and deformed tiebars are used to hold the joint tightly closed to limit the incidence of sympathetic cracking in adjacent lanes (Figure 9.6). Tiebars are commonly provided at 300 to 500 mm centres.

  It is preferable that the joint be placed normal to the longitudinal axis of the pavement, within the central third of the slab and not less than 1.5 m from a transverse contraction joint. In multilane paving the alignment may have to be altered to cater for skew in the adjacent contraction joints.

Figure 9.6: Example of PCP formed and tied transverse construction joint



Source: Adapted from Roads and Maritime (2016).

- Dowelled joints

  A construction joint can be formed at the location of a planned contraction joint in slabs with a minimum thickness of 200 mm, but it must be free to contract and therefore must not be tied. Load transfer by aggregate interlock would be minimal in an untied formed joint (either scabbled or corrugated) compared to that in an induced joint, and therefore dowels must be provided to carry out this function. Crack induction is not necessary, but positive joint debonding is required for horizontal movement together with subsequent widening and sealing of the formed joint.

  Because of the difficulties introduced by the use of dowels, especially in skewed joints, this method is unlikely to provide any benefit over the tied option and is therefore only recommended in unusual applications.

Jointed reinforced concrete pavements

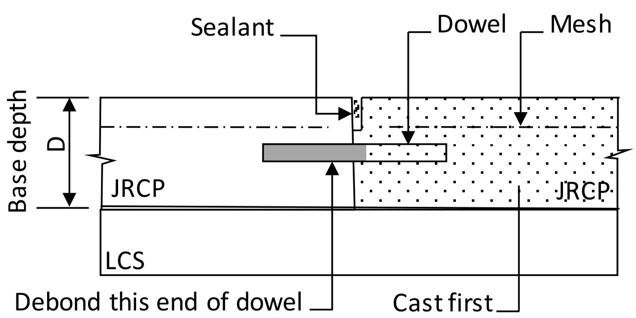
In JRCP, there are two options for the design of transverse construction joints:

- to form a dowelled joint at the location of a planned induced joint (Figure 9.7)
- to locate a tied joint within the standard slab length not less than 1.5 m away from the contraction joint.

  Construction of JRCP is necessarily geared towards the provision of dowels and therefore the first option is more feasible than in PCP. However, it is difficult to secure dowels in an accurate alignment through a formed joint and so the second option offers the benefit that tiebars do not need to be as accurately aligned.

  It is emphasised that contraction joints should not be interchanged with tied joints. All moving joints (i.e. contractions, isolations and expansions) should be continuous between free pavement edges.

Figure 9.7: Example of JRCP formed and tied transverse construction joint at a planned joint



Source: Adapted from Roads and Maritime (2015b).

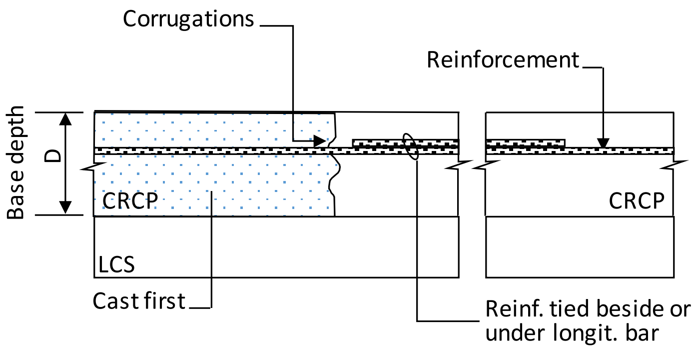
- Continuously reinforced concrete pavements

  In CRCP the longitudinal steel must continue through each construction joint. Supplementary steel is also provided to control the cracking pattern within the section closest to the previous pour. A corrugated joint, formed by a special header board, is provided to ensure effective load transfer (Figure 9.8).

Continuously reinforced concrete pavements

In CRCP, the longitudinal steel must continue through each construction joint. Supplementary longitudinal steel is also provided to control the cracking pattern within the vicinity of the construction joint. A corrugated joint, formed by a special header board, is provided to ensure effective load transfer (Figure 9.8).

Figure 9.8: Example of CRCP formed and tied transverse construction joint



Source: Adapted from Roads and Maritime (2015b).

9.7.4 Expansion and isolation joints

Concrete is far more capable of resisting compressive stresses than tensile stresses. It has been found that compressive failures or blowups are extremely rare in Australian pavements provided joint sealants are properly installed and maintained. Generally, regularly spaced expansion joints are not required within pavements except at fixed objects and asymmetrical intersections. At these discontinuities in the pavement some allowance has to be made for potentially high compressive forces by providing expansion (dowelled) or isolation (undowelled) joints.

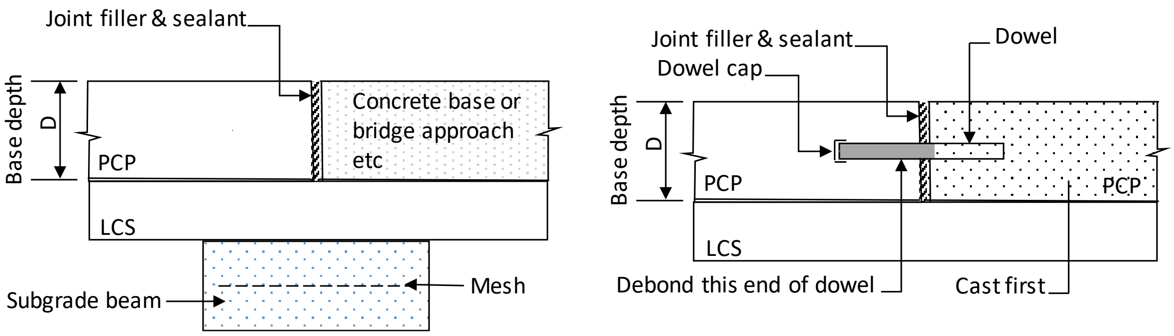
The term expansion joint is used in this Part only for such joints that are dowelled. For clarity, like joints which are undowelled are termed isolation joints. Their behaviour is different in that an expansion joint allows movement only along the axis of the dowels, whereas movement is possible at an isolation joint in all three axes.

The most common application of expansion joints is at bridge abutments. Isolation joints are required at intersections or junctions to prevent the development of stresses which could otherwise develop as a result of conflicting movements in the intersecting pavements.

Due to the absence of dowels or aggregate interlock, isolation joints theoretically require base or subbase thickening due to higher edge loadings. However, these thickenings cause significant construction difficulties and can induce unacceptable stresses by restraining slab contraction. For this reason, subgrade beams are commonly provided to reduce the stress on the subgrade and provide more support for the base at the isolation joint.

Examples of isolation and expansion joint details are given in Figure 9.9.

Figure 9.9: Example of PCP isolation joint with subgrade beam (left) and PCP expansion joint with dowel (right)



Source: Adapted from Roads and Maritime (2016).

9.7.5 Longitudinal joints

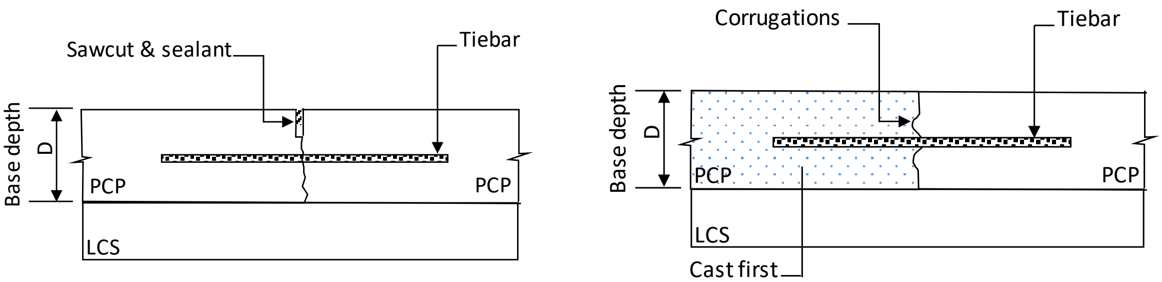
The purpose of longitudinal joints is to control longitudinal cracking by enabling hinge release of curling stresses. The joints are tied wherever possible to keep them firmly closed. In general, formed joints are left unsealed but sawn joints are sealed.

Longitudinal joints may be either (Figure 9.10):

- formed construction joints with corrugated faces tied together with tiebars
- induced for multi-lane placement, commonly by saw cutting to about one-third the slab depth.

Tiebars are provided purely to hold the faces together, and load transfer is provided by aggregate interlock (in sawn joints) or by corrugations (in formed joints).

Figure 9.10: Sawn and sealed (left) and formed PCP (right) longitudinal joint types



Source: Adapted from Roads and Maritime (2016).

Longitudinal joints should be provided to limit slab widths to about 4.3 m and should be tied up to width limits as specified in Section 9.2.1. Beyond these limits, the reinforcement design is likely to become excessive, in which case an untied joint should be provided at a suitable location in the pavement.

Where possible, untied joints should be located outside heavy vehicle wheelpaths, such as under a median or within a gore area, in which case it can be either an untied butt joint (as long as the transverse joints do not mismatch) or an isolation joint without a subgrade beam. If the untied joint falls near or within a heavy vehicle wheelpath, the pavement effectively becomes 'without shoulder' for thickness design purposes. As localised thickening is impractical (for reasons given above) an isolation joint with a subgrade beam is recommended.

9.7.6 Joint design

The objectives of joint design are to develop a jointing system which will control transverse and longitudinal cracking, and provide adequate load transfer across joints. Joints should be sealed (with exceptions as noted above) to minimise the intrusion of water and incompressible solids.

A joint layout should be designed which is compatible with the road geometry, the length of slab, the type of joint, and the construction method. Advice on joint design and layout can be found in Roads and Maritime *standard rigid pavement drawings* (RMS 2015a, 2015b, 2016).

10. Economic Comparison of Designs

10.1 General

In comparing various alternative pavement types and configurations, cost is a prime consideration. This Part 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, some details are available (e.g. Bennett and Moffatt 1995, Porter and Tinni 1993). 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.

In addition to the above, 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
- 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.

Alongside the economic considerations, designers are increasingly required to consider sustainability outcomes to enable the achievement of national (and state/territory) greenhouse gas emissions targets, of which transport plays a major role. This includes reducing emissions through not only decarbonising transport operations (the vehicles) but also through the design, construction and operation of transport infrastructure.

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 (Equation 67).

$$PWOC = C + \sum_i M_i (1+r)^{-x_i} - S(1+r)^{-z} \qquad 67$$

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
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
- 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 using 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 environmental 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, and the scale of the project and material standards. These may be assessed from experience with other 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 full-depth asphalt or cemented pavements, 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 placement 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 some details available (e.g. Bennett and Moffatt 1995, Porter and Tinni 1993).

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 distress modes, such as fatigue cracking of cemented material and lean-mix concrete subbases before the end of the design period, 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 in situ
- 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 next project, then the salvage value is equal to the cost in current dollars (say year 2025) for construction, in say, year 2045 of a pavement to subbase level (less any costs for tidying up the works, scarification, compaction, drainage renovation etc.), discounted to the evaluation year (2025).

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 next project, then the salvage value is equal to the cost in current dollars (say year 2025) for construction, in say, year 2045 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 2025).

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 analysis 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 need 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, Infrastructure Australia (2016) uses a discount rate of 7%. This rate is expressed in real terms, i.e. it excludes inflation. However, other agencies may require the use of other rates for other purposes. In most cases, it is desirable to carry out a sensitivity analysis with discount rates of 4% and 10%, which is also a requirement of Infrastructure Australia (2016).

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 Austroads (2004) and Thoresen and Roper (1996).

10.9 Surfacing service lives

Guidance on the range of expected service lives of surfacing can be found in *Part 3: Pavement Surfacings* of the Guide (Austroads 2025b). 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
- 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 *Part 3*: *Pavement Surfacings* of the Guide (Austroads 2025b) for each of the competing alternatives.

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.
- There is a lack of understanding of pavement distress mechanisms and terminal pavement condition.
- There is a lack of a perceived need to monitor pavement performance on a systematic basis.

This issue has been addressed with the establishment of an Austroads-sponsored project (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 cost 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
- 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 2000a, Youdale 2007, Austroads 2017b).

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
- 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 (2000b) 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.

12. Design of Lightly Trafficked Pavements

12.1 General

This chapter provides guidance on the structural design of new bituminous surfaced and rigid pavements for situations where design traffic is below that catered for in Sections 2 to 11. Such traffic loadings are common for roads and streets under the control of local government agencies, in remote areas of some road agencies or bikeways.

This chapter is applicable to the design of flexible pavements with design traffic in the range 10^3 to 10^5 ESA, and rigid pavements in the range 10^3 to 10^6 HVAG.

In the development of the design procedures for light traffic, emphasis has been placed on the following aspects:

- the potentially greater effect of environment
- the potentially higher variation in subgrade type and moisture conditions
- the lower traffic speeds in urban locations
- the potential for significant pavement damage resulting from a small number of passages of heavily overloaded vehicles.

The design procedures presented in this section, while in general reflecting an extension of the relevant procedures for moderate-to-heavily trafficked roads in this Part, differ to the extent necessary to incorporate these aspects. Note that due to these aspects, the distress modes of lightly trafficked roads commonly differ from moderate-to-heavily trafficked roads. In particular, asphalt fatigue does not appear to be a distress mode for lightly trafficked flexible pavements, nor erosion a distress mode for rigid pavements.

For the design of granular pavements with thin bituminous surfacings, the adoption of an empirically based procedure (Figure 12.2) is consistent with the approach adopted for higher traffic loadings in Figure 8.4.

12.2 Pavement design systems

12.2.1 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 2.3.1.

A trial pavement configuration may often be selected by judgement or by using the example design charts in this section.

12.3 Construction and maintenance considerations

The design procedures presented in this Part assume that appropriate standards of construction and maintenance practice will be adopted. Such standards are well-documented in specifications of Austroads member agencies and other Austroads publications.

The following text should be read in conjunction with Chapter 3.

12.3.1 Extent and type of drainage

Water on the pavement surface can cause problems for vehicles including hydroplaning, reduction in visibility due to spray, reduced availability of the pavement for vehicles and loss of stability for vehicles in floodways.

Pedestrians and cyclists require that sprays and splashes from passing vehicles are minimised, appropriate pit covers are used so as to not present a hazard for cycle wheels, velocities in gutters and channels are not excessive, and that there are safe inlets to underground or outlet facilities.

Factors to consider in the design of surface drainage for lightly trafficked roads are discussed by Giummarra (2005).

12.3.2 Use of boxed construction

It is often a desire of urban planners to reduce the visual impact of low traffic urban streets by requiring the road profile to closely follow the natural terrain. As a result, pavements are constructed at or close to existing surface levels and, hence, the use of boxed construction is more prevalent than on higher speed, more heavily trafficked roads.

Where boxed construction is used, consideration needs to be given to maintaining vehicular access to adjacent properties during construction.

With boxed construction the provision of adequate drainage, during construction and during service life, is particularly important. The provision of adequate crossfalls on the surfaces of the subgrade and subsequent pavement layers will facilitate run-off of both surface water during construction and water subsequently trapped in the pavement, hence minimising the potential for softening of the materials. Refer also to Section 12.4.2.

12.3.3 Availability of equipment

For the construction of lightly trafficked roads, there is likely to be a greater variation in equipment and supervisory expertise available than in the case of the construction of heavily trafficked roads.

If the road is at the higher end of the lightly trafficked road spectrum (e.g. collector roads) or part of a major residential development, equipment and supervisory expertise comparable with that used on the construction of major road projects, could be expected. For a lower hierarchical level road or less extensive development, the available equipment could be expected to be less numerous and smaller.

In the latter case, the designer needs to give consideration to the effects of reduced construction standards (e.g. greater surface level or thickness variations, lower compaction density) on the selection of pavement type and composition.

It should also be noted that in some urban areas the proximity of houses and other structures may limit the use of vibratory equipment, particularly high amplitude compaction equipment, and the physical size of some larger equipment may be impractical. The presence of overhanging trees may also limit the use of large equipment.

12.3.4 Use of staged construction

In urban areas a form of staged construction is often used on local access roads. In this case the construction of the final wearing surface is delayed until the heavy vehicle traffic supplying materials for residential construction has reduced significantly. During this period an initial seal (formerly called a primerseal) or two-layer asphalt surfacing may be used.

The advantage of this form of construction is that any loss of surface shape resulting from the construction traffic can be remedied prior to the final wearing surface being placed. The disadvantage is that kerb and channel normally constructed with the original pavement will be above the sealed pavement surface, thereby requiring temporary drainage measures to remove water from the pavement surface.

12.3.5 Environmental and safety constraints

Environmental conditions can have a more significant impact on the development of distress in lightly trafficked roads than moderate-to-heavily trafficked roads. Designers need to consider:

- the potential of moisture ingress to cause weakening of subgrades and to cause volume changes in expansive soils
- the potential of moisture ingress to cause weakening of unbound materials
- the influence of tree roots close to the pavement edge on the moisture regime
- the effect of prolonged exposure to the atmosphere and solar radiation on bituminous binders (Section 12.4.3).

12.3.6 Social considerations

The pavement surfacing selected for low traffic roads in urban areas should harmonise with the immediately surrounding area. Aspects such as low noise, compatibility with use by pedestrian and bicycle traffic and less stringent skid resistance requirements have an impact on this selection process.

12.3.7 Maintenance strategy

Once constructed, a local road network, particularly in urban areas, is unlikely to change significantly in alignment or level for many years – perhaps in excess of 100 years. Therefore it is important, when determining the appropriate pavement structure, that designers consider a future maintenance and rehabilitation strategy. Such consideration needs to take account of social constraints (e.g. impact on local residents in terms of noise and restricted property access) and physical constraints (e.g. the fixed levels or kerbing) on future work. Pavement levels should allow for drainage of crossovers and footpath areas. These constraints may largely determine a practical strategy. Maintenance and rehabilitation options for lightly trafficked roads are presented by Giummarra (2005) and Parts 5 and 7 of the Guide.

12.4 Environment

12.4.1 General

For roads and streets which are subject to low traffic loading, pavement distress is attributable to environmental effects to a greater extent than for higher traffic loading roads. However, pavement structure is still important for low traffic volumes as an inadequately designed and constructed pavement may lead to premature distress.

12.4.2 Moisture

The factors that influence the moisture regime within and/or beneath a pavement are detailed in Section 4.2. These factors may be more significant in lower traffic volume situations where wheel loadings often occur closer to the seal edge (or kerb) – the critical area of the pavement in respect of moisture effects. The situation may be further exacerbated in residential areas through excessive watering by residents of gardens, nature strips and irrigated landscaping along medians and verges.

12.4.3 Temperature

For lightly trafficked pavements the oxidation of bitumen can cause environmental deterioration to become significant.

Bitumen oxidises on exposure to air, becoming brittle. This process is accelerated by high temperatures and ultra-violet radiation (from sunlight). Brittleness leads to cracking of surface seals and ravelling of both surface seals and asphaltic surfaces. Trafficking has a beneficial effect by closing micro-cracks in bitumen films and surface voids in asphalt, hence constraining oxygen flow. In low traffic volume situations, especially in non-trafficked zones such as parking lanes, this beneficial effect is reduced, leading to earlier onset of the above-mentioned distresses. To counter this, the use of asphalt mixes which have lower air voids and higher bitumen contents may provide some benefit. Guidance may be found in Section 6.6 and Oliver (1995).

12.5 Subgrade evaluation

The following should be read in conjunction with Chapter 5.

12.5.1 Methods for estimating subgrade support value

Suggested spacing of test sites should vary between 60 to 120 m for urban projects, and up to 300 m for longer rural projects, with preferably no less than three test sites in any project. Where there is a variation along a project, at least three test sites should be considered for each subgrade, topography and drainage combination.

Procedures for estimation of the subgrade design California Bearing Ratio (CBR) are summarised in Figure 5.2.

In estimating subgrade design CBR, consideration should be also given to the following:

- For the design of urban streets, an adjacent existing street (or road) may be available for comparison purposes.
- For the low traffic situation in general, it is likely that knowledge of the soil types in a region coupled with knowledge of their previous performance as subgrades may provide considerable assistance in estimating a subgrade design CBR for a project.
- One consideration which is unique to the residential street situation is the possibility that the subgrade moisture content adjacent to the kerb may be artificially high due to excessive watering of gardens and nature strips by residents.
- Removal or subsequent planting of trees can significantly affect moisture content (removal resulting in an increase, planting resulting in a decrease).

12.6 Pavement materials

The following should be read in conjunction with Chapter 6.

12.6.1 Unbound granular materials

The quality and strength characteristics required for unbound granular materials depend upon a combination of factors such as:

- traffic loading (both number, type and loading of axle, tyre-pavement contact stresses)
- climate
- pavement configuration, cross-section and drainage
- whether the intended use is base or subbase
- strategic importance of the road.

For example, marginal or non-standard materials can more successfully be used for lightly trafficked roads in dry environments than moderate-to-heavily trafficked roads in wet environments. Such materials should only be used after consideration of:

- the documented performance history of the proposed material
- whole-of-life costs relative to standard materials
- the predicted traffic loading
- the climate at the site
- the moisture sensitivity of the proposed subgrade
- the quality and uniformity of the materials as shown by laboratory testing
- consequences of poor performance.

Specialist advice should be obtained on appropriate laboratory characterisation procedures for non-standard materials.

Marginal or non-standard materials are generally less stiff (have lower moduli) and are less durable than standard granular materials. In addition, both modulus and strength are usually more sensitive to moisture content. Hence, greater thicknesses are required to provide equivalent subgrade protection. However, while the extent of rutting of the subgrade is similar, the use of the non-standard materials may result in inferior performance due to greater rutting of the pavement materials under traffic loading. Therefore, controlling moisture entry into these pavements is a significant design consideration.

For pavements with thin bituminous surfacings, Figure 8.4 indicates higher quality materials are required near the pavement surface, with material having a CBR value of 80% or better being required for at least the top 100 mm of the pavement. Beneath this base layer, the lower strength requirements for subbase layers permit the use of materials which, for various reasons, would not be suitable for use in the base. In general, a much wider choice of test property limits can be permitted, provided the material, when compacted, has the required strength over the range of likely in-service moisture contents. Refer also to Section 12.4.2.

Lower standard base materials, with CBR ≥ 60%, may provide fit-for-purpose alternatives in lightly trafficked roads and dry environments. Use of such materials should be based on the same considerations detailed above for marginal and non-standard materials.

For unbound granular pavements with asphalt surfacings less than 100 mm thick, a minimum 100 mm of crushed rock base should be provided to:

- establish a working platform to enable adequate compaction of the asphalt
- in some situations, enhance subsurface drainage.

12.6.2 Cemented materials

The required properties of the materials stabilised with cement and cementitious binders may vary with the road class, design traffic and acceptable risk.

For moderate-to-heavily trafficked roads, the minimum 28-day Unconfined Compressive Strength (UCS) is 2 MPa to ensure a cemented material with less variable fatigue properties. When used in the design of new pavements, such materials are commonly covered by a minimum cover equivalent to 175 mm of asphalt to inhibit cracking of cemented materials reflecting to the surface. For some projects a strain alleviating membrane interlayer is also provided.

For some lightly trafficked roads, granular materials stabilised with cementitious binders to a UCS of 1–2 MPa have been used as there has been less concern about fatigue cracking causing detrimental effects on the life of thin bituminous surfacings.

12.6.3 Asphalt

The properties of asphalt for typical lightly trafficked roads differ from those required on heavier trafficked roads. This is especially so for granular pavements having thin asphalt surfacings and for pavements where the traffic loading is expected to be very low.

Asphalts for lightly trafficked roads are generally designed to be more flexible and durable and less permeable than those for heavier traffic applications. This reflects their use in thinner layers on more resilient pavements, the reduced likelihood of post-construction compaction and their common distress modes of cracking and ravelling which are related to the oxidation of the binder rather than to vehicle loads.

In order to achieve these attributes, asphalt mixes for light traffic applications are generally designed to have a lower air void content compared to asphalts for more heavily trafficked applications. This is typically achieved by using a finer aggregate grading and by incorporating 0.5% by mass additional binder relative to that used in normal duty mixes.

As with other asphalt mixes, asphalts for light duty applications also need to be well compacted, which is not always easy due to the use of thinner layers being laid on thinner more flexible pavements. The compactability of these mixes, however, can be improved by:

- an increased binder content
- a modification in the aggregate grading from a continuous grading to a gap grading
- the use of a softer grade of binder such as a Class 170 bitumen.

While the use of a softer binder in the asphalt improves compactability and resistance to cracking, it has been found that these asphalts are more susceptible to damage by heavier passenger vehicles with power steering when executing full-lock turns. The damage is confined to the surface and generally does not unduly diminish the life of the asphalt but it compromises the surface appearance. Therefore, if this is a concern then a stiffer binder (Class 320 bitumen) could be used.

Table 12.1 provides guidance on typical layer thicknesses used with various asphalt mix sizes.

Table 12.1: Typical asphalt layer thicknesses

Mix type	Nominal mix size (mm)	Typical layer thickness (mm)
Dense graded	7	25–35
	10	35–50
	14	45–70
	20	60–100
Stone mastic asphalt	7	25–35
	10	35–40
	14	50–60

Note: The adoption of layer thicknesses less than 30 mm should be avoided where the asphalt is to be laid during the cooler months of the year particularly in the southern Australian states and New Zealand. In those conditions, thinner layers will cool rapidly (e.g. in less than 10 minutes) thereby minimising the likelihood of achieving adequate compaction.

Fatigue cracking is a common distress mode for moderate-to-heavily trafficked roads. However, for lightly trafficked roads load-induced fatigue cracking is considered to be uncommon. Cracking of asphalt of lightly trafficked roads is often due to environmental factors such as:

- brittleness of the bitumen as a result of oxidisation, which is accelerated by high temperatures and ultra-violet radiation
- changes in volume of expansive subgrades due to moisture changes
- upheaval of the pavement due to tree roots.

The asphalt fatigue criteria (Equation 25) given in Chapter 6 is applicable to moderate-to-heavily trafficked roads. As fatigue cracking is not a common distress mode for lightly trafficked roads, this distress mode was not considered in the example design charts detailed in Section 12.8.

12.6.4 Concrete

Subbase concrete

The predominant role of the subbase concrete is the provision of a non-erodible subbase for a rigid pavement. For reasons discussed in Section 12.9, it is unlikely that there will be a need to make such a provision for lightly trafficked pavements. Where the need does exist, designers are referred to Section 6.8.2.

Base concrete

To provide adequate abrasion resistance to traffic, AS 3600 indicates that for pavements subject to pneumatic tyred vehicles exceeding 3 tonnes gross mass, a characteristic compressive strength of 32 MPa is required.

12.7 Design traffic

The text in this section should be read in conjunction with Chapter 7.

12.7.1 Procedure for determining total heavy vehicle axle groups

Short-term heavy loadings

The design procedures have been developed over many years using mechanistic modelling and in-service field performance data. Pavements are generally designed and constructed to last 20 years or more of trafficking, with the loading more or less evenly spread over the design period.

However, in some situations the pattern of loading differs markedly for that on which the procedures in this Part were based. For instance:

- Temporary pavements may be required to carry high daily traffic loadings but because of their limited design life (e.g. zero to two years) may have a relatively low design traffic loading.
- On some routes the haulage of grain or other produce results in large seasonal variations in traffic loadings.
- Some roads may have very high loadings on weekends or coinciding with major community or sporting events but very low loadings at other times.

In such situations the design traffic needs to be adjusted to allow for the greater impacts of these special loadings. For roads with intermittent or seasonal loadings, rather than the design traffic being calculated from the annual average daily heavy vehicle volume, the maximum daily heavy vehicle traffic per annum is used.

For temporary pavements with a design period of less than five years, the design traffic is calculated using a 10 to 20 year design period with zero traffic growth rate using the maximum daily heavy vehicle volume.

For minor and local access streets to be constructed in new residential subdivisions, the traffic generated by housing construction in the subdivision needs to be allowed for in the design traffic calculations.

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.

For lightly trafficked roads there is a greater likelihood of environmentally induced distress necessitating pavement rehabilitation in advance of any load-related distress. As such it may be prudent to opt for a shorter design period than that used for moderate-to-heavily trafficked roads.

Note that for lightly trafficked thin bituminous surfaced pavements, the rehabilitation at the end of the design period commonly only necessitates shaping and resurfacing rather than reconstruction. Whole-of-life costing (Chapter 10) may be useful in selecting the most cost-effective design period.

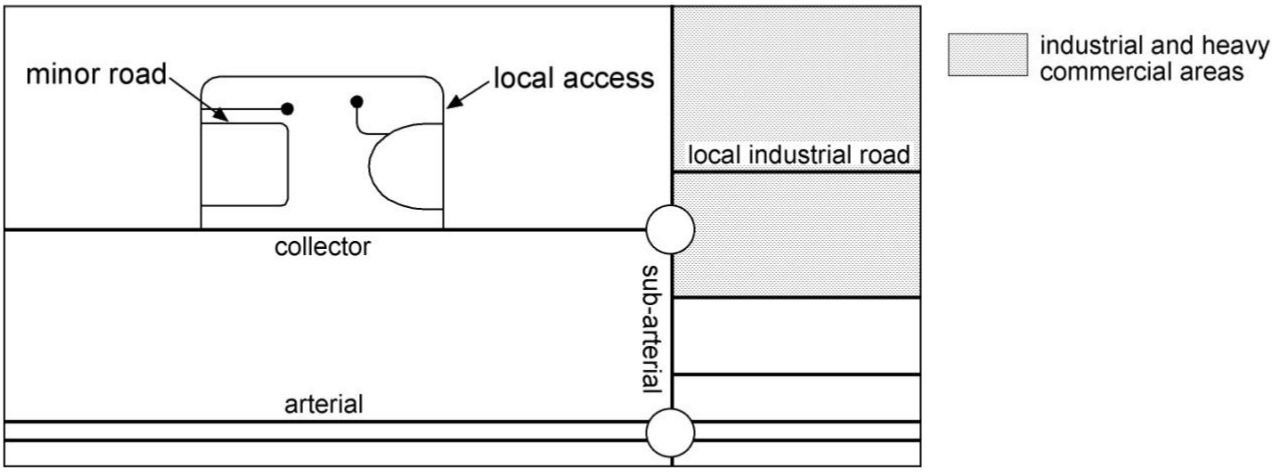
Estimating axle groups per heavy vehicle

The values for the average number of axle groups per heavy vehicle (N_{HVAG}) given in Appendix C are appropriate for moderate-to-heavily trafficked roads. For lightly trafficked roads, presumptive values given in Table 12.2 may be used.

Traffic volumes for lightly trafficked roads

In assessing expected traffic flows on lightly trafficked urban streets it is useful to categorise the streets in a hierarchical manner as indicated in Figure 12.1.

Figure 12.1: Lightly trafficked street categories



The cumulative HVAG for urban collectors, industrial roads, sub-arterials, and arterials are typically determined as detailed in Section 7.4. However, for more lightly trafficked roads Table 12.2 gives indicative values for heavy vehicle traffic volumes, both on a daily basis and also for 20 year and 40 year design periods.

The annual growth rate adopted for each street type and given as the cumulative growth factor (CGF) is considered to be representative.

Because there is considerable variation in both AADT and per cent heavy vehicles for streets of a given type, designers are urged to use all available information in arriving at estimates of these quantities for the specific design situation.

It is to be noted that in Table 12.2 two cases are presented for the Minor Street – single lane traffic and two lane traffic – with different Direction Factors (1.0 and 0.5 respectively) being applied to the two-way traffic to determine the design (single lane) traffic. Where the paved width of a street or the presence of parked vehicles is such that traffic travelling in both directions is likely to partially or fully use the same road space, consideration needs to be given to the appropriate Direction Factor (within the range 0.5 to 1.0).

Traffic on lightly trafficked rural roads is heavily dependent on local factors – predominantly land-use in the area served by the rural roads, but also actual (as distinct from designated) road function, e.g. does/will it form part of a convenient short cut/bypass for through traffic? Because of these (and like) factors, no attempt is made to provide guidance on indicative volumes of traffic for lightly trafficked roads in rural areas. Designers are encouraged to apply local knowledge when estimating volumes.

Table 12.2: Indicative heavy vehicle axle group volumes for lightly trafficked urban streets

Street type	AADT two-way	Heavy vehicles (%)	Initial daily heavy vehicles in design lane (single lane)	Design period (years)	Annual growth rate (%)	Cumulative growth factor[(1)]	Axle groups per heavy vehicle	Cumulative HVAG over design period	ESA/ HVAG	Indicative design traffic (ESA)
Minor with single lane traffic	30	3	0.9	20	0	20.0	2.00	13 140	0.45	6×10^3
				40	0	40.0	2.00	26 280	0.45	1.5×10^4
Minor with two lane traffic	90	3	1.35	20	0	20.0	2.00	19 710	0.45	9×10^3
				40	0	40.0	2.00	39 420	0.45	2×10^4
Local access with no buses	400	4	8	20	1	22.0	2.09	134 657	0.27	4×10^4
				40	1	48.9	2.09	298 963	0.27	8×10^4
Local access with buses	500	6	15	20	1	22.0	2.06	248 565	0.44	1.5×10^5
				40	1	48.9	2.06	551 862	0.44	2.5×10^5
Local access in industrial area	400	8	16	20	1	22.0	2.29	294 174	0.51	1.5×10^5
				40	1	48.9	2.29	653 122	0.51	3.5×10^5
Collector with no buses	1200	6	36	20	1.5	23.1	2.22	673 714	0.55	4×10^5
				40	1.5	54.3	2.22	1 581 109	0.55	9×10^5
Collector with buses	2000	7	70	20	1.5	23.1	2.15	1 273 100	0.64	8.5×10^5
				40	1.5	54.3	2.15	2 987 780	0.64	2×10^6

1 *For this case of below-capacity traffic volumes throughout the design period, values of the CGF for a range of annual growth rates and design periods are presented in Table 7.4.*

Note: Direction factor is 0.5, except for Minor Street with single lane traffic where DF = 1.0. Full indicative Traffic Load Distributions are listed in Appendix N.

12.7.2 Design traffic for flexible pavements

Damage to flexible pavements

For lightly trafficked roads, asphalt cracking is commonly due to environmental factors and load-induced fatigue cracking is uncommon. Hence for lightly trafficked pavements containing one or more bound layers, two distinct types of damage are considered:

- rutting and loss of surface shape
- fatigue damage to cemented material and lean-mix concrete.

Pavement damage in terms of standard axle repetitions

Presumptive traffic characteristics for lightly trafficked roads are presented in in Appendix N.

12.8 Design of flexible pavements

12.8.1 Mechanistic-empirical procedure

As described in Chapter 8, asphalt fatigue cracking is a recognised distress mode for moderate-to-heavily trafficked flexible pavements. However, for lightly trafficked roads load-induced fatigue cracking appears to be uncommon; cracking of asphalt of lightly trafficked roads is commonly due to environmental factors (refer Section 12.6.2). Consequently, for lightly trafficked roads there is no requirement to assess the fatigue life of asphalt layers.

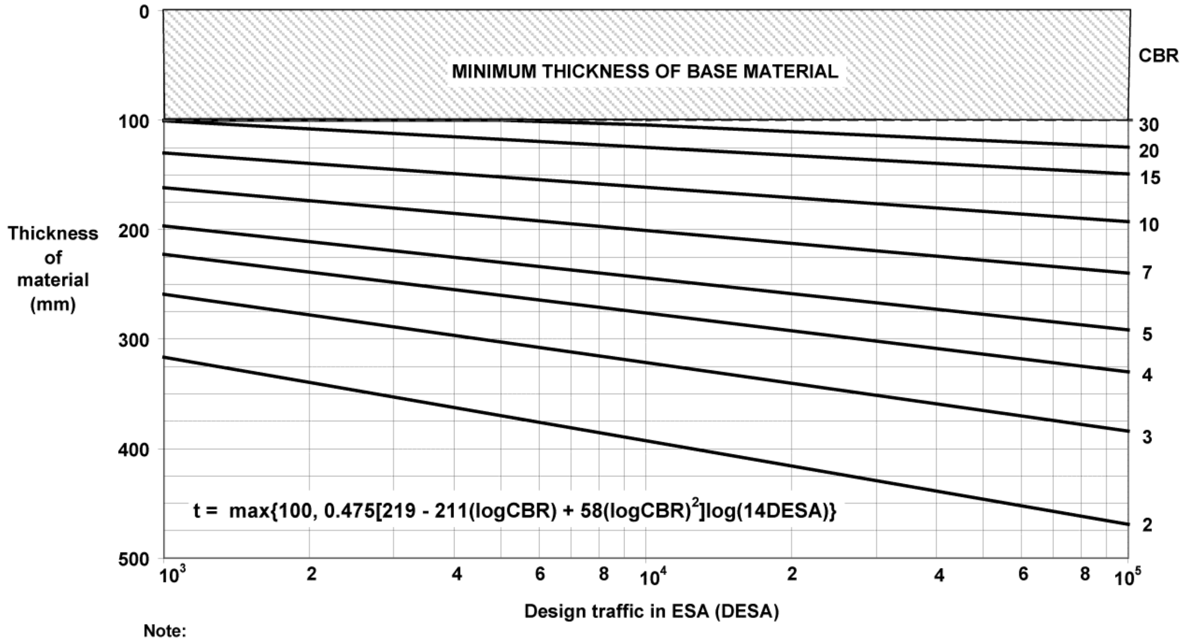
12.8.2 Empirical design of granular pavements with thin bituminous surfacing

Determination of basic thickness

Whereas Figure 8.4 is applicable to design traffic of 10^5 ESA or more, Figure 12.2 is applicable to lightly trafficked roads which are surfaced with either a bituminous seal or asphalt less than 40 mm thick.

Figure 12.2 is applicable where a minimum 100 mm thickness of base quality (CBR ≥ 80%) material is provided. However lower quality material may provide a fit-for-purpose alternative in some situations, as discussed in Section 12.6.1.

Figure 12.2: Example design chart for lightly trafficked granular pavements with thin bituminous surfacings



Note:
1. Appropriate local conditions, environmental and drainage issues must be considered in using these design curves.
2. Thin asphalt surfacings may be included in total granular thickness. However, the minimum thickness of the granular base is 100 mm.

Pavement composition

The composition of the pavement structure is made up by providing sufficient cover over the in situ subgrade and each successive material course. The thickness of cover required over a material is determined from its design CBR. If the design CBR value of a material is less than 30%, then the cover required to inhibit deformation is determined as for an in situ subgrade material, from Figure 12.2.

For a granular 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%) granular base material. This minimum base thickness is the thickness of cover required over material having a CBR equal to or greater than 30% (Section 6.2.1 and *Part 4A: Granular Base and Subbase Materials*, Austroads 2024b).

Note that the CBR test is not the sole measure to assess the adequacy of unbound granular materials (Section 6.2.1).

Beneath the granular layers, improved subgrade materials (including selected subgrade, lime-stabilised subgrade, select materials, capping layers or earthworks materials) may be used to provide the required cover of materials over the in situ subgrade. Bitumen, cement or other chemical stabilised materials are not considered improved subgrades in this document. The thicknesses of cover required over these materials are determined from their design CBR. In using Figure 12.2, improved subgrade materials normally have a maximum design CBR of 15%, irrespective of the measured CBR results. The process to determine the pavement composition is described in Section 8.3.2. Appendix L provides worked examples.

If the thin surfacing is dense graded asphalt or stone mastic asphalt, its thickness (< 40 mm) may be considered to contribute to the required total thickness over the in situ subgrade; however a minimum 100 mm thickness of base material needs to be provided. Other surfacing types (such as sprayed seals) are considered to make no contribution to the required thickness of material to inhibit deformation.

12.8.3 Mechanistic-empirical procedure – example charts

Values of input parameters implicit in design charts

To develop example design charts for lightly trafficked roads the following input parameters were used (Austroads 2008):

- Design period

  For the example design charts, traffic loading is expressed in terms of the design number of Equivalent Standard Axles (ESA). Hence, the example design charts are applicable to the normal range of design periods.

- Traffic load distributions

  In developing the example design charts, the traffic distributions given in Appendix N were used.

- Materials characterisation

  For asphalt-surfaced granular pavements, asphalt modulus values of 1000 MPa, 2000 MPa and 3500 MPa were adopted. The elastic characterisation of the granular materials was discussed in Section 8.2.2. Table 12.3 lists the pavement types included in the example design charts.

- Summary of input parameters

  The charts have been developed using the mechanistic-empirical procedure described above for the specific input parameters which are presented in Table 12.4. Before using these charts for the purpose of pavement design, designers should ensure that their use is appropriate to the design situation.

  The asphalt moduli appropriate for a specific project depends on the mix composition, operating asphalt temperature (WMAPT) and the design traffic speed. In the absence of project-specific data, Figure 12.3 provides guidance on asphalt moduli for size 14 mm dense graded asphalt mixes manufactured using Class 320 bitumen. Presumptive moduli for other asphalt mix sizes and binder types may be estimated from the values in Figure 12.3 and the relative moduli given in Table 6.14 and Table 6.15. The New Zealand bitumen standard (NZ Transport Agency 2022) uses penetration rather than viscosity classes. To assist in utilising Table 6.14 and Table 6.15, the New Zealand Class B80 and B60 bitumens are equivalent to Australian Class 170 and Class 320 bitumens, respectively.

Table 12.3: Catalogue of light traffic example design charts

Asphalt modulus (MPa)	Asphalt thickness (mm)	Granular thickness (mm)	Subgrade modulus (MPa)	Chart
1000	Varying	Varying	30	Figure 12.4
	Varying	Varying	50	Figure 12.5
	Varying	Varying	70	Figure 12.6
2000	Varying	Varying	30	Figure 12.7
	Varying	Varying	50	Figure 12.8
	Varying	Varying	70	Figure 12.9
3500	Varying	Varying	30	Figure 12.10
	Varying	Varying	50	Figure 12.11
	Varying	Varying	70	Figure 12.12

Table 12.4: 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	Presumptive traffic parameters in Appendix N
Distribution of loads on each type of axle group	Presumptive traffic parameters in Appendix N
Modulus of asphalt	1000 MPa, 2000 MPa and 3500 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 (Normal Standard Granular) with $\nu_H = \nu_V = 0.35$
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 Chapter 5
Relationship for asphalt fatigue	Not applicable for roads with design traffic loading less than 10^5 ESA
Relationship for permanent deformation	$N_{ESA} = \left[\frac{9150}{\mu\varepsilon}\right]^7$

Figure 12.3: Presumptive asphalt moduli of dense graded mixes with Class 320 binder for various heavy vehicle design speeds (km/h)

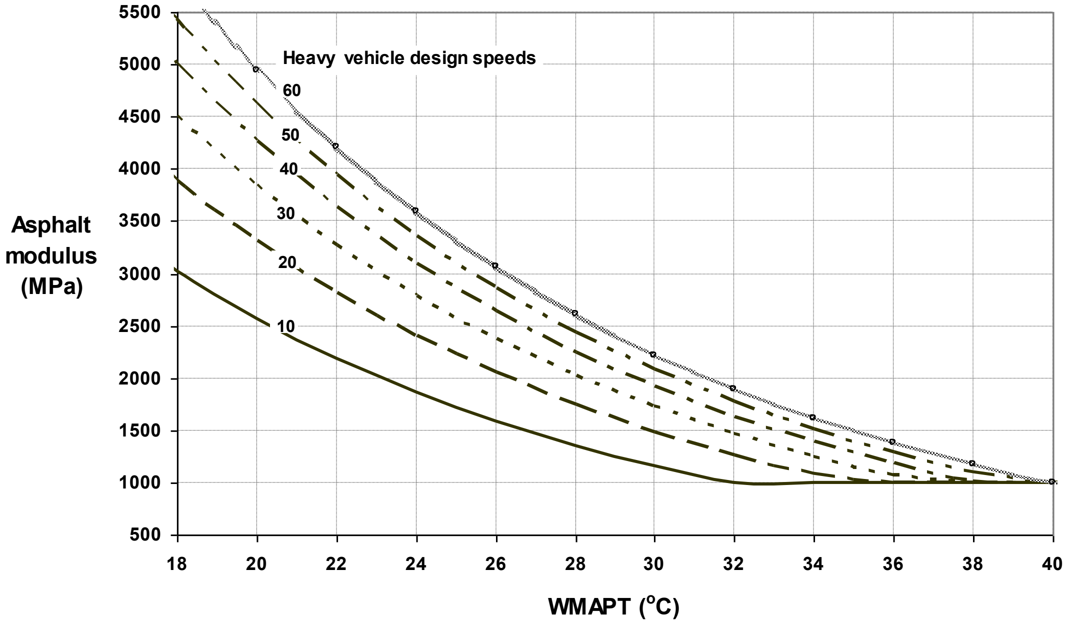
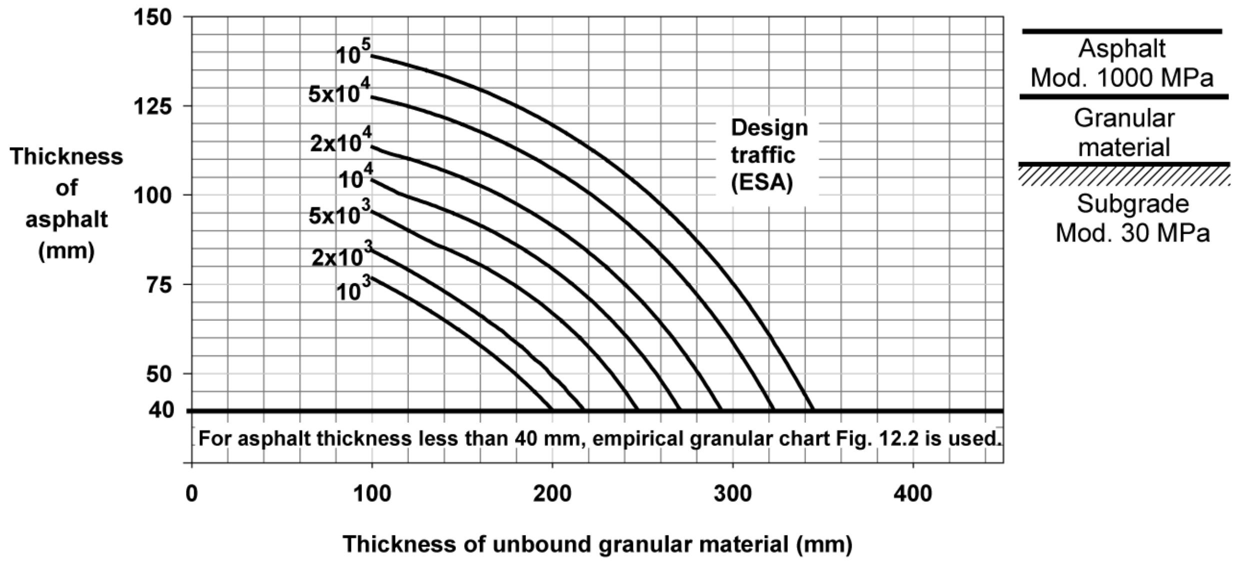


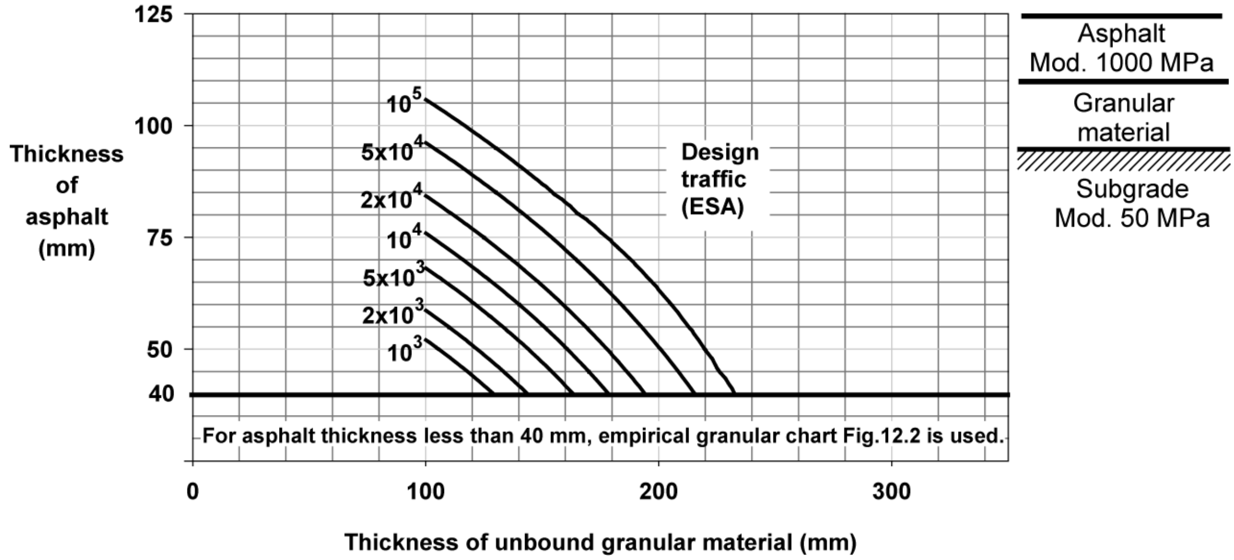
Figure 12.4: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 1000 MPa and subgrade modulus of 30 MPa



Note:

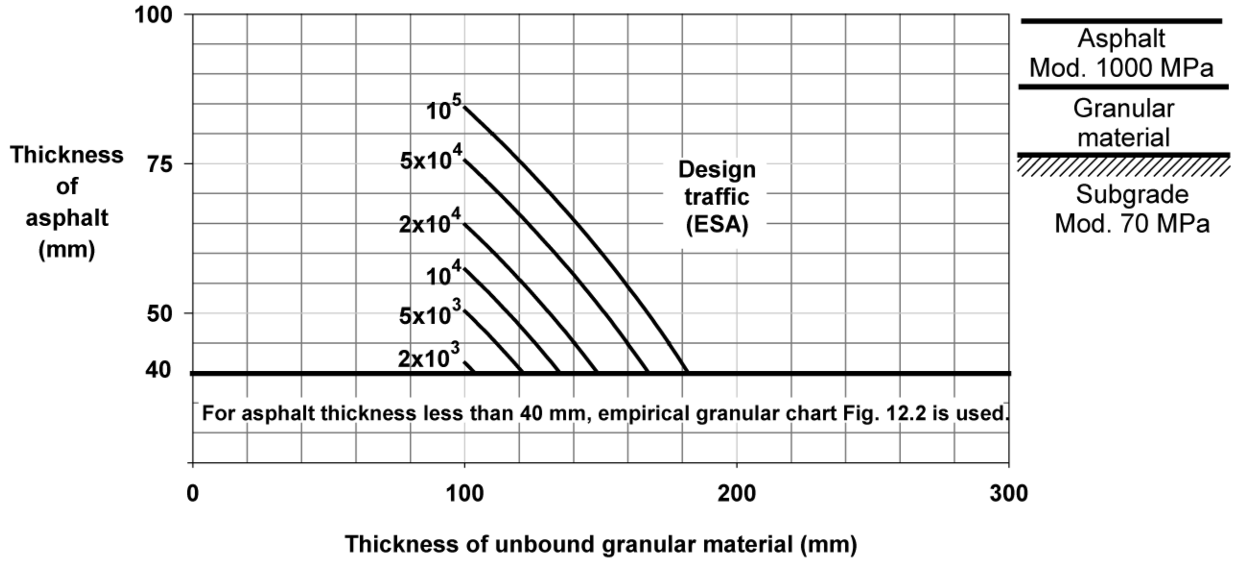
1. Allowance to be made for construction tolerances.
2. Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.

Figure 12.5: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 1000 MPa and subgrade modulus of 50 MPa



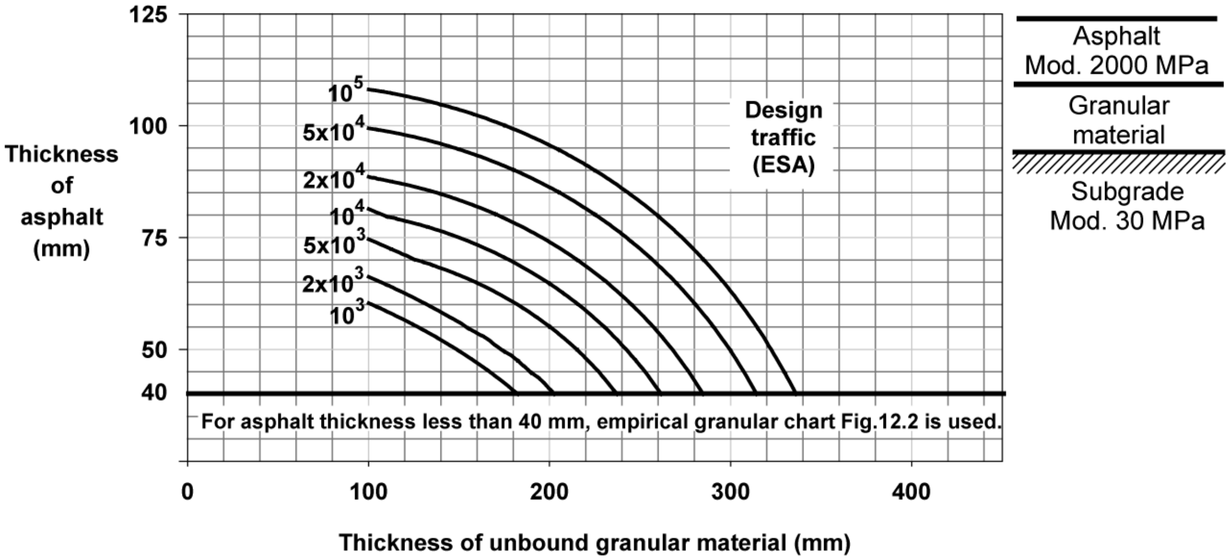
Note:
1. Allowance to be made for construction tolerances.
2. Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.

Figure 12.6: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 1000 MPa and subgrade modulus of 70 MPa



Note:
1. Allowance to be made for construction tolerances.
2. Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.

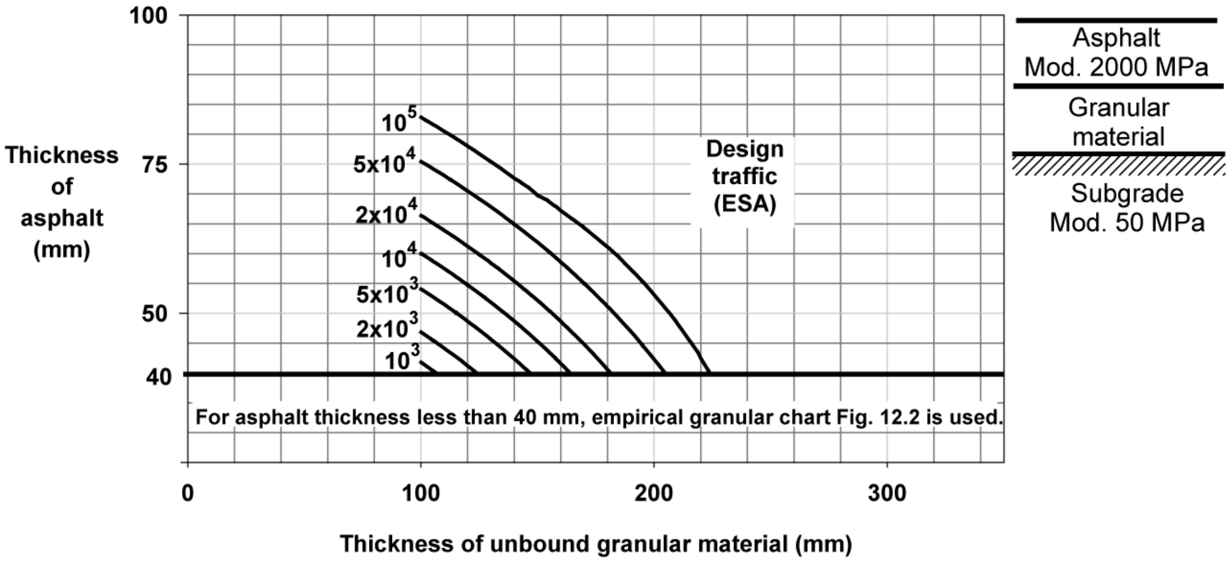
Figure 12.7: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 2000 MPa and subgrade modulus of 30 MPa



Note:

1. **Allowance to be made for construction tolerances.**
2. **Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.**

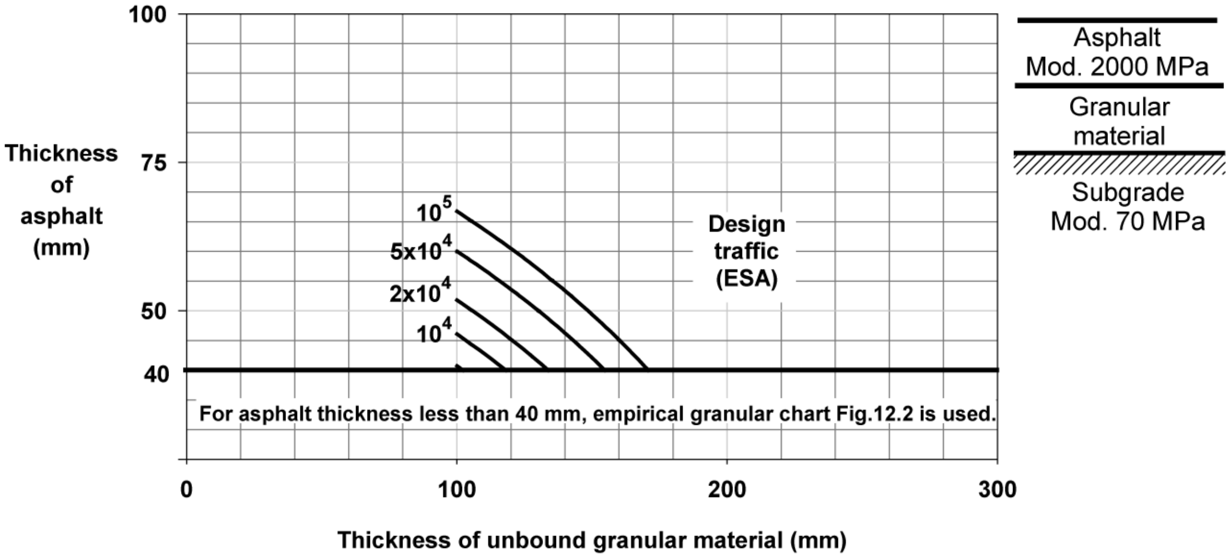
Figure 12.8: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 2000 MPa and subgrade modulus of 50 MPa



Note:

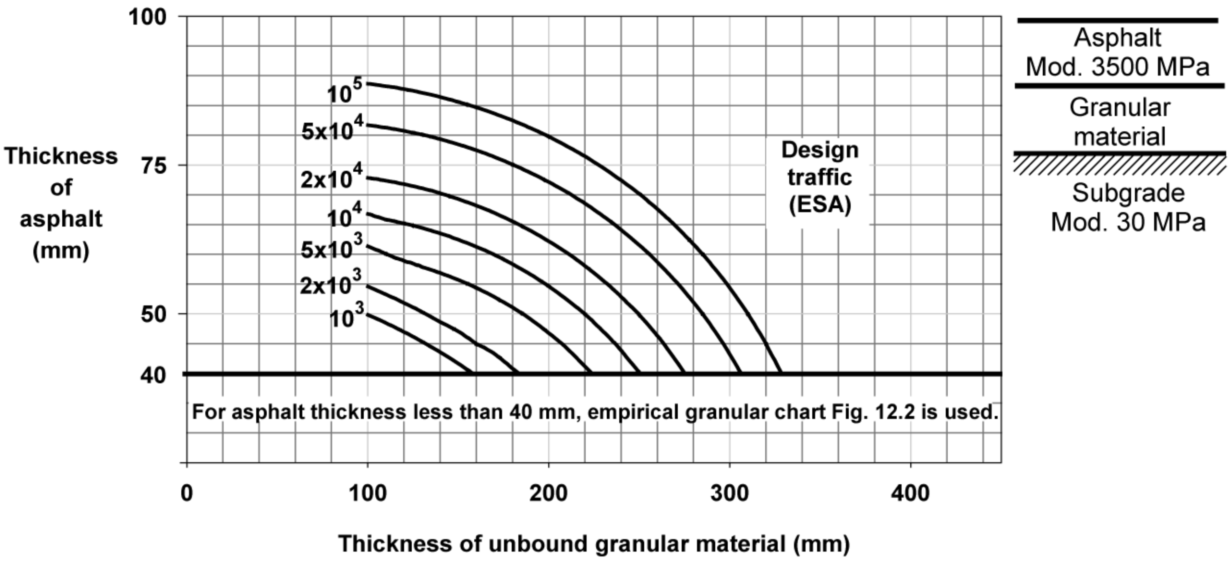
1. **Allowance to be made for construction tolerances.**
2. **Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.**

Figure 12.9: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 2000 MPa and subgrade modulus of 70 MPa



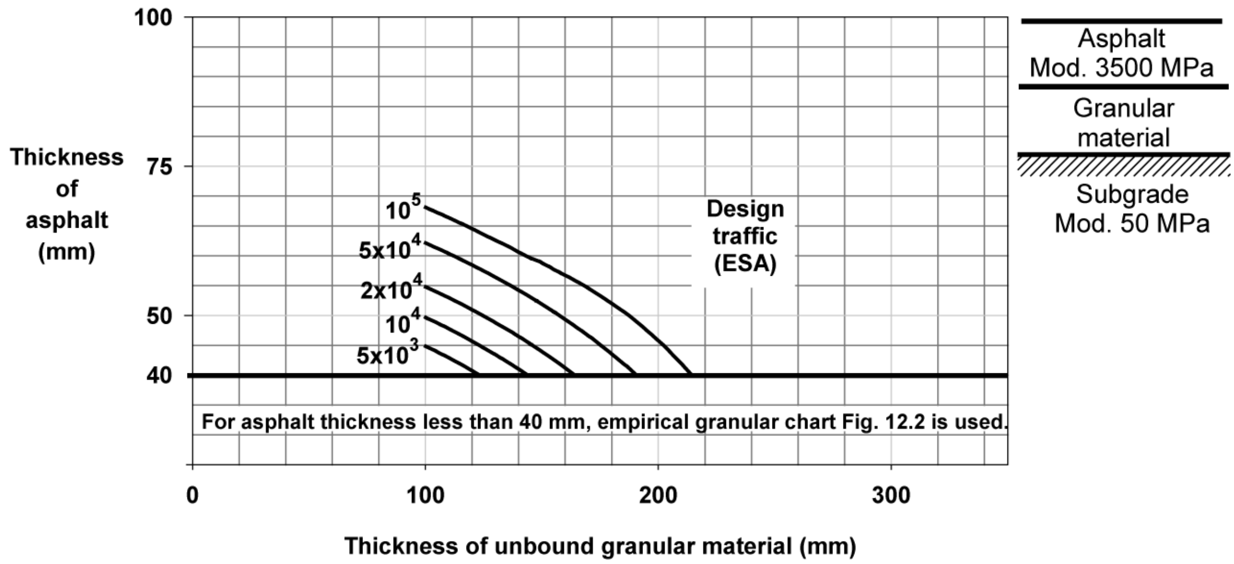
Note:
1. Allowance to be made for construction tolerances.
2. Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.

Figure 12.10: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 3500 MPa and subgrade modulus of 30 MPa



Note:
1. Allowance to be made for construction tolerances.
2. Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.

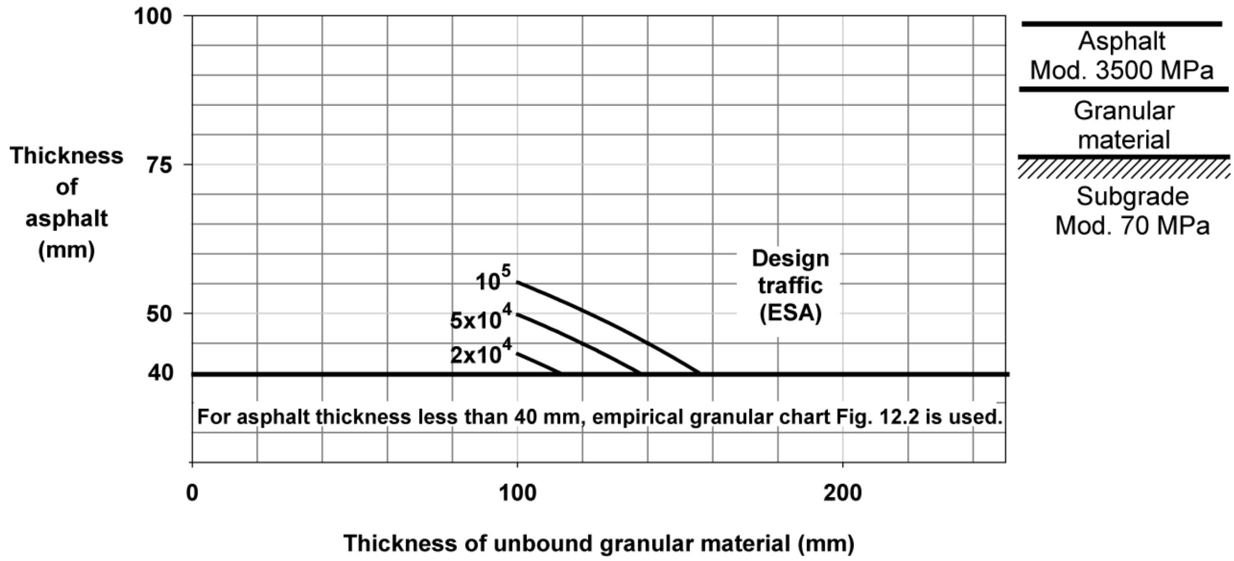
Figure 12.11: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 3500 MPa and subgrade modulus of 50 MPa



Note:

1. **Allowance to be made for construction tolerances.**
2. **Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.**

Figure 12.12: Example design chart for asphalt surfaced granular pavement with asphalt modulus of 3500 MPa and subgrade modulus of 70 MPa



Note:

1. **Allowance to be made for construction tolerances.**
2. **Permanent deformation is the only distress mode considered in the thickness design. Asphalt fatigue does not appear to be a distress mode for lightly trafficked roads.**

12.9 Design of rigid pavements

12.9.1 General

Chapter 3 provides the design procedure for design traffic loadings exceeding 10^6 HVAG. Two distress modes are considered in the base thickness design namely, fatigue cracking of the concrete base and subgrade/subbase erosion. These design procedures need to be modified for lightly trafficked roads. In particular, while erosion of subgrade/subbase is an important distress mode for more heavily trafficked roads, erosion is not normally of concern for lightly trafficked roads due to the combination of low axle repetitions and low vehicle speeds which reduces the likelihood of pumping of subbase or subgrade materials.

Designers may calculate the required base thickness using the procedures detailed below. Alternatively, example design charts have been developed using the procedures detailed in Section 12.9.4.

12.9.2 Pavement types

Base types

Section 9.2.1 describes the various concrete base types that may be used.

For cycleways, designed to cater for occasional use by service vehicles and crossings for residential property access, a 3 m wide concrete base with a 150 mm thickness of 32 MPa concrete, continuous lapped SL72 mesh and transverse joints sawcut at 3.5 m centres to a 50 mm depth has been used by the Roads and Maritime Services, NSW. Expansion joints are required at every fifth transverse joint (every 17.5 m). The concrete base is supported on 150 mm of dense graded granular base material.

Subbase types

The main functions of a subbase in a rigid pavement are:

- the provision of an erosion resistant layer under the concrete base
- assistance in the provision of a uniform support for the concrete base
- assistance in controlling volume changes in expansive subgrades
- the provision of a stable working platform upon which to construct the base.

The minimum subbase requirements in Table 9.1 are appropriate for moderate-to-heavily trafficked roads where erosion of fines from the subgrade or subbase may occur due to pumping under repeated load applications. However, for lightly trafficked roads, erosion is not normally a distress mode as discussed in Section 12.1 and hence these roads do not normally require a bound subbase to inhibit erosion. In most cases, a 100 mm granular subbase – typically crushed rock – will provide the remaining functions and this is all that should normally be specified for lightly trafficked rigid (concrete) streets.

The functions of the granular subbase can be effectively performed by in situ stabilisation of the subgrade provided a sufficient quantity of binder is used to ensure long-term strength gain.

12.9.3 Factors used in thickness determination

Effective subgrade strength

Where thin (< 150 mm) unbound granular subbases are used for lightly trafficked roads, the increase in effective subgrade strength is minor and can be ignored. For other subbase types and thicknesses the procedures detailed in Chapter 3 may be used to calculate the effective subgrade strength.

Base concrete strength

To provide adequate abrasion resistance to traffic, AS 3600 indicates that for pavements subject to pneumatic tyre vehicles exceeding 3 tonnes gross mass, a characteristic compressive strength of 32 MPa is required.

Where no information on flexural strength is available, designers may use a flexural strength of 4 MPa for thickness design purposes when a minimum characteristic compressive strength of 32 MPa is specified.

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, de-bonded, 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
- 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.

12.9.4 Base thickness design

General

As discussed in Section 12.1, erosion is not generally a distress mode for lightly trafficked roads due to the low number of load repetitions. Hence base thicknesses for lightly trafficked roads are only governed by the thickness required to inhibit fatigue cracking of the concrete base.

Minimum base thickness

For roads with design traffic less than 10^6 HVAG, a minimum base thickness of 125 mm would normally be adopted to allow passage of an occasional overloaded vehicle.

Example design charts

The example design charts (Figure 9.2 and Figure 9.3) given in Chapter 3 relate to roads with a design traffic loading of 10^6 HVAG or more. Example design charts for lightly trafficked rigid pavements are presented in Figure 12.13 to Figure 12.16. These charts are based on:

- design concrete flexural strength of 4 MPa
- load safety factors of 1.05 and 1.2 (Table 9.2 for associated project design reliabilities)
- with and without the provision of concrete shoulders
- a minimum base thickness of 125 mm.

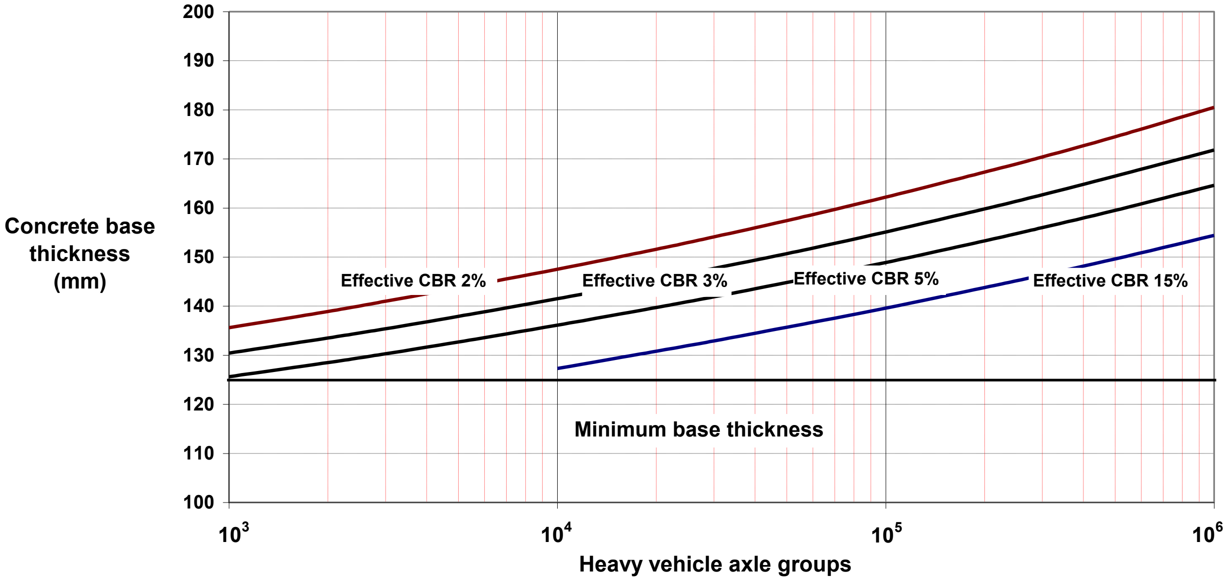
These example design charts have been developed using the data on the distributions of loads on axle groups for urban street traffic reported by Matthews and Mulholland (1994). The load distribution shown in Table 12.5 represents the 90th percentile high values for each street type (Figure 12.1). The base thicknesses were calculated for each street type and the maximum of these values was used in the design charts. Further details are given in Austroads (2008).

The charts allow designers to compare the design base thickness for different traffic volumes, effective subgrade strengths and load safety factors. The effective subgrade strengths (CBR of 2 to 15%) chosen for the example charts reflect the typical range used for design of lightly trafficked roads with thin unbound granular subbases.

Table 12.5: Traffic load distribution used in development of example design charts

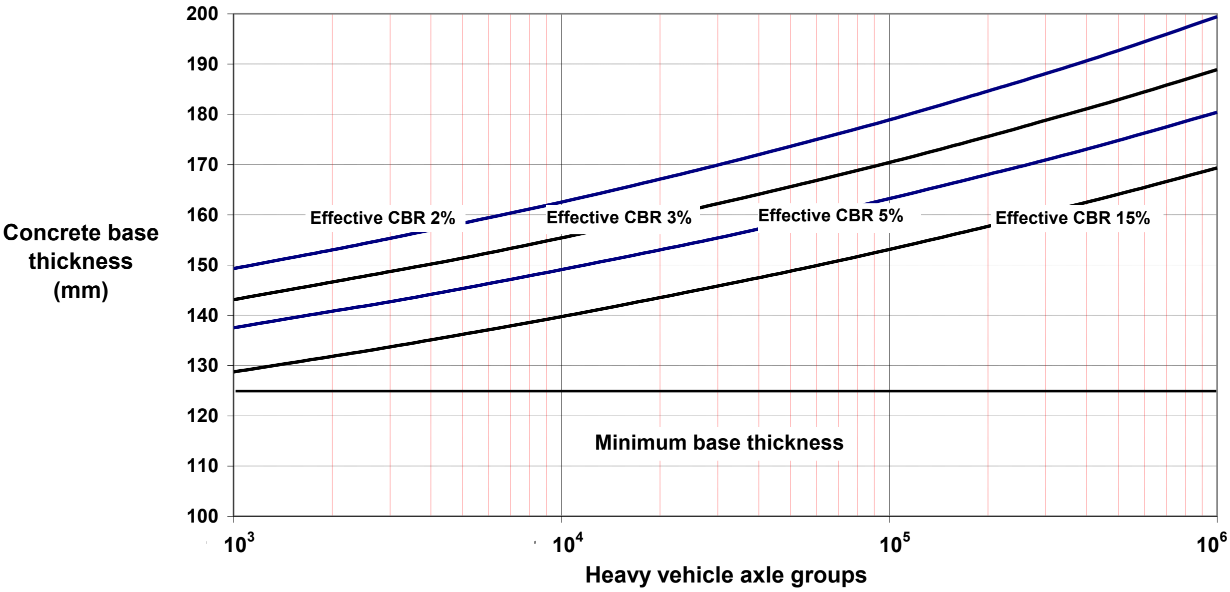
Type of heavy vehicle	Estimated % loading	Load on axle group (kN)		
		Single axle with single tyres (SAST)	Single axle with dual tyres (SADT)	Tandem axle with dual tyres (TADT)
2-axle rigid truck	Full	52.9	83.3	
	75	49.0	68.6	
	50	44.1	53.9	
	25	39.2	39.3	
	Empty	34.3	24.5	
3-axle rigid truck	Full	52.9		147.0
	75	49.0		119.1
	50	44.1		91.1
	25	39.2		62.7
	Empty	34.3		34.3
5-axle articulated truck	Full	39.2		147.0
	75	39.2		120.1
	50	39.2		93.1
	25	39.2		66.6
	Empty	39.2		39.2
2-axle bus	Full	63.7	83.8	
	50	51.9	75.5	
	Empty	39.2	68.6	

Figure 12.13: Example design chart for lightly trafficked roads with tied or integral concrete shoulders (LSF = 1.05)



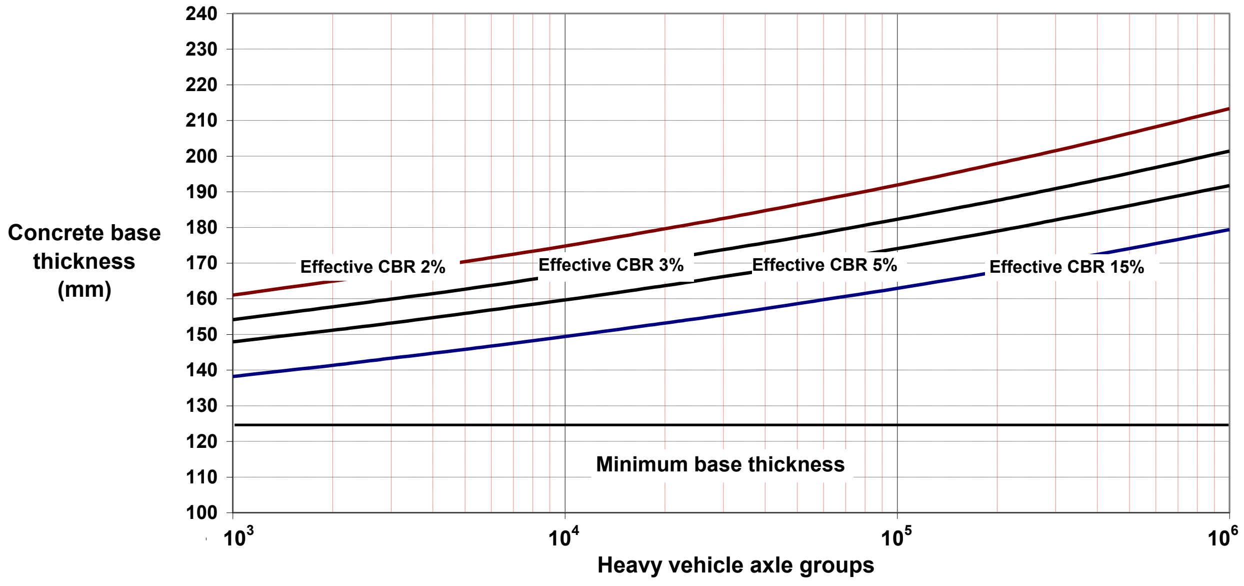
Note: Allowance to be made for construction tolerances.

Figure 12.14: Example design chart for lightly trafficked roads with tied or integral concrete shoulders (LSF = 1.2)



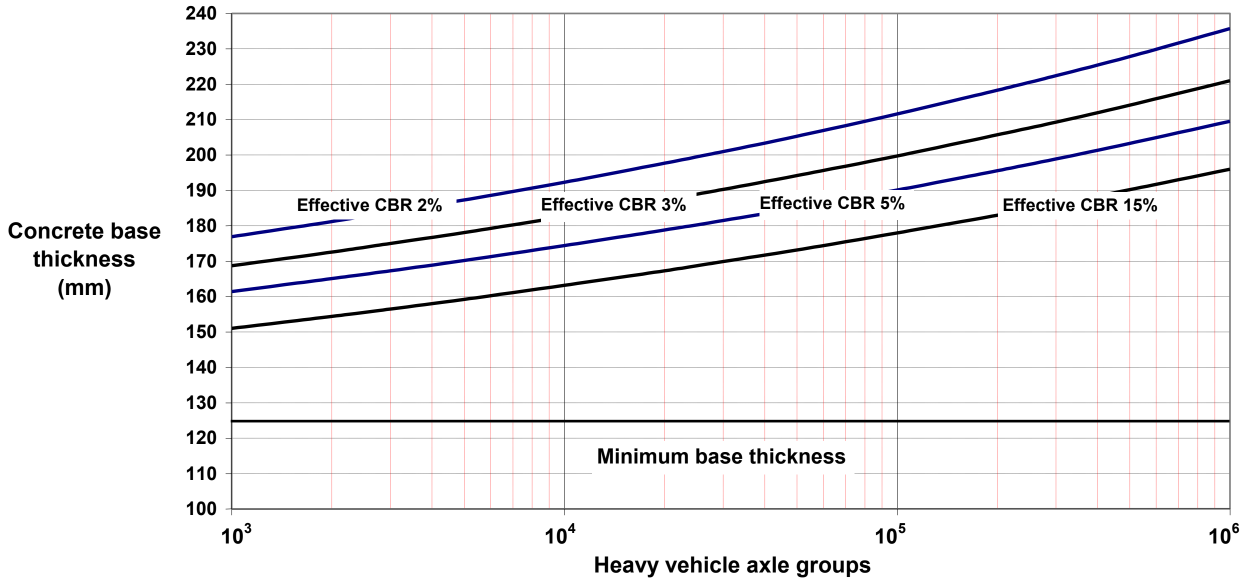
Note: Allowance to be made for construction tolerances.

Figure 12.15: Example design chart for lightly trafficked roads without tied or integral concrete shoulders (LSF = 1.05)



Note: Allowance to be made for construction tolerances.

Figure 12.16: Example design chart for lightly trafficked roads without tied or integral concrete shoulders (LSF = 1.2)



Note: Allowance to be made for construction tolerances.

12.9.5 Reinforcement design procedures

Jointed reinforced concrete pavement slabs are usually 8 to 15 m long, but lengths in the range of 10 to 12 m are recommended on the basis of economy and pavement performance.

For lightly trafficked streets and for slab lengths less than 15 m, Table 12.6 provides typical steel reinforcement requirements.

Table 12.6: Steel reinforcement for slabs up to 15 m long

Concrete base thickness (mm)	Minimum steel reinforcing fabric size
125	SL62
150	SL82
175	SL92
200	SL92

12.9.6 Joints

Joints are provided in rigid street pavements to control concrete shrinkage and warping caused by variations in temperature and moisture. Joints are also required to divide a pavement into suitable lengths and widths for construction purposes.

The objectives of joint design are to develop a jointing system which will control cracking and provide adequate load transfer across joints so that the pavement will have adequate riding qualities over its design life. Joints in major roads should be sealed to minimise the intrusion of water and incompressible solids into the joint.

These objectives can be accomplished by employing the following principles:

- For jointed unreinforced concrete pavements, contraction joints should be provided at a spacing not exceeding approximately 20 times the slab thickness with typical joint spacing of 4.2 m. This will provide adequate load transfer by aggregate interlock across the joint. The contraction joint is formed by providing a depth reduction, by saw cutting or wet forming, to a depth of 25% of the slab. Where the joint spacing is 4.5 m, dowels are required in all transverse contraction joints to provide effective load transfer.
- Jointed unreinforced base slabs should be kept approximately square, not exceeding a 3/2 ratio – otherwise steel reinforcement should be used to control the width of drying shrinkage cracks. Steel reinforcement should also be used where
  - longer jointed spacings (> 5 m) are desired
  - re-entrant corners are encountered
  - mismatched joints are unavoidable.
- Bound subbases improve joint performance by increasing the efficiency of aggregate interlock, particularly in more heavily trafficked roads (≥ 10^6 HVAG), but are not required for lesser trafficked roads.
- For jointed reinforced concrete pavements, with typical joint spacings of 8 to 12 m, dowels are required in all transverse contraction joints to provide effective load transfer – due to the opening width of the contraction joint.
- For cycleways, designed to cater for occasional use by service vehicles and crossings for residential property access, a 3 m wide concrete base with a 150 mm thickness of 32 MPa concrete, continuous lapped SL72 mesh and transverse joints sawcut at 3.5 m centres to a 50 mm depth has been used by Roads and Maritime Services, New South Wales. Expansion joints are required at every fifth transverse joint (every 17.5 m). The concrete base is supported on 150 mm of dense graded granular base material.
- At kerb returns, curved edges in cul-de-sacs, or at the perimeter of angle-parking areas, joints may form acute angles in the corners of slabs. In these cases, the potential for a crack to occur across the acute angle can be avoided by off-setting the joint at least 300 mm from the curved edge or corner. This will have the effect of removing the acute angle and, therefore, the potential for a crack to develop at that location. Designers are referred to RTA (2004) for more detailed guidance about treatment of slabs with acute angles.

- A longitudinal joint spacing in the range of 3.0 to 4.5 m, typically 3.7 m, provides longitudinal crack control. Steel tiebars are typically used to hold the joint tightly closed to provide effective load transfer. It is recommended that no more than four lanes be tied together. Longitudinal joints should be located away from concentrated heavy vehicle wheel paths. Longitudinal joints should be induced by a depth reduction of 33%, as they can take some time to crack. These joints are sometimes called warping joints as they relieve warping and curling stresses.
- Transverse expansion joints are required only at fixed objects, and at certain locations in intersections and where transverse contraction joints have not been sealed in streets with low traffic volumes.

The joint layout should be designed to be compatible with the road geometry, the length of slab, the type of joint, and the construction method and programming to be employed. Refer to Section 9.7 for further information about joint types and layouts.

To ensure that joints perform properly in-service it is necessary to employ appropriate construction techniques. Sealants should be properly installed according to their type, climatic conditions, and individual material specifications.

12.10 Implementation of design and collection of feedback

As discussed in Chapter 11, 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.

An example of the benefits of the systematic collection of pavement design, construction and maintenance data is provided in Jones and Vlasic (1991). Investigations of subdivisional and other selected pavement works provided local verification for the design procedure for granular pavements with thin bituminous surfacings.

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ATM-231:2023, *Deformation resistance of asphalt mixtures by the wheel tracking test.*

ATM-274:2023, *Characterisation of flexural stiffness and fatigue performance of bituminous mixes.*

ATS-3110:2023, *Supply of polymer modified binders.*

Appendix A Australasian Road Agency Pavement Design Manuals or Supplements

The following road agencies have published design manuals or supplements that translate the design guidance provided by Austroads into design practice reflecting local materials, environment, loading and pavement performance:

- Transport for New South Wales
- Department of Transport and Planning, Victoria
- Queensland Department of Transport and Main Roads
- Department for Infrastructure and Transport, South Australia
- Main Roads Western Australia
- New Zealand Transport Agency Waka Kotahi.

Reference to these documents may be required as appropriate.

Appendix B Weighted Mean Annual Pavement Temperature

The values contained in the following tables can be used to select a Weighted Mean Annual Pavement Temperature (WMAPT) (°C) for use in estimating asphalt moduli at in-service temperatures (Section 6.6). The values in the tables were calculated using the method detailed below using gridded (spatial) data from 2003 to 2022.

The following method is used to calculate the WMAPT for a specific location:

1. Obtain the monthly average daily maximum air temperatures and the monthly average daily minimum air temperatures from the Bureau of Meteorology (BoM) for a nearby weather station – www.bom.gov.au/climate/data. Alternatively, obtain gridded climate data http://www.bom.gov.au/climate/maps/averages/temperature/. In NZ, obtain data from the National Climate Database - https://cliflo.niwa.co.nz/
2. Calculate the monthly average air temperature (T_{air}) for each month by taking the average of the maximum and minimum air temperatures. If the climate agency provides mean temperature instead of maximum and minimum values, use that instead. This step will result in 12 average air temperature values, one for each month.
3. Using Equation A1 and the monthly average air temperatures from Step 2, calculate the temperature weighting factor (WF) for each month.
4. Average the 12 WFs obtained in Step 3.
5. Using the average WF from Step 4 and Equation A2, estimate the weighted mean annual air temperature (WMAAT). If the WMAAT is less than 2.72, do not use this method for calculating WMAPT. The WMAPT equation has mathematical limitations at low temperatures and should not be used.
6. Using the WMAAT from Step 5 and Equation A3, estimate the WMAPT. The WMAPT value should be rounded to the nearest whole number.

Shell Weighting Factors (based on Chart W of Shell *Pavement Design Manual*, Shell 1978) (Equation A1).

$$WF = 10^{(-1.224 + 0.06508 T_{air} - 0.000145 {T_{air}}^2)} \qquad \text{A1}$$

WMAAT from average WF (based on Chart W of Shell *Pavement Design Manual*) (Equation A2).

$$WMAAT = 19.66 + 16.91 \log WF + 0.3117 * (\log WF)^2 \qquad \text{A2}$$

Estimating WMAPT from WMAAT (Chart RT Shell *Pavement Design Manual*, 100 mm asphalt) (Equation A3).

$$WMAPT = -12.4 + \frac{6.32(WMAAT)}{\ln(WMAAT)} \qquad \text{A3}$$

Table 12.7: Weighted mean annual pavement temperatures for selected locations

Victoria	
Town	**WMAPT**
Bairnsdale	24
Ballarat	21
Benalla	26
Bendigo	25
Bright	23
Charlton	26
Dandenong	24
Dookie	26
Echuca	27
Frankston	24
Geelong	24
Horsham	25
Melbourne Region	25
Mildura	30
Nhill	25
Sale	24
Seymour	25
Swan Hill	28
Wangaratta	26
Warragul	23
Warrnambool	23
Wodonga	27
Yallourn	23

South Australia	
Town	**WMAPT**
Adelaide	28
Bordertown	25
Ceduna	28
Keith	25
Mt Gambier	23
Murray Bridge	27
Naracoorte	24
Port Augusta	31
Port Pirie	30
Renmark	29
Whyalla	29

Tasmania	
Town	**WMAPT**
Burnie	21
Campbell Town	19
Devonport	20
Geeveston	18
Hobart	21
Launceston	21
New Norfolk	19
Queenstown	18
St Helens	21
Scottsdale	21
Swansea	21

Western Australia	
Town	**WMAPT**
Albany	25
Broome	41
Bunbury	27
Cape Leeuwin	26
Carnarvon	35
Dampier	40
Esperance	27
Eucla	29
Fremantle	29
Geraldton	32
Kalgoorlie	31
Kununurra	42
Manjimup	24
Meekatharra	37
Merredin	30
Morawa	34
Mt Magnet	36
Narrogin	27
Newman	39
Norseman	28
Northam	30
Ongerup	25
Paraburdoo	41
Perth	30
York	29

NSW and ACT			
Town	**WMAPT**	**Town**	**WMAPT**
Albury	27	Liverpool	28
Armidale	23	Merimbula	25
Bathurst	24	Mittagong	23
Bega	25	Molong	26
Bellingen	30	Moree	33
Blayney	22	Moruya	26
Bourke	34	Moss Vale	23
Braidwood	22	Mudgee	27
Broken Hill	31	Murrurundi	26
Byron Bay	31	Murwillumbah	31
Campbelltown	28	Narooma	25
Canberra	24	Narrabri	32
Casino	31	Narrandera	29
Cessnock	29	Newcastle	29
Cobar	33	Nowra	27
Coffs Harbour	30	Nyngan	32
Cooma	20	Orange	23
Coonabarabran	28	Parkes	29
Coonamble	32	Parramatta	29
Cowra	28	Port Macquarie	29
Deniliquin	28	Queanbeyan	24
Dubbo	30	Richmond	29
Finley	28	Singleton	30
Forbes	29	Sydney Region	29
Gilgandra	30	Tamworth	28
Glen Innes	23	Taree	29
Gosford	29	Tenterfield	24
Goulburn	23	Thredbo	14
Grafton	31	Tumut	27
Griffith	29	Wagga Wagga	28
Gundagai	27	Walgett	34
Hay	30	Warialda	31
Inverell	28	Wellington	29
Katoomba	21	Wentworth	30
Kempsey	30	Wilcannia	33
Kiama	28	Wollongong	28
Kiandra	15	Wyong	28
Lismore	31	Yass	25
Lithgow	21	Young	26

Queensland			
Town	**WMAPT**	**Town**	**WMAPT**
Ayr	37	Julia Creek	40
Baralaba	35	Kingaroy	30
Barcaldine	37	Longreach	38
Beaudesert	31	Mackay	35
Biloela	34	Maryborough	33
Birdsville	39	Miles	33
Blackall	36	Mitchell	33
Bollon	35	Monto	32
Boulia	40	Mt Isa	39
Bowen	37	Nambour	32
Brisbane Region	32	Normanton	41
Bundaberg	34	Palmerville	38
Cairns	38	Pittsworth	29
Caloundra	32	Quilpie	37
Camooweal	40	Richmond	39
Cardwell	37	Rockhampton	36
Charleville	35	Roma	34
Charters Towers	36	Southport	32
Clermont	35	St George	34
Cloncurry	40	St Lawrence	35
Cooktown	38	Stanthorpe	25
Cunnamulla	36	Surat	34
Dalby	31	Tambo	34
Emerald	36	Taroom	34
Gayndah	33	Thargomindah	37
Georgetown	39	Toowoomba	28
Gladstone	35	Townsville	37
Goondiwindi	33	Urandangi	39
Gympie	32	Warwick	29
Herberton	31	Weipa	40
Hughenden	38	Windorah	39
Ipswich	32	Winton	39
Isisford	38		

New Zealand	
Town	**WMAPT**
Auckland	24
Christchurch	20
Dunedin	18
Gisborne	23
Greymouth	19
Hamilton	23
Invercargill	16
Kaikoura	20
Masterton	21
Napier	23
New Plymouth	22
Nelson	21
Oamaru	18
Palmerston North	22
Queenstown	17
Rotorua	21
Taupo	20
Tauranga	24
Timaru	18
Wanganui	22
Wellington	22
Westport	20
Whangarei	25

Northern Territory	
Town	**WMAPT**
Alice Springs	35
Barrow Creek	38
Daly Waters	40
Darwin	41
Katherine	41
Tennant Creek	40

Appendix C Example Determination of Cumulative Number of Heavy Vehicles Considering Capacity

The design example relates to an urban motorway comprising two carriageways, each with three traffic lanes.

Design parameters

Design period	40 years
Annual number of heavy vehicles in design lane in first year of opening to traffic	10^6
Average percentage of heavy vehicles	10%
Number of traffic lanes in the direction of the design lane	3
Lane distribution factor	0.65
Heavy vehicle growth rate	5%
Capacity flow rate (motorway)	2300 pc/h
Passenger car equivalent (E_{HV})	2.0 pc/HV

C.1 Calculation of annual number of heavy vehicles

Assuming the heavy vehicle (HV) growth applies for each year of the design period, the annual number of HV in the design lane for each year of the design period is calculated as shown in Table 12.8.

C.2 Calculation of maximum annual number of heavy vehicles

This motorway has a maximum hourly capacity of 2300 passenger cars per hour per lane (see Table 7.5). As the motorway has 10% HV and each HV affects the hourly volume the same as 2 cars (E_{HV} = 2.0), using Equation A7 the maximum daily number of vehicles travelling in the direction of the design lane is 151,000 considering there are 3 traffic lanes.

$$C_{veh} = \frac{24 \text{ x } 2300 \text{ x } 3}{\left(1 + \left(\frac{\%HV}{100}\right)\left(\mathrm{E_{HV}} - 1\right)\right)} = 151{,}000 \qquad \text{A4}$$

where

C_{veh} = capacity flow in vehicles per daily in direction of the design lane

$\%HV$ = average percentage of heavy vehicles in the direction of the design lane

E_{HV} = the number of passenger car equivalents per heavy vehicle

The maximum annual number of heavy vehicles in the design lane is

$$HV_{max} = \left(\frac{\%HV}{100}\right) x\ LDF\ x\ 365\ x\ 151{,}000 \quad \text{A5}$$

For $\%HV$ = 10% and LDF =0.65, the maximum annual number of HV in the design lane is 3.57 x 10^6 using th conservative assumption that this HV volume occurs every hour of every day of the year.

In this design example it is assumed that the percentage HV and growth do not change during the entire design period and that additional traffic lanes are not constructed. For other situations the above calculations need to be modified.

C.3 Adjusted annual number of heavy vehicles

As seen from Table 12.8, at Year 28 the calculated annual number of HV calculated without capacity constraints exceeds the maximum annual HV volume possible. Hence from Year 28 to the end of the design period, the annual HV volume needs to be limited to the maximum possible annual HV volume of 3.57× 10^6. These corrected annual HV volumes are listed in Table 12.8.

C.4 Cumulative number of heavy vehicles

By summing the annual HV volumes in the design lane corrected for capacity, the cumulative HV volume over the 40 year design period is 1.0 × 10^8 compared to 1.2 × 10^8 when capacity is not considered.

Table 12.8: Heavy vehicle calculations

Year	HV growth rate (%)	Annual number of HV (millions)	Maximum annual HV considering capacity flow rate (millions)	Annual number of HV corrected for capacity (millions)
1	–	1.00	3.57	1.00
2	5	1.05	3.57	1.05
3	5	1.10	3.57	1.10
4	5	1.16	3.57	1.16
5	5	1.22	3.57	1.22
6	5	1.28	3.57	1.28
7	5	1.34	3.57	1.34
8	5	1.41	3.57	1.41
9	5	1.48	3.57	1.48
10	5	1.55	3.57	1.55
11	5	1.63	3.57	1.63
12	5	1.71	3.57	1.71
13	5	1.80	3.57	1.80
14	5	1.89	3.57	1.89
15	5	1.98	3.57	1.98
16	5	2.08	3.57	2.08
17	5	2.18	3.57	2.18
18	5	2.29	3.57	2.29
19	5	2.41	3.57	2.41

Year	HV growth rate (%)	Annual number of HV (millions)	Maximum annual HV considering capacity flow rate (millions)	Annual number of HV corrected for capacity (millions)
20	5	2.53	3.57	2.53
21	5	2.65	3.57	2.65
22	5	2.79	3.57	2.79
23	5	2.93	3.57	2.93
24	5	3.07	3.57	3.07
25	5	3.23	3.57	3.23
26	5	3.39	3.57	3.39
27	5	3.56	3.57	3.56
28	5	3.73	3.57	3.57
29	5	3.92	3.57	3.57
30	5	4.12	3.57	3.57
31	5	4.32	3.57	3.57
32	5	4.54	3.57	3.57
33	5	4.76	3.57	3.57
34	5	5.00	3.57	3.57
35	5	5.25	3.57	3.57
36	5	5.52	3.57	3.57
37	5	5.79	3.57	3.57
38	5	6.08	3.57	3.57
39	5	6.39	3.57	3.57
40	5	6.70	3.57	3.57
Cumulative number of HV (millions)		121		101

Appendix D Characteristics of Traffic at Selected WIM Sites

As described in Chapter 7, the following characteristics are required to calculate the design traffic for a project:

- the average number of axle groups per heavy vehicle (N_{HVAG})
- the Traffic Load Distribution (TLD) for the project.

In the absence of project-specific weigh-in-motion (WIM) data, a presumptive TLD needs to be selected.

This appendix provides characteristics of WIM data obtained at many 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.

The data in Table 12.9 was supplied by road agencies. Note that there are some differences between road agencies in the methods used to identify valid and invalid WIM data and this may have influenced the results. Table 12.9 does not contain Queensland data. Instead, the Queensland Department of Transport and Main Roads (2021) provides a spreadsheet to generate traffic load distributions for pavement structural design purposes. Similarly, other road agencies may provide further data within their design manuals or supplements. The full TLD data for each of the sites listed in Table 12.9 is available at https://austroads.com.au/publications/pavement/agpt02.

Table 12.9: 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							
Jurisdiction	ID of WIM site	Road	Location	Speed limit (km/h)	Direction	Lanes in this direction	Lane surveyed	Survey year	Days in the survey	Lane ADT	% HVs	HVs surveyed	Axle groups per HV	SAST	SADT	TAST	TADT	TRDT	QADT	ESA per HVAG	ESA per HV
ACT	4200/4250	Federal Hwy	Majura ACT	100	N&S	3N3S	ALL	2014	Min 235	17150	10.3	500452	2.77	25.1	38.1	11	18.2	6.3	1.3	0.724	2.01
NSW	202	Bells Line of Road	North Richmond	80	E&W	1E1W	ALL	2015	Min 243	12283	4.4	135262	2.39	40.1	30.7	1.8	23.1	4.1	0.3	0.6	1.43
NSW	DE	Cobb Hwy	Deniliquin	100	N&S	1N1S	ALL	2015	Min 253	1788	17.3	92705	2.91	33.7	16.4	0.7	25.2	23.9	0	1.011	2.94
NSW	271/275	F3 Freeway	HVCS Mt White	110	N&S	1N1S	ALL	2014	Min 342	6269	97.4	2130434	3.02	32.4	9.5	0.7	34.1	23.3	0	0.876	2.65
NSW	800/850	Foreshore Rd	Botany	80	E&W	2E2W	ALL	2015	Min 316	34600	22.1	2689062	2.63	34.2	13.3	3.8	27.3	20	1.4	0.863	2.27
NSW	SH	Golden Hwy	Sandy Hollow	100	WB	1	1	2014	360	1092	17.5	68934	2.83	34.7	18	0.7	28.3	18.3	0	0.691	1.95
NSW	260/261	Great Western Hwy	HVCS Mt Boyce	60	E&W	1E1W	ALL	2014	Min 348	1330	97.6	454564	3.01	32.8	9.8	0.5	42.1	14.8	0.1	0.963	2.9
NSW	WD	Hume Hwy	Wodonga	100	N&S	2N2S	ALL	2014	176	25524	13.9	706162	3.01	32.8	11.5	0.5	28.1	27.2	0	0.926	2.78
NSW	4300	Hume Hwy	Holbrook	110	N&S	2N2S	ALL	2015	Min 275	7627	30.3	817929	3.26	30.5	6.3	0.1	28.4	34.7	0	0.97	3.16
NSW	220/222	Hume Hwy	HVCS Marulan	110	N&S	1N1S	ALL	2014	Min 332	4074	99	1390497	3.26	30.3	5.1	0.4	33.1	31.1	0.1	0.973	3.18
NSW	500	James Ruse Drive	Rosehill	70	N&S	3N3S	ALL	2015	Min 120	49993	7.6	1246135	2.41	39.6	28.2	1.9	21.2	9.1	0	0.679	1.64
NSW	300	M1 Pacific Mwy	Cowan	110	N&S	3N3S	ALL	2015	Min 208	82059	12.1	3123293	3.16	31	11.8	0.7	34	22.5	0	0.849	2.68
NSW	900	M4 Motorway	Eastern Ck	100	E&W	3E3W	ALL	2015	225	124816	14.9	4174389	2.53	37.8	21.8	1.7	27	11.7	0.1	0.64	1.62
NSW	291/292	M5 East Mwy	Kingsgrove	80	E&W	2E2W	ALL	2015	Min 341	110547	12.4	4837521	2.5	39.1	25.1	0.9	22	12.8	0.1	0.618	1.54
NSW	110	M7 Motorway	Prestons	100	N&S	2N2S	ALL	2015	Min 210	65203	17.4	3669523	2.8	34.1	15.3	1.7	31	17.9	0.1	0.907	2.54
NSW	120	M7 Motorway	Eastern Creek	100	N&S	2N2S	ALL	2015	Min 274	70820	15.5	3568068	2.7	34.7	19.4	2.3	29.5	13.9	0.3	1.041	2.81
NSW	130	M7 Motorway	Quakers Hill	100	N&S	2N2S	ALL	2015	Min 223	62825	13	2722810	2.49	38.2	17.4	2	30.9	11.4	0.2	1.059	2.63
NSW	MI	Monaro Hwy	Michelago	100	N&S	1N1S	ALL	2012	Min 270	5057	8.7	120268	2.58	38.3	25.4	0.5	24.5	11.4	0	0.776	2
NSW	700	New England Hwy	Branxton	100	N&S	2N2S	ALL	2015	Min 272	22132	14.4	890498	2.73	35.7	19.5	0.9	26.8	17.1	0	0.803	2.19
NSW	AR	New England Hwy	Armidale	100	N&S	1N1S	ALL	2014	336	3422	18.7	214827	2.76	35.8	19.5	0.5	25.3	18.9	0	0.714	1.97
NSW	TO	Newell Hwy	Tocumwal	80	N&S	1N1S	ALL	2015	Min 121	3395	24.6	112894	3.19	31.1	9.7	0.3	27.4	31.5	0	0.87	2.77
NSW	2600	Newell Hwy	Boggabilla	100	N&S	1N1S	ALL	2015	358	3879	33.3	461960	3.51	28.3	6.8	0.2	27.8	36.9	0	0.923	3.24
NSW	3800	Newell Hwy	Marsden	110	N&S	1N1S	ALL	2015	215	2048	39	171847	3.37	29.5	6.9	0.2	26.7	36.7	0	1.03	3.47
NSW	JE	Newell Hwy	Jerilderie	110	N&S	1N1S	ALL	2015	Min 358	2038	35.3	260475	3.38	29.4	7	0.2	27.6	35.8	0	1.048	3.54

Details of the WIM site										Characteristics of the traffic at the WIM site											
														Percentage distribution of axle group types							
Jurisdiction	ID of WIM site	Road	Location	Speed limit (km/h)	Direction	Lanes in this direction	Lane surveyed	Survey year	Days in the survey	Lane ADT	% HVs	HVs surveyed	Axle groups per HV	SAST	SADT	TAST	TADT	TRDT	QADT	ESA per HVAG	ESA per HV
NSW	CU	Olympic Hwy	Culcairn	100	SB	1	1	2015	271	1522	16.8	69238	2.85	34.8	17.6	0.3	23.3	24	0	0.956	2.72
NSW	CU	Olympic Hwy	Culcairn	100	N&S	1N1S	ALL	2014	Min 153	2912	19.2	133770	2.96	33.5	14.6	0.3	25.7	25.9	0	0.876	2.59
NSW	251	Pacific Hwy	HVCS 12 Mile Creek	100	SB	1	1	2014	352	1215	95.1	406809	3.15	31.3	6.4	0.5	32.4	29.4	0	1.029	3.24
NSW	400	Pacific Hwy	Jones Island	100	N&S	2N2S	ALL	2015	Min 261	16462	13.1	750851	3.18	31.1	7.2	0.4	30.6	30.8	0	1.146	3.64
NSW	283	Pacific Hwy	Brunswick Heads	110	N&S	2N2S	ALL	2015	Min 292	23791	14.2	1162545	2.86	34.5	16.1	0.6	26	22.9	0	0.807	2.3
NSW	203	Pennant Hills Rd	North Parramatta	60	N&S	2N2S	ALL	2014	Min 330	30466	6.6	689090	2.59	37	21.6	1.5	25	14.8	0	0.874	2.27
NSW	4000	Princes Hwy	Falls Ck	100	N&S	2N2S	ALL	2015	Min 168	23451	9	486017	2.27	43	35.9	1	16.6	3.5	0	0.454	1.03
NSW	19000	Putty Rd	East Kurrajong	80	N&S	1N1S	ALL	2014	Min 262	1489	10.1	40494	2.54	38.7	28.3	0.7	27.5	4.9	0	0.789	2
NSW	SM	Sturt Hwy	Merbein South	80	E&W	1E1W	ALL	2015	Min 245	2313	25.7	149421	3.34	29.8	9.2	0.1	27.1	33.8	0	0.955	3.19
NSW	600	Warringah Rd	Forestville	70	N&S	3N3S	ALL	2015	Min 130	69058	5.5	1038240	2.11	46.2	42.8	1.2	8.7	1.1	0	0.483	1.02
NZ	01N00628	SH1N	Tokoroa	100	I	1	1	2023	211	10175	18.6	185246	3.22	16.9	10.4	14.2	40.4	15.6	2.5	0.53	1.71
NZ	01N00628	SH1N	Tokoroa	100	D	1	1	2023	211	10175	18.6	184977	3.22	16.9	10.4	14.2	40.4	15.6	2.5	0.57	1.83
NZ	01N20463	SH1N	Drury	100	I	2	2	2023	364	29747	11.9	1265805	2.97	24.2	13.7	9.5	40.4	10.1	2.2	0.52	1.55
NZ	01N10463	SH1N	Drury	100	D	2	2	2023	364	29013	11.9	1158539	2.97	24.2	13.7	9.5	40.4	10.1	2.2	0.55	1.62
NZ	01S00285	SH1S	Waipara	100	I	1	1	2023	355	9191	17.2	283866	3.19	19.3	15.7	12.1	36.5	14.5	2	0.64	2.03
NZ	01S00285	SH1S	Waipara	100	D	1	1	2023	355	9191	17.2	250712	3.19	19.3	15.7	12.1	36.5	14.5	2	0.41	1.3
NZ	03500321	SH35	Hamanatua Bridge	70	I	1	1	2023	354	5266	5.9	47889	2.55	32.1	30.8	7.1	26.8	3.1	0.1	0.4	1.03
NZ	03500321	SH35	Hamanatua Bridge	70	D	1	1	2023	354	5266	5.9	54165	2.55	32.1	30.8	7.1	26.8	3.1	0.1	0.4	1.03
NZ	00500259	SH5	Eskdale	100	I	1	1	2023	261	4633	22.6	130560	2.95	19.1	12.9	14.8	40.9	11.1	1.2	0.71	2.11
NZ	00500259	SH5	Eskdale	100	D	1	1	2023	364	29013	11.9	1158539	2.97	24.2	13.7	9.5	40.4	10.1	2.2	0.55	1.62
NZ	00210166	SH2	Kairua	100	I	2	2	2023	361	17839	8.4	553502	2.77	22.6	17.8	13.4	35.2	9.2	1.8	0.5	1.38
NZ	00220166	SH2	Kairua	100	D	2	2	2023	361	17542	8.4	491046	2.77	22.6	17.8	13.4	35.2	9.2	1.8	0.55	1.52
NZ	01S00401	SH1S	Rakaia	100	I	1	1	2023	346	14875	15.4	367666	3.13	18.8	15.7	13.2	37.7	12.1	2.5	0.47	1.48
NZ	01S00401	SH1S	Rakaia	100	D	1	1	2023	346	14875	15.4	404200	3.13	18.8	15.7	13.2	37.7	12.1	2.5	0.61	1.9
SA	PIN	Rn 1000 / Stuart Highway	Pimba (5 Km South Of Pimba)	110	North	1	1	2014	320		22.9	31019	4.79	20.7	3.5	0.2	38.9	36.8	0	1.901	9.11

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SA	PIS	Rn 1000 / Stuart Highway	Pimba (5 Km South Of Pimba)	110	South	1	1	2014	320		24	30841	4.8	20.6	3.4	0.2	39.5	36.3	0	1.044	5.01
SA	IKE	Rn 2000 / Eyre Highway	Iron Knob (8.4 Km West Of Intersection With Lincoln Highway)	110	East	1	1	2014	363		27.8	34995	3.98	24.9	3.7	0.2	35.6	35.6	0	1.165	4.64
SA	IKW	Rn 2000 / Eyre Highway	Iron Knob (8.4 Km West Of Intersection With Lincoln Highway)	110	West	1	1	2014	365		30.5	38032	3.96	25	3.9	0.3	35	35.8	0	1.145	4.53
SA	OWN	Rn 3400 / Barrier Highway	Oodla Wirra (5 Km North Of Oodla Wirra)	110	North	1	1	2014	363		30	30472	3.86	25.7	3.8	0.2	34.3	36	0	1.115	4.31
SA	OWS	Rn 3400 / Barrier Highway	Oodla Wirra (5 Km North Of Oodla Wirra)	110	South	1	1	2014	363		30.1	30921	3.88	25.6	3.1	0.2	34.4	36.6	0	1.294	5.02
SA	MOE	Rn 4500 / S.E.Highway	Monarto (11.4 Km East Of Callington - One Lane)	110	East	2	Kerb lane	2014	351		16.1	271042	3.21	30.9	6.3	0.3	31.4	31.1	0	1.054	3.38
SA	MOW	Rn 4500 / S.E.Highway	Monarto (11.4 Km East Of Callington - One Lane)	110	West	2	Kerb lane	2014	353		14.1	247798	3.18	31.3	8.1	0.2	29.4	31	0	1.094	3.47
SA	WI1	Rn 5424 / Portriver Expressway	Wingfield <Lane 1>	90	East	2	Kerb lane	2014	333		14.4	456302	2.95	32.7	8.4	1.2	34.2	23.5	0	1.052	3.1
SA	WI4	Rn 5424 / Portriver Expressway	Wingfield <Lane 4>	90	West	2	Kerb lane	2014	348		13.2	472107	2.87	33.6	9.9	1.3	33.8	21.5	0	0.989	2.84
SA	WI2	Rn 5424 / Portriver Expressway	Wingfield <Lane 2>	90	East	2	Median lane	2014	356		2.9	63638	2.99	32.6	14.5	0.9	30.2	21.8	0	0.834	2.49
SA	WI3	Rn 5424 / Portriver Expressway	Wingfield <Lane 3>	90	West	2	Median lane	2014	355		5.3	115473	3.08	31.7	8.6	0.8	33.7	25.2	0	0.922	2.84
SA	TEB	Rn 7200 / Sturt Highway	Truro (6.5 Km West Of Truro)	110	East	2	Overtaking lane	2014	358		7.4	15263	3.47	28.5	4.6	0.3	33.1	33.5	0	1.224	4.25
SA	BTE	Rn 7800 / Dukes Highway	Bordertown (Western End Of By-Pass)	110	East	1	1	2014	174		28.4	70156	3.45	28.9	3.4	0.1	30.7	36.9	0	1.007	3.48
SA	BTW	Rn 7800 / Dukes Highway	Bordertown (Western End Of By-Pass)	110	West	1	1	2014	172		29.9	74504	3.47	28.7	3	0.1	30.5	37.7	0	1.236	4.29

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SA	NAN	Rn 8000 / Riddoch Highway	Naracoorte (20 Km North Of Naracoorte)	110	North	1	1	2014	358		12.8	45514	3.17	31.4	7.3	0.2	31.2	29.9	0	1.138	3.6
SA	NAS	Rn 8000 / Riddoch Highway	Naracoorte (20 Km North Of Naracoorte)	110	South	1	1	2014	364		17	61688	3.21	31.1	8.1	0.1	29.9	30.7	0	0.813	2.61
VIC	gis	Calder Fwy	Macedon Ranges	110	N	2	1	2016	362	8686	8.1	256166	2.8	34.2	12.8	0.9	31	21.1	0	0.72	2.05
VIC	gss	Calder Fwy	Macedon Ranges	110	S	2	1	2018	245	9533	7.7	178747	2.8	34.3	14.8	0.9	28.7	21.1	0.1	0.916	2.59
VIC	gsf	Calder Fwy	Macedon Ranges	110	S	2	2	2019	307	3207	1.8	17613	2.9	33.5	28.6	0.7	22.4	14.8	0	0.629	1.84
VIC	csn	Calder Hwy	Loddon	100	N	1	1	2023	359	1928	13.5	93209	3.3	29.3	6.3	0.6	29.8	33.7	0.2	0.533	1.77
VIC	css	Calder Hwy	Loddon	100	S	1	1	2023	355	1964	13.8	96391	3.4	29.1	5.5	0.6	29.4	35.1	0.2	0.762	2.57
VIC	ton	Goulburn Valley Hwy	Berrigan	80	N	1	1	2017	365	1872	21.2	145214	3.3	29.3	7.5	0.4	29.7	33	0	0.747	2.5
VIC	tos	Goulburn Valley Hwy	Berrigan	80	S	1	1	2017	364	1878	19.2	131109	3.4	28.5	4	0.4	31.4	35.5	0	0.715	2.47
VIC	gfn	Goulburn Valley Hwy	Strathbogie	100	N	2	1	2016	343	3069	11.2	117939	3.3	29.8	5.9	0.5	30.1	33.7	0	0.831	2.73
VIC	gn2	Goulburn Valley Hwy	Strathbogie	100	N	2	2	2018	315	395	2.7	3339	3.2	30.8	8	0.5	28.4	32.3	0	0.723	2.31
VIC	gs2	Goulburn Valley Hwy	Strathbogie	100	S	2	2	2023	335	630	10	21082	3.4	28.7	6.3	0.8	29.9	34.2	0.2	0.638	2.17
VIC	gfs	Goulburn Valley Hwy	Strathbogie	100	S	2	1	2019	347	3207	15.4	171487	3.3	29.7	5.1	0.5	30.1	34.5	0.1	0.654	2.18
VIC	wan	Hume Fwy	Mitchell	100	N	2	1	2018	337	9339	23.3	734504	3.3	30.1	6.6	0.3	29.9	32.9	0.1	0.879	2.89
VIC	waf	Hume Fwy	Mitchell	100	N	2	2	2018	342	3083	4.4	46904	3.2	30.8	14.1	0.2	26.4	28.4	0.1	0.796	2.56
VIC	wal	Hume Fwy	Mitchell	100	S	2	1	2018	301	9288	23	641931	3.3	30.1	6.2	0.3	29.8	33.6	0.1	0.87	2.86
VIC	wsf	Hume Fwy	Mitchell	100	S	2	2	2023	339	3813	2.8	36296	3.4	29.3	7.2	0.4	29.6	33.1	0.3	0.624	2.1
VIC	hsg	Hume Fwy	Wangaratta	100	S	2	1	2023	347	3025	5.4	56975	3.3	30.1	8.6	0.6	29.2	31.1	0.4	0.768	2.5
VIC	wd1	Hume Fwy	Wodonga City	110	N	2	1	2017	363	10871	15.4	607699	3.2	31	6.6	0.7	30.4	31.3	0.1	0.898	2.83
VIC	wd2	Hume Fwy	Wodonga City	110	N	2	2	2017	363	3881	5	70144	3.2	30.9	6	0.4	30.4	32	0.2	0.849	2.71

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VIC	wd3	Hume Fwy	Wodonga City	110	S	2	1	2017	363	12603	14.4	660398	3.2	30.8	6.2	0.6	30.3	32	0.1	0.793	2.52
VIC	me1	Monash Fwy	Greater Dandenong	100	E	4	1	2023	325	18747	6.1	369533	2.5	36.9	18.2	2.2	29.7	12.6	0.5	0.455	1.16
VIC	me2	Monash Fwy	Greater Dandenong	100	E	4	2	2023	325	19708	7.5	480093	2.7	35.4	15.2	1.8	31.1	15.9	0.5	0.647	1.75
VIC	me3	Monash Fwy	Greater Dandenong	100	E	4	3	2023	354	18795	12.2	814069	2.4	41.1	33.1	1.1	17.3	7.3	0.1	0.455	1.08
VIC	me4	Monash Fwy	Greater Dandenong	100	E	4	4	2022	361	18633	11	740864	2.4	40.7	32.7	1	18.1	7.4	0	0.354	0.84
VIC	mw1	Monash Fwy	Greater Dandenong	100	W	4	1	2023	342	21799	10.6	791239	2.3	42.9	36.1	1.3	15.6	4.1	0	0.384	0.87
VIC	mw2	Monash Fwy	Greater Dandenong	100	W	4	2	2023	343	19778	12.9	873536	2.4	40.5	30.5	1.3	19	8.4	0.3	0.3	0.73
VIC	mw3	Monash Fwy	Greater Dandenong	100	W	4	3	2023	341	17360	13.4	792326	2.5	39.5	28.6	1.1	19.7	10.8	0.3	0.49	1.2
VIC	mw4	Monash Fwy	Greater Dandenong	100	W	4	4	2023	342	17787	12.6	764584	2.5	38.7	27.7	1	20.8	11.6	0.1	0.344	0.87
VIC	ge1	Princes Fwy West	Wyndham	100	E	3	1	2017	364	10136	14.3	528121	2.8	34.9	15.7	1.2	30.4	17.6	0.2	0.846	2.35
VIC	ge2	Princes Fwy West	Wyndham	100	E	3	2	2023	329	15717	7.9	407753	2.9	33	12.3	1.3	31.9	21.2	0.3	1.007	2.94
VIC	gw1	Princes Fwy West	Wyndham	100	W	3	1	2023	357	9899	14	493811	2.8	34.1	14.1	2.1	32	17.4	0.3	0.638	1.76
VIC	gw2	Princes Fwy West	Wyndham	100	W	3	2	2023	357	16153	8.2	474877	2.9	33.3	13.6	1.3	31.6	19.9	0.3	0.626	1.8
VIC	ple	Princes Hwy East	Baw Baw	110	E	2	1	2017	162	10604	4.9	83489	2.8	34.4	13.5	1	33.4	17.6	0.2	0.662	1.87
VIC	plf	Princes Hwy East	Baw Baw	110	E	2	2	2023	353	5273	2.8	53011	3	32.7	15.7	1.1	32.3	18.1	0.1	0.677	2
VIC	pfy	Princes Hwy East	Baw Baw	110	W	2	1	2023	322	9352	9.6	290170	2.9	33.5	14.2	1.4	33.3	17.4	0.2	0.738	2.12
VIC	pff	Princes Hwy East	Baw Baw	110	W	2	2	2023	322	4581	1.7	25311	2.9	33.5	22.4	1.2	28.5	14.3	0.1	0.551	1.59
VIC	pws	Princes Hwy West	Glenelg	100	S	1	1	2023	352	1601	18.8	105955	3.7	26.5	6.8	0.4	26.3	39.6	0.2	0.892	3.3
VIC	sme	Sturt Hwy	Merbein	100	E	1	1	2022	365	1222	20.3	90733	3.7	26.7	3.8	0.5	29.9	39.1	0.1	0.641	2.36
VIC	smw	Sturt Hwy	Merbein	100	W	1	1	2017	301	1211	24	87436	3.5	28	5.1	0.3	28.6	38	0	0.747	2.65
VIC	51-1	Western Fwy	Moorabool	110	W	2	1	2019	324	8131	14.3	377346	3.3	28.6	14.3	2.2	28.8	25.2	1	0.687	2.24
VIC	wme	Western Hwy	Pyrenees	100	E	2	1	2018	73	2944	23.4	50363	3.2	31	8.5	0.2	27.8	32.5	0	0.925	2.97
VIC	wmf	Western Hwy	Pyrenees	100	E	2	2	2022	365	917	3.3	11085	3.5	28.6	4.6	0.2	31	35.6	0	0.651	2.26
VIC	46-1	Western Ring Rd	Brimbank	100	N	4	1	2017	365	26058	13	1233485	2.7	35.2	14	1.6	30.8	18.1	0.2	0.958	2.61
VIC	46-2	Western Ring Rd	Brimbank	100	N	4	2	2017	364	25814	8.3	784004	2.7	35.8	14.9	1.4	30.8	17	0.1	0.976	2.62
VIC	46-3	Western Ring Rd	Brimbank	100	N	4	3	2018	371	18559	4.9	337625	2.8	34.8	16.4	1.3	30.7	16.7	0.1	1.232	3.41

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VIC	45-1	Western Ring Rd	Brimbank	100	S	4	1	2019	270	9309	16.9	424042	2.7	34.9	11.1	2	32.8	19	0.1	0.721	1.95
VIC	45-2	Western Ring Rd	Brimbank	100	S	4	2	2018	347	21711	9.5	717910	2.7	35.7	13.7	1.3	31	18	0.3	0.801	2.16
VIC	45-3	Western Ring Rd	Brimbank	100	S	4	3	2019	275	23121	7.1	448523	2.7	35.1	14.5	1.5	31.1	17.6	0.2	0.698	1.91
VIC	45-4	Western Ring Rd	Brimbank	100	S	4	4	2018	347	17142	5.6	335592	2.9	33.5	13.7	1.4	32.9	18.4	0.1	0.745	2.13
VIC	wm1	Western Ring Rd	Moreland	100	E	4	1	2018	364	13872	13.1	658939	2.6	37.4	15.5	1.7	29.2	16.2	0.1	0.72	1.84
VIC	wm2	Western Ring Rd	Moreland	100	E	4	2	2018	364	23316	13.8	1168276	2.8	34.7	13.1	1.2	29.4	21.4	0.1	1.112	3.1
VIC	wm4	Western Ring Rd	Moreland	100	W	4	1	2018	365	13869	3.9	196056	2.8	34.6	14	1.1	31.8	18.3	0.1	1	2.8
VIC	wm3	Western Ring Rd	Moreland	100	W	4	2	2018	365	21611	9.2	725961	2.8	34.3	12.5	1.1	30.7	21.3	0.1	0.978	2.76
WA	H050	Geraldton-Mt Magnet Rd	SLK8.43, Geraldton	110	West	1	1	2007	990	594	19.7	115880	3.81	23.1	10.9	3.1	25.9	36.9	0	0.977	3.73
WA	H005	Great Eastern Highway	SLK102.66, Northam	110	East	1	1	2007	649	995	26.8	173131	3.54	27.1	9.1	1.1	32.7	30	0	0.918	3.25
WA	H006	Great Northern Highway	SLK35, Muchea	110	North	1	1	2006	554	3098	21.8	374205	3.64	24.3	10.3	3.1	32.3	29.8	0.1	0.986	3.59
WA	H006	Great Northern Highway	SLK30, Bullsbrook	110	South	1	1	2007	232	3857	18.8	168524	3.6	24.5	9.7	3.2	32.8	29.6	0.1	1.118	4.02
WA	H015	Kwinana Freeway	SLK56.84, Mandurah	100	South	2	1	2011	58	10663	10.1	62623	2.76	33.8	20.7	2.4	24.6	18.5	0	1.031	2.85
WA	H015	Kwinana Freeway	SLK69.05, Pinjarra	110	South	2	1	2011	59	6683	11.7	46252	2.98	30.6	16.1	3	27.1	23.2	0	0.957	2.85
WA	H021	Reid Highway	SLK22.65, Middle Swan	70	East	2	1	2011	59	7031	11.9	49323	2.45	38.1	24.9	2.6	23.5	10.9	0	0.773	1.9
WA	H018	Roe Highway	SLK13.03, Jandakot	100	West	2	1	2007	61	11916	13.2	95221	2.63	35.5	20	2.5	25.8	16.1	0	0.756	1.99
WA	H008	South Coast Highway	SLK468.4, Esperance	110	East	1	1	2007	432	485	21.2	44391	4.14	20.1	8.3	4.1	24.5	43	0.1	1.392	5.77
WA	H009	South Western Highway	SLK204.79, Kirup	110	West	1	1	2007	942	1396	16.5	216959	3.57	24.8	10.4	3.2	35.7	25.7	0.1	1.455	5.19
WA	H009	South Western Highway	SLK79.29, Waroona	110	South	1	1	2007	579	2930	14	237305	3.13	29.5	14.5	2.5	30.7	22.7	0.1	0.921	2.88
WA	Kunu	Victoria Highway	Kununurra	110	West	1	1	2006	200	309	15.9	9828	4.04	24.5	11.7	0.2	33.8	29.7	0	1	4.04

Appendix E Adjustment of Design Traffic for Anticipated Increases in Load Magnitude

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:

$$X \times N_{DT} \times P_i \times P_{ij} \text{ and } L_{ij}$$

where X is that fraction of the cumulative number of axle groups (N_{DT}) which traverse the pavement during the first Q years, P_i is the proportion of all axle groups which are of type i, and P_{ij} is the proportion of axle group type i which has load magnitude L_{ij}.

The second listing is of the following pairs of quantities:

$$(1-X) \times N_{DT} \times P_i \times P_{ij} \text{ and } (1+0.01H) \times L_{ij}$$

For the case where there is uniform growth in the volume of traffic throughout the design period of R% per annum, then (Equation A9).

$$X = \frac{(1+0.01R)^Q - 1}{(1+0.01R)^P - 1} \qquad \text{A6}$$

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 (Equation A7).

$$N_{DT} \times P_i \times P_{ij} \text{ and } L_{ij} \qquad \text{A7}$$

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 (Equation A8).

$$X_i \times N_{DT} \times P_i \times P_{ij} \text{ and } L_{ij}$$
$$\text{and } (1 - X_i) \times N_{DT} \times P_i \times P_{ij} \text{ and } (1 + 0.01H_i) \times L_{ij} \qquad \text{A8}$$

where

i = the values for those axle group types whose loads are anticipated to increase

X_i = that fraction of the cumulative number of type i axle groups which traverse the pavement during the first Q years

H_i = the anticipated % increase in the loads on axle group type i after the Q^{th} year

Appendix F Traffic Load Distribution

This appendix provides an example traffic load distribution to assist in explaining the design methods in this Part.

Table 12.10: Example traffic load distribution

Axle group load (kN)	Axle group type				
	SAST %	SADT %	TAST %	TADT %	TRDT %
10	0.2804	3.4730	0.0354	0.1444	0.0050
20	7.8270	8.6960	0.2377	0.5755	0.1568
30	15.4600	23.4600	0.2763	0.6242	0.3290
40	15.7100	21.9300	0.5755	1.9770	1.3170
50	29.9400	16.8000	2.8890	6.4960	4.1670
60	23.2900	9.6060	10.2700	9.5110	7.4190
70	6.5020	6.5000	16.8100	10.9400	9.7770
80	0.7943	4.6230	16.6100	9.7690	8.3380
90	0.1087	2.9690	15.9500	7.6110	6.1500
100	0.0354	1.3930	14.4200	7.2420	5.0290
110	0.0174	0.4098	9.7740	6.2670	3.7010
120	0.0174	0.1158	5.9030	5.9520	3.2980
130	0.0174	0.0244	2.9430	5.8780	3.1470
140			1.5390	6.5340	3.3610
150			0.8439	8.0300	4.0080
160			0.4279	5.7170	4.1150
170			0.2308	3.5540	4.8190
180			0.1367	1.8630	6.0970
190			0.0723	0.8535	7.7330
200			0.0555	0.3331	8.4330
210				0.0801	5.1360
220				0.0322	2.3390
230				0.0160	0.7764
240					0.2503
250					0.0905
260					0.0080
270					
280					
290					
300					
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

Measure	Value
N_{HVAG}	2.5
ESA/HVAG	0.7
ESA/HV	1.8

Appendix G Pavement Damage in Terms of Equivalent Standard Axles

G.1 Evaluation of number of equivalent standard axle (ESA) repetitions per axle group

The design traffic used in the empirical design of unbound granular pavements with thin bituminous surfacings (Figure 8.4) is expressed in terms of ESAs. Additionally, when considering rutting and loss of surface shape in the mechanistic-empirical design of pavements the design traffic is considered also in units of ESAs (Chapter 5). The consideration of the post-cracking phase of cemented materials and lean-mix concretes is similarly conducted using units of ESAs.

The damage caused by a specific axle group type-load combination can be expressed as follows (Equation A9).

$$DAMAGE_{ij} = \frac{DAMAGE_{ij}}{DAMAGE_{SA}} \times DAMAGE_{SA} \qquad \text{A9}$$

$$= ESA_{ij} \times DAMAGE_{SA}$$

where

$DAMAGE_{ij}$	=	damage caused by axle group type i with load level L_{ij}
$DAMAGE_{SA}$	=	damage caused by the Standard Axle
ESA_{ij}	=	the number of Equivalent Standard Axle repetitions (or passages of the Standard Axle) which causes the same amount of damage as a single passage of axle group type i with load level L_{ij}

The term $DAMAGE_{SA}$ is directly evaluated within the pavement design procedures. ESA_{ij} is evaluated – for each axle group type i with load level L_{ij} – in the manner now described.

Because the overall pavement damage and rutting and loss of surface shape 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 each axle group type the load which is considered to cause the same damage as the Standard Axle. These loads are listed in Table 7.7 and Table 7.8.

Denoting this axle group load (which causes the same damage as a Standard Axle) as the axle group's Standard Load (SL), $ESA_{ij}$is evaluated as follows (Equation A10).

$$ESA_{ij} = \left(\frac{L_{ij}}{SL_i}\right)^4 \qquad \text{A10}$$

where

ESA_{ij}	=	the number of Equivalent Standard Axle repetitions (or passages of the Standard Axle) which causes the same amount of damage as a single passage of axle group type i with load level L_{ij}
SL_i	=	Standard Load for axle group type i (from Table 7.7 and Table 7.8)
L_{ij}	=	j^{th}load magnitude on the axle group type i

From the design Traffic Load Distribution (TLD) the average number of ESA per heavy vehicle axle group (ESA/HVAG) can be readily determined (Equation A11).

$$ESA/HVAG = \sum_{all\ i,j} \left[P_i \times P_{ij} \times \left(\frac{L_{ij}}{SL_i} \right)^4 \right] \quad \text{A11}$$

where

$ESA/HVAG$	=	average number of Equivalent Standard Axles per Heavy Vehicle Axle Group for the TLD
P_i	=	proportion of all axle groups within the TLD which are of axle group type i
P_{ij}	=	proportion of axle group type i which have loads of magnitude L_{ij}
L_{ij}	=	j^{th} load magnitude in the distribution of loads begin carried by axle group type i
SL_i	=	Standard Load for axle group type i (from Table 7.7 and Table 7.8)

G.2 Specification of design traffic loading and its calculation

The design traffic used in the empirical design of unbound granular pavements with thin bituminous surfacings and when considering rutting and loss of surface shape in the mechanistic-empirical design of pavements can now be formally stated as follows:

> *The design traffic loading for empirical flexible pavement design and for considering rutting and loss of pavement surface shape in mechanistic-empirical flexible pavement design is the cumulative traffic over the design period expressed in terms of the number of Equivalent Standard Axle (ESA) repetitions which causes the same damage as the cumulative traffic.*

The designer can readily determine the design number of ESAs, DESA, as follows (Equation A12).

$$DESA = N_{DT} \times ESA/HVAG \quad \text{A12}$$

where

$DESA$	=	design number of Equivalent Standard Axles
$ESA/HVAG$	=	average number of Equivalent Standard Axles per Heavy Vehicle Axle Group for the TLD
N_{DT}	=	cumulative heavy vehicle axle groups traversing the design lane during the design period

Appendix H Example of Design Traffic Calculations

Design parameters:

Design period:	20 years
Annual Average Daily Traffic (AADT):	5350
Direction factor:	0.5
Percentage heavy vehicles:	4%
Lane distribution factor:	1.0
Heavy vehicle growth rate (compound):	4%
Traffic project load distribution:	Table 12.11

H.1 Total number of heavy vehicle axle groups

The initial daily heavy vehicles in the design lane is (Equation A13).

$$N_i = AADT \times DF \times \%HV/100 \times LDF \tag{A13}$$

where

N_i = initial daily heavy vehicles traversing the design lane (Section 7.4.4)

$AADT$ = 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 heavy vehicles (Section 7.4.4)

LDF = Lane Distribution Factor (Section 7.4.3)

From the above-mentioned design parameters:

$AADT$ = 5350

DF = 0.5

$\%HV$ = 4%

LDF = 1.0

Hence,

$$N_i = 5350 \times 0.5 \times 4/100 \times 1.0 = 107 \text{ heavy vehicles}$$

Assuming below capacity traffic volumes throughout the design period, the cumulative heavy vehicles traversing the design lane during the design period is (Equation A14).

$$N_{HV} = 365 \times CGF \times N_i \quad \text{A14}$$

where

N_{HV} = cumulative number of heavy vehicles traversing the design lane during the design period (Section 7.4.5)

CGF = cumulative growth factor (Section 7.4.5)

N_i = initial daily heavy vehicles traversing the design lane (Section 7.4.4)

The Cumulative Growth Factor (CGF) is calculated as follows using R the annual growth rate (4%) and the Design Period P (20 years) (Equation A15).

$$\text{Cumulative Growth Factor } (CGF) = \frac{(1 + 0.01 \times R)^P - 1}{0.01 \times R} \quad \text{for } R > 0 \quad \text{A15}$$

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

Hence,

$$CGF = \frac{(1+0.01\times4)^{20}-1}{0.01\times4} = 29.8$$

The cumulative heavy vehicles traversing the design lane during the design period is, therefore,

$$N_{HV} = 365 \times 29.8 \times 107 = 1.16 \times 10^6 \text{ heavy vehicles}$$

As this number is considerably below 10^7, a check need not be made to determine whether the volume of heavy vehicles exceeds the capacity of the design lane (Section 7.4.5).

The cumulative number of heavy vehicle axle groups over the design period is calculated using Equation A16.

$$N_{DT} = N_{HV} \times N_{HVAG} \quad \text{A16}$$

where

N_{DT} = cumulative number of heavy vehicle axle groups in the design lane during the design period (Section 7.4.7)

N_{HV} = cumulative number of heavy vehicles traversing the design lane during the design period (Section 7.4.5)

N_{HVAG} = average number of axle groups per heavy vehicle (Section 7.4.7)

As stated in Section 7.4.7, the average number of axle groups per heavy vehicle (N_{HVAG}) may be obtained from:

- weigh-in-motion survey data
- vehicle classification counts
- presumptive values (e.g. Appendix D).

In this example weigh-in-motion data given in Table 12.11 is available to calculate N_{HVAG}.

From Table 12.11, 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. That is, on average, each 100 HVAG of loading includes 40 steer axles. To estimate the 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 HVAG of loading result from the passage of 40 heavy vehicles. Thus, the average number of axle groups per heavy vehicle is 100/40 = 2.5.

Using Equation A16, the cumulative heavy vehicle axle groups is calculated as:

$$N_{DT} = 1.16 \times 10^6 \times 2.5 = 2.9 \times 10^6 \text{ HVAGs}$$

H.2 Design traffic for flexible pavements

H.2.1 Estimating equivalent standard axles per heavy vehicle axle group

The Design Traffic for use in the empirical design of unbound granular pavements with thin bituminous surfacings (Section 7.6.1) is expressed in terms of the number of Equivalent Standard Axles (ESA). When considering rutting and loss of surface shape in the mechanistic-empirical design procedure, the Design Traffic is also expressed in units of ESA.

To calculate the ratio ESA/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 ESA/HVAG values are calculated as follows:

- The Traffic Load Distribution (TLD) for the project (Table 12.11) is used to calculate the proportion of axles of each axle group type and load. These proportions are given in Table 12.12.
- The ESA of overall pavement and deformation damage caused by each axle group type is then calculated using Equation 37. These ESA damage values are given in Table 12.13.
- The average ESA/HVAG is then calculated by multiplying the ESA values for each axle group and of each axle group type (Table 12.13) by its frequency of occurrence (Table 12.12). These values are given in Table 12.14. The average ESA/HVAG is the sum of the weighted ESA values in this table – i.e. 0.80.

Table 12.11: Project traffic load distribution

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		0.0000	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				0.0000	0.0254
340				0.0045	0.0181
350					
Proportion of each axle group	0.3799	0.2171	0.0216	0.2591	0.1223

Table 12.12: Project traffic 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	0.004731	0.000022	0.000252	0.000005
20	0.051391	0.022213	0.000206	0.001761	0.000129
30	0.068445	0.044885	0.000271	0.003650	0.000309
40	0.075951	0.039061	0.000288	0.009748	0.001275
50	0.097778	0.029135	0.000975	0.020016	0.006018
60	0.065016	0.018018	0.002950	0.026727	0.011542
70	0.016605	0.013604	0.003877	0.026491	0.011978
80	0.002921	0.016667	0.003750	0.022171	0.010536
90	0.000449	0.013838	0.002858	0.017514	0.007981
100	0.000218	0.007770	0.002143	0.013841	0.005316
110	0.000049	0.003654	0.002110	0.011351	0.003817
120	0.000049	0.001990	0.001006	0.010748	0.003303
130	0.000033	0.000945	0.000502	0.011120	0.003025
140	0.000020	0.000410	0.000258	0.012213	0.003235
150	0.000000	0.000106	0.000188	0.015935	0.003776
160	0.000000	0.000054	0.000071	0.014805	0.004181
170	0.000000	0.000000	0.000067	0.012888	0.004654
180	0.000000	0.000017	0.000027	0.008809	0.006046
190	0.000000	0.000000	0.000022	0.006839	0.007627
200	0.000000	0.000000	0.000006	0.004416	0.008828
210	0.000000	0.000000	0.000000	0.003094	0.006405
220	0.000000	0.000000	0.000000	0.002149	0.004531
230	0.000000	0.000000	0.000000	0.001094	0.002470
240	0.000000	0.000000	0.000000	0.000547	0.001773
250	0.000000	0.000000	0.000000	0.000420	0.001095
260	0.000000	0.000000	0.000000	0.000195	0.000737
270	0.000000	0.000000	0.000000	0.000195	0.000762
280	0.000000	0.000000	0.000000	0.000035	0.000374
290	0.000000	0.000000	0.000000	0.000024	0.000239
300	0.000000	0.000000	0.000000	0.000000	0.000198
310	0.000000	0.000000	0.000000	0.000029	0.000050
320	0.000000	0.000000	0.000000	0.000012	0.000031
330	0.000000	0.000000	0.000000	0.000000	0.000031
340	0.000000	0.000000	0.000000	0.000012	0.000022
350	0.000000	0.000000	0.000000	0.000000	0.000000
	0.3799	0.2171	0.0216	0.2591	0.1223

Table 12.13: ESA for each axle group load of each axle group type

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	0.003	0.000	0.000
30	0.103	0.020	0.013	0.002	0.001
40	0.324	0.063	0.041	0.008	0.002
50	0.792	0.153	0.100	0.019	0.006
60	1.642	0.316	0.207	0.039	0.012
70	3.043	0.586	0.383	0.072	0.022
80	5.191	1.000	0.653	0.123	0.037
90	8.315	1.602	1.046	0.198	0.060
100	12.673	2.441	1.594	0.301	0.091
110	18.555	3.574	2.334	0.441	0.133
120	26.280	5.063	3.305	0.624	0.189
130	36.197	6.973	4.552	0.860	0.260
140	48.687	9.379	6.123	1.157	0.350
150	64.160	12.360	8.069	1.524	0.461
160	83.057	16.000	10.445	1.973	0.597
170	105.85	20.391	13.312	2.515	0.761
180	133.04	25.629	16.731	3.160	0.957
190	165.16	31.817	20.771	3.924	1.188
200	202.78	39.063	25.501	4.817	1.458
210	246.48	47.481	30.997	5.855	1.773
220	296.88	57.191	37.336	7.053	2.135
230	354.66	68.321	44.602	8.425	2.551
240	420.48	81.000	52.879	9.989	3.024
250	495.06	95.367	62.259	11.760	3.560
260	579.15	111.57	72.834	13.758	4.165
270	673.52	129.75	84.702	16.000	4.844
280	778.98	150.06	97.965	18.505	5.602
290	896.37	172.68	112.73	21.294	6.446
300	1026.6	197.75	129.10	24.387	7.382
310	1170.4	225.47	147.19	27.804	8.417
320	1328.9	256.00	167.12	31.569	9.557
330	1503.0	289.53	189.01	35.704	10.809
340	1693.6	326.25	212.99	40.233	12.180
350	1901.8	366.36	239.17	45.179	13.677

Table 12.14: ESA for each axle group load of each axle group type

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	0.007	0.001	0.000	0.000	0.000
40	0.025	0.002	0.000	0.000	0.000
50	0.077	0.004	0.000	0.000	0.000
60	0.107	0.006	0.001	0.001	0.000
70	0.051	0.008	0.001	0.002	0.000
80	0.015	0.017	0.002	0.003	0.000
90	0.004	0.022	0.003	0.003	0.000
100	0.003	0.019	0.003	0.004	0.000
110	0.001	0.013	0.005	0.005	0.001
120	0.001	0.010	0.003	0.007	0.001
130	0.001	0.007	0.002	0.010	0.001
140	0.001	0.004	0.002	0.014	0.001
150	0.000	0.001	0.002	0.024	0.002
160	0.000	0.001	0.001	0.029	0.002
170	0.000	0.000	0.001	0.032	0.004
180	0.000	0.000	0.000	0.028	0.006
190	0.000	0.000	0.000	0.027	0.009
200	0.000	0.000	0.000	0.021	0.013
210	0.000	0.000	0.000	0.018	0.011
220	0.000	0.000	0.000	0.015	0.010
230	0.000	0.000	0.000	0.009	0.006
240	0.000	0.000	0.000	0.005	0.005
250	0.000	0.000	0.000	0.005	0.004
260	0.000	0.000	0.000	0.003	0.003
270	0.000	0.000	0.000	0.003	0.004
280	0.000	0.000	0.000	0.001	0.002
290	0.000	0.000	0.000	0.001	0.002
300	0.000	0.000	0.000	0.000	0.001
310	0.000	0.000	0.000	0.001	0.000
320	0.000	0.000	0.000	0.000	0.000
330	0.000	0.000	0.000	0.000	0.000
340	0.000	0.000	0.000	0.000	0.000
350	0.000	0.000	0.000	0.000	0.000
				Average ESA/HVAG	0.80

H.2.2 Design traffic loading calculation in ESA

The design traffic loading for the design of unbound granular pavements with thin bituminous surfacings using the empirical design procedure, and for the consideration of rutting and loss of surface shape in the mechanistic-empirical design procedure, is the total number of ESA during the design period that causes the same damage as the cumulative traffic. The design number of ESA loading, DESA, is calculated using Equation 38.

$$DESA = ESA/HVAG \times N_{DT}$$

$$= 0.8 \times 2.9 \times 10^6$$

$$= 2.3 \times 10^6$$

H.2.3 Design traffic loading calculation for bound materials

When considering the damage to asphalt, cemented material and lean-mix concrete using the mechanistic-empirical design procedure, the design traffic 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 group load for each group type.

Consequently, the design traffic for considering the damage to asphalt, cemented material and lean-mix concrete using the mechanistic-empirical design procedure comprises:

- cumulative HVAG, 2.9×10^6 (Appendix H.1)
- the project Traffic Load Distribution (Table 12.11).

H.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, to consider reliability a Load Safety Factor is used to change 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, Appendix H.1)
- the project Traffic Load Distribution (Table 12.11), with axle group loads increased by the selected Load Safety Factor for the desired project reliability (Section 9.3.6).

Appendix I Procedures for Evaluation of Pavement Damage Due to Specialised Vehicles

I.1 Introduction

This appendix provides guidance on the damage to road pavements caused by the passage of vehicle loads that do not comply with prescriptive regulations for heavy vehicles in the areas of mass, dimensions, and configurations. Such vehicles may include mobile cranes and specialised vehicles with unusual configurations of axle wheel loads, tyre loads and types. Such damage assessments may be used to assess whether a permit should be issued for the use of the vehicle on the road network.

In undertaking the damage evaluation, due consideration needs to be given to the different pavement configurations anticipated to be trafficked by the specialised vehicle as the damaging effect varies with pavement type and distress mode. Usually the assessment would include the complete range of pavement types expected in-service along the route defined by the specialised vehicle permit application.

As pavement damage varies with pavement type and distress mode, guidance is given below of the procedure to use according to the pavement type.

I.2 Granular pavements with thin bituminous surfacings

The empirical Scala and Potter (1981) deflection-based method was developed from field deflection measurements of bituminous surfaced granular pavements. A review of alternative assessment methods by Jameson (2004) recommended it be used for granular pavements with thin bituminous surfacings.

The Scala and Potter method is simple to use and equations have been fitted (Jameson 2004) to the Scala and Potter design chart which enable the procedure to be completed using spreadsheets. A minor modification to the Equation for L_{TO} has been made such that the procedure calculates a Load Equivalence Factor of 1 for a Standard Axle (Chapter 8) fitted with commonly used 11R22.5 tyres. (The Load Equivalence Factor is the ESA of pavement damage due to a single pass of an axle). Note that an 11R22.5 tyre is a radial (R) tyre with a nominal tyre section width of 11 inches (279 mm) and a nominal rim diameter of 22.5 inches (572 mm). This is the most commonly used Australian heavy vehicle tyre.

The following steps are used to estimate the ESA of damage due to a single pass of a specialised vehicle, the so-called Load Equivalence Factor (LEF):

1. Divide the vehicle into individual single axles and estimate the load on each axle (L_A).
2. Estimate the tyre contact width (TCW) for each tyre on each axle in millimetres using the nominal tyre section width ($NTSW$) and the following relationships (Equation A17 and Equation A18).

if $NTSW$ < 300 mm: $$TCW = 2.1705NTSW - 0.002115NTSW^2 - 217 \quad \text{A17}$$

if $NTSW$ >= 300 mm: $$TCW = 1.3322NTSW - 0.0005515NTSW^2 - 109 \quad \text{A18}$$

3. Estimate that load on each tyre (L_{TO}) in kN which, when the tyre is acting in isolation, will produce the same maximum pavement deflection as the Standard Axle. This is done using the tyre contact width (TCW) and the following relationship (Equation A19).

$$L_{TO} = 18.47 + 0.03843TCW \qquad \text{A19}$$

4. With each tyre on the axle supporting a load L_{TO}, determine the ratio (d_A/d_T) of the maximum deflection under the axle to the maximum pavement deflection under an isolated tyre. This ratio is determined as follows:

Select the position on the axle where the maximum surface deflection is expected.

a. For each tyre on the axle, determine the transverse distance (TD) (tyre centreline to tyre centreline expressed in units of contact width) of the tyre from the position of the selected tyre and calculate the corresponding normalised deflection (ND) from the following relationship (Equation A20).

$$ND = \frac{1}{1 - 0.085826TD + 1.755TD^2 - 0.7555TD^3 + 0.1389TD^4} \qquad \text{A20}$$

b. Sum the normalised deflections obtained. The resulting number is the ratio d_A/d_T.

5. Determine that load per tyre (L_{T1}) which will produce a maximum pavement deflection under the axle equal to the maximum pavement deflection under the Standard Axle. Using a linear relationship between load and deflection, gives (Equation A21):

$$L_{T1} = {}^{L_{TO}}\!/_{(d_A/d_T)} \qquad \text{A21}$$

6. Determine the load on the axle (L_{A1}) which produces the same damage as the Standard Axle from the product of the number of tyres on the axle (n_T) and the load per tyre (L_{T1}), using Equation A22.

$$L_{A1} = n_T L_{T1} \qquad \text{A22}$$

7. Determine the ratio of damage caused by the axle when the axle load is L_A to the damage caused by the Standard Axle by applying the fourth power law. Following convention, which defines the damage ratio as the Load Equivalence Factor (LEF) gives (Equation A23).

$$LEF_{LA} = (L_A/L_{A1})^4 \qquad \text{A23}$$

8. Having determined the LEF for each axle of the vehicle by the above method, a LEF for the vehicle (ratio of damage caused by one load repetition of the vehicle to the damage caused by one load repetition of a Standard Axle) is determined by simply summing the LEFs for each axle of the vehicle using Equation A24.

$$LEF_V = \sum_{i=1}^{NAX} LEF_{LAi} \qquad \text{A24}$$

where

LEF_V	=	the Load Equivalence Factor for the vehicle
LEF_{LAi}	=	the Load Equivalence Factor for the i^{th} axle
NAX	=	the number of axles on the vehicle.

Having calculated the LEF of the specialised vehicle and knowing the load repetitions over its service life, the total Equivalent Standard Axle (ESA) of loading due to the specialised vehicle may be calculated and compared to the loading in ESA due to highway traffic without the specialised vehicle operating.

An example of the use of the procedure to calculate the LEF of a mobile crane is given in the Attachment to this appendix.

I.3 Flexible pavements which include bound materials

Mechanistic-empirical procedures are used to estimate the damage to flexible pavements with one or more bound layers. The method has been adapted (Jameson 2004) from the mechanistic-empirical procedures for flexible pavements (Chapter 8).

The procedure involves calculation of the critical strains under each isolated axle in the group and the determination of the number of repetitions of a Standard Axle that would cause the same damage as one pass of the individual axle. The total damage caused by a single pass of a specialised vehicle is the sum of the damage due to each axle. This method assumes critical strains are not influenced by adjacent axles. In applying the method, due consideration needs to be given to the different pavement configurations for which damage is to be assessed. Usually this would include the complete range of pavement types expected to be in-service along the route defined by the specialised vehicle permit application.

The following analysis steps describe the procedure to estimate the damage for a selected pavement configuration:

1. Select a pavement configuration.
2. Determine the elastic parameters of pavement layers and the subgrade (Chapter 8).
3. Divide the specialised vehicle into individual single axles and estimate the load on each tyre of each axle (L_T) and the spacing between tyres.
4. For each tyre on the specialised vehicle calculate the radius of the loaded area from the net contact area (NCA) for the tyre.

The tyres for mobile cranes commonly range from 16R25 to 20.5R25 in size. For these tyres the following relationships are used to estimate the tyre-pavement net contact area (Equation A25 and Equation A26).

16.00R25 Tyre

$$NCA = (79070 - 27.885P)\left(1 + 0.656\left(\frac{L_T}{60} - 1\right)\right) \quad \text{A25}$$

20.5R25 Tyre

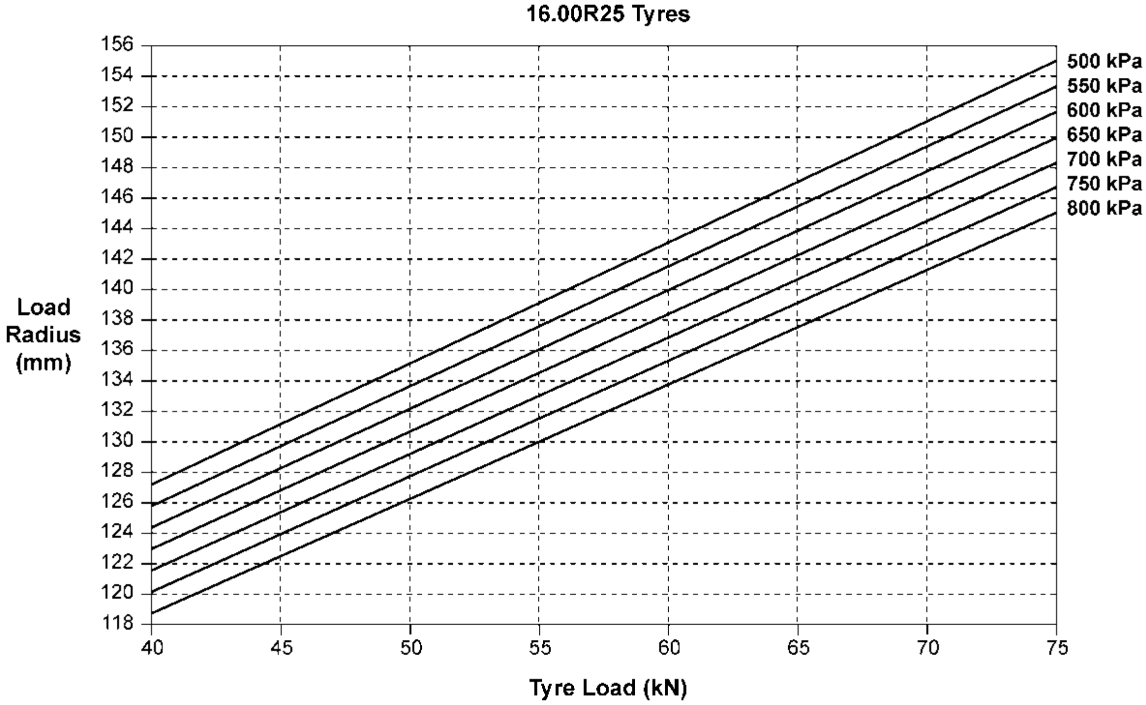
$$NCA = (92000 - 36.424P)\left(1 + 0.656\left(\frac{L_T}{60} - 1\right)\right) \quad \text{A26}$$

where

NCA	=	net contact area (mm^2) at tyre load L_T and inflation pressure P
P	=	tyre inflation pressure (kPa)
L_T	=	tyre load (kN)

For tyre sizes between 16R25 and 20.5R25 sizes, the NCA may be interpolated. The load radii obtained from some specialised vehicles are shown in Figure I.1.

Figure I.1: Variation in tyre load radius of 16R25 and 20.5R25 tyres with tyre load and inflation pressure



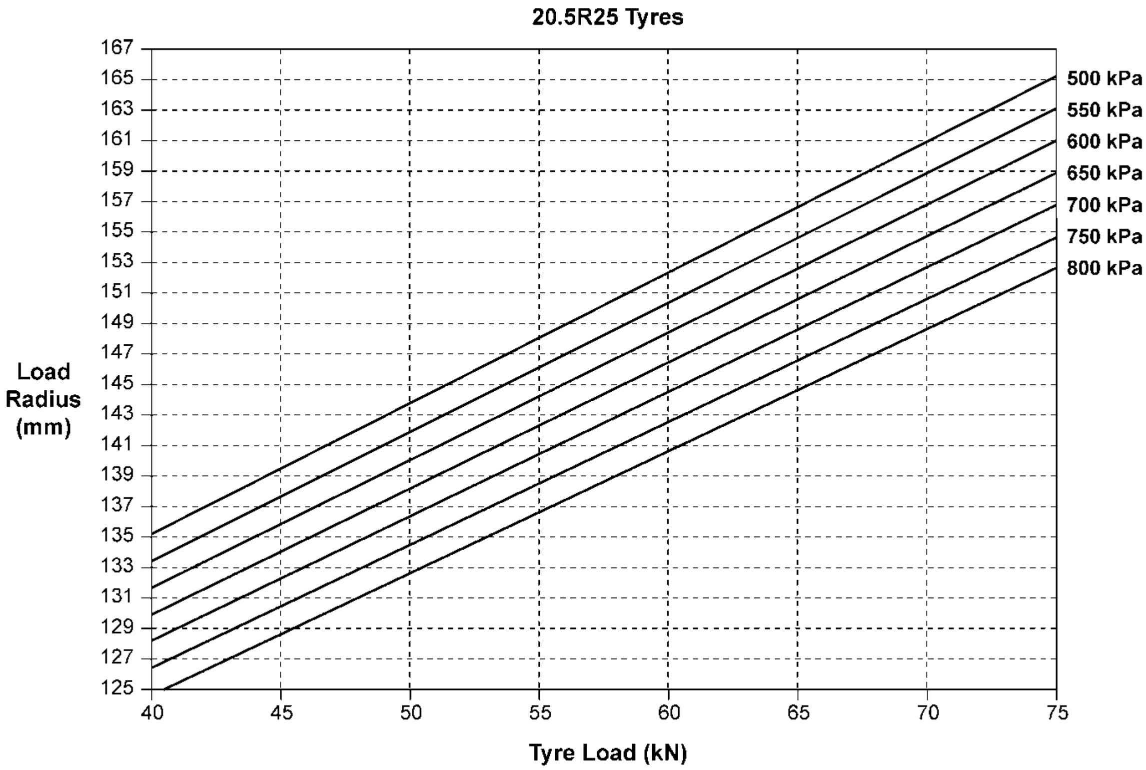


Table 12.15 and Table 12.16 provide load radii for other wide single tyres and for dual tyres at various axle loads and tyre inflation pressures based on the data collected by a COST project (COST 2001, Table 4.16) and assuming the net contact area is 69% of the gross contact area. For other tyre types (i.e. where the tyre section width is outside the 16 to 20.5 inch range, or the nominal diameter is not 25 inches) the contact areas need to be measured.

Table 12.15: Load radii for various wide single tyres

Tyre size	Tyre load (kN)							
	35		40		45		50	
	Radius (mm)	Pressure[1] (kPa)	Radius (mm)	Pressure[1] (kPa)	Radius (mm)	Pressure[1] (kPa)	Radius (mm)	Pressure[1] (kPa)
385/55R22.5	108.3	775	109.1	900	–	–	–	–
385/65R22.5	110.4	775	111.3	900	112.7	1000	–	–
425/65R22.5	120.7	625	121.7	725	118.5	800	124.2	900
445/65R22.5	125.8	600	127.7	675	128.3	775	129.8	850

1 Tyre inflation pressures are values for operating conditions, i.e. 'hot'.

Table 12.16: Load radii for various dual tyres

Tyre size	Tyre load (kN)							
	20		22.5		25		28	
	Radius (mm)	Pressure[1] (kPa)	Radius (mm)	Pressure[1] (kPa)	Radius (mm)	Pressure[1] (kPa)	Radius (mm)	Pressure[1] (kPa)
11R22.5	92.0	650	93.1	725	93.4	825	94.5	950
12R22.5	97.2	600	98.2	675	99.1	750	99.9	875
295/80R22.5	98.9	600	101.1	675	101.9	750	102.9	875
315/80R22.5	101.3	550	102.1	625	102.9	700	103.5	825

1 Tyre inflation pressures are values for operating conditions, i.e. 'hot'.

The load radius for a tyre of a Standard Axle is 92.1 mm (Chapter 8).

5. Calculate the tyre-pavement contact stress by dividing the tyre load by the contact area (Equation A27).

$$CS = \frac{L_T \times 10^6}{\pi R^2} \qquad \text{A27}$$

where

CS = tyre-pavement contact stress (kPa)

L_T = tyre load (kN)

R = radius of loaded area (mm)

Note that the tyre-pavement contact stress for a Standard Axle is 750 kPa (Section 8.2).

6. Determine locations in the pavement for the calculation of critical strains as follows:
   a. bottom of each asphalt, cemented material or lean-mix concrete layer
   b. top of in situ subgrade and any improved subgrade material.
7. Input the above parameters into a suitable linear elastic model (e.g. AustPADS, CIRCLY). Determine the maximum vertical strains at the top of the subgrade and any improved subgrade and the maximum horizontal tensile strain at the bottom of each asphalt, cemented material and lean-mix concrete layer. These strains should be calculated under both the Standard Axle and each specialised vehicle axle.

8. For each distress mode, calculate the relative damage of each axle as follows (Equation A28).

$$RDm = \left(\frac{\text{strain under specialised vehicle}}{\text{strain under Standard Axle}}\right)^{m} \qquad \text{A28}$$

where

RDm = relative damage – i.e. the damage resulting from a single passage of the specialised vehicle axle relative to the damage caused by a single passage of the Standard Axle – where the load damage exponent is m

m = load damage exponent for the damage:

- for permanent deformation damage $m = 4$
- for asphalt fatigue damage $m = 5$
- for cemented material and lean-mix concrete fatigue damage $m = 12$

9. For each distress mode, sum the relative damage, RDm, of all axles to obtain the relative damage for the specialised vehicle.

Having calculated the relative damage to a single pass of the specialised vehicle and knowing the load repetitions over its service life, the total damage due to the specialised vehicle may be calculated.

An example of the use of the procedure to calculate the damage caused by a mobile crane is given in the Attachment to this appendix.

I.4 Rigid pavements

For specialised vehicles with two tyres on each axle and a spacing between these tyres similar to normal highway traffic, the pavement damage due to the specialised vehicle may be assessed by adapting the procedures for the design of new rigid pavements (Chapter 3).

The process follows the design steps similar to those used in the design of new rigid pavements (Table 9.3):

1. Select a rigid pavement type for analysis, representing atypical or weakest part of the road network on which the specialised vehicle will operate, including whether concrete shoulders exist.
2. Determine the Effective Subgrade Strength from Figure 9.1.
3. Select the 28-day characteristic flexural strength of the concrete base f_{cf}.
4. Select the desired project reliability and hence the load safety factor from Table 9.2.
5. Select an analysis period over which the pavement damage is to be assessed. Commonly this is the service life or period of loading of the specialised vehicle. Calculate the cumulative number of load repetitions of each axle group of the specialised vehicle over the analysis period.
6. Obtain the load of each individual axle of the specialised vehicle and hence from Step 5 the expected load repetitions of each axle group load.
7. Assuming that each axle of the specialised vehicle produces the same damage as a Single Axle Single Tyre (SAST) of the same load, calculate the allowable repetitions in terms of fatigue using Equation 58 and Equation 59.
8. Calculate the ratio of the expected fatigue repetitions (Step 6) to the allowable fatigue repetitions for each axle of the specialised vehicle. Multiply by 100 to determine the percentage fatigue. Sum the damage of the individual axles to calculate the total fatigue damage.

9. Assuming that each axle of the specialised vehicle produces the same damage as a Single Axle Single Tyre (SAST) of the same load, calculate from Equation 61 the allowable repetitions in terms of erosion.
10. Calculate the ratio of the expected fatigue repetitions (Step 6) to the allowable erosion repetitions for each axle of the specialised vehicle. Multiply by 100 to determine the percentage erosion. Sum the damage of the individual axles to calculate the total erosion damage.

An example of the use of the procedure to calculate damage caused by a mobile crane is given in the Attachment.

For specialised vehicles with other tyre configurations there is an additional step in the analysis. For each axle of the specialised vehicle, the load on a SAST which causes the same damage needs to be determined; in other words an equivalent SAST load is determined. The damage calculations then proceed as outlined in Steps 1 to 10 above. For each specialised vehicle axle, the Pickett and Ray influence charts (Yoder and Witczak 1975) may be used to estimate the equivalent SAST load this being the load on a SAST that causes the same stress as the specialised vehicle axle.

Note that where very limited load repetitions of the specialised vehicle are anticipated and the calculated damage due to a single pass of the vehicle is high, consideration may need to be given to evaluating the effect on pavement damage of restricting the use of the vehicle to periods of the day when the damage is the lowest. This will require the calculation of the thermal stresses in the pavement (Yoder and Witczak 1975).

Attachment – Example of the Use of Evaluation Procedures for Specialised Vehicles

Introduction

This Attachment gives examples of the use of the evaluation procedures for the following specialised vehicle:

- vehicle is a six-axle mobile crane
- each axle is fitted with two type 20.5R25 tyres, with tyre centres spaced 2640 mm apart
- Table 1 lists the axle loads
- all tyres are inflated to a pressure of 551 kPa.

Table 1: Axle loads

Axle	Tyre type	Tyre inflation pressure (kPa)	Axle load (kN)
Axle 1	20.5R25	551	116.7
Axle 2	20.5R25	551	119.2
Axle 3	20.5R25	551	117.7
Axle 4	20.5R25	551	119.2
Axle 5	20.5R25	551	120.1
Axle 6	20.5R25	551	121.1

The crane is proposed to be operated over a 10-year period and it is assumed that, on average, there is one crane load repetition per day for each section of pavement.

Granular pavements with thin bituminous surfacings

The damage in Equivalent Standard Axle (ESA) to sprayed seal granular pavements due to a single load repetition per day of the mobile crane over a 10-year period of operation was calculated as follows:

Step

1. The vehicle is divided into six axles with axle loads (L_A) as given in Table 1.
2. The tyres fitted to the crane are type 20.5R25, with a nominal tyre section width (NTSW) of $20.5 \times 25.4 = 521$ mm. Using Equation A17, the tyre contact width (TCW) of each tyre is

$$TCW = 1.3322 \times 521 - 0.0005515 \times 521^2 - 109 = 435 \text{ mm}$$

3. The load on each tyre that produces the same maximum deflection as a Standard Axle is

$$L_{TO} = 18.47 + 0.03843 \times 435 = 35.2 \text{ kN}$$

4. With each tyre on the axle supporting a load L_{TO}, the ratio (dA/dT) of the maximum deflection under the axle to the maximum deflection under an isolated tyre is calculated as follows:
   a. The position of the expected maximum deflection is under the two tyres of each axle.
   b. The transverse distance (TD) between tyres expressed in units of contact width is:

Tyre 1:
$TD = 0/435 = 0$

Tyre 2:
$TD = 2640/435 = 6.07$

Using Equation A20 the normalised deflection (ND) is calculated:

Tyre 1:

$$ND = \frac{1}{1 - 0.085826 \times 0 + 1.755 \times 0^2 - 0.7555 \times 0^3 + 0.1389 \times 0^4} = 1$$

Tyre 2:

$$ND = \frac{1}{1 - 0.085826 \times 6.07 + 1.755 \times 6.07^2 - 0.7555 \times 6.07^3 + 0.1389 \times 6.07^4} = 0.012$$

c. Thus $(dA/dT) = 1 + 0.012$.

5. The load per tyre (L_{T1}) which will produce a maximum deflection equal to the maximum deflection under a Standard Axle is

$$L_{T1} = L_{TO}/\,(d_T/d_A) = 35.3/1.012 \ = 34.88 \text{ kN}$$

6. The load on the axle (L_{A1}) which produces the same damage as the Standard Axle is the load per tyre (L_{T1}) multiplied by the number of tyres

$$L_{A1} = 2 \times 34.88 = 69.76 \text{ kN}$$

7. For each axle calculate the LEF_A as follows from the ratio of axle load (L_A) to the load on the axle which produces the same deflection as a Standard Axle (L_{A1}):

LEF for Axle 1 $LEF_{A1} = (116.7/69.76)^4 = 7.8$

LEF for Axle 2 $LEF_{A2} = (119.2/69.76)^4 = 8.5$

LEF for Axle 3 $LEF_{A3} = (117.7/69.76)^4 = 8.1$

LEF for Axle 4 $LEF_{A4} = (119.2/69.76)^4 = 8.5$

LEF for Axle 5 $LEF_{A5} = (120.1/69.76)^4 = 8.8$

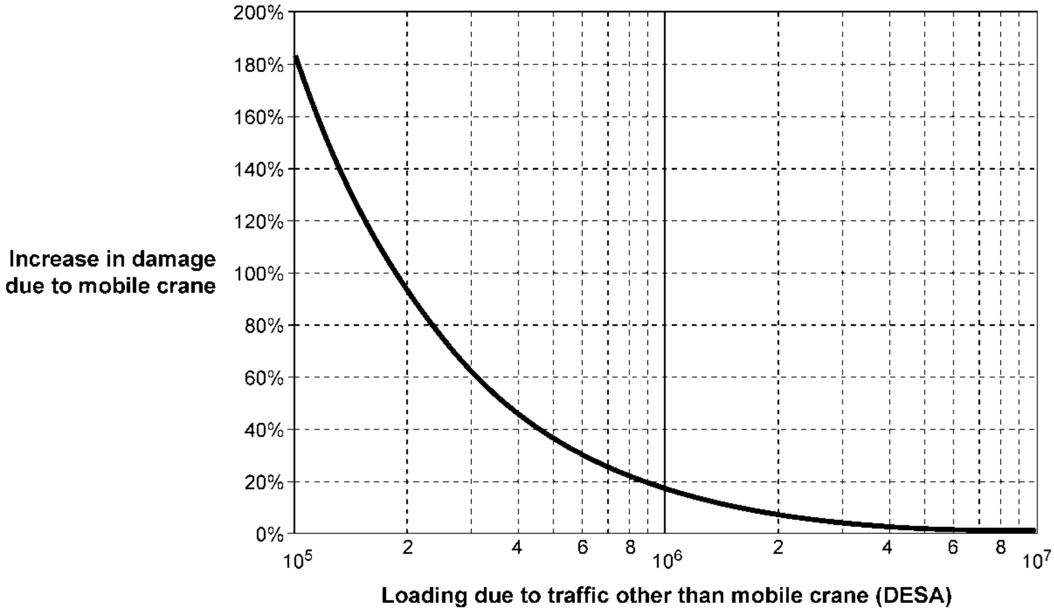
LEF for Axle 6 $LEF_{A6} = (121.1/69.76)^4 = 9.1$

The total LEF_V for the mobile crane is the sum of the LEF_{LA} values of each axle which is equal to 51. That is, each pass of the mobile crane produces the same pavement damage as 51 passes of a Standard Axle. Note the LEF_V is the ESA of damage of a single pass of the crane as the LEF is calculated assuming damage is related to the 4th power of the pavement deflection.

Over the 10-year period, the number of load repetitions to the pavement is 10 x 365 = 3650. Hence the ESA of damage due to the mobile crane over the 10-year period is the number of load repetitions (3650) times the ESA of damage per pass (51), that is 3650 x 51 = 1.9×10^5 ESA.

The damage due to the mobile crane may be compared to the damage due to other highway vehicles over its 10-year service period. As shown in Figure 1 the percentage increase in damage due to the mobile crane is greater if the crane travels along lightly trafficked roads.

Figure 1: Increase in damage due to mobile crane



Asphalt surfaced granular pavements

The damage to an asphalt surfaced granular pavement due to a single pass of the mobile crane was calculated following the steps in Appendix I.3:

Step

1. The network of roads along which the mobile crane will travel includes asphalt surfaced granular pavements following pavement shown in Table 2 was selected to assess the damage due to the crane on this pavement type (note the process needs to be repeated for each relevant pavement type).

Table 2: Asphalt surfaced granular pavement composition

Material type	Thickness (mm)
Asphalt	80
Unbound granular material	450
Subgrade, CBR = 5%	Semi-infinite

2. Elastic parameters for pavement layers and subgrade (Chapters 5 and 6):

Asphalt: A modulus of 3000 MPa was adopted for the analysis as this was typical of the values being used in the locations where the crane will be used.

Subgrade:

CBR = 5%

E_V = 50 MPa – Section 5.6

E_H = 25 MPa

$\nu_V = \nu_H$ = 0.45 – Section 5.6

$f = E_V/(1+\nu_V)$ = 34.5

Elastic parameters for top granular sublayer:

Divide the total granular layer thickness (450 mm) into five equi-thick layers with the top granular sublayer being the minimum of

$E_{V\ \text{top granular sublayer}} = E_{V\ \text{subgrade}} \times 2^{(\text{total granular thickness}/125)} = 50 \times 2^{(450/125)} = 606$ MPa (see Section 8.2.3)

$E_{V\ \text{top granular sublayer}}$ = 330 MPa – (from Table 6.4 for 80 mm asphalt E = 3000 MPa)

E_V = Minimum (330 MPa, 606 MPa) = 330 MPa

E_H = 165 MPa

$\nu_V = \nu_H$ = 0.35

f = $E_V/(1+\nu_V)$ = 244.4

Elastic parameters for other granular sublayers:

Divide the total granular layer thickness into five equi-thick sublayers (Section 8.2.3), each of thickness 450/5 = 90 mm.

Calculate the ratio of moduli of adjacent layers:

$$R = \left[\frac{E_{V\ \text{top granular sublayer}}}{E_{V\ \text{subgrade}}}\right]^{\frac{1}{5}} = \left[\frac{330}{50}\right]^{\frac{1}{5}}$$

i.e. R = 1.459

Therefore, elastic parameters of the first granular sublayer on top of the subgrade are:

E_{V1} = $R \times E_{V\ \text{subgrade}}$ = 1.459 × 50 = 72.9 MPa

E_{H1} = 0.5 × E_{V1} = 36.4 MPa

$\nu_V = \nu_H$ = 0.35

f = $E_{V1}/(1+\nu_V)$ = 54.0

Elastic properties of other sublayers are calculated similarly using the elastic properties of the underlying sublayer and are listed in the Table 3.

Table 3: Elastic properties of pavement materials

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f
		E_V	E_H	ν_V	ν_H	
Asphalt	80	3000	3000	0.4	0.4	2142
Granular	90	330	165	0.35	0.35	244.4
Granular	90	226.3	113.1	0.35	0.35	167.6
Granular	90	155.1	77.6	0.35	0.35	114.9
Granular	90	106.4	53.2	0.35	0.35	78.8
Granular	90	72.9	36.4	0.35	0.35	54.0
Subgrade	Semi-infinite	50	25	0.45	0.45	34.5

3. Loads on each tyre of each axle of the mobile crane and spacing between tyres on each axle are shown in Table 4.

Table 4: Loads on axles and spacing between tyres

Crane axle	Tyre type	Tyre inflation pressure (kPa)	Tyre loads (kN)	Spacing between tyres (mm)
Axle 1	20.5R25	551	58.35	2640
Axle 2	20.5R25	551	59.58	2640
Axle 3	20.5R25	551	58.84	2640
Axle 4	20.5R25	551	59.58	2640
Axle 5	20.5R25	551	60.07	2640
Axle 6	20.5R25	551	60.56	2640

4. The net contact area of each crane tyre was calculated using Equation A26 and the area, and the load's circular radius is shown in Table 5.

Table 5: Contact stresses and radii of tyres

Crane axle	Tyre inflation pressure (kPa)	Tyre load (kN)	Net contact area (mm^2)	Load radius (mm)	Contact stress (kPa)
Axle 1	551	58.35	70 631	149.9	826
Axle 2	551	59.58	71 595	151.0	832
Axle 3	551	58.84	71 016	150.4	829
Axle 4	551	59.58	71 595	151.0	832
Axle 5	551	60.07	71 981	151.4	834
Axle 6	551	60.56	72 367	151.8	837

5. Calculate the tyre-pavement contact stress by dividing the tyre load by the contact area (Equation A27). These stresses are shown in Table 5.
6. The locations to calculate the strains are
   a. horizontal tensile strain at the bottom of the asphalt under the middle of the crane tyre
   b. vertical compressive strain at the top of subgrade under the middle of crane tyre.
7. Using a suitable linear elastic model (e.g. AustPADS, CIRCLY), calculate the critical strains under each axle and under a Standard Axle. These strains are listed in Table 6.

Table 6: Calculated critical; strains

Crane axle	Load radius (mm)	Contact stress (kPa)	Asphalt strain (microstrain)	Subgrade strain (microstrain)
Axle 1	149.9	826	443	976
Axle 2	151.0	832	448	997
Axle 3	150.4	829	446	986
Axle 4	151.0	832	448	997
Axle 5	151.4	834	450	1004
Axle 6	151.8	837	452	1013
Standard Axle	92.1	750	324	604

8. Calculate the relative damage resulting from each axle using Equation A28 with a damage exponent (m) of 5 for asphalt fatigue and 4 for permanent deformation. Table 7 shows the calculated damages.

Table 7: Relative damage resulting from each axle

Crane axle	Relative damage in terms of asphalt fatigue (RD5)	Relative damage in terms of permanent deformation (RD4)
Axle 1	4.8	6.8
Axle 2	5.1	7.4
Axle 3	4.9	7.1
Axle 4	5.1	7.4
Axle 5	5.2	7.6
Axle 6	5.3	7.9
Total	30	44

9. Sum the relative damage for each distress mode to obtain the damage due to a single pass of the mobile crane:
    a. In terms of asphalt fatigue, one pass of the mobile crane produces the same damage as 30 passes of the Standard Axle.
    b. In terms of permanent deformation, one pass of the mobile crane produces the same damage as 44 passes of the Standard Axle.
10. Over the 10-year period, the number of crane load repetitions to the pavement is 10 x 365 = 3650. Hence the damage due to the crane over the 10-year period is
    a. the same asphalt fatigue damage as 3650 x 30 = 1.1×10^5 repetitions of the Standard Axle.
    b. the same permanent deformation damage as 3650 x 44 = 1.6×10^5 repetitions of the Standard Axle.

In terms of asphalt fatigue, the number of allowable repetitions of the Standard Axle due to all traffic during the design period can be calculated using Equation 25 as:

$$\frac{SF}{RF}\left[\frac{6918(0.856V_b + 1.08)}{E^{0.36}\mu\varepsilon}\right]^5 = \frac{6}{6} \times \left[\frac{6918 \times (0.856 \times 11 + 1.08)}{3000^{0.36} \times 324}\right]^5 = 3.1 \times 10^5$$

In terms of permanent deformation, the number of allowable repetitions of the Standard Axle can be calculated using Equation 3 as:

$$\left[\frac{9150}{\mu\varepsilon}\right]^7 = \left[\frac{9150}{604}\right]^7 = 1.8 \times 10^8$$

The loading due to the crane can then be compared to the allowable loadings for each distress mode (Table 8).

Table 8: Crane loading compared to allowable loadings

Distress mode	Allowable repetitions of the Standard Axle	Design traffic due to the crane over 10 years	Percentage of life consumed by the crane over 10 years
Asphalt fatigue	3.1×10^5	1.1×10^5	36%
Permanent deformation	1.8×10^8	1.6×10^5	< 1%

The overall life of the pavement is governed by the allowable loading in terms of asphalt fatigue. It was concluded that over its 10-year service life, the crane will consume about 36% of the life of this pavement.

Note that as the percentage damage varies markedly with the pavement configuration, the above analysis needs to be undertaken on the full range of pavement types the crane will load.

Rigid pavements

The damage to a rigid (concrete) pavement due to a single load repetition per day of the mobile crane over a 10-year period of operation was calculated following the steps in Table 9.3 as modified in Appendix I.4:

1. The network of roads along which the mobile crane will travel includes rigid pavements and the pavement shown in Table 9 was selected to assess the damage due to the crane on this pavement type (note the analysis needs to be repeated for all relevant rigid pavement types).

Table 9: Rigid pavement composition

Material type	Thickness (mm)
Plain concrete pavement base	230
Lean-mix concrete subbase	150
Subgrade, CBR = 5%	Semi-infinite

2. Using the design subgrade CBR of 5%, from Figure 9.1 the Effective Subgrade CBR is 75% for 150 mm lean-mix concrete subbase.
3. The 28-day characteristic flexural strength of the concrete base f_{cf} is 4.5 MPa.
4. The desired project reliability is 95% and hence the load safety factor is 1.3 (Table 9.2).
5. The cumulative number of passes of the mobile crane over 10 years is 3650.
6. There are six axles on the mobile crane with the axle loads shown in Table 10.

Table 10: Loads on axles and expected repetitions

Axle	Axle load (kN)	Expected repetitions
Axle 1	116.7	3650
Axle 2	119.2	3650
Axle 3	117.7	3650
Axle 4	119.2	3650
Axle 5	120.1	3650
Axle 6	121.1	3650

These expected load repetitions are recorded in the first three columns of Table 11.

For each crane axle the number of allowable repetitions in terms of fatigue is calculated using Equation 58 and Equation 59, assuming each axle of the crane produces the same damage as a Single Axle Single Tyres (SAST) with the same axle load. The results are shown in the Table 11.

For each crane axle the ratio of the expected fatigue repetitions to the allowable fatigue repetitions is calculated and multiplied by 100 to determine percentage fatigue consumed over the 10-year analysis period. The total fatigue damage of the six-axle crane is 18% as shown in Table 11.

7. For each crane axle load the allowable repetitions in terms of erosion distress is calculated using Equation 61 as shown in Table 11.

   The ratio of the expected erosion repetitions to the allowable repetitions was calculated and multiplied by 100 to determine percentage erosion consumed over the 10-year analysis period. The total erosion damage of the 6 axle crane is 3%, as shown in the Table 11.

   For this pavement configuration, the governing distress mode due to traffic other than the mobile crane is commonly erosion. In this case, the erosion damage due to cranes would be more significant in evaluating the effect of the crane operation on the life of this rigid pavement than fatigue. Hence the 10-year operation of the crane will consume about 3% of the pavement life.

   Note that as the percentage damage varies markedly with the pavement configuration, the above analysis needs to be undertaken on the full range of pavement types the crane will load.

Table 11: Loads on axles (SAST) and expected repetitions

Axle group load (kN)	Design load (kN)	Expected repetitions	Equivalent stress 0.66		Erosion factor 1.76	
			Fatigue analysis		Erosion analysis	
			Allowable repetitions	Damage (%)	Allowable repetitions	Damage (%)
116.7	151.7	3 650	163 516	2.23	764 226	0.48
119.2	155.0	3 650	119 512	3.05	653 177	0.56
117.7	153.0	3 650	143 724	2.54	717 066	0.51
119.2	155.0	3 650	119 512	3.05	653 177	0.56
120.1	156.1	3 650	107 189	3.41	618 361	0.59
121.1	157.4	3 650	94 987	3.84	582 440	0.63
			Total	18.13	Total	3.32

Appendix J Effect of Asphalt Thickness on Fatigue Life of Asphalt-Surfaced Pavements

For a given design traffic, asphalt modulus, thickness of granular, cemented material or lean-mix concrete base or subbase, and subgrade modulus, two thicknesses of asphalt can provide the same theoretical fatigue performance.

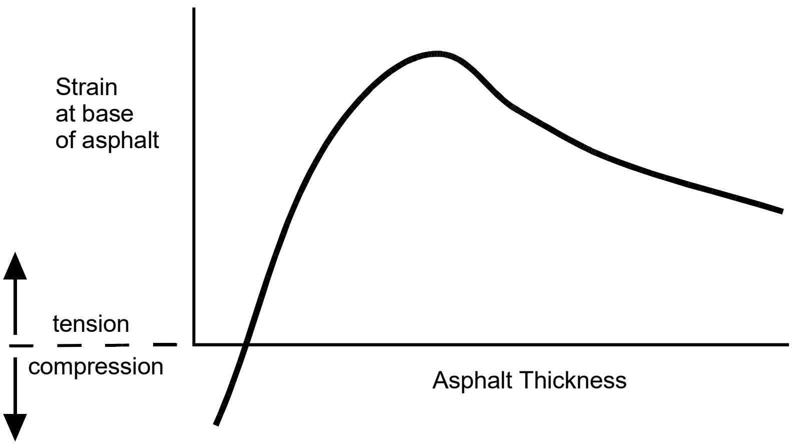
The reason for this can be explained by examining the relationship between asphalt thickness and horizontal strain at the bottom of the asphalt, induced by a Standard Axle load for a given composition of underlying material. A typical relationship is illustrated in Figure J.1.

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 the magnitude 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 mm to 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 J.1: General relationship between asphalt thickness and horizontal strain at the base of an asphalt layer



Appendix K Examples of Use of the Mechanistic-Empirical Procedure for Flexible Pavements

This appendix gives examples of the use of mechanistic-empirical procedures for the design of the following three flexible pavement types:

- sprayed seal surfaced unbound granular pavement
- full depth asphalt pavement
- asphalt pavement containing a heavily bound cemented material subbase
- sprayed seal surfaced lightly bound base pavement.

The following design parameters are used:

- subgrade design CBR = 5%
- design traffic for 30 year design life = 10^7 HVAG.

The traffic load distribution (TLD) is in accordance with the example distribution (see Appendix F). Using a Weighted Mean Annual Pavement Temperature of 28 °C and design traffic speed of 60 km/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%.

K.1 Sprayed seal surfaced unbound granular pavement

Following the steps in Table 8.1, Table 8.2 and Table 8.3:

Step 1

Try pavement composition shown in Table 12.17.

Table 12.17: Candidate pavement: sprayed seal surfaced unbound granular pavement

Material type	Thickness (mm)
Sprayed seal surface	–
Unbound granular material	475
Subgrade, design CBR = 5%	Semi-infinite

Step 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 = $E_V/(1+\nu_V)$ = 50/1.45 = 34.5

Step 3

Properties of top granular sublayer:

$E_{V \text{ top granular sublayer}}$ is the minimum of Equation 42 in Section 8.2.3 and the value indicated in Table 6.5 (assuming high standard crushed rock).

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

= 696 MPa (Section 8.2.3)

$E_{V \text{ top granular sublayer}}$ = 500 MPa (Table 6.5)

$E_{V \text{ top granular sublayer}}$ = minimum (696 MPa, 500 MPa) = 500 MPa

$E_{H \text{ top granular sublayer}}$ = 250 MPa

$\nu_V = \nu_H$ = 0.35 – Table 6.3

f = $E_V/(1+\nu_V)$ = 500/1.35 = 370.4

Step 4

Other granular sublayers

Divide the total granular layer thickness into five equi-thick sublayers (Section 8.2.3), each 475/5 = 95 mm thick.

Calculate the ratio of adjacent sublayers.

$$R = (500/50)^{1/5} = 1.585$$

Therefore, elastic parameters of the first granular sublayer on top of the subgrade are:

E_{V1} = $R \times E_{V\text{ subgrade}}$ = 1.585 × 50 = 79 MPa

E_{H1} = 0.5 × E_{V1} = 39.5 MPa

$\nu_V = \nu_H$ = 0.35

f = $E_{V1}/(1+\nu_V)$ = 79/1.35 = 58.5

Elastic properties of other sublayers are calculated similarly using the elastic properties of the underlying sublayer and are listed in Table 12.18.

Table 12.18: Elastic properties of unbound granular material sublayers

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f
		E_V	E_H	ν_V	ν_H	
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

Steps 5 to 7 are not relevant.

Step 8

Permanent deformation allowable loading (Equation 3).

$$N = \left[\frac{9150}{\mu\varepsilon}\right]^7 \qquad \text{A29}$$

where

$\mu\varepsilon$ = the vertical compressive strain (in terms of microstrain), developed under a Standard Axle, at the top pf the subgrade

N = the allowable number of repetitions of a Standard Axle at this strain before an unacceptable level of pavement surface deformation develops (units of ESAs)

Steps 9 and 10 are not relevant.

Step 11

N_{DT} = 10^7 HVAG

TLD is Table 12.10 in Appendix F

Step 12

Using the example distribution in Appendix F, ESA/HVAG = 0.7.

Therefore using Equation 38:

$$\text{DESA} = 0.7 \times 10^7$$

Step 13

Standard Axle load is represented as:

Tyre-pavement 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).

Step 14

Critical locations to calculate strains are:

- top of subgrade directly beneath the inner tyre load of one of the dual tyre sets
- top of subgrade midway between one of the dual tyre sets.

Step 15

Maximum vertical compressive strain from CIRCLY is 906 $\mu\varepsilon$, located midway between the two loaded tyres.

Step 16 to 18 are not relevant.

Step 19

Permanent deformation allowable loading – from Step 7:

$$N = \left[\frac{9150}{906}\right]^7 = 1.1 \times 10^7 ESA$$

Step 20

The allowable loading is 1.1×10^7 ESA compared to the design traffic 0.7×10^7 ESA from Step 11. The candidate structure is acceptable in terms of permanent deformation.

Steps 21 to 30 are not relevant.

Step 31

As the candidate structure is acceptable with regards to all relevant performance criteria, the trial pavement composition is acceptable.

K.2 Full depth asphalt pavement

Following the steps in Table 8.1, Table 8.2 and Table 8.3:

Step 1

Try pavement composition shown in Table 12.19.

Table 12.19: Candidate pavement: full depth asphalt pavement

Material type	Thickness (mm)
Size 14 mm asphalt, E = 2200 MPa	50
Size 20 mm asphalt, E = 2500 MPa	200
Subgrade, design CBR = 5%	Semi-infinite

Step 2

Subgrade as in Appendix K.1.

Steps 3 to 5 are not relevant.

Step 6

Size 14 mm asphalt:

$$E_V = E_H = 2200 \text{ MPa}$$

$$\nu_V = \nu_H = 0.4$$

$$f = 1571$$

Size 20 mm asphalt:

$$E_V = E_H = 2500 \text{ MPa}$$

$$\nu_V = \nu_H = 0.4$$

$$f = 1786$$

Elastic properties for all material are listed in Table 12.20.

Table 12.20: Elastic properties of full depth asphalt pavement structure

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f
		E_V	E_H	ν_V	ν_H	
Size 14 mm asphalt	50	2200	2200	0.4	0.4	1571
Size 20 mm asphalt	200	2500	2500	0.4	0.4	1786
Subgrade	Semi-infinite	50	25	0.45	0.45	34.5

Step 7

Step 7 is not relevant.

Step 8

Subgrade strain criterion as in Appendix K.1.

Step 9

Step 9 is not relevant.

Step 10

Asphalt fatigue criterion (Equation 25):

For the sake of brevity, the upper asphalt layer is not examined in this example.

Size 20 mm asphalt:

$$N = \frac{SF}{RF}\left[\frac{6918(0.856 \times 11 + 1.08)}{2500^{0.36}\mu\varepsilon}\right]^5$$

Assuming volume of bitumen (V_b) of 11%.

Shift Factor, SF = 6 and Reliability Factor, RF = 9 (Table 6.17).

Step 11

N_{DT}	=	10^7 HVAG
TLD	is	Table 12.10 in Appendix F

Step 12

Design ESA loading, DESA as in Appendix K.1.

Step 13

Standard Axle load as in Appendix K.1.

Step 14

Critical locations to calculate strains are:

- top of subgrade
- bottom of asphalt layer.

All the above strains are calculated directly beneath the inner loaded tyres and midway between the loaded tyres.

Step 15

Critical strains resulting from Standard Axle load calculated by CIRCLY as:

- asphalt – maximum horizontal tensile strain of 175 $\mu\varepsilon$, located midway between the loaded tyres
- subgrade – maximum vertical compressive strain of 438 $\mu\varepsilon$, located beneath the inner loaded tyre.

Step 16

Single axle with single tyres represented as:

Tyre-pavement contact stress	=	800 kPa
Load radius	=	102.4 mm

Two circular areas separated centre-to-centre 2130 mm (Figure 8.2).

Step 17

Critical location to calculate strains is:

- bottom of asphalt layer.

The above strain is calculated directly beneath one of the loaded tyres of the single axle with single tyres.

Step 18

Critical strain resulting from single axle with single tyres with 53 kN load calculated by CIRCLY as:

- asphalt – maximum horizontal tensile strain of 142 $\mu\varepsilon$.

Step 19

Permanent deformation allowable loading – from Step 7:

$$N = \left[\frac{9150}{438}\right]^7 = 1.7 \times 10^9\ ESA$$

Step 20

The allowable loading is 1.7 × 10^9 ESA compared to the design traffic 7 × 10^6 ESA from Step 12. The candidate structure is acceptable in terms of rutting and loss of surface shape.

Step 21

Asphalt material is present in the candidate structure, therefore Steps 22 to 28 must be repeated for the asphalt material.

Step 22

Steps 23 to 27 are repeated for each axle group type present in the distribution:

- single axle with single tyres/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.

Step 23

The expected repetitions of each load level for each axle group type is calculated in Table 12.21 to Table 12.25 by entering the data from the TLD (Table 12.10 in Appendix F) in the first three columns of the tables, together with the design traffic in HVAG into the fourth column. The expected repetitions of each load/group type combination is the product of columns 2, 3 and 4.

Table 12.21: Calculation of expected repetitions – single axle/single tyre (SAST) – full depth asphalt pavement

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
10	0.002804	0.393	10 000 000	11 020
20	0.07827	0.393	10 000 000	307 601
30	0.1546	0.393	10 000 000	607 578
40	0.1571	0.393	10 000 000	617 403
50	0.2994	0.393	10 000 000	1 176 642
60	0.2329	0.393	10 000 000	915 297
70	0.06502	0.393	10 000 000	255 529
80	0.007943	0.393	10 000 000	31 216
90	0.001087	0.393	10 000 000	4 272
100	0.000354	0.393	10 000 000	1 391
110	0.000174	0.393	10 000 000	684
120	0.000174	0.393	10 000 000	684
130	0.000174	0.393	10 000 000	684

Table 12.22: Calculation of expected repetitions – single axle/dual tyres (SADT) – full depth asphalt pavement

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
10	0.03473	0.191	10 000 000	66 334
20	0.08696	0.191	10 000 000	166 094
30	0.2346	0.191	10 000 000	448 086
40	0.2193	0.191	10 000 000	418 863
50	0.168	0.191	10 000 000	320 880
60	0.09606	0.191	10 000 000	183 475
70	0.065	0.191	10 000 000	124 150
80	0.04623	0.191	10 000 000	88 299
90	0.02969	0.191	10 000 000	56 708
100	0.01393	0.191	10 000 000	26 606
110	0.004098	0.191	10 000 000	7 827
120	0.001158	0.191	10 000 000	2 212
130	0.000244	0.191	10 000 000	466

Table 12.23: Calculation of expected repetitions – tandem axle group/single tyre (TAST) – full depth asphalt pavement

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
10	0.000354	0.009	10 000 000	32
20	0.002377	0.009	10 000 000	214
30	0.002763	0.009	10 000 000	249
40	0.005755	0.009	10 000 000	518
50	0.02889	0.009	10 000 000	2 600
60	0.1027	0.009	10 000 000	9 243
70	0.1681	0.009	10 000 000	15 129
80	0.1661	0.009	10 000 000	14 949
90	0.1595	0.009	10 000 000	14 355
100	0.1442	0.009	10 000 000	12 978
110	0.09774	0.009	10 000 000	8 797
120	0.05903	0.009	10 000 000	5 313
130	0.02943	0.009	10 000 000	2 649
140	0.01539	0.009	10 000 000	1 385
150	0.008439	0.009	10 000 000	760
160	0.004279	0.009	10 000 000	385
170	0.002308	0.009	10 000 000	208
180	0.001367	0.009	10 000 000	123
190	0.000723	0.009	10 000 000	65
200	0.000555	0.009	10 000 000	50

Table 12.24: Calculation of expected repetitions – tandem axle group/dual tyres (TADT) – full depth asphalt pavement

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
10	0.001444	0.259	10 000 000	3 740
20	0.005755	0.259	10 000 000	14 905
30	0.006242	0.259	10 000 000	16 167
40	0.01977	0.259	10 000 000	51 204
50	0.06496	0.259	10 000 000	168 246
60	0.09511	0.259	10 000 000	246 335
70	0.1094	0.259	10 000 000	283 346
80	0.09769	0.259	10 000 000	253 017
90	0.07611	0.259	10 000 000	197 125
100	0.07242	0.259	10 000 000	187 568
110	0.06267	0.259	10 000 000	162 315
120	0.05952	0.259	10 000 000	154 157
130	0.05878	0.259	10 000 000	152 240
140	0.06534	0.259	10 000 000	169 231
150	0.0803	0.259	10 000 000	207 977
160	0.05717	0.259	10 000 000	148 070
170	0.03554	0.259	10 000 000	92 049
180	0.01863	0.259	10 000 000	48 252
190	0.008535	0.259	10 000 000	22 106
200	0.003331	0.259	10 000 000	8 627
210	0.000801	0.259	10 000 000	2 075
220	0.000322	0.259	10 000 000	834
230	0.00016	0.259	10 000 000	414

Table 12.25: Calculation of expected repetitions – triaxle axle group/dual tyres (TRDT) – full depth asphalt pavement

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
10	0.00005	0.148	10 000 000	74
20	0.001568	0.148	10 000 000	2 321
30	0.00329	0.148	10 000 000	4 869
40	0.01317	0.148	10 000 000	19 492
50	0.04167	0.148	10 000 000	61 672
60	0.07419	0.148	10 000 000	109 801
70	0.09777	0.148	10 000 000	144 700
80	0.08338	0.148	10 000 000	123 402
90	0.0615	0.148	10 000 000	91 020
100	0.05029	0.148	10 000 000	74 429
110	0.03701	0.148	10 000 000	54 775
120	0.03298	0.148	10 000 000	48 810
130	0.03147	0.148	10 000 000	46 576
140	0.03361	0.148	10 000 000	49 743
150	0.04008	0.148	10 000 000	59 318
160	0.04115	0.148	10 000 000	60 902
170	0.04819	0.148	10 000 000	71 321
180	0.06097	0.148	10 000 000	90 236
190	0.07733	0.148	10 000 000	114 448
200	0.08433	0.148	10 000 000	124 808
210	0.05136	0.148	10 000 000	76 013
220	0.02339	0.148	10 000 000	34 617
230	0.007764	0.148	10 000 000	11 491
240	0.002503	0.148	10 000 000	3 704
250	0.000905	0.148	10 000 000	1 339
260	0.00008	0.148	10 000 000	118

Step 24

The allowable repetitions of each axle group type and load level is calculated in Table 12.26 to Table 12.30. As a first stage the critical asphalt strain for each load level is estimated for a single constituent axle of each axle group type using Equation 45.

$$\boldsymbol{\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SAST,53}}{53} = \frac{L_{ij}}{n} \times \frac{142}{53}}$$

for axles within SAST, TAST groups

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SADT,80}}{80} = \frac{L_{ij}}{n} \times \frac{175}{80}$$

for axles within SADT, TADT and TRDT groups

For example, the critical asphalt strain developed under an axle within a triaxle group (TRDT) with a total group load of 60 kN is calculated as:

$$\mu\varepsilon_{\text{TRDT},60} = \frac{60}{3} \times \frac{175}{80} = 43.8\ \mu\varepsilon$$

The allowable repetitions of each axle group type and load magnitude are then calculated using Equation 46 (Section 8.2.6):

$$N_{ij} = \frac{1}{n} \times \frac{SF}{RF}\left[\frac{6918(0.856V_b + 1.08)}{E^{0.36}\mu\varepsilon_{ij}}\right]^5$$

For example, the allowable repetitions of a triaxle group (TRDT) with a total group load of 60 kN are calculated as:

$$N_{\text{TRDT},60} = \frac{1}{3} \times \frac{6}{9}\left[\frac{6918(0.856 \times 11 + 1.08)}{2500^{0.36} \times 43.8}\right]^5 = 2.13 \times 10^9$$

The allowable repetitions of all axle group/load combinations are shown in Table 12.26 to Table 12.30.

Steps 25 and 26

The damage resulting from each axle load is calculated as the number of expected repetitions of the load divided by the allowable number of repetitions, as shown in the last column of Table 12.26.

Table 12.26: Calculation of asphalt damage – single axle/single tyre (SAST) – full depth asphalt pavement

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	11 020	1	26.8	7.46E+10	1.48E-07
20	307 601	1	53.6	2.33E+09	1.32E-04
30	607 578	1	80.4	3.07E+08	1.98E-03
40	617 403	1	107.2	7.28E+07	8.48E-03
50	1 176 642	1	134.0	2.39E+07	4.93E-02
60	915 297	1	160.8	9.59E+06	9.54E-02
70	255 529	1	187.5	4.44E+06	5.76E-02
80	31 216	1	214.3	2.28E+06	1.37E-02
90	4 272	1	241.1	1.26E+06	3.38E-03
100	1 391	1	267.9	7.46E+05	1.87E-03
110	684	1	294.7	4.63E+05	1.48E-03
120	684	1	321.5	3.00E+05	2.28E-03
130	684	1	348.3	2.01E+05	3.40E-03
				Total SAST damage	0.239

Table 12.27: Calculation of asphalt damage – single axle/dual tyres (SADT) – full depth asphalt pavement

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	66 334	1	21.9	2.06E+11	3.23E-07
20	166 094	1	43.8	6.42E+09	2.59E-05
30	448 086	1	65.6	8.46E+08	5.30E-04
40	418 863	1	87.5	2.01E+08	2.09E-03
50	320 880	1	109.4	6.58E+07	4.88E-03
60	183 475	1	131.3	2.64E+07	6.94E-03
70	124 150	1	153.1	1.22E+07	1.02E-02
80	88 299	1	175.0	6.27E+06	1.41E-02
90	56 708	1	196.9	3.48E+06	1.63E-02
100	26 606	1	218.8	2.06E+06	1.29E-02
110	7 827	1	240.6	1.28E+06	6.13E-03
120	2 212	1	262.5	8.26E+05	2.68E-03
130	466	1	284.4	5.54E+05	8.42E-04
				Total SADT damage	0.078

Table 12.28: Calculation of asphalt damage – tandem axle group/single tyre (TAST) – full depth asphalt pavement

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	32	2	13.4	1.19E+12	2.67E-11
20	214	2	26.8	3.73E+10	5.74E-09
30	249	2	40.2	4.91E+09	5.06E-08
40	518	2	53.6	1.17E+09	4.45E-07
50	2 600	2	67.0	3.82E+08	6.81E-06
60	9 243	2	80.4	1.53E+08	6.02E-05
70	15 129	2	93.8	7.10E+07	2.13E-04
80	14 949	2	107.2	3.64E+07	4.11E-04
90	14 355	2	120.6	2.02E+07	7.10E-04
100	12 978	2	134.0	1.19E+07	1.09E-03
110	8 797	2	147.4	7.41E+06	1.19E-03
120	5 313	2	160.8	4.80E+06	1.11E-03
130	2 649	2	174.2	3.21E+06	8.24E-04
140	1 385	2	187.5	2.22E+06	6.24E-04
150	760	2	200.9	1.57E+06	4.83E-04
160	385	2	214.3	1.14E+06	3.38E-04
170	208	2	227.7	8.40E+05	2.47E-04
180	123	2	241.1	6.31E+05	1.95E-04
190	65	2	254.5	4.82E+05	1.35E-04
200	50	2	267.9	3.73E+05	1.34E-04
				Total TAST damage	0.008

Table 12.29: Calculation of asphalt damage – tandem axle group/dual tyres (TADT) – full depth asphalt pavement

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	3 740	2	10.9	3.29E+12	1.14E-09
20	14 905	2	21.9	1.03E+11	1.45E-07
30	16 167	2	32.8	1.35E+10	1.19E-06
40	51 204	2	43.8	3.21E+09	1.59E-05
50	168 246	2	54.7	1.05E+09	1.60E-04
60	246 335	2	65.6	4.23E+08	5.82E-04
70	283 346	2	76.6	1.96E+08	1.45E-03
80	253 017	2	87.5	1.00E+08	2.52E-03
90	197 125	2	98.4	5.57E+07	3.54E-03
100	187 568	2	109.4	3.29E+07	5.70E-03
110	162 315	2	120.3	2.04E+07	7.95E-03
120	154 157	2	131.3	1.32E+07	1.17E-02
130	152 240	2	142.2	8.86E+06	1.72E-02
140	169 231	2	153.1	6.11E+06	2.77E-02
150	207 977	2	164.1	4.33E+06	4.80E-02
160	148 070	2	175.0	3.14E+06	4.72E-02
170	92 049	2	185.9	2.32E+06	3.97E-02
180	48 252	2	196.9	1.74E+06	2.77E-02
190	22 106	2	207.8	1.33E+06	1.66E-02
200	8 627	2	218.8	1.03E+06	8.39E-03
210	2 075	2	229.7	8.05E+05	2.58E-03
220	834	2	240.6	6.38E+05	1.31E-03
230	414	2	251.6	5.11E+05	8.11E-04
				Total TADT damage	0.271

Table 12.30: Calculation of asphalt damage – triaxle group/dual tyres (TRDT) – full depth asphalt pavement

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	74	3	7.3	1.66E+13	4.44E-12
20	2 321	3	14.6	5.20E+11	4.46E-09
30	4 869	3	21.9	6.85E+10	7.11E-08
40	19 492	3	29.2	1.63E+10	1.20E-06
50	61 672	3	36.5	5.33E+09	1.16E-05
60	109 801	3	43.8	2.14E+09	5.13E-05
70	144 700	3	51.0	9.91E+08	1.46E-04
80	123 402	3	58.3	5.08E+08	2.43E-04
90	91 020	3	65.6	2.82E+08	3.23E-04
100	74 429	3	72.9	1.66E+08	4.47E-04
110	54 775	3	80.2	1.03E+08	5.30E-04
120	48 810	3	87.5	6.69E+07	7.30E-04
130	46 576	3	94.8	4.48E+07	1.04E-03
140	49 743	3	102.1	3.10E+07	1.61E-03
150	59 318	3	109.4	2.19E+07	2.71E-03
160	60 902	3	116.7	1.59E+07	3.84E-03
170	71 321	3	124.0	1.17E+07	6.08E-03
180	90 236	3	131.3	8.81E+06	1.02E-02
190	114 448	3	138.5	6.72E+06	1.70E-02
200	124 808	3	145.8	5.20E+06	2.40E-02
210	76 013	3	153.1	4.08E+06	1.86E-02
220	34 617	3	160.4	3.23E+06	1.07E-02
230	11 491	3	167.7	2.59E+06	4.44E-03
240	3 704	3	175.0	2.09E+06	1.77E-03
250	1 339	3	182.3	1.70E+06	7.86E-04
260	118	3	189.6	1.40E+06	8.45E-05
				Total TRDT damage	0.105

Step 27

The damage resulting from each axle group type is the sum of the damage caused by each load level for the group. The damage from each axle group type is shown in Table 12.31.

Table 12.31: Total group asphalt fatigue damage – full depth asphalt pavement

Axle group type	Total group damage
SAST	0.239
SADT	0.078
TAST	0.008
TADT	0.271
TRDT	0.105

Step 28

Total damage to asphalt is the sum of damage resulting from each axle group type:

$$\text{Total asphalt damage} = 0.239 + 0.078 + 0.008 + 0.271 + 0.105 = 0.701$$

Step 29 is not relevant.

Step 30

Total damage for asphalt layer is less than 1, therefore the trial pavement is acceptable.

K.3 Asphalt pavement containing heavily bound cemented material subbase

Following the steps in Table 8.1, Table 8.2 and Table 8.3:

Step 1

Try pavement composition shown in Table 12.32.

Table 12.32: Candidate pavement: pavement with cemented material subbase

Material type	Thickness (mm)
Size 14 mm asphalt, E = 2200 MPa	50
Size 20 mm asphalt, E = 2500 MPa	125
Heavily cemented material, E = 3000 MPa	200
Granular material	200
Subgrade, design CBR = 5%	Semi-infinite

The design flexural strength of the cemented material is 1.2 MPa. Both modulus and flexural strength of the cemented material were determined from laboratory tests.

As the thickness of asphalt over the cemented material is greater than or equal to 175 mm, the post-cracking phase of heavily bound cemented material life may be considered (Section 8.2.7).

Step 2

Subgrade as in Appendix K.1.

Step 3

Properties of top granular sublayer – pre-cracking heavily bound cemented material phase:

$E_{V \text{ top granular sublayer}}$ is the minimum of Equation 42 in Section 8.2.3 and the value indicated in Table 6.5 (assuming high standard crushed rock).

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

$$= 151.57 \text{ MPa (Section 8.2.3)}$$

$$E_{V \text{ top granular sublayer}} = 210 \text{ MPa (Table 6.5)}$$

$$E_{V \text{ top granular sublayer}} = \text{minimum (151.57 MPa, 210 MPa)} = 151.57 \text{ MPa}$$

$$E_{H \text{ top granular sublayer}} = 75.79 \text{ MPa}$$

$$\nu_V = \nu_H = 0.35$$

$$f = 112.28$$

Properties of top granular sublayer – post-cracking heavily bound cemented material phase:

The modulus of the granular material is dependent upon its stress environment, which is, in part, dependent upon the overlying materials – including the cracked heavily bound cemented material. An iterative approach is therefore needed to determine the moduli of the cracked cemented material and the granular material. This is covered in Step 5.

Step 4

Other granular sublayers – pre-cracking heavily bound cemented material phase:

Divide the total granular layer thickness into five equi-thick sublayers (Section 8.2.3) each of thickness 200/5 – 40 mm.

Calculate the ratio of adjacent sublayers:

$$R = (152/50)^{1/5} = 1.249$$

Sublayer elastic properties calculation procedure is as shown in Appendix K.1.

Step 5

<u>Cemented, pre-cracking heavily bound cemented material phase:</u>

$$E_V = 3000 \text{ MPa (Table 12.32)}$$

$$\nu_V = \nu_H = 0.2$$

Granular and cemented materials, post-cracking heavily bound cemented material phase:

The maximum vertical modulus of the cracked cemented material is limited by the modulus of the underlying support, i.e. the modulus of the granular material.

The modulus of the granular material is also dependent upon its stress environment, which is, in part, dependent upon the overlying materials – including the cracked heavily bound cemented material.

An iterative approach is therefore needed to determine the moduli of the cracked cemented material and the granular material.

From Step 3, the subgrade modulus of 50 MPa limits the vertical modulus of the top granular sublayer to 152 MPa.

The modulus of the cracked cemented material is dependent upon the stress environment enabled by the thickness and moduli of the overlying asphalt materials and the cracked cemented material. The higher the cracked cemented material modulus, the lower the permissible top granular modulus.

The maximum modulus of the cracked heavily bound cemented material is the minimum of 600 MPa and the value provided in Table 6.8. The combined thickness of asphalt overlaying the cracked cemented material is 175 mm, and the effective modulus of the combined asphalt layers, E_e, can be calculated using Equation 5 in Section 6.2.3.

$$E_e = \left[\frac{\sum_i h_i E_i^{1/3}}{T}\right]^3$$

$$= \left[\frac{50 \times 2200^{1/3} + 125 \times 2500^{1/3}}{50 + 125}\right]^3$$

$$= 2412 \text{ MPa}$$

Table 6.8 shows that this amount of overlying material limits the modulus of the cracked heavily bound cemented material to 448 MPa.

With an upper bound of the cracked cemented material modulus determined, the maximum effective modulus of combined material overlying the granular material can be determined.

$$E_e = \left[\frac{\sum_i h_i E_i^{1/3}}{T}\right]^3$$

$$= \left[\frac{50 \times 2200^{1/3} + 125 \times 2500^{1/3} + 200 \times 448^{1/3}}{50 + 125 + 200}\right]^3$$

$$= 950 \text{ MPa}$$

Table 6.5 shows that the maximum modulus of the top granular sublayer achievable with 375 mm material with effective modulus of 950 MPa is 210 MPa. This value exceeds the maximum modulus of the top granular sublayer permissible by the underlying support to the granular layer of 152 MPa determined above.

Therefore, the design modulus of the top granular sublayer is 152 MPa, and the design modulus of the cracked cemented material is 448 MPa.

The elastic characteristics of the post-cracked heavily bound cemented material are:

E_V = 448 MPa (Section 8.2.7)

E_H = 224 MPa

$\nu_V = \nu_H$ = 0.35 (Section 8.2.7)

f = 332

No sublayering.

The moduli of the remaining granular sublayers can be determined. Divide the total granular layer thickness into five equi-thick sublayers (Section 8.2.3) each of thickness 200/5 – 40 mm.

Calculate the ratio of adjacent sublayers:

$$R = (152/50)^{1/5} = 1.249$$

Sublayer elastic properties calculation procedure is as shown in Appendix K.1.

Step 6

Asphalt

Size 14 mm asphalt:

$E_V = E_H$ = 2200 MPa

$\nu_V = \nu_H$ = 0.4

Size 20 mm asphalt:

$E_V = E_H$ = 2500 MPa

$\nu_V = \nu_H$ = 0.4

Elastic properties for all material are listed in Table 12.33.

Step 7

Step 7 is not relevant.

Table 12.33: Elastic properties of pavement with cemented material subbase

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f
		E_V	E_H	ν_V	ν_H	
Size 14 mm asphalt	50	2200	2200	0.4	0.4	–
Size 20 mm asphalt	125	2500	2500	0.4	0.4	–
Heavily bound cemented material Pre-cracked/post-cracked	200	3000/448	3000/224	0.2/0.35	0.2/0.35	–/332
Granular	40	152	76	0.35	0.35	112.6
Granular	40	122	61	0.35	0.35	90.4
Granular	40	98	49	0.35	0.35	72.6
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	34.48

Step 8

Subgrade strain criterion as in Appendix K.1.

Step 9

Heavily bound cemented material fatigue criterion.

Calculate the fatigue constant K of the in-service fatigue relationship using the design modulus of 3000 MPa and the design flexural strength of 1.2 MPa (Equation 14):

$$K = 240FS + \frac{919300}{E} - 285$$
$$= 240 \times 1.2 + \frac{919300}{3000} - 285 = 309$$

Using Equation 13 check the K value does not exceed the maximum values of K (K_{max} = 345).

Using K = 309 and Equation 10, the in-service fatigue relationship is:

$$N = RF \times \left(\frac{309}{\mu\varepsilon}\right)^{12}$$

Reliability Factor, RF = 0.5 (Table 6.9).

Step 10

Asphalt fatigue criterion as in Appendix K.2.

For the sake of brevity, the upper asphalt layer is not examined in this example.

Step 11

N_{DT} = 10^7 HVAG

TLD is Table 12.10 in Appendix F

Step 12

Design ESA loading, DESA as in Appendix K.1.

Step 13

Standard Axle load as in Appendix K.1.

Step 14

Critical locations to calculate strains are:

- top of subgrade
- bottom of asphalt layer
- bottom of cemented material layer.

All of the above strains are calculated directly below the inner of the loaded tyres and midway between the loaded tyres.

Step 15

Critical strains resulting from Standard Axle load as follows.

Pre-cracking heavily bound cemented material phase:

- asphalt – maximum horizontal tensile strain of 18 $\mu\varepsilon$, located under the inner loaded tyre
- cemented material – maximum horizontal tensile strain of 88 $\mu\varepsilon$, located between the loaded tyres
- subgrade – maximum vertical compressive strain of 211 $\mu\varepsilon$, located between the loaded tyres.

Post-cracking heavily bound cemented material phase:

- asphalt – maximum horizontal tensile strain of 161 $\mu\varepsilon$, located under the inner loaded tyre
- subgrade – maximum vertical compressive strain of 393 $\mu\varepsilon$, located between the loaded tyres.

Step 16

Single axle with single tyres load as in Appendix K.2.

Step 17

Critical locations to calculate strains under single axle with single tyres with 53 kN load as follows.

Pre-cracking heavily bound cemented material phase:

- bottom of asphalt
- bottom of heavily bound cemented material.

Post-cracking heavily bound cemented material phase:

- bottom of asphalt.

Step 18

Critical strains resulting from single axle with single tyres with 53 kN as follows.

Pre-cracking heavily cemented material phase:

- asphalt – maximum horizontal tensile strain of 24 με
- cemented material – maximum horizontal tensile strain of 64 με.

Post-cracking heavily cemented material phase:

- asphalt – maximum horizontal tensile strain of 153 με.

Step 19

Permanent deformation allowable loading – from Step 8:

Pre-cracking heavily bound cemented material phase:

$$N_{1stS} = \left[\frac{9150}{211}\right]^{7} = 2.88 \times 10^{11}\ ESA$$

Post-cracking heavily bound cemented material phase:

$$N_{2ndS} = \left[\frac{9150}{393}\right]^{7} = 3.71 \times 10^{9} ESA$$

Step 20

As the post-cracking phase of the heavily bound cemented materials is being considered, the total allowable ESA is determined in Step 29.

Step 21

Asphalt and heavily bound cemented materials are present in the candidate structure, therefore Steps 22 to 28 must be repeated for both the asphalt and the cemented material.

Asphalt Fatigue Damage

Step 22 (asphalt)

Steps 23 (asphalt) to 27 (asphalt) are repeated for each axle group type present in the distribution:

- single axle with single tyres/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.

Step 23 (asphalt)

The expected repetitions of each load level for each axle group type as in Appendix K.2.

Step 24 (asphalt)

Pre-cracking cemented material phase

The allowable loading of each axle group type and load level is calculated in Table 12.34. As a first stage the critical asphalt strain for each load level is estimated for a single constituent axle of each axle group type using Equation 45.

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SAST,53}}{53} = \frac{L_{ij}}{n} \times \frac{24}{53}$$ for axles within SAST, TAST groups

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SADT,80}}{80} = \frac{L_{ij}}{n} \times \frac{18}{80}$$ for axles within SADT, TADT and TRDT groups

For example, the critical asphalt strain developed under an axle within a triaxle group (TRDT) with a total group load of 120 kN is calculated as:

$$\mu\varepsilon_{\mathrm{TRDT,120}} = \frac{120}{3} \times \frac{18}{80} = 9.00\ \mu\varepsilon$$

The allowable repetitions of each axle group type and load magnitude is then calculated using Equation 46 (Section 8.2.6):

$$N_{ij} = \frac{1}{n} \times \frac{SF}{RF}\left[\frac{6918(0.856V_b + 1.08)}{E^{0.36}\mu\varepsilon_{ij}}\right]^5$$

For example, the allowable repetitions of a triaxle group (TRDT) with a total group load of 120 kN is calculated as:

$$N_{\mathrm{TRDT,120}} = \frac{1}{3} \times \frac{6}{9}\left[\frac{6918(0.856 \times 11 + 1.08)}{2500^{0.36} \times 9.00}\right]^5 = 5.81 \times 10^{12}$$

The allowable repetitions of all axle group/load combinations are shown in Table 12.34 to Table 12.38.

Table 12.34: Calculation of asphalt damage for pre-cracking cemented material phase – SAST

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	11 020	1	4.5	5.58E+14	1.98E-11
20	307 601	1	9.1	1.65E+13	1.86E-08
30	607 578	1	13.6	2.21E+12	2.75E-07
40	617 403	1	18.1	5.30E+11	1.16E-06
50	1 176 642	1	22.6	1.75E+11	6.74E-06
60	915 297	1	27.2	6.92E+10	1.32E-05
70	255 529	1	31.7	3.22E+10	7.95E-06
80	31 216	1	36.2	1.66E+10	1.88E-06
90	4 272	1	40.8	9.11E+09	4.69E-07
100	1 391	1	45.3	5.40E+09	2.58E-07
110	684	1	49.8	3.36E+09	2.03E-07
120	684	1	54.3	2.18E+09	3.14E-07
130	684	1	58.9	1.45E+09	4.71E-07
				Total SAST damage	3.30E-05

Table 12.35: Calculation of asphalt damage for pre-cracking cemented material phase – SADT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	66 334	1	2.3	1.60E+16	4.15E-12
20	166 094	1	4.5	5.58E+14	2.98E-10
30	448 086	1	6.8	7.08E+13	6.33E-09
40	418 863	1	9.0	1.74E+13	2.40E-08
50	320 880	1	11.3	5.59E+12	5.74E-08
60	183 475	1	13.5	2.30E+12	7.99E-08
70	124 150	1	15.8	1.05E+12	1.19E-07
80	88 299	1	18.0	5.45E+11	1.62E-07
90	56 708	1	20.3	2.99E+11	1.90E-07
100	26 606	1	22.5	1.79E+11	1.49E-07
110	7 827	1	24.8	1.10E+11	7.13E-08
120	2 212	1	27.0	7.17E+10	3.08E-08
130	466	1	29.3	4.77E+10	9.78E-09
				Total SADT damage	9.00E-07

Table 12.36: Calculation of asphalt damage for pre-cracking cemented material phase – TAST

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	32	2	2.3	8.00E+15	3.98E-15
20	214	2	4.5	2.79E+14	7.67E-13
30	249	2	6.8	3.54E+13	7.02E-12
40	518	2	9.1	8.25E+12	6.28E-11
50	2 600	2	11.3	2.79E+12	9.31E-10
60	9 243	2	13.6	1.11E+12	8.35E-09
70	15 129	2	15.8	5.23E+11	2.89E-08
80	14 949	2	18.1	2.65E+11	5.64E-08
90	14 355	2	20.4	1.46E+11	9.85E-08
100	12 978	2	22.6	8.73E+10	1.49E-07
110	8 797	2	24.9	5.38E+10	1.64E-07
120	5 313	2	27.2	3.46E+10	1.54E-07
130	2 649	2	29.4	2.34E+10	1.13E-07
140	1 385	2	31.7	1.61E+10	8.61E-08
150	760	2	34.0	1.13E+10	6.70E-08
160	385	2	36.2	8.28E+09	4.65E-08
170	208	2	38.5	6.09E+09	3.41E-08
180	123	2	40.8	4.55E+09	2.70E-08
190	65	2	43.0	3.50E+09	1.86E-08
200	50	2	45.3	2.70E+09	1.85E-08
				Total TAST damage	1.07E-06

Table 12.37: Calculation of asphalt damage for pre-cracking cemented material phase – TADT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	3 740	2	1.1	3.20E+17	1.17E-14
20	14 905	2	2.3	8.00E+15	1.86E-12
30	16 167	2	3.4	1.13E+15	1.43E-11
40	51 204	2	4.5	2.79E+14	1.84E-10
50	168 246	2	5.6	9.35E+13	1.80E-09
60	246 335	2	6.8	3.54E+13	6.96E-09
70	283 346	2	7.9	1.67E+13	1.69E-08
80	253 017	2	9.0	8.72E+12	2.90E-08
90	197 125	2	10.1	4.90E+12	4.02E-08
100	187 568	2	11.3	2.79E+12	6.71E-08
110	162 315	2	12.4	1.76E+12	9.24E-08
120	154 157	2	13.5	1.15E+12	1.34E-07
130	152 240	2	14.6	7.76E+11	1.96E-07
140	169 231	2	15.8	5.23E+11	3.24E-07
150	207 977	2	16.9	3.73E+11	5.57E-07
160	148 070	2	18.0	2.72E+11	5.44E-07
170	92 049	2	19.1	2.03E+11	4.55E-07
180	48 252	2	20.3	1.49E+11	3.23E-07
190	22 106	2	21.4	1.15E+11	1.93E-07
200	8 627	2	22.5	8.93E+10	9.66E-08
210	2 075	2	23.6	7.03E+10	2.95E-08
220	834	2	24.8	5.49E+10	1.52E-08
230	414	2	25.9	4.42E+10	9.38E-09
				Total TADT damage	3.13E-06

Table 12.38: Calculation of asphalt damage for pre-cracking cemented material phase – TRDT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	74	3	0.8	1.05E+18	7.07E-17
20	2 321	3	1.5	4.52E+16	5.14E-14
30	4 869	3	2.3	5.33E+15	9.13E-13
40	19 492	3	3.0	1.41E+15	1.38E-11
50	61 672	3	3.8	4.33E+14	1.42E-10
60	109 801	3	4.5	1.86E+14	5.90E-10
70	144 700	3	5.3	8.21E+13	1.76E-09
80	123 402	3	6.0	4.41E+13	2.80E-09
90	91 020	3	6.8	2.36E+13	3.86E-09
100	74 429	3	7.5	1.45E+13	5.15E-09
110	54 775	3	8.3	8.71E+12	6.29E-09
120	48 810	3	9.0	5.81E+12	8.40E-09
130	46 576	3	9.8	3.80E+12	1.23E-08
140	49 743	3	10.5	2.69E+12	1.85E-08
150	59 318	3	11.3	1.86E+12	3.18E-08
160	60 902	3	12.0	1.38E+12	4.42E-08
170	71 321	3	12.8	9.99E+11	7.14E-08
180	90 236	3	13.5	7.65E+11	1.18E-07
190	114 448	3	14.3	5.74E+11	1.99E-07
200	124 808	3	15.0	4.52E+11	2.76E-07
210	76 013	3	15.8	3.49E+11	2.18E-07
220	34 617	3	16.5	2.81E+11	1.23E-07
230	11 491	3	17.3	2.21E+11	5.19E-08
240	3 704	3	18.0	1.82E+11	2.04E-08
250	1 339	3	18.8	1.46E+11	9.17E-09
260	118	3	19.5	1.22E+11	9.73E-10
				Total TRDT damage	1.22E-06

Post-cracking cemented material phase

The allowable loading of each axle group type and load level is calculated in Table 12.39 to Table 12.43.

Steps 25 and 26 (asphalt)

The asphalt fatigue damage resulting from each axle load is calculated as the number of expected repetitions of the load divided by the allowable number of repetitions, as shown in the last column of Table 12.34 to Table 12.38 for the pre-cracking phase, and Table 12.39 to Table 12.43 for the post-cracking phase.

Step 27 (asphalt)

The asphalt fatigue damage resulting from each axle group type is the sum of the damage caused by each load level of the group. These are summarised for both cemented material phases in Table 12.44.

Table 12.39: Calculation of asphalt damage for post-cracking cemented material phase – SAST

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	11 020	1	28.9	5.11E+10	2.16E-07
20	307 601	1	57.7	1.61E+09	1.91E-04
30	607 578	1	86.6	2.11E+08	2.87E-03
40	617 403	1	115.5	5.01E+07	1.23E-02
50	1 176 642	1	144.3	1.65E+07	7.15E-02
60	915 297	1	173.2	6.61E+06	1.39E-01
70	255 529	1	202.1	3.05E+06	8.37E-02
80	31 216	1	230.9	1.57E+06	1.99E-02
90	4 272	1	259.8	8.70E+05	4.91E-03
100	1 391	1	288.7	5.13E+05	2.71E-03
110	684	1	317.5	3.19E+05	2.14E-03
120	684	1	346.4	2.06E+05	3.31E-03
130	684	1	375.3	1.38E+05	4.95E-03
				Total SAST damage	0.347

Table 12.40: Calculation of asphalt damage for post-cracking cemented material phase – SADT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	66 334	1	20.1	3.14E+11	2.11E-07
20	166 094	1	40.3	9.69E+09	1.71E-05
30	448 086	1	60.4	1.28E+09	3.50E-04
40	418 863	1	80.5	3.05E+08	1.38E-03
50	320 880	1	100.6	9.99E+07	3.21E-03
60	183 475	1	120.8	4.00E+07	4.58E-03
70	124 150	1	140.9	1.85E+07	6.70E-03
80	88 299	1	161.0	9.52E+06	9.28E-03
90	56 708	1	181.1	5.29E+06	1.07E-02
100	26 606	1	201.3	3.11E+06	8.54E-03
110	7 827	1	221.4	1.94E+06	4.04E-03
120	2 212	1	241.5	1.25E+06	1.76E-03
130	466	1	261.6	8.40E+05	5.55E-04
				Total SADT damage	0.051

Table 12.41: Calculation of asphalt damage for post-cracking cemented material phase – TAST

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	32	2	14.4	8.31E+11	3.83E-11
20	214	2	28.9	2.55E+10	8.38E-09
30	249	2	43.3	3.38E+09	7.35E-08
40	518	2	57.7	8.05E+08	6.44E-07
50	2 600	2	72.2	2.62E+08	9.91E-06
60	9 243	2	86.6	1.06E+08	8.75E-05
70	15 129	2	101.0	4.90E+07	3.09E-04
80	14 949	2	115.5	2.50E+07	5.97E-04
90	14 355	2	129.9	1.39E+07	1.03E-03
100	12 978	2	144.3	8.23E+06	1.58E-03
110	8 797	2	158.8	5.10E+06	1.73E-03
120	5 313	2	173.2	3.30E+06	1.61E-03
130	2 649	2	187.6	2.22E+06	1.20E-03
140	1 385	2	202.1	1.53E+06	9.07E-04
150	760	2	216.5	1.08E+06	7.02E-04
160	385	2	230.9	7.84E+05	4.91E-04
170	208	2	245.4	5.78E+05	3.59E-04
180	123	2	259.8	4.35E+05	2.83E-04
190	65	2	274.2	3.32E+05	1.96E-04
200	50	2	288.7	2.57E+05	1.95E-04
				Total TAST damage	0.011

Table 12.42: Calculation of asphalt damage for post-cracking cemented material phase – TADT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	3 740	2	10.1	4.90E+12	7.64E-10
20	14 905	2	20.1	1.57E+11	9.50E-08
30	16 167	2	30.2	2.05E+10	7.89E-07
40	51 204	2	40.3	4.84E+09	1.06E-05
50	168 246	2	50.3	1.60E+09	1.05E-04
60	246 335	2	60.4	6.40E+08	3.85E-04
70	283 346	2	70.4	2.98E+08	9.52E-04
80	253 017	2	80.5	1.52E+08	1.66E-03
90	197 125	2	90.6	8.43E+07	2.34E-03
100	187 568	2	100.6	5.00E+07	3.75E-03
110	162 315	2	110.7	3.10E+07	5.24E-03
120	154 157	2	120.8	2.00E+07	7.70E-03
130	152 240	2	130.8	1.34E+07	1.13E-02
140	169 231	2	140.9	9.27E+06	1.83E-02
150	207 977	2	150.9	6.58E+06	3.16E-02
160	148 070	2	161.0	4.76E+06	3.11E-02
170	92 049	2	171.1	3.51E+06	2.62E-02
180	48 252	2	181.1	2.64E+06	1.83E-02
190	22 106	2	191.2	2.01E+06	1.10E-02
200	8 627	2	201.3	1.56E+06	5.54E-03
210	2 075	2	211.3	1.22E+06	1.70E-03
220	834	2	221.4	9.68E+05	8.62E-04
230	414	2	231.4	7.76E+05	5.34E-04
				Total TADT damage	0.179

Table 12.43: Calculation of asphalt damage for post-cracking cemented material phase – TRDT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	74	3	6.7	2.54E+13	2.91E-12
20	2 321	3	13.4	7.94E+11	2.92E-09
30	4 869	3	20.1	1.05E+11	4.66E-08
40	19 492	3	26.8	2.48E+10	7.85E-07
50	61 672	3	33.5	8.13E+09	7.58E-06
60	109 801	3	40.3	3.23E+09	3.40E-05
70	144 700	3	47.0	1.50E+09	9.67E-05
80	123 402	3	53.7	7.69E+08	1.61E-04
90	91 020	3	60.4	4.27E+08	2.13E-04
100	74 429	3	67.1	2.52E+08	2.95E-04
110	54 775	3	73.8	1.57E+08	3.49E-04
120	48 810	3	80.5	1.02E+08	4.81E-04
130	46 576	3	87.2	6.81E+07	6.84E-04
140	49 743	3	93.9	4.70E+07	1.06E-03
150	59 318	3	100.6	3.33E+07	1.78E-03
160	60 902	3	107.3	2.41E+07	2.52E-03
170	71 321	3	114.0	1.78E+07	4.00E-03
180	90 236	3	120.8	1.33E+07	6.76E-03
190	114 448	3	127.5	1.02E+07	1.12E-02
200	124 808	3	134.2	7.88E+06	1.58E-02
210	76 013	3	140.9	6.18E+06	1.23E-02
220	34 617	3	147.6	4.90E+06	7.07E-03
230	11 491	3	154.3	3.92E+06	2.93E-03
240	3 704	3	161.0	3.17E+06	1.17E-03
250	1 339	3	167.7	2.59E+06	5.18E-04
260	118	3	174.4	2.13E+06	5.57E-05
				Total TRDT damage	0.070

Table 12.44: Total group asphalt fatigue damage for both cemented material phases

Axle group type	Total group asphalt fatigue damage	
	Pre-cracking phase	Post-cracking phase
SAST	3.30×10^{-5}	0.347
SADT	9.00×10^{-7}	0.051
TAST	1.07×10^{-6}	0.011
TADT	3.13×10^{-6}	0.179
TRDT	1.22×10^{-6}	0.070

Step 28 (asphalt)

Total asphalt fatigue damage is the sum of damage resulting from each axle group type:

Pre-cracking heavily bound cemented material phase:	$3.30 \times 10^{-5} + 9.00 \times 10^{-7} + 1.07 \times 10^{-6} + 3.13 \times 10^{-6} + 1.22 \times 10^{-6} = 3.93 \times 10^{-5}$
Post-cracking heavily bound cemented material phase:	0.347 + 0.051 + 0.011 + 0.179 + 0.070 = 0.658

Heavily bound cemented material fatigue damage

Step 22 (heavily bound cemented material)

Steps 23 (heavily bound cemented material) to 27 (heavily bound cemented material) are repeated for each axle group type present in the distribution:

- single axle with single tyres/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.

Step 23 (heavily bound cemented material)

The expected repetitions of each load level for each axle group type as in Appendix K.2.

Step 24 (heavily bound cemented material)

Pre-cracking cemented material phase

The allowable loading of each axle group type and load level is calculated in Table 12.45. As a first stage the critical cemented material strain for each load level is estimated for a single constituent axle of each axle group type using Equation 45.

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SAST,53}}{53} = \frac{L_{ij}}{n} \times \frac{64}{53}$$ for axles within SAST, TAST groups

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SADT,80}}{80} = \frac{L_{ij}}{n} \times \frac{88}{80}$$ for axles within SADT, TADT and TRDT groups

For example, the critical cemented material strain developed under an axle within a tandem group with dual tyres (TADT) with a total group load of 130 kN is calculated as:

$$\mu\varepsilon_{\mathrm{TADT,130}} = \frac{130}{2} \times \frac{88}{80} = 71.5\ \mu\varepsilon$$

The allowable repetitions of each axle group type and load magnitude is then calculated using Equation 47:

$$N_{ij} = \frac{1}{n} \times RF \times \left(\frac{K}{\mu\varepsilon_{ij}}\right)^{12} = \frac{1}{n} \times 0.5 \times \left(\frac{309}{\mu\varepsilon_{ij}}\right)^{12}$$

For example, the allowable repetitions of a tandem group with dual tyres (TADT) with a total group load of 130 kN is calculated as:

$$N_{\mathrm{TADT,130}} = \frac{1}{2} \times 0.5 \times \left(\frac{309}{71.5}\right)^{12} = 1.06 \times 10^{7}$$

The allowable repetitions of all axle group/load combinations are shown in Table 12.45 to Table 12.49.

Table 12.45: Calculation of heavily bound cemented material damage for pre-cracking phase – SAST

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	11 020	1	12.1	3.85E+16	2.86E-13
20	307 601	1	24.2	9.39E+12	3.28E-08
30	607 578	1	36.2	7.48E+10	8.12E-06
40	617 403	1	48.3	2.35E+09	2.63E-04
50	1 176 642	1	60.4	1.61E+08	7.32E-03
60	915 297	1	72.5	1.80E+07	5.10E-02
70	255 529	1	84.5	2.86E+06	8.94E-02
80	31 216	1	96.6	5.74E+05	5.44E-02
90	4 272	1	108.7	1.39E+05	3.07E-02
100	1 391	1	120.8	3.92E+04	3.55E-02
110	684	1	132.8	1.26E+04	5.43E-02
120	684	1	144.9	4.42E+03	1.55E-01
130	684	1	157.0	1.69E+03	4.05E-01
				Total SAST damage	0.882

Table 12.46: Calculation of heavily bound cemented material damage for pre-cracking phase – SADT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	66 334	1	11.0	1.21E+17	5.50E-13
20	166 094	1	22.0	2.95E+13	5.64E-09
30	448 086	1	33.0	2.27E+11	1.97E-06
40	418 863	1	44.0	7.20E+09	5.82E-05
50	320 880	1	55.0	4.94E+08	6.49E-04
60	183 475	1	66.0	5.55E+07	3.31E-03
70	124 150	1	77.0	8.72E+06	1.42E-02
80	88 299	1	88.0	1.76E+06	5.03E-02
90	56 708	1	99.0	4.27E+05	1.33E-01
100	26 606	1	110.0	1.21E+05	2.20E-01
110	7 827	1	121.0	3.85E+04	2.03E-01
120	2 212	1	132.0	1.35E+04	1.63E-01
130	466	1	143.0	5.18E+03	8.99E-02
				Total SADT damage	0.878

Table 12.47: Calculation of heavily bound cemented material damage for pre-cracking phase – TAST

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	32	2	6.0	8.70E+19	3.66E-19
20	214	2	12.1	1.92E+16	1.11E-14
30	249	2	18.1	1.53E+14	1.62E-12
40	518	2	24.2	4.70E+12	1.10E-10
50	2 600	2	30.2	3.29E+11	7.90E-09
60	9 243	2	36.2	3.74E+10	2.47E-07
70	15 129	2	42.3	5.77E+09	2.62E-06
80	14 949	2	48.3	1.18E+09	1.27E-05
90	14 355	2	54.3	2.88E+08	4.98E-05
100	12 978	2	60.4	8.04E+07	1.62E-04
110	8 797	2	66.4	2.58E+07	3.41E-04
120	5 313	2	72.5	8.98E+06	5.91E-04
130	2 649	2	78.5	3.46E+06	7.66E-04
140	1 385	2	84.5	1.43E+06	9.69E-04
150	760	2	90.6	6.19E+05	1.23E-03
160	385	2	96.6	2.87E+05	1.34E-03
170	208	2	102.6	1.39E+05	1.49E-03
180	123	2	108.7	6.96E+04	1.77E-03
190	65	2	114.7	3.65E+04	1.78E-03
200	50	2	120.8	1.96E+04	2.55E-03
				Total TAST damage	0.013

Table 12.48: Calculation of heavily bound cemented material damage for pre-cracking phase – TADT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	3 740	2	5.5	2.47E+20	1.51E-17
20	14 905	2	11.0	6.04E+16	2.47E-13
30	16 167	2	16.5	4.65E+14	3.48E-11
40	51 204	2	22.0	1.47E+13	3.47E-09
50	168 246	2	27.5	1.01E+12	1.66E-07
60	246 335	2	33.0	1.14E+11	2.17E-06
70	283 346	2	38.5	1.79E+10	1.59E-05
80	253 017	2	44.0	3.60E+09	7.03E-05
90	197 125	2	49.5	8.75E+08	2.25E-04
100	187 568	2	55.0	2.47E+08	7.59E-04
110	162 315	2	60.5	7.88E+07	2.06E-03
120	154 157	2	66.0	2.77E+07	5.56E-03
130	152 240	2	71.5	1.06E+07	1.43E-02
140	169 231	2	77.0	4.36E+06	3.88E-02
150	207 977	2	82.5	1.91E+06	1.09E-01
160	148 070	2	88.0	8.78E+05	1.69E-01
170	92 049	2	93.5	4.24E+05	2.17E-01
180	48 252	2	99.0	2.14E+05	2.26E-01
190	22 106	2	104.5	1.12E+05	1.98E-01
200	8 627	2	110.0	6.04E+04	1.43E-01
210	2 075	2	115.5	3.36E+04	6.17E-02
220	834	2	121.0	1.92E+04	4.34E-02
230	414	2	126.5	1.13E+04	3.67E-02
				Total TADT damage	1.265

Table 12.49: Calculation of heavily bound cemented material damage for pre-cracking phase – TRDT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	74	3	3.7	1.92E+22	3.86E-21
20	2 321	3	7.3	5.51E+18	4.21E-16
30	4 869	3	11.0	4.02E+16	1.21E-13
40	19 492	3	14.7	1.24E+15	1.57E-11
50	61 672	3	18.3	8.95E+13	6.89E-10
60	109 801	3	22.0	9.82E+12	1.12E-08
70	144 700	3	25.7	1.52E+12	9.51E-08
80	123 402	3	29.3	3.15E+11	3.91E-07
90	91 020	3	33.0	7.57E+10	1.20E-06
100	74 429	3	36.7	2.12E+10	3.52E-06
110	54 775	3	40.3	6.88E+09	7.96E-06
120	48 810	3	44.0	2.40E+09	2.04E-05
130	46 576	3	47.7	9.10E+08	5.12E-05
140	49 743	3	51.3	3.80E+08	1.31E-04
150	59 318	3	55.0	1.65E+08	3.60E-04
160	60 902	3	58.7	7.55E+07	8.07E-04
170	71 321	3	62.3	3.69E+07	1.93E-03
180	90 236	3	66.0	1.85E+07	4.88E-03
190	114 448	3	69.7	9.61E+06	1.19E-02
200	124 808	3	73.3	5.25E+06	2.38E-02
210	76 013	3	77.0	2.91E+06	2.61E-02
220	34 617	3	80.7	1.66E+06	2.09E-02
230	11 491	3	84.3	9.80E+05	1.17E-02
240	3 704	3	88.0	5.86E+05	6.33E-03
250	1 339	3	91.7	3.57E+05	3.75E-03
260	118	3	95.3	2.25E+05	5.26E-04
				Total TRDT damage	0.113

Steps 25 and 26 (heavily bound cemented material)

The cemented material damage resulting from each axle load is calculated as the number of expected repetitions of the load divided by the allowable number of repetitions, as shown in the last column of Table 12.45 to Table 12.49.

Step 27 (heavily bound cemented material)

The cemented material damage resulting from each axle group type is the sum of the damage caused by each load level of the group. These are summarised for both cemented material phases in Table 12.50.

Table 12.50: Total group heavily bound cemented material fatigue damage for pre-cracking phase

Axle group type	Total group cemented material fatigue damage
SAST	0.882
SADT	0.878
TAST	0.013
TADT	1.265
TRDT	0.113

Step 28 (heavily bound cemented material)

Total damage to cemented material is the sum of damage resulting from each axle group type:

$$0.882 + 0.878 + 0.013 + 1.265 + 0.113 = 3.151$$

As this damage exceeds 1.0, design traffic exceeds the allowable loading for the cemented material.

Allowable Loading in ESA

Step 29

As the thickness of asphalt 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 8.2.7). That is, the allowable loading is the sum of the loading to fatigue cracking of the cemented material plus the loading for each distress mode (i.e. asphalt fatigue, permanent deformation) post-cracking (Section 8.2.7). This calculation requires the allowable loadings to be converted into units of ESA.

Pre-cracking phase:

Heavily bound cemented material fatigue:

Equation 53 is used to estimate the allowable ESA loading (A_{ESA}) for the heavily bound cemented material using the total damage, D, calculated in Step 28 (heavily bound cemented material), the design traffic, N_{DT}, (HVAG) of 10^7 and the average $ESA/HVAG$ determined for the design traffic load distribution of 0.7:

$$N_C = \frac{N_{DT} \times ESA/HVAG}{D} = \frac{10^7 \times 0.7}{3.151} = 2.23 \times 10^6 ESA$$

Asphalt fatigue:

$$N_{1stA} = \frac{N_{DT} \times ESA/HVAG}{D} = \frac{10^7 \times 0.7}{3.93 \times 10^{-5}} = 1.78 \times 10^{11} ESA$$

Permanent deformation: from Step 19:

$$N_{1stS} = 2.88 \times 10^{11} ESA$$

Post-cracking phase:

Asphalt fatigue:

$$N_{2ndA} = \frac{N_{DT} \times ESA/HVAG}{D} = \frac{10^7 \times 0.7}{0.658} = 1.07 \times 10^7 ESA$$

Permanent deformation: from Step 19:

$$N_{2ndS} = 3.71 \times 10^9 ESA$$

As discussed in Section 8.2.7, the total allowable loading of the pre-cracking and post-cracking phases are:

Permanent deformation allowable loading using Equation 55:

$$N_S = N_C + \left(1 - \frac{N_C}{N_{1stS}}\right) \times N_{2ndS} = 2.23 \times 10^6 + \left(1 - \frac{2.23 \times 10^6}{2.88 \times 10^{11}}\right) \times 3.71 \times 10^9 = 3.71 \times 10^9 ESA$$

Asphalt fatigue allowable loading using Equation 54:

$$N_A = N_C + \left(1 - \frac{N_C}{N_{1stA}}\right) \times N_{2ndA} = 2.23 \times 10^6 + \left(1 - \frac{2.23 \times 10^6}{1.79 \times 10^{11}}\right) \times 1.07 \times 10^7 = 1.29 \times 10^7 ESA$$

The design number of ESA, DESA from Step 11:

$$DESA = 7 \times 10^6$$

As the allowable loading for both asphalt fatigue and permanent deformation exceed the design traffic, the candidate pavement is acceptable.

K.4 Sprayed seal surfaced lightly bound cemented material base pavement

Following the steps in Table 8.1, Table 8.2 and Table 8.3:

Step 1

Try pavement composition shown in Table 12.51.

Table 12.51: Candidate pavement: sprayed seal surfaced lightly bound cemented material base pavement

Material type	Thickness (mm)
Sprayed seal surface	–
Lightly bound cemented material	260
Unbound granular material	200
Subgrade, design CBR = 5%	Semi-infinite

The lightly bound cemented material is manufactured from granular materials with a laboratory soaked CBR of greater than 30%.

Step 2

Subgrade as in Appendix K.1.

Step 3

The modulus of the granular material is dependent upon the modulus of the overlying lightly bound cemented material.

Additionally, the maximum vertical modulus of the lightly bound cemented material is limited by the modulus of the underlying support, i.e. the modulus of the granular material. An iterative approach is necessary.

The subgrade support underlying the granular material will limit the maximum modulus that can be considered. Using Equation 42 in Section 8.2.3, the maximum modulus of the top granular material sublayer allowed by the subgrade support is calculated.

$E_{V\ top\ granular\ sublayer}$ is the minimum of Equation 42 in Section 8.2.3 and the value indicated in Table 6.4 (assuming normal standard crushed rock).

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

$$= 152 \text{ MPa (Section 8.2.3)}$$

$$E_{V\ \text{top granular sublayer}} = 350 \text{ MPa (Table 6.4, assuming normal standard crushed rock)}$$

$$E_{V\ \text{top granular sublayer}} = \text{minimum (152 MPa, 350 MPa)} = 152 \text{ MPa}$$

$$E_{H\ \text{top granular sublayer}} = 76 \text{ MPa}$$

$$\nu_V = \nu_H = 0.35 \text{ (Table 6.3)}$$

An iterative approach is needed to determine the moduli of the lightly bound cemented material and the granular material. This is covered in Step 7.

Step 4

Covered in Step 7.

Steps 5 and 6 are not relevant.

Step 7

The maximum modulus of the lightly bound cemented material is limited by the modulus of the underlying support, i.e. the modulus of the granular material.

From Section 6.6.2, the maximum permissible design modulus, for a material manufactured from a granular material with a laboratory soaked CBR of 30% or greater, is limited to the value obtained from Figure 6.6 reflecting the degree of support provided by the underlying material.

The maximum lightly bound cemented modulus permitted by the maximum granular material top sublayer modulus of 152 MPa is 600 MPa.

With an upper bound limit of the lightly bound cemented material modulus determined, the maximum modulus of material overlying the granular material can be determined, as 600 MPa.

Table 6.4 shows that 260 mm of 600 MPa material limits the top granular material sublayer modulus to 188 MPa. This value exceeds the maximum modulus of the top granular sublayer permissible by the underlying support to the granular layer of 152 MPa determined above.

Therefore, the design modulus of the top granular sublayer is 152 MPa, and the design modulus of the lightly bound cemented material is 600 MPa.

The moduli of the remaining granular sublayers are determined by calculating the ratio of adjacent sublayers:

$$R = (152/50)^{1/5} = 1.249$$

The minimum base thickness of lightly bound cemented material is determined using Equation 44.

$$\begin{aligned} t_{min} &= \text{maximum}\left[200, 250 + 35\log_{10}\left(\frac{DESA}{5 \times 10^6}\right) - 0.25\left(E_{V\ \text{underlying material}} - 150\right)\right] \\ &= \text{maximum}\left[200, 250 + 35\log_{10}\left(\frac{0.7 \times 10^7}{5 \times 10^6}\right) - 0.25(152 - 150)\right] \\ &= 255 \text{ mm} \end{aligned}$$

The candidate lightly bound cemented material thickness of 260 mm exceeds this 255 mm minimum base thickness, and therefore the material is suitable as base with the material staying in a micro-cracked only state during its life.

$E_V = E_H$ = 600 MPa

$\nu_V = \nu_H$ = 0.35

f = 444.4

Table 12.52: Elastic properties of all materials

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f
		E_V	E_H	ν_V	ν_H	
Lightly bound cemented material	260	600	600	0.35	0.35	444.4
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

Step 8

Subgrade strain criterion as in Appendix K.1.

Steps 9 to 10 are not relevant.

Step 11

N_{DT} = 10^7 HVAG

TLD is Table 12.10 in Appendix F

Step 12

$$\text{DESA} = 0.7 \times N_{DT}$$
$$= 7 \times 10^{6} \text{ ESA}$$

Step 13

Standard axle load as in Appendix K.1.

Step 14

Critical locations to calculate strains are:

- top of subgrade directly beneath the inner tyre load of one of the dual tyre sets
- top of subgrade midway between one of the dual tyre sets.

Step 15

Maximum vertical compressive strain from CIRCLY is 827 με, located midway between the two loaded tyres.

Steps 16 to 18 are not relevant.

Step 19

Permanent deformation allowable loading – from Step 8:

$$N = \left[\frac{9150}{827}\right]^{7} = 2.0 \times 10^{7} ESA$$

Step 20

The allowable loading is 2.0×10^{7} ESA compared to the design traffic 0.7×10^{7} ESA from Step 12. The candidate structure is acceptable in terms of permanent deformation.

Steps 21 to 31 are not relevant.

Appendix L Examples of Use of the Empirical Design Charts for Granular Pavements with Thin Bituminous Surfacings

This appendix gives examples of the use of empirical design charts for granular pavements with a thin bituminous surfacing:

- Examples 1 and 2 illustrate the use of Figure 8.4 for moderate to heavily trafficked roads.
- Example 3 illustrates the use of Figure 12.2 for lightly trafficked roads.

L.1 Example 1: Utilising unbound granular materials

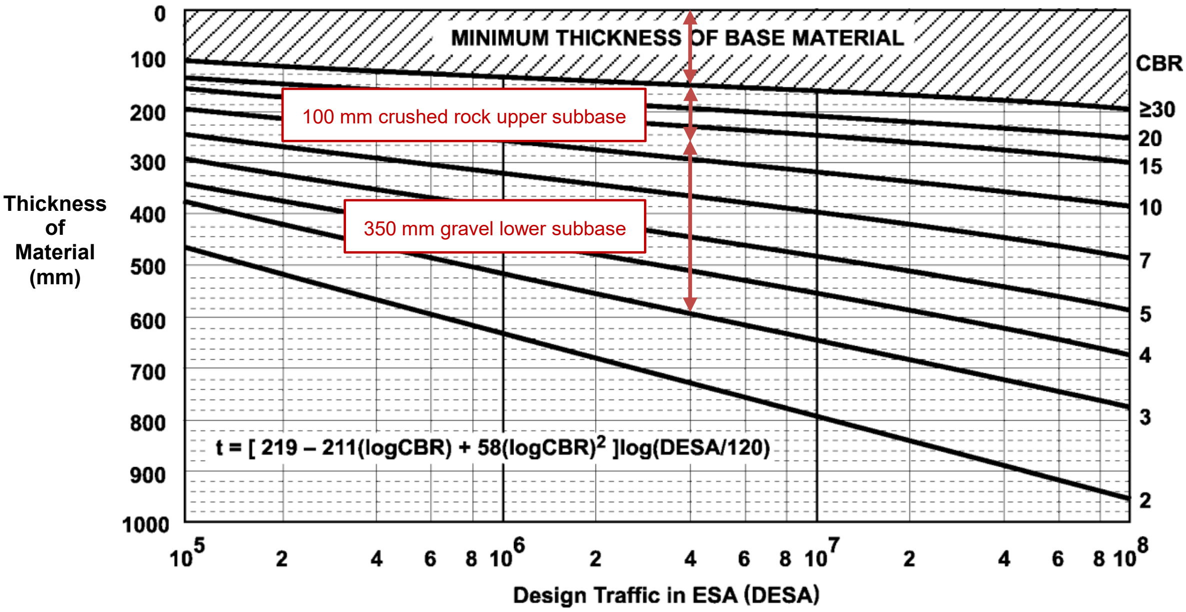
This design example utilises various qualities of granular materials and is based on the following design parameters:

- sprayed bituminous seal surface
- subgrade design CBR = 3%
- design traffic = 4 × 10^6 ESA.

The design process is described in Section 8.3 and illustrated in Figure 8.3.

Utilising Figure 8.4 (reproduced below), the total thickness of material over a subgrade with a design CBR = 3% is 600 mm, as illustrated in Figure L.1.

Figure L.1: Example 1 use of Figure 8.4 to obtain total thickness of material over subgrade and the pavement composition



To evaluate the material qualities required to provide this 600 mm thickness, consider the properties of the three granular materials available for use:

- crushed rock base (CBR ≥ 80%)
- crushed rock upper subbase (CBR ≥ 30%)
- gravel lower subbase (CBR ≥ 15%).

As seen from Figure L.1, the top 150 mm of granular material needs to be base quality material with a minimum CBR of 80%. Of the three available materials only the crushed rock base is suitable for this layer.

The material immediately below the base layer needs to have a CBR of at least 30%. Both the base and upper subbase materials are suitable. As the upper subbase quality is lower in cost, it is decided to utilise it rather than the base material. The minimum practical layer thickness is 100 mm, so it is decided to utilise the upper subbase material between 150 mm and 250 mm below the sprayed seal surface.

At a depth of 250 mm below the surface, the pavement material requires a minimum design CBR of about 13% to inhibit deformation. Although all three available granular materials meet this minimum strength requirement, the gravel is selected due to its lower cost. This layer is 350 mm (600–250 mm) thick. To ensure adequate compaction, it is placed in three layers.

Table 12.53 summarises the Example 1 pavement composition.

Table 12.53: Example 1 final design

Material type	Thickness (mm)
Sprayed seal surface	–
Crushed rock – base	150
Crushed rock – upper subbase	100
Gravel – lower subbase (three layers)	350
Subgrade, design CBR = 3%	–

L.2 Example 2: Utilising crushed rocks and selected or improved subgrade materials

Like Example 1, this example is based on the following design parameters:

- sprayed bituminous seal surface
- subgrade design CBR = 3%
- design traffic = 4 × 10^6 ESA.

It utilises various qualities of crushed rock and improved subgrade material with a laboratory measured CBR of 7%.

The iterative steps to determine the pavement composition are described in Section 8.3.2.

Step 1 Trial selected subgrade thickness

A trial selected subgrade thickness of 200 mm is chosen.

Step 2 Design CBR of selected subgrade

The design CBR of improved subgrade is the minimum of (1) 15%, (2) the value determined from CBR tests, in this case 7%, and (3) the value determined from the support provided by the underlying material (i.e. in situ subgrade, improved or selected subgrade material) as follows:

$$CBR_{\text{selected or improved subgrade}} = CBR_{\text{underlying material}} \times 2^{\left(\frac{\text{thickness of selected or improved subgrade}}{150}\right)}$$

$$= 3 \times 2^{(200/150)} = 8\%$$

The adopted design CBR of the improved subgrade material is the minimum of 15%, 7% and 8%. Hence a design CBR of 7% is adopted.

Step 3 Select granular material types and thicknesses

From Figure L.2, improved subgrade with a design CBR of 7% requires 370 mm cover of granular materials. The properties of the available granular materials are:

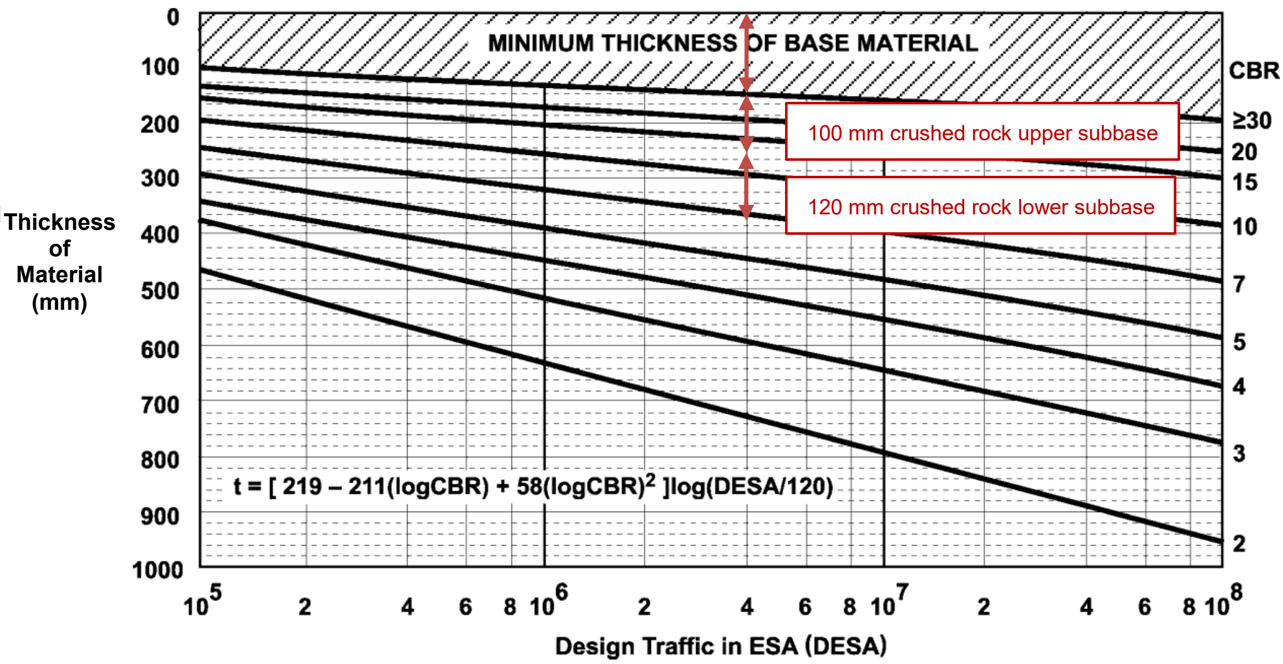
- crushed rock base (CBR ≥ 80%)
- crushed rock upper subbase (CBR ≥ 30%)
- crushed rock lower subbase (CBR ≥ 15%).

A 150 mm thickness of crushed rock base is proposed. Therefore there is a need for an additional 220 mm (370 mm – 150 mm) of granular material in addition to the base.

The material immediately below the granular base layer needs to have a CBR of at least 30%; hence a 100 mm thickness of upper subbase quality crushed rock is adopted.

Below the upper subbase layer, at a depth of 250 mm below the surface, material with a minimum CBR of about 13% is required. It is proposed to adopt a 120 mm thickness of lower subbase quality crushed rock below the upper subbase material and above the improved subgrade material.

Figure L.2: Example 2 use of Figure 8.4 to obtain total thickness of material with improved or selected subgrade and the pavement composition



Step 4 Check whether the total thickness of cover over in situ subgrade is adequate

Table 12.54 summarises the trial pavement option.

Table 12.54: Example 2 trial design

Material type	Thickness (mm)
Sprayed seal surface	–
Crushed rock – base	150
Crushed rock – upper subbase	100
Crushed rock – lower subbase	120
Improved subgrade material, design CBR = 7%	200
In situ subgrade, design CBR = 3%	–

The total thickness of cover over the in situ subgrade is 570 mm. However, this thickness is inadequate as Figure L.2 indicates the required minimum thickness over the in situ subgrade (CBR = 3%) is 600 mm.

Step 5 Increase improved subgrade thickness and repeat steps 1 to 4

The improved subgrade thickness is increased to 230 mm. The design CBR of the improved subgrade material design CBR remains at 7%, despite the increase in its thickness. Hence the granular materials types and thicknesses remain unchanged. The final pavement design is given in Table 12.55.

Table 12.55: Example 2 final design

Material type	Thickness (mm)
Sprayed seal surface	–
Crushed rock – base	150
Crushed rock – upper subbase	100
Crushed rock – lower subbase	120
Improved subgrade material, design CBR = 7%	230
In situ subgrade, design CBR = 3%	–

L.3 Example 3: Utilising crushed rocks and lime-stabilised subgrade materials

This example illustrates the design of lightly trafficked pavement based on the following design parameters:

- sprayed bituminous seal surface
- subgrade design CBR = 3%
- design traffic = 10^4 ESA.

The design process is described in Section 12.8.2 and the Figure 12.2 design chart.

This design example utilises crushed rock and lime-stabilised subgrade. Based on appropriate testing, as defined in the *Part 4D: Stabilised Materials* (Austroads 2019a), a long-term CBR strength of 10% has been adopted for the lime-stabilised subgrade using 4% lime.

The iterative steps to determine the pavement composition are described in Section 8.3.2.

Step 1 Trial lime-stabilised subgrade thickness

A trial lime-stabilised subgrade thickness of 150 mm is selected.

Step 2 Design CBR of stabilised subgrade

The design CBR of the lime-stabilised subgrade is the minimum of (1) 15%, (2) the value determined from CBR tests, in this case 10%, and (3) the value determined from the support provided by the underlying material (i.e. in situ subgrade or improved or selected subgrade material) as follows:

$$CBR_{\text{stabilised subgrade}} = CBR_{\text{underlying material}} \times 2^{\left(\frac{\text{thickness of stabilised subgrade}}{150}\right)} = 3 \times 2^{(150/150)} = 6\%$$

Hence the adopted design CBR of the lime-stabilised subgrade material is 6%.

Step 3 Design CBR of lime-stabilised subgrade

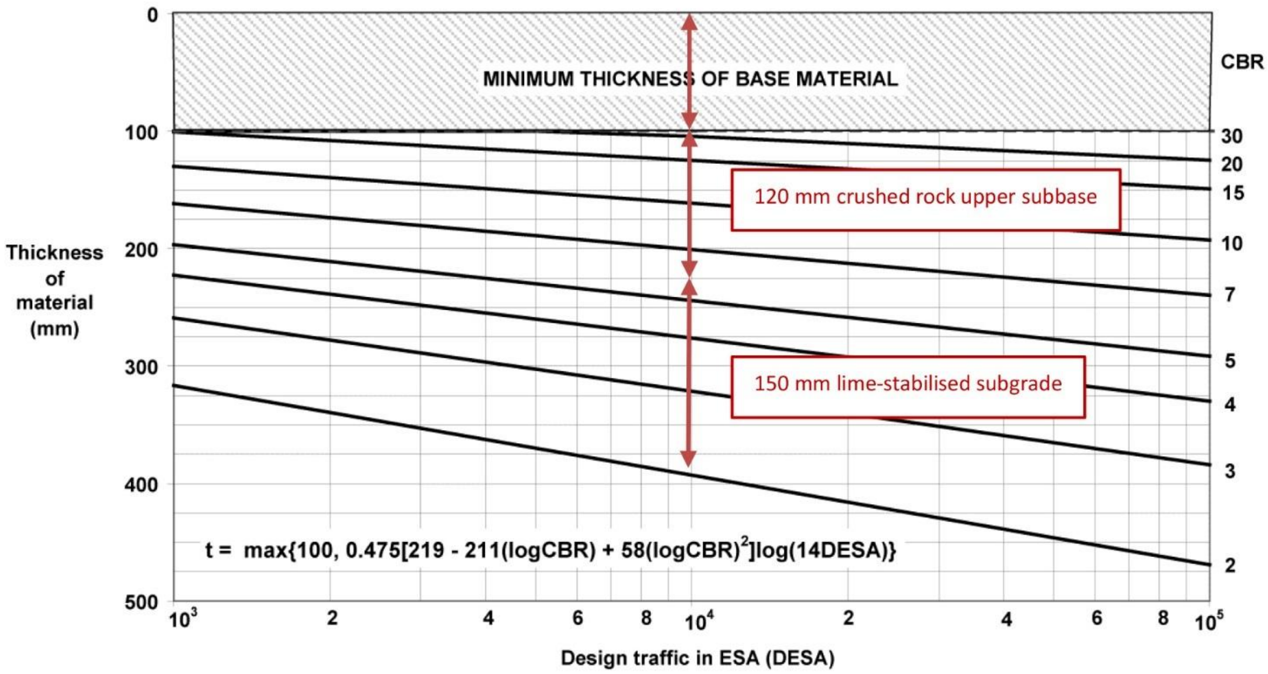
From Figure L.3, a stabilised subgrade with a design CBR of 6% requires a minimum 220 mm of cover. The properties of the available granular materials are:

- crushed rock base (CBR ≥ 80%)
- crushed rock upper subbase (CBR ≥ 30%)
- crushed rock lower subbase (CBR ≥ 15%).

From Figure L.3, a 100 mm thickness of crushed rock base is proposed. Therefore there is a need for an additional 120 mm (220 mm – 100 mm) of granular material in addition to the base.

The material immediately below the granular base layer needs to have a CBR at least 30%; hence a 120 mm thickness of upper subbase quality crushed rock is adopted.

Figure L.3: Example 3 use of Figure 12.2 to obtain total thickness of material with lime-stabilised subgrade and the pavement composition



Step 4 Check whether the total thickness of cover over in situ subgrade is adequate

Table 12.56 summarises the pavement option. The total thickness of cover over the in situ subgrade is 370 mm. As Figure L.3 indicates a minimum thickness of 320 mm is required over the in situ subgrade, the pavement design is acceptable.

Table 12.56: Example 3 final design

Material type	Thickness (mm)
Sprayed seal surface	–
Crushed rock – base	100
Crushed rock – upper subbase	120
Lime-stabilised subgrade, design CBR = 6%	150
In situ subgrade, design CBR = 3%	–

Appendix M Examples of Use of the Design Procedure for Rigid Pavements

This appendix gives examples of the use of the base thickness design procedure for a plain concrete pavement (PCP) without dowelled joints 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 load distribution (TLD) is in accordance with the example distribution in Appendix F.

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 subbase (LCS) is required for the design traffic (4×10^7 HVAG).
4. From Figure 9.1, for a 150 mm LCS and a subgrade design CBR of 5%, effective subgrade design 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 Appendix F is entered in the first three columns of Table 12.57 to Table 12.61 (one table for each axle group type in the TLD), together with the design traffic in HVAG into column 4. The expected repetitions of each load on each axle group type (column 5) are then calculated as the product of the entries in columns 2, 3 and 4. The axle load value and the corresponding expected repetitions are then transferred to columns 1 and 3 of the damage calculation tables (Table 12.62 to Table 12.66, one table for each axle group type).
9. Using Equation 58 and Equation 59, allowable repetitions in fatigue distress mode are calculated for the SAST axle group and are entered into column 4 of Table 12.62 to Table 12.66.
10. The ratio of the expected fatigue repetitions and the allowable repetitions is calculated and multiplied by 100 before entering into column 5 of Table 12.62 to Table 12.66.
11. Allowable repetitions for erosion are calculated using Equation 61 and are entered into column 6 of Table 12.62 to Table 12.66.
12. The ratio of the expected erosion repetitions and the allowable repetitions is calculated and multiplied by 100 before entering into column 7 of Table 12.62 to Table 12.66.
13. Steps 9 to 12 are repeated for each load on this axle group up to a load level where the allowable load repetitions exceed 10^8.
14. Total percentage fatigue and erosion are calculated for all relevant loads in this axle group.
15. Steps 9 to 14 are repeated for all other axle group types (i.e. TAST, SADT, TADT, TRDT and QADT).
16. Total percentages for fatigue and erosion for all axle group types are calculated.
17. Total percentage for fatigue = 124.69 + 0.19 + 0.46 + 0.00 + 0.00 = 125%.

18. Total percentage for erosion = 5.98 + 46.42 + 34.39 + 32.07 + 0.52 = 119%.
19. As in Step 16 both fatigue and erosion were not less than or equal to 100%, it is necessary to increase the base thickness and repeat steps 7 to 16 until both the total percentage fatigue and erosion are less than 100%.
20. Selected final base thickness = 215 mm.
21. Total percentage fatigue = 66%.
22. Total percentage erosion = 93%.
23. Selected final base thickness = 215 mm exceeds the minimum base thickness requirement (200 mm from Table 9.7).

Table 12.57: Calculation of expected repetitions – single axle/single tyre (SAST)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
130	0.000174	0.393	40 000 000	2 735
120	0.000174	0.393	40 000 000	2 735
110	0.000174	0.393	40 000 000	2 735
100	0.000354	0.393	40 000 000	5 565
90	0.001087	0.393	40 000 000	17 088
80	0.007943	0.393	40 000 000	124 864
70	0.06502	0.393	40 000 000	1 022 114
60	0.2329	0.393	40 000 000	3 661 188
50	0.2994	0.393	40 000 000	4 706 568
40	0.1571	0.393	40 000 000	2 469 612
30	0.1546	0.393	40 000 000	2 430 312
20	0.07827	0.393	40 000 000	1 230 404
10	0.002804	0.393	40 000 000	44 079

Table 12.58: Calculation of expected repetitions – tandem axle group/single tyre (TAST)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
200	0.000555	0.009	40 000 000	200
190	0.000723	0.009	40 000 000	260
180	0.001367	0.009	40 000 000	492
170	0.002308	0.009	40 000 000	831
160	0.004279	0.009	40 000 000	1 540
150	0.008439	0.009	40 000 000	3 038
140	0.01539	0.009	40 000 000	5 540
130	0.02943	0.009	40 000 000	10 595
120	0.05903	0.009	40 000 000	21 251
110	0.09774	0.009	40 000 000	35 186
100	0.1442	0.009	40 000 000	51 912
90	0.1595	0.009	40 000 000	57 420
80	0.1661	0.009	40 000 000	59 796
70	0.1681	0.009	40 000 000	60 516
60	0.1027	0.009	40 000 000	36 972

Table 12.59: Calculation of expected repetitions – single axle/single tyres (SADT)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
130	0.000244	0.191	40 000 000	1 864
120	0.001158	0.191	40 000 000	8 847
110	0.004098	0.191	40 000 000	31 309
100	0.01393	0.191	40 000 000	106 425
90	0.02969	0.191	40 000 000	226 832
80	0.04623	0.191	40 000 000	353 197
70	0.06500	0.191	40 000 000	496 600
60	0.09606	0.191	40 000 000	733 898
50	0.1680	0.191	40 000 000	1 283 520
40	0.2193	0.191	40 000 000	1 675 452
30	0.2346	0.191	40 000 000	1 792 344
20	0.08696	0.191	40 000 000	664 374
10	0.03473	0.191	40 000 000	265 337

Table 12.60: Calculation of expected repetitions – tandem axle group/dual tyres (TADT)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
230	0.000160	0.259	40 000 000	1 658
220	0.000322	0.259	40 000 000	3 336
210	0.000801	0.259	40 000 000	8 298
200	0.003331	0.259	40 000 000	34 509
190	0.008535	0.259	40 000 000	88 423
180	0.01863	0.259	40 000 000	193 007
170	0.03554	0.259	40 000 000	368 194
160	0.05717	0.259	40 000 000	592 281
150	0.08030	0.259	40 000 000	831 908
140	0.06534	0.259	40 000 000	676 922
130	0.05878	0.259	40 000 000	608 961
120	0.05952	0.259	40 000 000	616 627
110	0.06267	0.259	40 000 000	649 261
100	0.07242	0.259	40 000 000	750 271
90	0.07611	0.259	40 000 000	788 500
80	0.09769	0.259	40 000 000	1 012 068
70	0.1094	0.259	40 000 000	1 133 384
60	0.09511	0.259	40 000 000	985 340

Table 12.61: Calculation of expected repetitions – triaxle group/dual tyres (TRDT)

Axle group load (kN)	Proportion of loads (%/100)	Proportion of axle group (%/100)	Design traffic (HVAG)	Expected repetitions
260	0.00008	0.148	40 000 000	474
250	0.000905	0.148	40 000 000	5 358
240	0.002503	0.148	40 000 000	14 818
230	0.007764	0.148	40 000 000	45 963
220	0.02339	0.148	40 000 000	138 469
210	0.05136	0.148	40 000 000	304 051
200	0.08433	0.148	40 000 000	499 234
190	0.07733	0.148	40 000 000	457 794
180	0.06097	0.148	40 000 000	360 942
170	0.04819	0.148	40 000 000	285 285
160	0.04115	0.148	40 000 000	243 608
150	0.04008	0.148	40 000 000	237 274
140	0.03361	0.148	40 000 000	198 971
130	0.03147	0.148	40 000 000	186 302
120	0.03298	0.148	40 000 000	195 242
110	0.03701	0.148	40 000 000	219 099

Table 12.62: Calculation of allowable repetitions and damaged caused by each group load – SAST

Axle group load (kN)	Design load (kN)	Expected repetitions	Equivalent stress 0.751		Erosion factor 1.865	
			Fatigue analysis		Erosion analysis	
			Allowable repetitions	Damage (%)	Allowable repetitions	Damage (%)
130	169.0	2 735	2 974	91.96	162 052	1.69
120	156.0	2 735	11 812	23.15	269 239	1.02
110	143.0	2 735	47 245	5.79	483 605	0.57
100	130.0	5 565	196 228	2.84	977 194	0.57
90	117.0	17 088	1 808 199	0.95	2 419 503	0.71
80	104.0	124 864	Unlimited	0.00	9 462 726	1.32
70	91.0	1 022 114	Unlimited	0.00	978 333 565	0.10
			Total fatigue %	124.69	Total erosion %	5.98

Table 12.63: Calculation of allowable repetitions and damaged caused by each group load – TAST

Axle group load (kN)	Design load (kN)	Expected repetitions	Equivalent stress 0.751		Erosion factor 2.456	
			Fatigue analysis		Erosion analysis	
			Allowable repetitions	Damage (%)	Allowable repetitions	Damage (%)
200	260.0	200	196 228	0.10	14 262	1.40
190	247.0	260	494 164	0.05	19 043	1.37
180	234.0	492	1 808 199	0.03	25 860	1.90
170	221.0	831	16 212 762	0.01	35 814	2.32
160	208.0	1 540	Unlimited	0.00	50 762	3.03
150	195.0	3 038	Unlimited	0.00	73 978	4.11
140	182.0	5 540	Unlimited	0.00	111 567	4.97
130	169.0	10 595	Unlimited	0.00	175 728	6.03
120	156.0	21 251	Unlimited	0.00	293 156	7.25
110	143.0	35 186	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
			Total fatigue %	0.19	Total erosion %	46.42

Table 12.64: Calculation of allowable repetitions and damaged caused by each group load – SADT

Axle group load (kN)	Design load (kN)	Expected repetitions	Equivalent stress 1.064		Erosion factor 2.465	
			Fatigue analysis		Erosion analysis	
			Allowable repetitions	Damage (%)	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	1 864	601 365	0.31	163 912	1.14
120	156.0	8 847	5 962 495	0.15	272 483	3.25
110	143.0	31 309	Unlimited	0.00	489 873	6.39
100	130.0	106 425	Unlimited	0.00	991 416	10.73
90	117.0	226 832	Unlimited	0.00	2 462 593	9.21
80	104.0	353 197	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	733 898	Unlimited	0.00	Unlimited	0.00
			Total fatigue %	0.46	Total erosion %	34.39

Table 12.65: Calculation of allowable repetitions and damaged caused by each group load – TADT

Axle group load (kN)	Design load (kN)	Expected repetitions	Equivalent stress 0.889		Erosion factor 2.456	
			Fatigue analysis		Erosion analysis	
			Allowable repetitions	Damage (%)	Allowable repetitions	Damage (%)
240	312.0	0	Unlimited	0.00	293 156	0.00
230	299.0	1 658	Unlimited	0.00	389 462	0.43
220	286.0	3 336	Unlimited	0.00	530 027	0.63
210	273.0	8 298	Unlimited	0.00	743 295	1.12
200	260.0	34 509	Unlimited	0.00	1 083 356	3.19
190	247.0	88 423	Unlimited	0.00	1 662 739	5.32
180	234.0	193 007	Unlimited	0.00	2 746 341	7.03
170	221.0	368 194	Unlimited	0.00	5 082 000	7.25
160	208.0	592 281	Unlimited	0.00	11 519 479	5.14
150	195.0	831 908	Unlimited	0.00	42 456 964	1.96
140	182.0	676 922	Unlimited	0.00	Unlimited	0.00
			Total fatigue %	0.00	Total erosion %	32.07

Table 12.66: Calculation of allowable repetitions and damaged caused by each group load – TRDT

Axle group load (kN)	Design load (kN)	Expected repetitions	Equivalent stress 0.733		Erosion factor 2.452	
			Fatigue analysis		Erosion analysis	
			Allowable repetitions	Damage (%)	Allowable repetitions	Damage (%)
270	351.0	0	Unlimited	0.00	2 845 859	0.00
260	338.0	474	Unlimited	0.00	4 236 980	0.01
250	325.0	5 358	Unlimited	0.00	6 786 246	0.08
240	312.0	14 818	Unlimited	0.00	12 191 794	0.12
230	299.0	45 963	Unlimited	0.00	27 021 679	0.17
220	286.0	138 469	Unlimited	0.00	101 961 571	0.14
210	273.0	304 051	Unlimited	0.00	Unlimited	0.00
200	260.0	499 234	Unlimited	0.00	Unlimited	0.00
			Total fatigue %	0.00	Total erosion %	0.52

Appendix N Traffic Load Distributions for Lightly Trafficked Roads

This appendix provides example traffic load distributions for different types of lightly trafficked roads. The distributions were derived from data reported by Matthews and Mulholland (1994).

Table 12.67: Example traffic load distribution – collector with buses

Axle group load (kN)	Axle group type		
	SAST %	SADT %	TADT %
24.5	0.0000	13.7184	0.0000
34.3	14.4086	0.0000	0.0000
39.2	29.8925	13.7184	18.4615
44.1	12.4731	0.0000	0.0000
49.0	10.3226	0.0000	0.0000
51.9	3.0108	0.0000	0.0000
52.9	23.6559	0.0000	0.0000
53.9	0.0000	13.7184	0.0000
63.7	6.2366	0.0000	0.0000
68.6	0.0000	17.3285	0.0000
75.5	0.0000	5.0542	0.0000
83.3	0.0000	36.4621	0.0000
93.1	0.0000	0.0000	22.3077
147.0	0.0000	0.0000	48.0769
Total	**100.0000**	**100.0000**	**100.0000**
Proportion of each axle group (%)	46.407	27.645	25.948

Measure	Value
N_{HVAG}	2.15
ESA/HVAG	0.64
ESA/HV	1.37

Table 12.68: Example traffic load distribution – collector with no buses

Axle group load (kN)	Axle group type		
	SAST %	**SADT %**	**TADT %**
24.5	0.0000	16.4122	0.0000
34.3	16.4080	0.0000	11.1498
39.2	30.5987	16.4122	18.4669
44.1	14.1907	0.0000	0.0000
49.0	11.7517	0.0000	0.0000
52.9	27.0510	0.0000	0.0000
53.9	0.0000	16.4122	0.0000
68.6	0.0000	20.2290	0.0000
83.3	0.0000	30.5344	0.0000
93.1	0.0000	0.0000	22.2997
147.0	0.0000	0.0000	48.0836
Total	**100.0000**	**100.0000**	**100.0000**
Proportion of each axle group (%)	45.100	26.200	28.700

Measure	Value
N_{HVAG}	2.22
ESA/HVAG	0.55
ESA/HV	1.23

Table 12.69: Example traffic load distribution – local access in industrial area

Axle group load (kN)	Axle group type		
	SAST %	**SADT %**	**TADT %**
24.5	0.0000	26.5957	0.0000
34.3	22.8311	0.0000	13.2979
39.2	39.9543	26.5957	13.2979
44.1	20.0913	0.0000	0.0000
49.0	5.7078	0.0000	0.0000
52.9	11.4155	0.0000	0.0000
53.9	0.0000	26.5957	0.0000
68.6	0.0000	13.2979	0.0000
83.3	0.0000	6.9149	0.0000
93.1	0.0000	0.0000	23.4043
147.0	0.0000	0.0000	50.0000
Total	**100.0000**	**100.0000**	**100.0000**
Proportion of each axle group (%)	43.713	18.762	37.525

Measure	Value
N_{HVAG}	2.29
ESA/HVAG	0.51
ESA/HV	1.16

Table 12.70: Example traffic load distribution – local access with buses

Axle group load (kN)	Axle group type		
	SAST %	SADT %	TADT %
24.5	0.0000	14.8241	0.0000
34.3	18.1443	0.0000	24.7863
39.2	39.3814	14.8241	24.7863
44.1	15.2577	0.0000	0.0000
49.0	5.9794	0.0000	0.0000
51.9	12.1649	0.0000	0.0000
52.9	3.0928	0.0000	0.0000
53.9	0.0000	14.8241	0.0000
62.7	0.0000	0.0000	12.8205
63.7	5.9794	0.0000	0.0000
66.6	0.0000	0.0000	24.7863
68.6	0.0000	29.6482	0.0000
75.5	0.0000	14.8241	0.0000
83.3	0.0000	11.0553	0.0000
91.1	0.0000	0.0000	12.8205
Total	**100.0000**	**100.0000**	**100.0000**
Proportion of each axle group (%)	48.500	39.800	11.700

Measure	Value
N_{HVAG}	2.06
ESA/HVAG	0.44
ESA/HV	0.91

Table 12.71: Example traffic load distribution – local access with no buses

Axle group load (kN)	Axle group type		
	SAST %	SADT %	TADT %
24.5	0.0000	26.6862	0.0000
34.3	28.5115	0.0000	24.8619
39.2	33.3333	26.6862	24.8619
44.1	23.8994	0.0000	0.0000
49.0	9.4340	0.0000	0.0000
52.9	4.8218	0.0000	0.0000
53.9	0.0000	26.6862	0.0000
62.7	0.0000	0.0000	12.7072
66.6	0.0000	0.0000	24.8619
68.6	0.0000	13.1965	0.0000
83.3	0.0000	6.7449	0.0000
91.1	0.0000	0.0000	12.7072
Total	**100.0000**	**100.0000**	**100.0000**
Proportion of each axle group (%)	47.748	34.134	18.118

Measure	Value
N_{HVAG}	2.09
ESA/HVAG	0.27
ESA/HV	0.56

Table 12.72: Example traffic load distribution – minor road

Axle group load (kN)	Axle group type		
	SAST %	SADT %	TADT %
44.1	85.8000	0.0000	0.0000
52.9	14.2000	0.0000	0.0000
53.9	0.0000	80.1120	0.0000
83.1	0.0000	19.8880	0.0000
91.1	0.0000	0.0000	100.0000
Total	**100.0000**	**100.0000**	**100.0000**
Proportion of each axle group (%)	50.000	35.700	14.300

Measure	Value
N_{HVAG}	2.00
ESA/HVAG	0.45
ESA/HV	0.89

Austroads' **Guide to Pavement Technology Part 2: Pavement Structural Design** provides advice for the structural design of sealed road pavements. It includes the assessment of input parameters needed for design, design methods for flexible and rigid pavements and gives guidance on the economic comparisons of alternative pavement designs. The advice has been generally developed from the approaches followed by the Austroads member agencies. However, as it encompasses the wide range of materials and conditions found in Australia and New Zealand, some parts are broadly based.

Guide to Pavement Technology Part 2

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