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Guide to Pavement Technology Part 5

Pavement Evaluation and Treatment Design

Austroads

Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design

Austroads

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Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design

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Abstract

Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design provides advice for the investigation of existing sealed road pavements and the selection and design of pavement strategies/treatments. It covers pavement investigation, testing and evaluation, identification of causes and modes of distress, and treatment options.

Knowledge of pavement technology is of critical importance for all transport agencies in Australia and New Zealand. Austroads and others (e.g. state 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 guide that will be a readily available, accessible 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.

The advice has been generally developed from the approaches followed by Austroads' member authorities. However, as it encompasses the wide range of materials and conditions found in Australia and New Zealand, some parts are broadly based. Treatment selection is related to availability of materials and knowledge of their performance in any particular locality.

Keywords

Pavement rehabilitation, pavement evaluation, pavement design, overlay, pavement treatment

Edition 4.1 published November 2019
Correction to removed and replaced existing unbound granular material value in Table L.1.

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- Editorial changes and updates to technical changes throughout.
- Changes to mechanistic-empirical design of strengthening treatments for flexible pavements.
- New design process for calculating granular overlays using Traffic Speed Deflectometer deflections.
- Deletion of asphalt overlay design charts.

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About Austroads

Austroads is the peak organisation of Australasian road transport and traffic 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
- Roads Corporation Victoria
- Queensland Department of Transport and Main Roads
- Main Roads Western Australia
- Department of Planning, Transport and Infrastructure South Australia
- Department of State Growth Tasmania
- Department of Infrastructure, Planning and Logistics Northern Territory
- Transport Canberra and City Services Directorate, Australian Capital Territory
- The Department of Infrastructure, Transport, Cities and Regional Development
- Australian Local Government Association
- New Zealand Transport Agency.

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1. Introduction

This Part of the Austroads *Guide to Pavement Technology* is intended to give the practitioner an overview of the pavement engineering techniques and procedures associated with the periodic maintenance and rehabilitation treatments commonly used for sealed pavements. This includes the identification of distress observed on the surface, the analysis of the distress and mechanisms causing it and the design of treatments aimed at restoring the pavement to a suitable condition. Guidance on unsealed pavements is contained in *Guide to Pavement Technology Part 6: Unsealed Pavements* (Austroads 2009a).

The Part contains descriptions of the following topics:

- causes and modes of pavement distress
- inspection and testing at the project level
- evaluation of pavement defects and test results
- selection of rehabilitation treatments
- thickness design of structural overlays and stabilisation treatments.

The subject area is often considered part of the broader area of asset management, but for the purposes of the Austroads publications library, a distinction has been made between pavement management at the network level and the project level focus of this Part. The reader is referred to the Austroads *Guide to Asset Management* for information on network level pavement management.

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
  - Part 4I: Earthworks Materials
  - Part 4J: Aggregate and Source Rock
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  - Part 4L: Stabilising Binders
- Part 5: Pavement Evaluation and Treatment Design
- Part 6: Unsealed Pavements
- Part 7: Pavement Maintenance
- Part 8: Pavement Construction
- Part 10: Subsurface Drainage.

It is emphasised that this document should be used only as a **guide** and not as a limiting or standard pavement rehabilitation specification. Considerable judgement is required to define the scope and nature of investigations of existing pavements and to select the parameters for incorporation into both the evaluation and design processes. In addition, some road agencies have published design manuals or supplements that translate the design guidance provided by Austroads into practice reflecting local materials, environments, loadings and pavement performance.

1.1 Overview of the Rehabilitation Design Process

The pavement rehabilitation design process is illustrated in Figure 1.1 and Figure 1.2. In essence, the process consists of the following steps:

- determine the purpose and scope of the project (Section 2)
- conduct a visual inspection and collate existing information (Section 3)
- undertake testing to establish the cause(s) and mode(s) of distress and to assist in selecting treatment options (Section 4 to Section 6)
- select treatment options considering functional and structures needs (Section 7 and Section 8)
- calculate the thicknesses of strengthening treatments (Section 9 to Section 12)
- compare the whole-of-life costs of treatment options (Section 13)
- report findings.

This Part assumes a need to treat a site that has already been identified through network inspections and forward works programming. However, it could be found after investigations are completed and interpreted that structural rehabilitation is not necessary as alternative treatments offer lower whole-of-life costs. The steps below outline the process needed to evaluate the most appropriate treatment and associated design assumptions:

1. **Preliminary visual and desktop investigation** – Undertaking the initial visual inspection of pavement defects: considering historical information about pavement type, maintenance, high-speed condition data (such as roughness, rutting and texture), performance, and past traffic; and then assembling information on the existing pavement. The road topography and geometry should be considered, particularly in regard to drainage. Site constraints such as those imposed by kerb and channel as well as underground utilities should also be considered. This data should be examined to identify any deficiencies or missing information.
2. **Testing pavement and in situ materials** – Testing the in situ pavement, including (but not limited to): deflection testing, test pits, dynamic cone penetrometer and laboratory testing any extracted pavement materials.
3. **Causes and modes of distress** – Conducting a root cause analysis by reviewing the results of the investigation to determine the distress modes and cause or causes of the pavement failure.
4. **Preliminary treatment design options** – Reviewing the failure modes and cause or causes to identify appropriate rehabilitation treatments. Associated design assumptions should also be identified. A key assessment is the ability of in situ materials to resist the accumulation of plastic deformation. The adequacy of pavement depths and subgrade strength should also be established. Some initial treatment designs should be developed for comparison. These options could range from a simple patch repair and reseal, a full width stabilisation, or an overlay.
5. **Comparison and treatment selection** – Comparing treatment options to determine which is the most appropriate based on: engineering (design life expected); appetite for risk; cost and budget. Select these treatments for the next stage (steps 6 and 7 below).
6. **Finalise the design** – Material parameters used in the modelling are based on the derived laboratory mix design results. However, consideration should be given to the ability to construct the pavement to achieve the assumed properties.
7. **Finalise design report.**

Figure 1.1: Stage 1: pavement evaluation

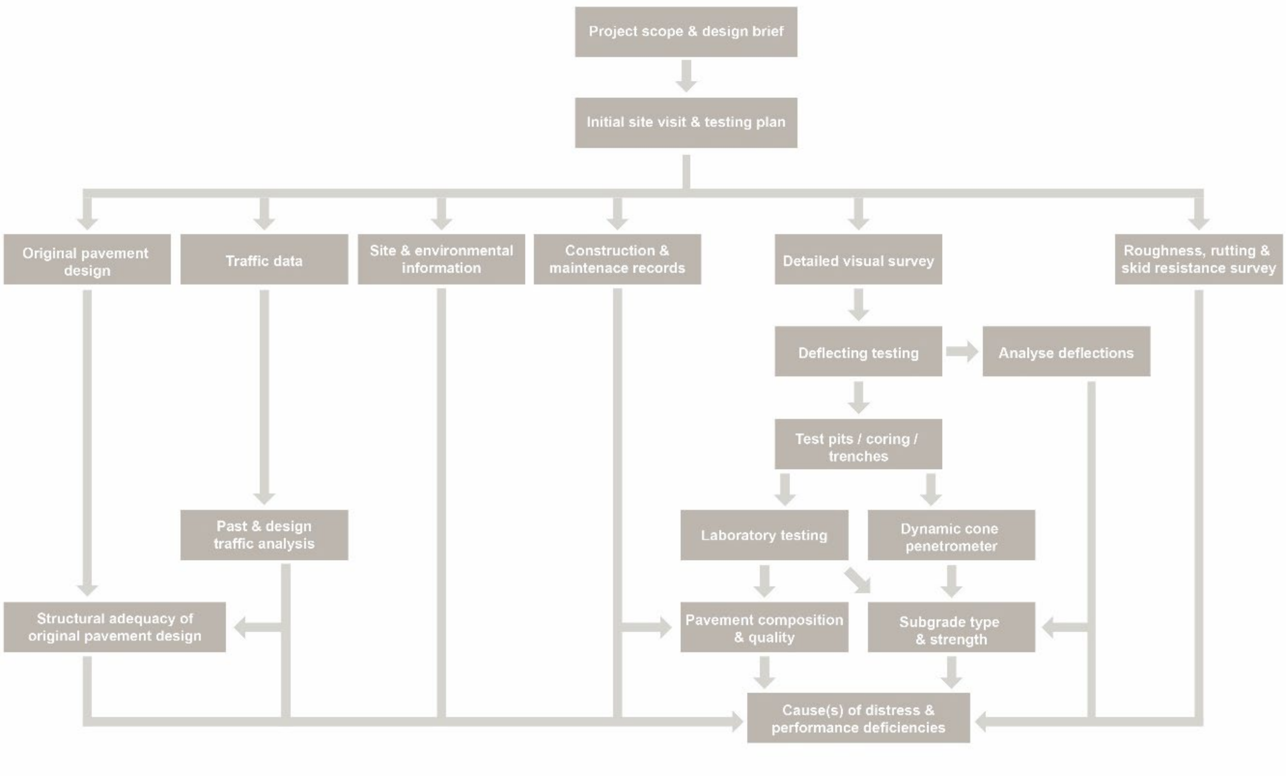
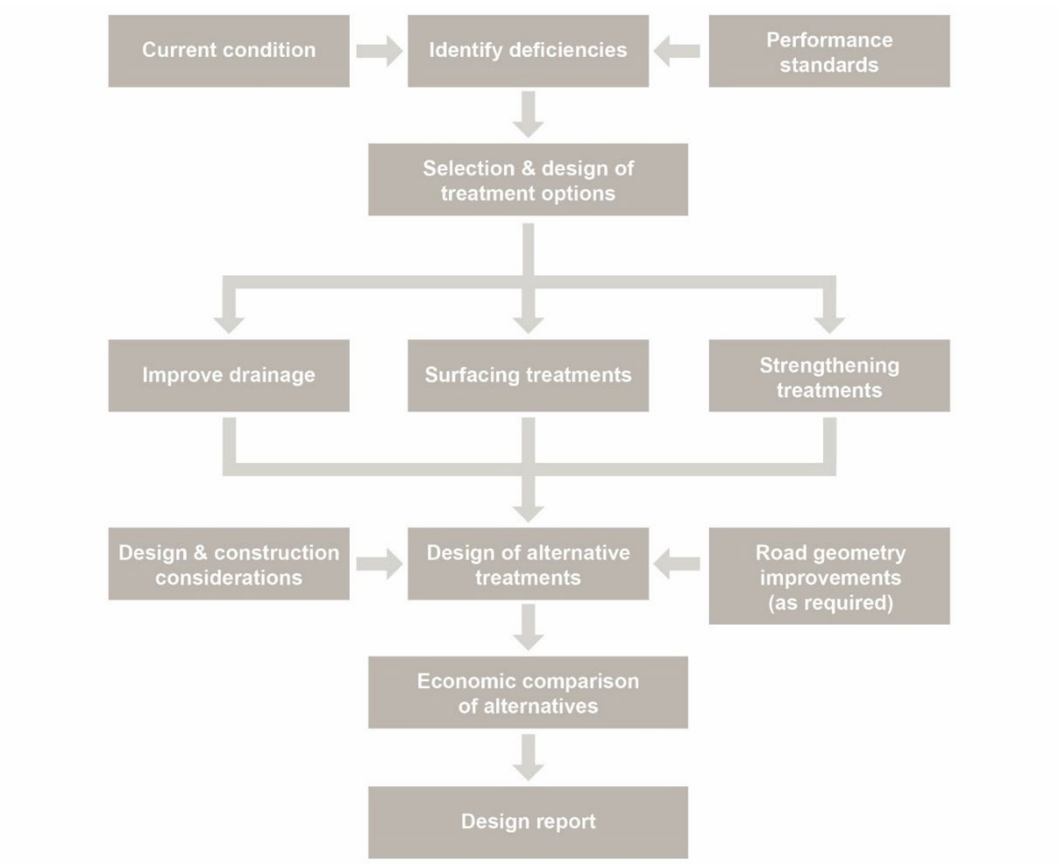


Figure 1.2: Stage 2: selection and design of treatments



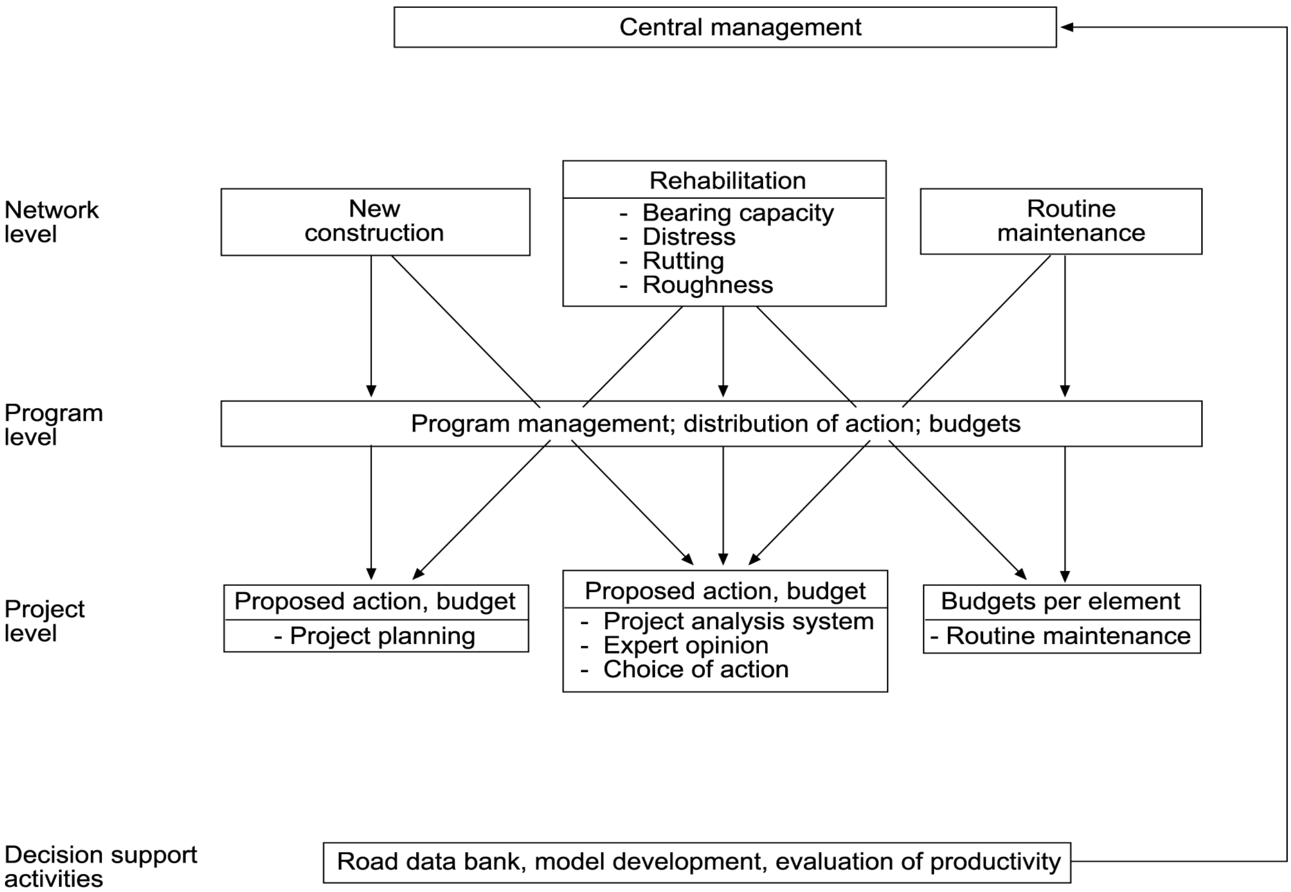
2. Project Definition

2.1 Project Scope

The first stage in any engineering endeavour is to understand 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, but more often it will be necessary to define these issues at the project planning stage.

For some road agencies the project scope will have been decided based on the agency's asset management system. A typical example of a process for identifying a project for detailed design as part of the overall asset management system is illustrated in Figure 2.1.

Figure 2.1: Designing project level treatments as part of the overall asset management process



Source: Organisation for Economic Co-operation and Development (OECD) (1994).

Asset management systems often contain only limited data, hence, the network and program level analyses undertaken as part of those systems can produce only broad treatment types. Therefore, treatment selection at a project level, based on more detailed data and an engineering assessment of the causes of the pavement distress, often leads to different treatments being implemented from the broad treatment types generated by network and program level analyses.

Since the project scope provides a framework for the required pavement investigation, analysis and design, it is crucial that it be clearly defined and understood by everyone involved in the project. It ensures that the focus is on developing the right solution for the problem.

In defining the scope of a project, it is essential to identify the purpose of the project and any constraints (e.g. budget, timing, product or process availability) which might influence the planning process. For example, the purpose may be to improve the functional performance (roughness, skid resistance) of the pavement or the structural performance (strength, modulus) of the pavement. In the former case, the treatment options will revolve around improving the surface characteristics of the pavement whilst, in the latter case, treatments such as overlays, recycling or stabilisation will need to be considered.

In either case, the desired life of the treatment will need to be determined if it has not already been specified by the client within the project brief. It is important that resources are not wasted, providing a costly long-life treatment when a cheaper, short-term, or interim, treatment would suffice. Equally, providing a short-term treatment may be inefficient where subsequent intervention is likely to be difficult or expensive. In some cases, it may be necessary to develop a ‘staged’ treatment plan if funding is not immediately available to implement the full treatment.

Before any pavement rehabilitation option can be applied, it is necessary to first identify the pavement distress and its cause(s). Clearly, the selection of the most appropriate treatment should 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 rehabilitation works must be determined because this will control many factors in the process, including the extent of pavement investigation works and the type of treatment that can be adopted. An economic comparison is recommended when comparing short-term, interim or staged approaches to long-term treatment solutions.

Table 2.1 provides a list of some suggested issues or questions that might be considered when scoping a project. In working through this list, a more complete understanding of the nature and extent of the design task should be obtained, including the client expectations of remedial treatments and constraints on treatment types.

Table 2.1: Project scope

Project size	• Type and length of road • Road geometry
Current deficiencies	• Visual manifestations (type, extent and severity) of the deficiencies • Possible causes (e.g. is the site location an influence on observed deficiencies) • Reasons why rehabilitation is being considered
Treatment objectives	• Project objectives and priorities (level of service, functional or structural improvement, level of future maintenance commitment) • Nature of any other road improvements or changes planned or to be incorporated at the time of rehabilitation, e.g. widenings, development proposals
Timing	• Timing and duration of investigations • Timing and duration of construction works • Staging of any or all of the investigation, design and construction phases
Funding	• Funding available for the investigation and design • Funding available for the construction works • Economic considerations – costs both initial and maintenance, service life, user benefits
Critical success factors	• Project timing, funding, constructability, innovation, public relationships, treatment risk
Treatment options	• Road agency policy or preferences • Acceptable risk of inadequate treatment performance • Alternatives and their evaluation • Trials

2.2 Background Data

While consideration of background data may be deferred until the pavement investigation phase (refer to Section 3), it is often useful and cost-effective to conduct a preliminary data search at the time of scoping the project to gain a better understanding of the project. Table 2.2 lists some of the more common background datasets.

Table 2.2: Background data

Road usage	• Nature of road users:
	- motorised road vehicles – commercial, private
	- cyclists, small-wheeled users
	- pedestrians – ages, level of mobility
	- services, e.g. garbage, cleaning, maintenance
	  - utilities
	  - underground services and facilities
	  - user management devices
	• Levels of usage – numbers, loads, time distribution, rate of change
	• Traffic records
	• Management of users during investigations and construction
	• Network significance
	• Other usage e.g. flood levee, floodway, stabilising beam

Site	• Climate both during construction and throughout the design period • Geology and terrain • Hydrology, rainfall and evaporation, drainage and depth to water table • Land use, e.g. industrial, commercial, residential or rural • Access • Traffic management during construction • Geometry – overhead heights, levels, widths, alignment, cross-section • Foundations and stability • Hazards, overhead obstructions • Utility services in the existing subgrade • Regional characteristics • Change – past, current and future site environment
Environment	• Planning regulations and considerations • Energy and resource conservation • Potential for reusing pavement materials • Hazards • Pollution, e.g. air, noise, water, visual, vibratory, waste disposal, erosion • Protection
Safety	• Ability to undertake investigations • Ability to construct treatments • Accidents – history • Levels of service – past, current, future, rate of change, standards, skid resistance, ride quality, visibility, wet and dry road characteristics • Driver and pedestrian behaviour
Pavement	• Condition – functional and structural • Configuration • Composition • Cross-section • Past maintenance • Past traffic • History – performance • Availability of local materials, equipment and labour

2.3 Investigation and Design Proposal

Prior to undertaking an investigation and design it is advisable to formulate the methodology to be followed and the extent of engineering to be undertaken. This is often presented as a project proposal.

A detailed project proposal normally contains some or all of the following:

- project title
- client and contract type
- project description – location, extent, nature
- project objectives
- project considerations or understanding of issues
- scope of services
- methodology
- client-supplied information
- quality requirements and standards
- hold points and client liaison
- deliverables
- timeframe
- project resources
- costs.

2.4 Design Report Content and Structure

The report should contain sufficient investigation information and a clearly documented decision-making process supported by modelling for a reader to be confident an appropriate design process has been used.

In addition to the information in the design proposal (Section 2.3), the design report commonly includes:

- background data (Section 2.2, Section 3.2)
- design period and design traffic loadings
- strip map of pavement visual condition, description of existing surface and subsurface drainage and other site characteristics (Section 3.3)
  Photographs of condition are useful.
- results of pavement deflection testing, roughness, rutting and other investigatory tests on the pavement surface (Section 4)
- results from pavement coring, pits and trench, including dynamic cone penetrometer testing of the subgrade (Section 5)
- existing pavement structure, classification and quality of materials from laboratory testing (Section 5)
- causes and modes of distress (Section 6)
- treatment options, including descriptions of how each address the cause of distresses (Section 7, Section 8)
- influence of design and construction issues on treatment selection (Section 7.7, Section 8.8)
- thickness design calculations for structural treatments (Section 9 to Section 12)
- comparison of whole-of-life costs of treatment options (Section 13).

2.5 Acceptable Risk and Project Reliability

Because of the many factors which must be evaluated in the design of rehabilitation treatments and which influence performance to varying degrees, there is no absolute certainty that the desired performance will be achieved.

It is appropriate that the acceptable level of risk varies with the function of the road for which the treatment is being designed, such that the level of acceptable risk diminishes with the importance of the road and the scale of the project. For some projects a higher risk of premature treatment distress is accepted than would be considered appropriate for new construction. For instance, improving pavement support in specific locations may be foregone for economic reasons and this may increase the risk of premature distress.

The risk of poor performance can be controlled, to some extent, in the selection of design parameters and the degree of conservatism in the values adopted for each of these parameters. For the empirical design of granular overlays (Section 9), a simple method of achieving a more conservative overlay design thickness is to adopt a value for the total traffic over the design period higher than that which is anticipated. It is suggested that the use of a value of up to two times the anticipated traffic may be warranted for projects with a higher-than-normal risk of poor performance.

For the mechanistic-empirical design of other structural treatments for flexible pavements (Section 10) and for concrete overlays (Section 11 and Section 12.3), the risk depends on the project reliability selected for the project. The project reliability is defined as follows (Austroads 2018a):

> The Project Reliability is the probability that the pavement treatment, 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) constructed road pavement which is subsequently rehabilitated as an entity.

The desired project reliability is the chance that a structural treatment being considered will outlast its design traffic assuming that:

- the treatment is designed in accordance with the procedures in this Part
- the treatment is constructed in accordance with standard specifications
- the materials used meet standard specifications requirements.

The desired project reliability is chosen by the road agency or designer. Typical project reliability levels are given in Table 2.3.

Table 2.3: Typical project reliability levels for rehabilitation projects

Road type	Project reliability (%)
Freeway, motorway, major highways or other heavy-duty pavements	95
Other than above where lane AADT > 2000	90
Other than above where lane AADT < 2000	85

Reliability procedures have yet to be developed for:

- thickness design of bitumen stabilisation treatments (Section 10.9)
- asphalt overlays on concrete pavements (Section 12.2).

For these treatments, past experience of projects in the vicinity provides insight in terms of the likelihood that the treatment will outlast the design traffic.

3. Pavement Data and Inspection

3.1 General

The nature of the investigative process is governed by its purpose. If this is simply an upgrading to accommodate an anticipated change in usage and is not associated with any significant signs of distress, then the evaluation may only involve:

- confirmation of current condition by visual assessment
- structural evaluation for possible pavement strengthening.

Alternatively, if the purpose is to rehabilitate a pavement showing signs of distress, as is more commonly the case, the evaluation process becomes more complex. In broad terms, it is based on an assessment of some aspect of the current pavement condition with respect to the required condition. Condition can be considered to have two aspects:

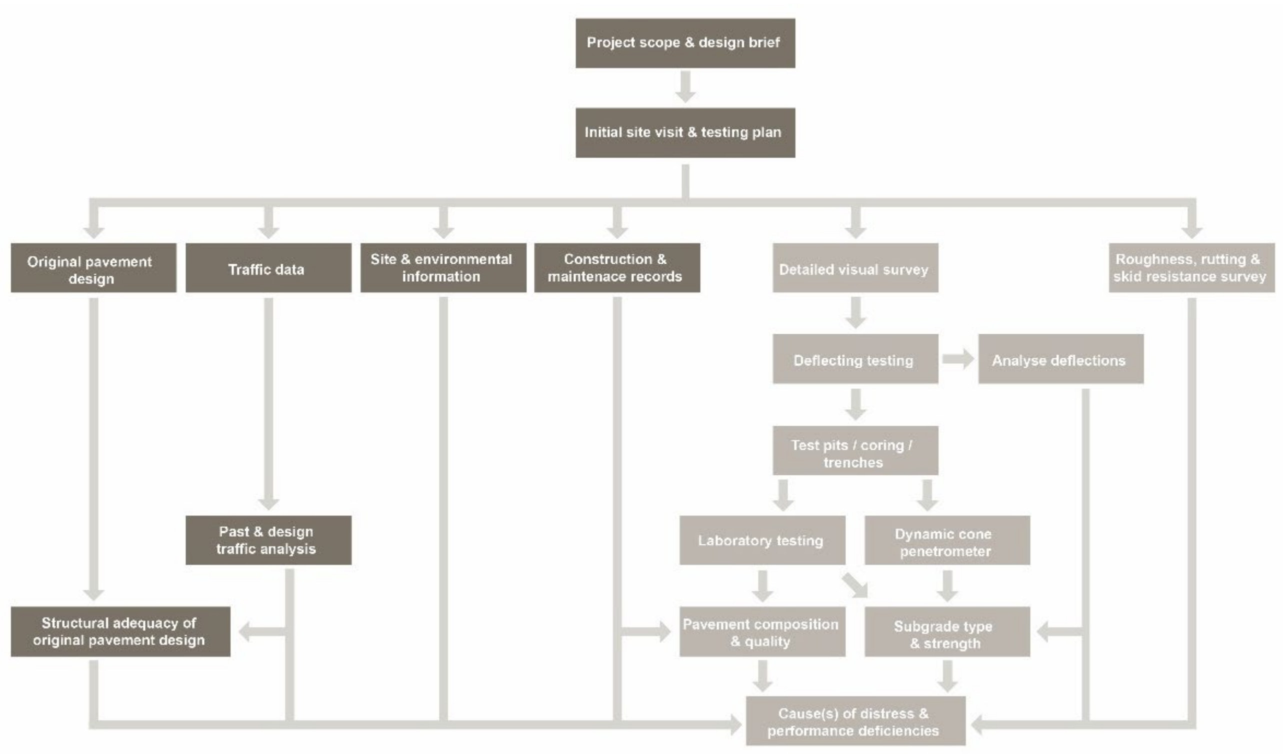
- **Functional** – the level of service provided by the pavement to the user.
- **Structural** – the pavement's load carrying capacity.

Road users are usually only interested in the functional condition or serviceability of a pavement. However, a loss in functional condition can often be explained by a loss of structural capacity. Hence, in selecting rehabilitation treatments, designs need to reflect both the functional condition and the structural condition.

It should be noted that pavement condition often varies substantially with time, hence, condition data has a limited life span. In addition, the data can be influenced by climatic conditions at the time at which it was collected. Consequently, the data used to analyse the pavement and select treatments needs to be relevant and fit for the purpose. It is important that all data can be accurately related to a common datum/chainage system.

This section provides a brief overview of the activities or observations which the evaluating team would make (Figure 3.1).

Figure 3.1: Design steps discussed in Section 3



3.2 Historical Data

The quality of historical records has a significant influence on the effectiveness of the investigation process, particularly those relating to the design and construction details.

A considerable quantity of data, relevant to the determination of an appropriate rehabilitation strategy, may already be available from a variety of historical records.

3.2.1 Original Pavement Design

The original pavement design data assists in determining whether the distress is due to the selection of inappropriate design parameters rather than deficiencies in the as-constructed pavement or the performance expectations. Among the design data of interest are:

- pre-construction testing, including laboratory test results
- the basis of design
- pavement structure and materials
- drainage design
- geological maps
- discussions with relevant field staff.

There is, however, no guarantee that the pavement was constructed in accordance with the design, and this should be confirmed by either 'as-constructed' detail or field testing.

3.2.2 Construction Details

A thorough understanding of the previous construction process is useful. Information on material types, construction methods and experiences may hold the key to the pavement's performance. For a major investigation of premature failure, sources of such details include:

- diaries, reports and correspondence
- quality control test results for materials and methods
- 'as-constructed' plans or sketches
- weather records for the construction period
- recollections of the supervisors/constructors.

3.2.3 Maintenance and Rehabilitation Records

Maintenance works tend to be less well-documented than major rehabilitation or construction projects. If records are available, however, they may yield the following information of significance to the nature and priority of the rehabilitation:

- level of current maintenance expenditure and its rate of increase over the previous five years
- nature of past pavement distress
- past maintenance, surfacing history or rehabilitation treatments and their effectiveness
- comparison of actual and anticipated performance following these maintenance treatments.

3.2.4 Climatic Conditions

Rainfall records, both in terms of intensity and distribution, may be of value when related to the original design assumptions and the development of pavement distress. The climatic conditions during pavement construction is also useful, if available.

Consideration also needs to be given to the influence of pavement temperatures on performance. For instance, some surfacing types may not perform adequately when high turning and braking stresses are applied at high temperatures.

3.2.5 Effect of Traffic on Past Performance

Original pavement design data together with data on traffic loading since construction may be used to determine whether the pavement has met or exceeded its design traffic loading.

Deterioration of pavements due to traffic loadings is dependent on both the magnitude of individual wheel or axle loads and the total number of repetitions of these loads. In the case of cemented materials in flexible pavements and rigid pavements, the magnitude of individual axle group loads assumes a greater importance. In evaluating an existing pavement, analysis of the total traffic since initial construction or the last rehabilitation may be useful.

3.3 Field Survey

3.3.1 Introduction

It is important that the site inspection is completed competently and with the assistance of good local knowledge or records of the road section's loading and maintenance history. This inspection will be aimed at gathering two major types of information:

- visual condition data
- site analysis and environment data (see Section 3.3.3).

In order to enable comparison of different datasets (previous data or different types of assessment), a reliable coordination or mapping system must be used during these inspections. GPS technology is inexpensive and sufficiently accurate, and GPS coordinates offer good value for this purpose. In the absence of this technology, an accurate chainage reading or measurement relative to the road agency's road referencing system or to fixed features (road junction, railway crossing) can also be used.

3.3.2 Visual Condition Data

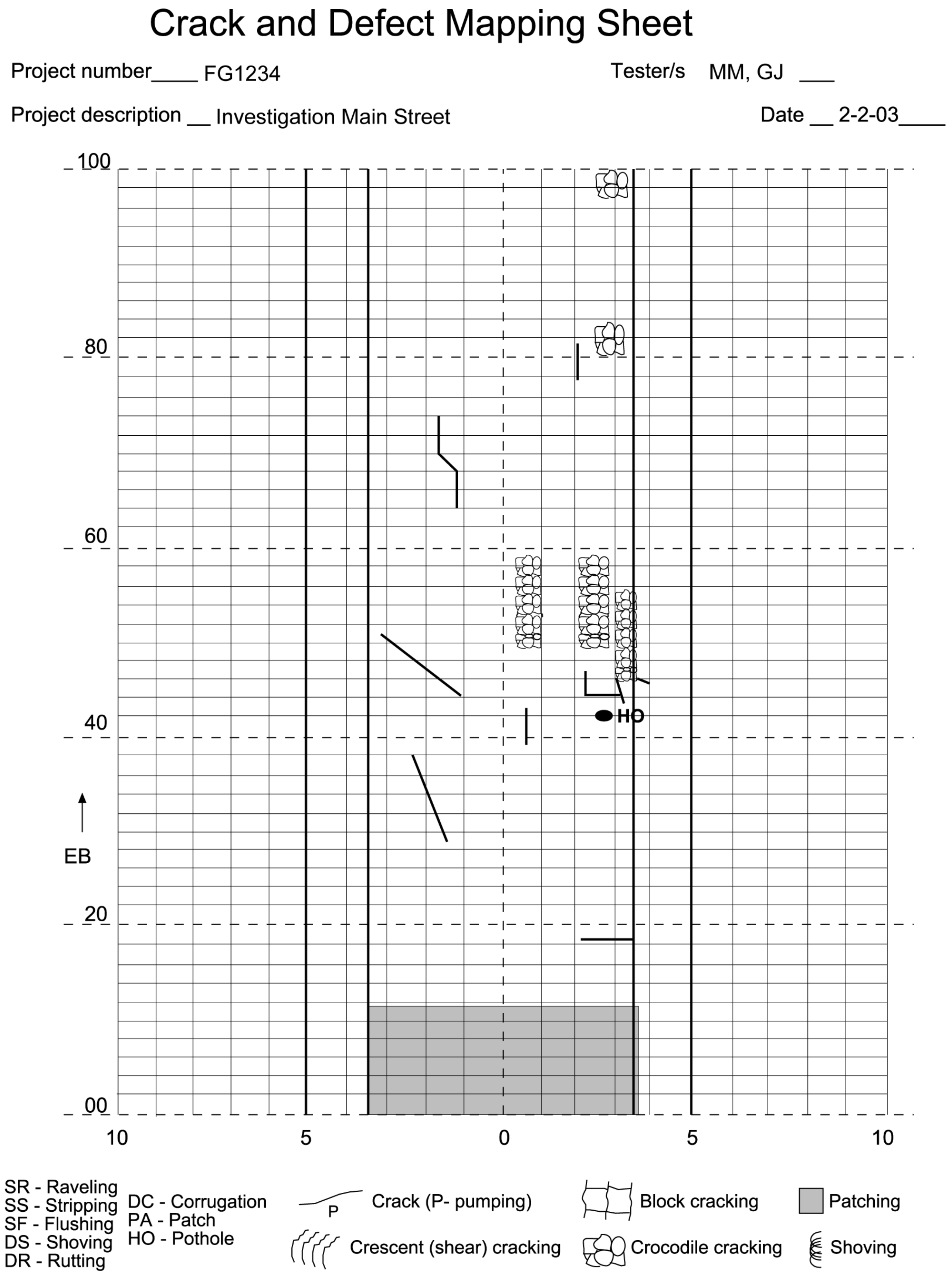
A visual survey of the pavement condition is one of the simplest and often most valuable datasets. Indeed, managers of smaller road networks would use this as the major source of information on their pavements' condition and rarely use field testing, usually for reasons of cost. The level of detail and rigour of a visual survey can vary significantly depending on the method used e.g. a drive-through, automated survey and manual mapping.

When a visual survey is conducted, there is a minimum level of information required to make knowledgeable decisions regarding rehabilitation needs and treatments. This information includes:

- Distress type – identify types and causes of physical distress existing in the pavement.
- Distress severity – record the level of severity for each distress type and possible causes.
- Distress extent and location – record the location and relative area of the project affected by each combination of distress type and severity.

A visual survey should be recorded in a systematic way and plotted on a map against a reference system, with further reference to key features along the roadway, e.g. water and drainage, vegetation, cuts and fills, structures. Mapping against GPS coordinates is desirable as it allows close correlation of different condition datasets, e.g. linking visual data to pavement deflections. Figure 3.2 is an example of a flexible pavement visual survey map. It is useful to photograph all observations significant to the rehabilitation design.

Figure 3.2: Example of a defect mapping sheet for a flexible pavement



Source: Queensland Department of Transport and Main Roads (2006).

In relation to the visual condition of a sprayed bituminous seal surfacing, it is necessary to appreciate its function. The main purpose of providing a seal is to protect and waterproof the pavement and to provide a durable and safe surface to travel on. Sprayed seals[1] are, over time, subject to surface condition deterioration and the onset of distress and traffic safety-related issues. Understanding the modes and causes of distress of sprayed bituminous seals is useful in determining maintenance requirements for a pavement. The level of seal distress due to structural deficiencies is an important consideration.

When a sprayed seal surface has been adequately designed, constructed and placed on a sound pavement, then the remaining service life, and hence maintenance requirement, of the surfacing is influenced by:

- age of the seal
- size and quality of the aggregate
- durability and performance of the binder
- climatic conditions
- traffic volume and composition
- site conditions e.g. curves, intersections
- maintenance practices.

To provide a reliable indication of seal condition, the following criteria have been used in the past:

- Cracking – loss of waterproofing and indication of the binder losing its elastic properties.
- Loss of aggregate – indication of the binder hardening and losing its ability to retain the aggregate.
- Binder condition – a direct assessment of binder condition and the most difficult of all the assessments. Binder condition is affected by temperature and an assessor needs to take the prevailing temperature into account and try to relate the binder condition to 'normal' service temperatures.
- Binder level – a high binder level can increase the life of a seal but when the bitumen is at or near the top of stone, the loss of texture can result in poor skid resistance.
- Maintenance patching – provides an indication of the general state of the pavement and possible future problems.

It is essential that the position of the relevant section of road be accurately described on the survey map.

For flexible pavements, the survey should map the following:

- surfacing and pavement type, e.g. sprayed seal (single or double application), asphalt, microsurfacing
- surface deterioration – stripping, ravelling, flushing
- deformation – rutting, shoving
- cracking, including existing construction joints
- edge defects and shoulder condition
- potholes
- patching
- condition of subsoil and surface drainage (whenever possible).

For rigid pavements, the survey should map the following:

- pavement type, e.g. plain jointed concrete
- cracking – plastic and structural
- surface texture deficiencies
- joint sealant failures and joint pumping
- joint and crack spalling
- joint stepping.

1 In New Zealand, the term 'chip seal' is used. 'Chip' refers to the description of the aggregate particles used in sealing works.

Details and examples of these distress types are given in Appendix A.

In the late 1980s, Austroads issued a *Guide to the Visual Assessment of Pavement Condition* (Austroads 1987) which presented a comprehensive and detailed description of the most common types of pavement and bituminous surfacing distress. The Guide included a large number of photographs illustrating typical forms of distress, similar to those in Appendix A.

3.3.3 Other Site and Environment Information

In addition to the condition of the road pavement itself, there is a need to take note of a number of factors which might explain the current pavement condition, and which would have a bearing on the selection of a rehabilitation treatment or strategy.

Geology, topography and climate

Soil/rock exposures in cuts and drains are often useful in correlating geological and pedological map information to the actual pavement and can provide a useful insight into the nature and extent of subgrade soils/rock.

The location of swampy areas, deep fills, cuts, cut-to-fill transitions, etc. should be noted as these can be associated with various types of pavement distress. Information should also be sought about the depth of the water table below the pavement.

The intensity and distribution of rainfall at the site needs to be considered as these may provide additional insight into the extent to which pavement distress is related to moisture in the pavement and/or to seasonal moisture variations within the subgrade.

Consideration should also be given to in-service pavement temperatures and their likely influence on pavement performance. Also, where there is doubt about the engineering properties of any pavement, subgrade, shoulder or verge materials, laboratory testing is relatively inexpensive and can yield valuable information.

Road geometry and cross-section

The geometry of a road refers to its horizontal alignment, longitudinal profile and cross-sectional shape. The cross-sectional detail includes crossfalls, widths of pavement, shoulders and verges and subsurface and surface drainage provisions.

The reasons why road geometry may be considered substandard include the following:

- traffic capacity below current traffic volume or likely to be inadequate for traffic volumes within the proposed design life of the rehabilitation treatment
- edge breaks reducing the effective seal width
- interim widenings creating inconsistent/irregular crossfalls
- changes in design standards since the construction of the existing pavement.

In these circumstances, possible alterations to the road geometry need to be considered when designing the rehabilitation treatment. Normally the designer undertaking the pavement rehabilitation investigation is concerned with information affecting pavement performance. Assessment of the adequacy of geometry is not usually within scope of a pavement rehabilitation designer’s investigation. If assessment of the geometry or drainage is required, it is recommended that a specialist designer, or designers, be engaged to undertake this part of the investigation.

Nevertheless, where possible, information about the road's horizontal and vertical alignment, as well as cross-sectional information, including type of cross-sections, should be collected. For pavement rehabilitation investigations the following details should be noted for the current road and its proposed future states:

- superelevation and crossfall (e.g. to identify high side)
- cross-section element widths (e.g. lane, shoulders, verge)
- the width of pavement types and their extents (e.g. shoulder versus through lanes, longitudinal variations)
- location of longitudinal construction joints relative to the wheelpaths
- drainage details (of subsurface and surface drainage systems).

Guidance on road geometry standards is provided in the Austroads *Guide to Road Design Part 3: Geometric Design* (Austroads 2016).

Site constraints

The visual inspection of the pavement should identify any constraints on the type and extent of rehabilitation treatments which can be undertaken, e.g. whether an overlay thickness is restricted by level controls imposed by kerb and channel, safety barriers, median barriers or clearance to overhead structures or services (e.g. bridge soffits).

Requirements for re-establishing access to adjoining properties also need to be noted.

Drainage

In considering the design of any rehabilitation treatment, the potential influence of surface water and the subsurface moisture conditions must be considered. Moisture-related pavement distress and failures account for a large proportion of pavement rehabilitation expenditure. Any source of water infiltration into the pavement, subgrade, verge and shoulders needs to be considered as influential.

The visual survey will often identify moisture-related distress in the existing pavement, the cause of which needs to be addressed if any rehabilitation treatment is to achieve its intended purpose. Distress types in flexible pavement, which may be caused or accelerated by moisture, include stripping, rutting, loss of surface shape (depressions), fatigue cracking and potholes. Moisture-related distresses in rigid pavements include pumping, cracking, joint deterioration and corner breaks. It is equally important to recognise when pavement distress is not moisture-related, so that funds are not unnecessarily expended on drainage improvements when they are not required.

The following are typical conditions which may contribute to moisture-related distress, and hence, need to be identified either during the visual survey, by reference to historical records or by subsequent investigation:

- shallow and/or ineffective table drains
- blocked or ineffective subsurface drains
- cracked or misaligned kerb and channel
- blocked kerbside inlet pits
- permeable shoulders and medians
- impermeable aggregate in drainage layers
- impermeable shoulders in boxed construction without subsurface drainage
- concentrated surface sheet flow from adjacent terrain
- water ponding on the pavement, shoulders or verges or depressions capable of retaining surface water
- ground water seepage, particularly in cuttings
- presence of cracks and other surface discontinuities capable of allowing ingress of moisture
- accumulation of moisture in sag curves
- moisture accumulation at changes of pavement type or thickness, including permeability reversals
- high moisture contents due to soil suction of fine-grained cohesive soils in wet environments
- presence of underground services, e.g. water mains or sewers.

The following are some of the major checks/tests required for evaluating a moisture control system:

- Visually inspect table drains, cut-off drains, kerb and channel, inlet pits and pipe outlets.
- Excavate trenches or pits or probe to check whether moisture is entering the pavement through the surface, shoulders, subgrade or adjacent terrain and to evaluate materials. Test for moisture content variations in pavement materials.
- Note evidence of seepage from the pavement surface (i.e. stains, pumping, etc.) and embankments after rain. These may indicate malfunctions in the pavement drainage system or other impediments to the ready egress of moisture from the pavement (e.g. permeability reversals in the pavement structure, boxed construction with impermeable shoulders).
- Correlate pavement deflection variations with results of visual inspection and rut depth measurements.

Inspection techniques can comprise several methods of varying complexity. For more detailed information on subsurface drainage refer to *Guide to Pavement Technology Part 10: Subsurface Drainage* (Austroads 2009b) and National Cooperative Highway Research Program (NCHRP) (1994,1997). For more detailed information on drainage of surface water refer to *Guide to Road Design Part 5A: Drainage: Road Surface, Networks, Basins and Subsurface* (Austroads 2018f).

Normally the designer undertaking the pavement rehabilitation investigation is concerned with information affecting pavement drainage. Except for pavement drainage, assessment of the adequacy of the drainage system is not usually within the scope of the pavement rehabilitation designer's investigation. If assessment of the drainage not directly related to the pavement is required, it is recommended that a specialist drainage designer, or designers, be engaged to undertake this part of the investigation. Maintenance personnel are a good source of information on the performance of a drainage system.

Utilities and services

Underground (and overhead) services should be located for workplace, health and safety reasons (e.g. during investigations so that they can be avoided when excavating trenches/test pits, for construction). Although accurate information on the location and depth of services such as gas and water mains, telephone and electrical cabling can be difficult to obtain, experience indicates that it is almost always prudent and cost-effective to do so.

Information on services can be important both in understanding the reasons for existing pavement conditions and in planning any rehabilitation treatment. For example, in situ stabilisation may not be viable if there is a lack of cover to some services.

4. Investigative Testing on the Pavement Surface

4.1 Introduction

Once the site inspection is complete, the following issues need to be addressed:

- Is the road section in need of treatment?
- If so, what further tests are required to determine urgency, priority and the nature of the treatment?
- Is pavement testing required to explain the current pavement condition or design of the treatment?

In most cases consideration of these issues will lead to a plan for testing and investigation.

If the pavement is considered to be sound or needs some minor or routine works as a holding action, it is unlikely that any further tests will be required. It may well be put on some form of watch list, so that the surveillance teams doing periodic or routine inspections can pay particular attention to the progression of one or more defects.

If the pavement requires rehabilitation, more tests will usually be needed so that the causes of the distress can be identified and the most appropriate treatments can be determined (Figure 4.1). Identification of the mode or modes of distress evident in the pavement and their associated cause or causes is a very important phase in treatment selection. If the modes or causes of distress are incorrectly assessed, then it is very unlikely that the selected rehabilitation treatment will be effective, regardless of the quality of its design and construction (see Section 6).

The type, severity and extent of visual distress provides the most immediate and direct indicator of both the modes of distress and associated causes. Evaluation of visual condition should include:

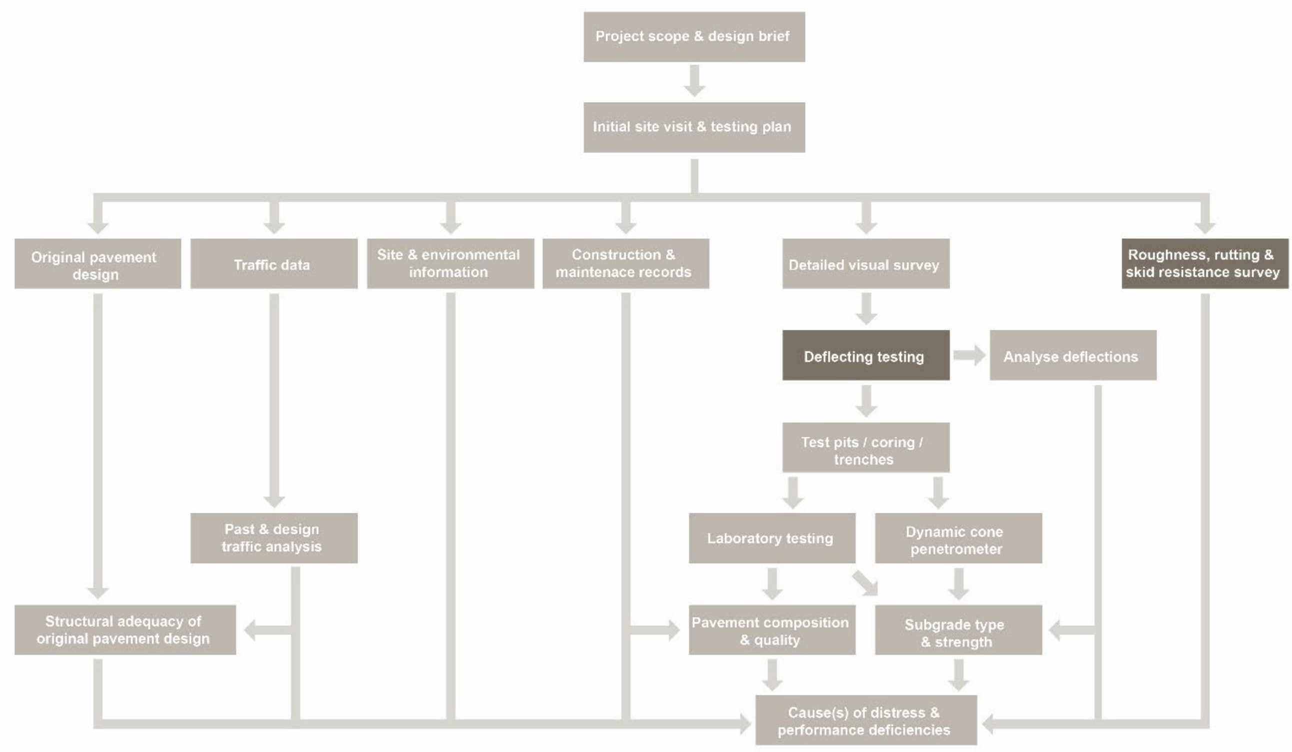
- identification of patterns in the type and distribution of distress
- links between types of distress and other recorded features e.g. cut/fill boundaries, drainage and vegetation
- links between distress and other data recorded e.g. deflections, construction history and geology
- links between distress and traffic loading, e.g. where distress is related to road grade, points of entry and exit of heavy vehicles.

In evaluating a pavement, it is important to determine the adequacy of the surface and subsurface drainage systems (see Section 3.3) in preventing or controlling the infiltration of moisture and to determine the extent to which pavement performance relies on the integrity of the drainage systems.

In addition, it is necessary to analyse the collected test data (refer Section 4.2) in search of necessary correlations between:

- the visual condition and the test data on the existing pavement
- the various factors that are known to affect the performance of pavements.

Figure 4.1: Design steps discussed in Section 4



4.2 Types of Forensic Testing

In seeking information to make decisions about the rehabilitation treatment, one or more of the following types of testing may be required:

- roughness or ride quality
- rut depth
- cracking
- skid resistance
- surface texture
- surface deflection
- pavement composition and material quality (see Section 5).

The structure of the testing program should reflect the relative significance of the data in developing a pavement model that is consistent with past pavement performance. The more complete and truer to life the model, the greater the likelihood of developing appropriate and optimal pavement treatments.

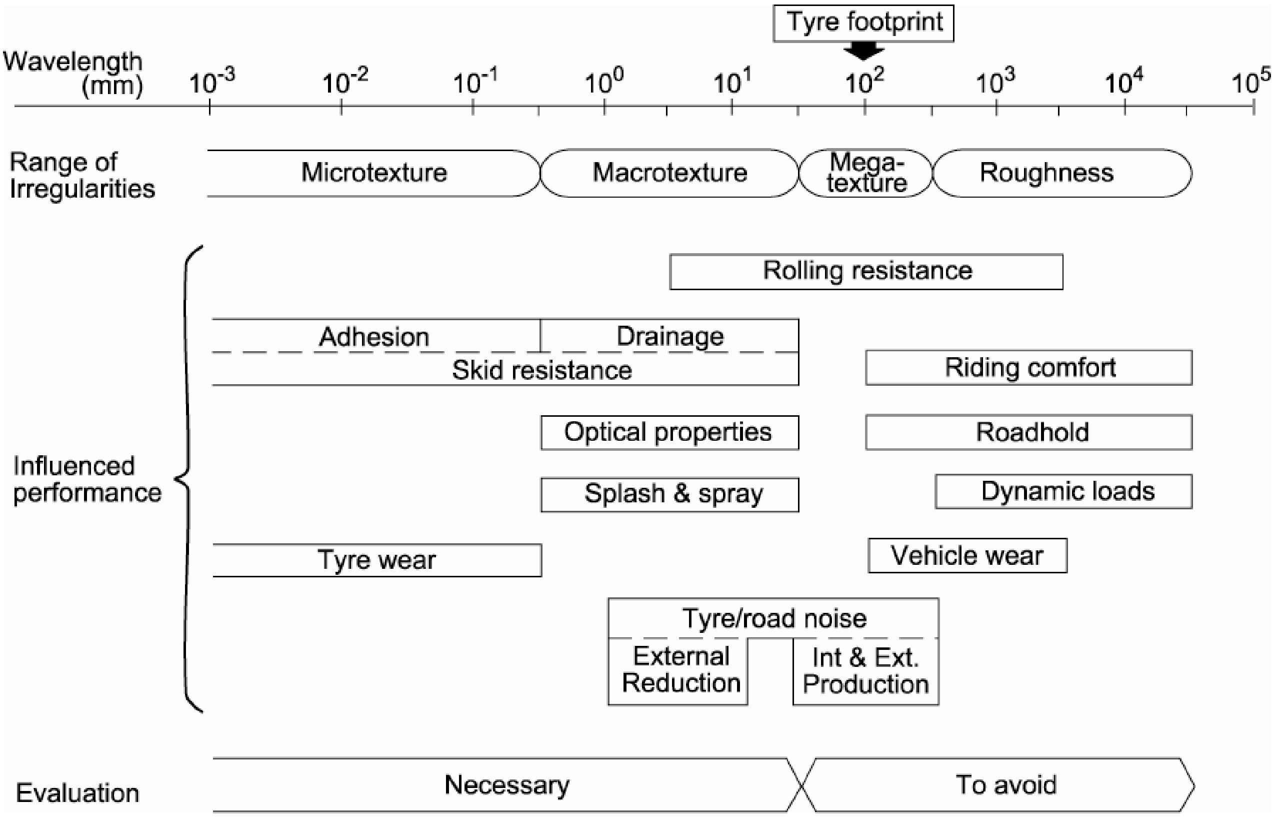
4.3 Roughness

Road roughness is a pavement condition parameter, which characterises deviations in a road surface from the intended longitudinal profile. It is widely used to rate the road condition at a network level because of its effect on vehicle dynamics and hence, vehicle operating costs, driver comfort and dynamic pavement loading.

Roughness measurements are usually taken as part of a routine or cyclical network testing program. They are generally analysed at a network level and this data is used to identify sections for project level analysis. Details of roughness measurement are given below and in more detail in the *Guide to Asset Management Technical Information Part 15: Technical Supplements* (Austroads 2018b). An overview of key concepts is given below.

Current methods of measuring roughness reflect surface irregularities with wavelengths between 0.5 m and 50 m (Figure 4.2) along each of the two wheelpaths in a traffic lane.

Figure 4.2: Influence of the wavelength of surface irregularities on pavement and vehicle performance



Source: Descornet (1989).

Current practice is to measure the longitudinal profile in both wheelpaths in a selected lane with an inertial laser profilometer (Figure 4.3) and to mathematically model the response of a hypothetical vehicle (quarter-car simulation) to the longitudinal wheelpath profile.

Austroads has determined that, at a road network level, roughness should always be reported in terms of the International Roughness Index of the measurement lane (Lane IRI_{qc}), commonly referred to simply as IRI (m/km) in Austroads test method AG:AM/T001-16. This is the standard, composite IRI value representing the roughness of a traffic lane within a section of road. It is determined by averaging two individual single wheelpath IRI_{qc} values obtained separately in each wheelpath of a lane. The simulated travel speed of the IRI quarter-car model (IRI_{qc}) is 80 km/h.

Historically, roughness values in Australasia were derived from the physical response of a vehicle to a road surface using a response type device that reported roughness in terms of NAASRA Roughness Meter Counts (NRM, counts/km). The wheelpath profiles measured by a profilometer were correlated to the response of an NRM; as such, roughness results determined by a profilometer can be reported in units of counts/km as if they had been measured by an NRM. A reasonable correlation between the two roughness indices is provided by the following relationship (Austroads 2018b) (Equation 1).

$$\text{NAASRA counts/km} = 26.49 \times (\text{lane-IRI}) - 1.27 \qquad (1)$$

Results are normally processed and reported at 100 m intervals for each lane surveyed, with IRI results reported to no more than two decimal places.

Figure 4.3: Network survey vehicle complete with Inertial laser profilometer

Source: Australian Road Research Board (n.d.).

4.4 Rutting

Rutting is a form of deformation typically evident in flexible pavements; it is caused by the passage of loaded wheels over the pavement surface. It is manifested as a longitudinal depression along wheelpaths. Rutting may occur in one or both wheelpaths (Figure 4.4).

Figure 4.4: Rutting in a wheelpath

Source: Local Government and Municipal (LGAM) knowledge base.

Rut depth is the maximum vertical pavement displacement in the transverse profile, either across a wheelpath (wheelpath rut depth), or across a lane width (lane rut depth), measured from a reference datum taken at 90 degrees to the road edge. Measurement of the rut depth gives an indication of the surface and structural condition of the pavement and also provides an indicator of potential aquaplaning problems.

At the project level, it is common to manually measure rut depth using a 1.2 metre or 2-metre-long straight edge and depth wedge (Figure 4.5). Where the width of the rut exceeds the length of the straight edge, the straight edge will not fully span the rut profile between its highest points. Therefore, for a given rut, the measured depth may vary with the length of the straight edge.

Figure 4.5: Straightedge and wedge manual rut depth measurement

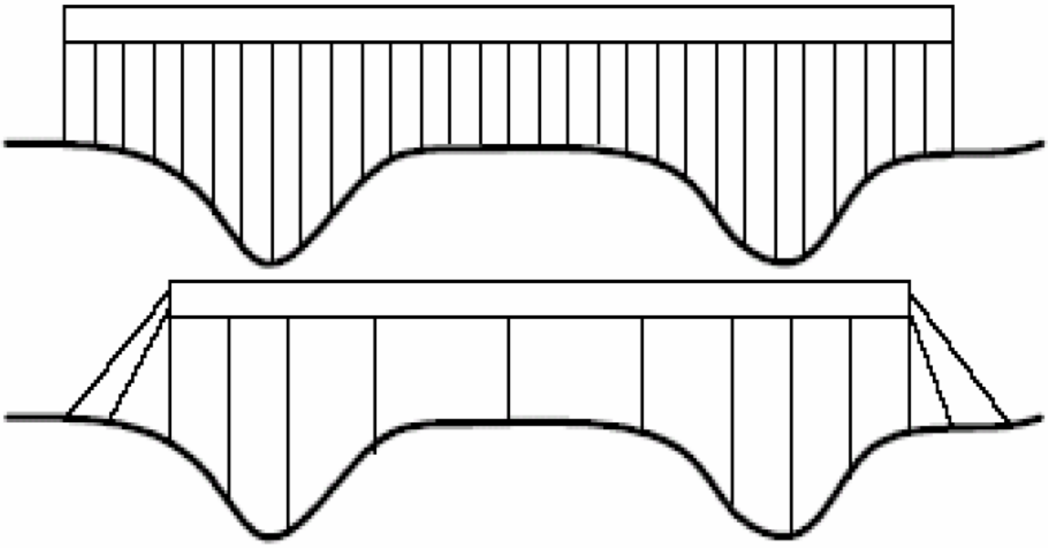
Source: Australian Road Research Board.

The use of non-contact automated rutting (or transverse profile) measuring technologies has been increasing for some time, and it is now generally agreed that rutting and roughness measurements can be automatically collected at highway speed provided there are sufficient sensors. Automated methods for rutting surveys use a number of vehicle-mounted lasers, such as ultrasonic sensors or the Digital Laser Profiler (DLP) in multi-laser profilometer form, as shown in Figure 4.6 and illustrated in Figure 4.7, in conjunction with taut wire or straight edge models of determining rut depth (Figure 4.8). The Austroads standard test method for collecting rutting data with a multi-laser profilometer (Test Method AG:AM/T009-16), recommends the use of at least 11 laser sensors. These devices are typically capable of measuring transverse profiles as close as 50 mm spacings while the host vehicle travels at normal traffic speeds.

Figure 4.6: An example of a multi-laser profilometer

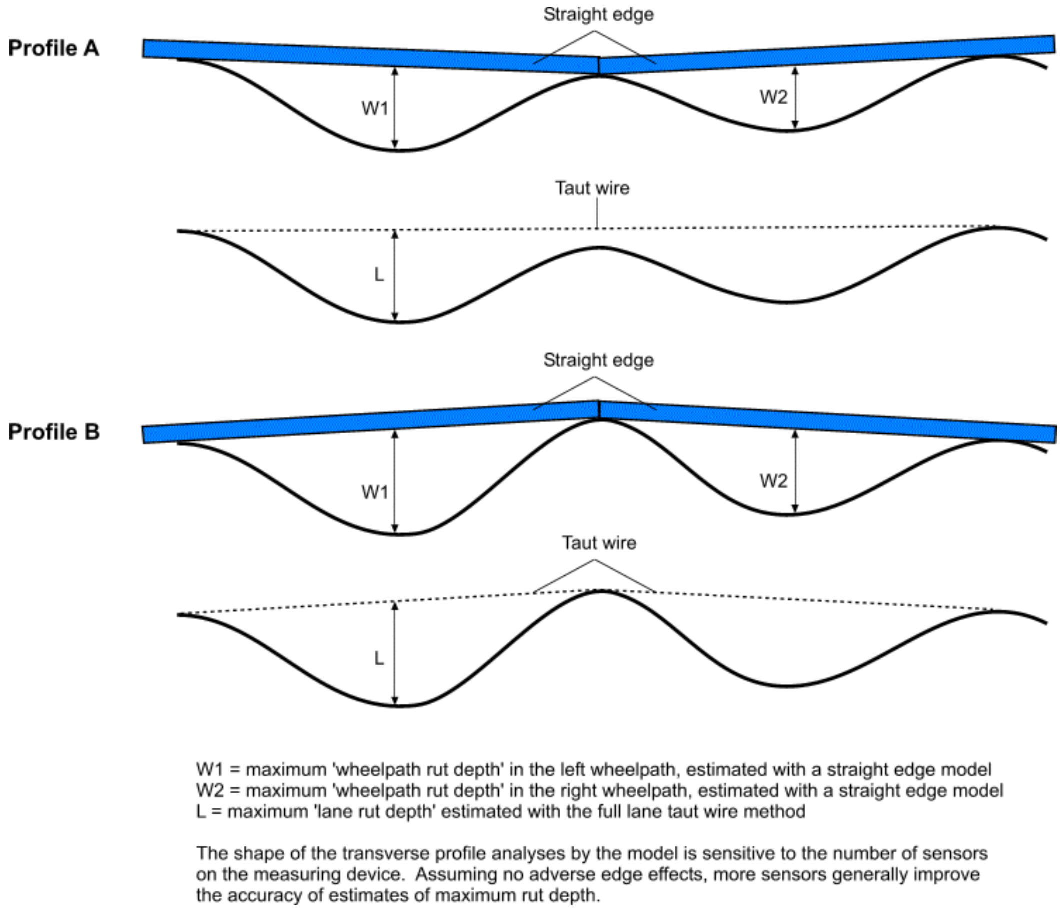
Source: Australian Road Research Board.

Figure 4.7: Various sensor arrays for high speed transverse profile data capture



Source: Transfund New Zealand (2003).

Figure 4.8: Straight edge and taut wire models for automated rut depth measurement



Source: Austroads (2007).

Ideally the measurement should be able to:

- distinguish rutting from other types of deformation, i.e. heave, shear, shoving
- locate the rut or ruts within the lane i.e. inner wheelpath or outer wheelpath
- determine the magnitude of rutting.

Guide to Asset Management Technical Information Part 15: Technical Supplements (Austroads 2018b), discusses the different definitions of rutting and the methods of measurement in more detail.

4.5 Cracking

A crack is an unplanned break or discontinuity in the integrity of the pavement surface, usually a narrow opening or partial fracture, often indicating vertical fracturing of the pavement, not necessarily extending through the entire thickness of a course or pavement. A crack may be caused either by environmental factors or due to the action of traffic loading. Cracks allow the ingress of water and can be exacerbated by the ingress of water. The term 'cracking' is used for the process of development of a crack, and as a collective noun describing the coverage of cracks over a pavement (Austroads 2015).

As detailed and illustrated in Appendix A and Austroads (2018b), cracks may be linear (transverse or longitudinal), interconnected (crocodile or block) or irregular (meandering, diagonal, crescent and edge cracking), single and isolated or in groups, with varying spacing between them. Except on continuously reinforced concrete pavements, cracks almost invariably represent a pavement defect.

A visual survey of the cracking (Section 3.3.2) is a common means of collecting cracking data. The level of detail and rigour of a visual survey can vary significantly between projects according to the importance of the data in determining the cause of distress and the treatment option. Figure 3.2 is an example of a method of recording the cracking data.

As described in Austroads (2018b), automated methods of cracking data collection may be 'fully automated' or 'partially automated':

- Partially automated survey methods employ high definition video and digital cameras. These typically detect medium cracking (3 mm) and above. They involve manual interpretation of video images and data recording.
- With 'fully automated' methods such as Roads and Maritime Services (Roads and Maritime) NSW RoadCrack (see Figure 4.9), vehicle-mounted cameras capture images of the road surface, and crack recognition software processes the images either in real time while the vehicle progresses or after completion of the survey.
- The laser crack measuring system (LCMS) is a 'fully automated' method capable of detecting cracks down to a width of around 2 mm, depending on the surface type. It captures data on cracking over a pavement surface width of 4 m continuously along the length of the road (see Figure 4.10). The continuously recorded road data is processed into user defined images that are automatically analysed using proprietary algorithms developed by the manufacturer of the equipment. The end result is a continuous crack map, which identifies the location and width of the crack on the pavement surface.

Figure 4.9: Roads and Maritime RoadCrack automated crack survey device

Source: Australian Road Research Board

Figure 4.10: LCMS automated crack survey device

Source: Australian Road Research Board.

4.6 Skid Resistance

Skid resistance is a condition parameter that characterises the contribution that a road surface makes to the level of friction available at the contact patch between a road surface and vehicle tyre during acceleration, braking and cornering manoeuvres. The friction level depends upon both the microtexture of the aggregate in the surfacing, the macrotexture (surface texture) of the surfacing as well as the presence and thickness of any water film on the surface.

If the available level of friction at any of the contact patches between tyre and road surface is insufficient for the manoeuvre being attempted then the driver of the vehicle is likely to lose control of their vehicle, which may in turn result in a collision with another road user and/or highway related infrastructure.

Skid resistance measurements are used to establish whether there are other non-structural reasons for road surface treatment.

The three most common devices used to measure skid resistance in Australia and New Zealand are:

- SCRIM (Sideways-force Coefficient Routine Investigation Machine) or similar device as shown in Figure 4.11 and Figure 4.12
- portable pendulum tester (PPT) also referred to as the British pendulum tester (BPT) or portable skid resistance tester (PSRT) as shown in Figure 4.13
- Grip Tester as shown in Figure 4.14.

Guide to Asset Management Technical Information Part 15: Technical Supplements (Austroads 2018b), describes these devices and their methods of operation in more detail.

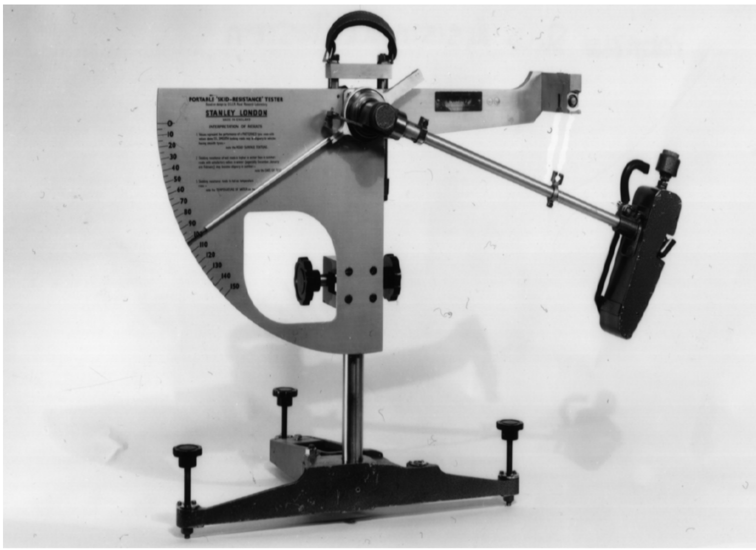
Figure 4.11: Truck-mounted skid resistance tester

Source: Australian Road Research Board

Figure 4.12: Instrumented test wheel

Source: Australian Road Research Board

Figure 4.13: The British pendulum tester



Source: Road Research Laboratory (1969).

Figure 4.14: Grip Tester

Measurement mechanism exposed **Grip tester in operation**

Source: Private communication Department of Planning, Transport and Infrastructure South Australia.

4.7 Surface Texture

The texture of a road surface makes a vital contribution to the total available level of friction at the contact patch between vehicle tyre and road surface, and particularly so at higher vehicle speeds and/or in wet conditions.

In project-level pavement rehabilitation, texture depth measurements are used to establish whether there are other non-structural reasons for treating the section.

Texture depth is an indicator of the space through which water may escape from the interface between a tyre and the road surface. It is an important factor affecting skid resistance at high traffic speeds, as without sufficient texture depth, vehicle aquaplaning can occur. Texture depth also affects tyre-road interaction noise and surface spray.

Currently in Australia and New Zealand, two methods have been used to measure macrotexture:

- volumetric, using the sand patch test
- surface profile measurements using lasers.

The two methods use the same underlying principle, namely measurement of the variation in height (profile) of the surface; the methods do not necessarily directly relate to one another. In addition, it has been found that the relationship between the two methods can be surfacing-specific.

Some laser profilers measure texture depths as Mean Profile Depths (MPD) or, by use of correlation factors, as equivalent sand patch texture depths expressed in millimetres.

Guide to Asset Management Technical Information Part 15: Technical Supplements (Austroads 2018b), describes these devices and their methods of operation.

4.8 Use of Ground Penetrating Radar

For projects where pavement thicknesses or type of construction vary significantly, it may be necessary to use ground penetrating radar (GPR) to supplement the information from coring and pits. GPR is a non-destructive testing technique (Figure 4.15 and Figure 4.16) that may be useful in pavement evaluation.

Typically, GPR can provide information about changes in pavement construction, layer thicknesses and defects/features within the pavement. The usefulness of the information obtained from ground radar is largely a function of four factors (Highways Agency 2008).

- the electrical properties (dielectric constant and the conductivity) of the materials forming the pavement
- the type of GPR equipment employed
- the processing software and analysis
- the methodology including calibration procedures employed.

GPR systems transmit electromagnetic signals (i.e. microwave energy) into the ground and receive reflections from subsurface features. They measure the strength of these reflections and the time it takes for them to come back to the surface to produce an ‘image’ of the subsurface.

A GPR survey is relatively quick and non-invasive. However, once a survey has been completed, the raw data needs to be analysed and interpreted to yield useful information. This may take some time and requires specific expertise. Like all non-destructive testing (NDT) methods, calibration against physical measurements, such as those obtained from trenches or cores, is strongly recommended to make the results as representative as possible. That is, additional fieldwork (e.g. coring, trenching) and/or destructive testing is normally required to validate the GPR results. As a consequence, GPR is a complementary tool (e.g. to deflection testing and fieldwork) rather than a technique which can be relied upon on its own.

For pavement investigations, the transverse location of tests may be similar to deflection testing. For instance, the outer wheelpath in the outer lane may be targeted with a single run in one direction, or multiple runs can be used to investigate other wheelpaths.

Figure 4.15: Example of a ground-coupled antenna with distance-measuring wheel set-up behind a vehicle

Source: Private communication with Queensland Department of Transport and Main Roads (TMR) (2010).

Figure 4.16: Example of an air-coupled antenna mounted on the front of a vehicle

Source: Plati et al. (2010).

Advantages of using GPR include the following:

- It is non-destructive (for the GPR survey and investigation itself).
- Changes in pavements can be identified to allow investigations to be better targeted and to proceed with more confidence.
- Longitudinal and sometimes transverse changes can be identified. In particular, the pavement can be sectioned with more confidence.
- Pavement investigations are better optimised.

How well a particular GPR system can achieve the above depends on the characteristics of the system used, the expertise of the operator(s) and the expertise of those who analyse the data.

Disadvantages of using GPR include the following:

- It is costly.
- Field investigations are normally required to obtain accurate results. That is, the use of GPR does not eliminate the need for the other fieldwork, investigations and testing (e.g. trenching, sampling and laboratory testing).
- GPR surveys must not be carried out when it is raining or when standing water is present on the surface of the pavement. This is because a film of surface water may affect the radar signal, making interpretation of the data more difficult.
- GPR cannot penetrate metal, such as closely spaced reinforcement or highly conductive materials, and therefore is not used for continuously reinforced concrete pavements.
- Analysis is time-consuming, which affects the cost and the timeframe of the investigation.
- Analysis and interpretation require input from a GPR specialist and the person or team that is undertaking the pavement investigation.

It should be noted, however, that the technology is still developing – productivity and techniques may improve and costs may reduce with time.

4.9 Surface Deflection of Flexible Pavements

4.9.1 General

The surface deflection of a flexible pavement under an applied load is an important indicator of its structural condition. It is also an important parameter in the design of structural treatments and, together with the pavement layer thicknesses, in the back-analysis of existing flexible pavements to estimate the pavement layer and subgrade moduli (see Section 10).

As an indicator of structural condition, deflections aid the selection of appropriate structural rehabilitation treatments if any is required, by identifying:

- the structural adequacy of the overall pavement
- homogeneous lengths of pavement which might be treated similarly
- areas of weak pavement (inadequate thickness, poor quality pavement materials, soft subgrade) requiring specific treatment (e.g. patching)
- areas for more detailed pavement investigation.

Because of the capability of some methods to measure deflection rapidly, it is possible to use deflections to characterise a substantial length of pavement in a relatively short period of time.

For some projects, specifically where the condition of the heavily trafficked lane will dictate the rehabilitation treatment for all lanes, it may be appropriate to record deflection measurements only in the heavily trafficked lane and then mainly in the outer wheelpath. In other situations, where the requirements for strengthening vary across the pavement, it may be necessary to measure deflections in both the outer and inner wheelpaths and in all lanes, in order to enable consideration of localised strengthening. In particular, measurements in both wheelpaths are useful in identifying areas where moisture ingress from the shoulder is affecting pavement performance.

4.9.2 Methods of Testing

Two methods of deflection measurement are commonly used in Australia and New Zealand. The Benkelman Beam (Figure 4.17) and the Deflectograph (Figure 4.18, Figure 4.19) operate on a similar principle of a rolling wheel load to obtain pavement surface deflection readings. However, the ancillary equipment associated with each of these devices, such as the vehicles used to apply the test load, differs slightly. The response recorded by each device varies depending on the composition of the pavement being tested.

Structural analysis and granular overlay procedures given in Section 9 were developed to make use of deflections measured with a Benkelman Beam and Deflectograph under an 80 kN standard axle load.

Austroads test method AG:AM/T007-11 *Pavement Deflection Measurement with a Deflectograph* describes the Deflectograph testing procedure .

Figure 4.17: Benkelman Beam with load truck and data logger to record measured deflection bowl

Source: Private email from Greg Arnold (November 2018).

Figure 4.18: Host truck with loaded rear axle and deflection measurement sled in front of rear axle

Source: Highways Agency (2008).

Figure 4.19: Deflectograph with measurement beam shortly after starting a deflection bowl measurement

Source: Highways Agency (2008).

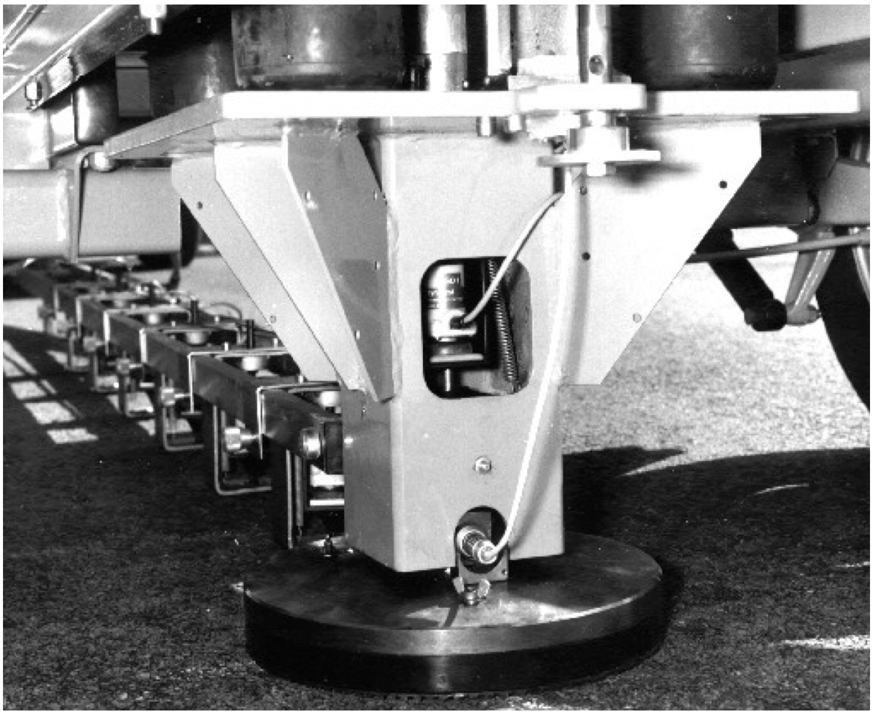
A third method of deflection measurement – Falling Weight Deflectometer (FWD, Figure 4.20) – is increasingly being used in Australasia. Its principle of operation differs markedly from that of the other methods, using a falling weight in lieu of a rolling loaded wheel to load the pavement and geophones in lieu of a deflection beam to measure pavement deflections at specific distances (offsets) from the point of application of the load (Figure 4.21). Austroads test method AG:AM/T006-11 *Pavement Deflection Measurement with a Falling Weight Deflectometer (FWD)* describes the testing procedure.

The empirical process for the design of granular overlays on flexible pavements (Section 9) uses FWD deflections measured with a plate contact stress of 566 kPa applied to a 300 mm diameter plate and a load pulse duration of about 25 milliseconds. These procedures use the deflection at the centre of the FWD loading plate (D_0). Note that if the FWD loading plate contact stress during measurement differs from 566 kPa (40 kN load), deflections need to be normalised to a stress of 566 kPa, usually assuming deflections are linearly related to load level. To minimise errors of adjustment, the contact stress during testing should be as close as practicable to the standard stress and differ from 566 kPa by no more than 15%.

Figure 4.20: General view of a FWD

Source: Australian Road Research Board.

Figure 4.21: FWD loading plate and geophones to measure pavement response to load



Source: COST Transport Program (1999).

Deflections may be measured with a Heavy Weight Deflectometer (HWD), and these measurements may differ from FWD values at the same contact stress. As the structural analysis of flexible pavements and procedures are based on FWD deflections, HWD deflections may need to be standardised for load to FWD values.

If FWD deflections are measured for use with the mechanistic-empirical procedure (MEP) for strengthening design of flexible pavements (Section 10), contact stresses higher than the standard 566 kPa may be required to reduce measurement errors for stiff pavements with very low deflections. Layer moduli may be back-calculated from the measured FWD deflections (see Section 10.7). In this event, it is necessary to measure the maximum deflection and deflections at six or more offsets from the centre of the loading plate. Commonly used offsets are 0 mm, 200 mm, 300 mm, 450 mm, 600 mm, 900 mm and 1500 mm.

Whilst the FWD allows more precise estimates of the deflection bowl parameters at a site, the Deflectograph is highly productive and provides an extensive coverage of the variation of pavement strength between wheelpaths and along the project length.

The use of newly-developed Traffic Speed Deflectometers (TSD) (Austroads 2017b) (Figure 4.22) for road network strength evaluation is progressing, and the use of this data for the design of strengthening treatments is emerging. These deflection devices evaluate the response to load by measuring the deflection velocity of the pavement under a load applied on a single axle through dual rear wheels. The procedures in this Part assume an axle load of 10 tonnes is used, and the deflections are standardised to a 50 kN load on a set of dual tyres. Doppler laser sensors measure the vertical velocity of the pavement surface as it moves down under deflection, rather than measuring displacement directly.

As specified in Austroads test method AG:AM/T017-16 *Pavement Data Collection with Traffic Speed Deflectometer (TSD) Device,* a minimum of seven laser sensors is required. The TSD used in Australasia is currently fitted with seven sensors, six of which measure the deflection velocity at the following locations in front of the rear axle (located at 0 mm): 100 mm, 200 mm, 300 mm, 450 mm, 600 mm, 900 mm. The remaining sensor acts as a reference and is located 3500 mm in front of the rear axle where it is assumed that the pavement is unloaded, and therefore, there is no vertical velocity response.

Currently in Australasia, the vertical deflection bowl is estimated by analysis of the vertical and horizontal deflection velocities using the area under the curve approach.

Knowing the vertical velocity of the pavement (from the Doppler lasers) and the horizontal velocity of the TSD allows the slope of the deflection to be determined (Krarup et al. 2006). The deflection of the pavement can then be calculated using one of several analysis methods. In Australasia, a numerical method based on the area under the deflection slope curve (Muller & Roberts 2013) has been selected as the preferred method to calculate the surface deflections.

Testing is carried out continuously and reported at a 10 m or greater interval, with measurements being made in the outer wheelpath.

Figure 4.22: Traffic speed deflectometer

Source: Australian Road Research Board

In selecting between deflection devices, the following should be noted:

- The Benkelman Beam is used for point testing at user-specified locations, and both the outer and inner wheelpaths may be tested. It is a manually operated, low-cost, mechanical testing system. For stiff pavements, the bowl measurements are affected by the movement of the reference beam in the deflection bowl. As such, the use of Benkelman Beam maximum deflections in this Part is limited to the design of granular overlays on flexible pavements.
- The Deflectograph is, in essence, a pair of automated Benkelman Beams, mounted under a truck, which allows point testing at user-specified locations in the outer and inner wheelpaths. The deflection beams measure deflections relative to a reference frame, automatically pulled along the pavement by the truck. Commonly, deflections are measured at 4–5 m intervals along a test section. Hence, the Deflectograph is well-suited to testing roads that vary significantly in stiffness as the closely spaced deflection measurements enable identification of local weak areas of pavement requiring patching/reconstruction. However, Deflectograph deflection bowls are affected by deflection of the reference frame under the influence of the front and rear truck axle loads. As a consequence, procedures are not provided in this Part for the design of strengthening treatments using the mechanistic-empirical procedure using Deflectograph measured bowls. However, procedures are provided in Section 9 for the design of granular overlays for flexible pavements using Deflectograph maximum deflections. Deflectograph maximum deflections may also assist in identifying homogeneous sections for use in the mechanistic-empirical procedure (Section 10).
- The TSD measures pavement response virtually continuously travelling at highway speeds, usually at 80 km/h. In terms of designing strengthening treatments at the project level, interim procedures are provided in Section 9 for the design of granular overlays for flexible pavements. Although procedures have yet to be developed to back-calculate pavement and subgrade moduli from the TSD responses, the TSD may assist in identifying homogeneous sections for use in the mechanistic-empirical procedure (Section 10).
- The FWD provides the most accurate means of measuring the ‘true’ deflection at any single point as, unlike Benkelman Beam and Deflectograph deflections, the measured deflections are not influenced by movement of the reference frame within the deflection bowl. As such, it is the only deflection device that enables flexible pavement strengthening treatments to be designed using the mechanistic-empirical procedure (Section 10). However, it is not as cost-effective as the Deflectograph in measuring deflections at closely spaced intervals if this is required to accurately characterise pavements with variable stiffness.

The *Guide to Asset Management Technical Information Part 15: Technical Supplements* (Austroads 2018b) provides a detailed description of measurement and the above applications.

4.9.3 Selection of Test Sites

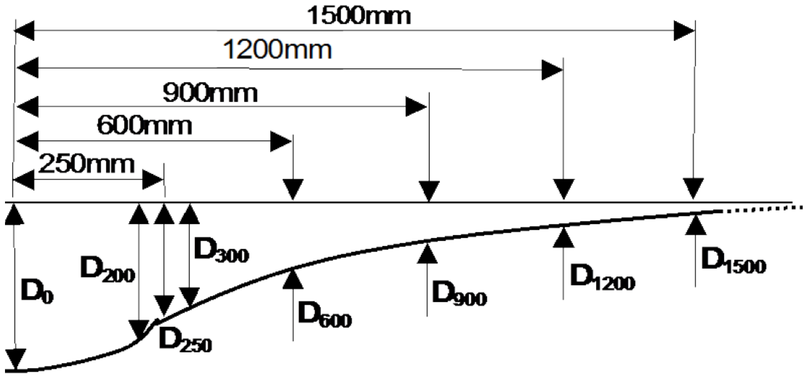
When testing with the Deflectograph and TSD, it is necessary to select only the transverse location of wheelpath locations, because the longitudinal spacing of test sites is a function of equipment geometry and test speed. If practical, wheelpath positions should be selected keeping in mind any proposed changes to the road alignment. The close spacing of deflection test sites obtained using the Deflectograph is very useful in isolating weak sections for local strengthening as part of the overall rehabilitation treatment.

For Benkelman Beam and FWD testing, the spacing of individual test sites in a given section of road is arranged so that the general pattern of deflections over the whole section of road can be identified and sub-sections with consistent deflections can be defined. An adequate number of results for each sub-section, at least 10 and preferably upwards of 30, should be obtained for the purposes of statistical analysis. Normally the spacing used lies between 5 and 100 m, with wider spacings being used for more uniform pavement conditions. Again, if practical, wheelpath positions should be selected, keeping in mind any proposed changes to the road alignment.

4.9.4 Response to Load

Deflection of the pavement surface under an applied load can be visualised as a depression in the pavement's surface, termed a 'deflection bowl'. By convention, the magnitude and shape of the bowl is described in terms of the deflection value 'D' and its offset from the centre of the applied load. Thus 'D_0' is the deflection at the centre of the bowl (which is also the site of the maximum deflection), 'D_{200}' is the deflection 200 mm offset from the centre, and similarly for other values, as shown in Figure 4.23.

Figure 4.23: Schematic of pavement surface deflection bowls (not to scale)



Source: Adapted from Austroads (2008a).

In this Part, response to load is characterised in terms of:

- maximum pavement surface deflections (D_0) measured with Benkelman Beam, which may be taken as the total deflection minus the residual deflection, that is, the rebound deflection
- maximum pavement surface deflections (D_0) measured with Deflectograph, TSD or FWD
- deflection bowls measured with the FWD.

4.9.5 Measurement of Pavement Temperature

For flexible pavements containing asphalt, variations in temperature result in significant changes in the asphalt modulus and, therefore, in the deflection of the pavement. The temperature of the asphalt is typically recorded at a mid-depth in the asphalt during deflection testing so that appropriate adjustments can be made during the analysis and design phases. If weather conditions vary during the deflection survey, several measurements of temperature must be taken, and different adjustments made for corresponding sections of the test length.

The frequency at which temperatures are measured needs to be considered. More frequent measurements are required when the temperature is changing quickly. In addition, more frequent measurements are required for pavements with thick asphalt layers than for those with thin layers, as the deflections on thick asphalt layers are more highly influenced by the asphalt temperature.

To adjust measured deflections for temperature, the existing asphalt thickness is required. Asphalt thicknesses may be obtained from existing records or measured by coring the pavement or excavating pits in the pavement.

4.10 Surface Deflection of Rigid Pavements

4.10.1 Surface Deflection Data

Compared to flexible pavements, deflection testing has a much more limited application for rigid pavements; typically, it is used to assess:

- the ability of joints to transfer loads between adjacent slabs
- the presence of voids under joints and cracks.

In these situations, testing can be undertaken using a Benkelman Beam but is more commonly undertaken using a FWD. Each method has its advantages and disadvantages for a particular application as discussed in Part 5D of the Austroads *Guide to Asset Management* (Austroads 2008b).

Slab flexural stresses at or near a joint or crack are governed by the ability of the joint to transfer load between adjacent slabs; the lower the load transfer, the higher the stresses and hence the lower the fatigue life. This ability of a joint to transfer load between adjacent slabs is defined as the load transfer efficiency as described in Equation 2.

$$\text{LTE} = 100 \times \frac{d_u}{d_l} \qquad 2$$

where

LTE = load transfer efficiency (%)

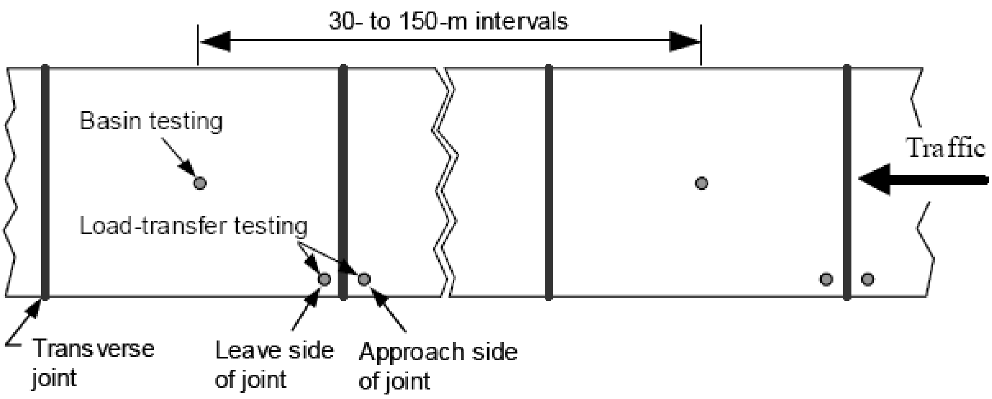
d_u = deflection of the slab on the unloaded side of the joint

d_l = deflection of the adjacent loaded slab

For jointed plain concrete pavements and jointed reinforced concrete pavements, vertical deflections at the approach and departure sides of a joint are measured using the FWD to evaluate the load transfer efficiency and to detect the presence of voids under the slabs where a lean-mix concrete subbase is not used. In addition, mid-slab deflection (basin deflection) is sometimes measured (refer to Figure 4.24 and Figure 4.25).

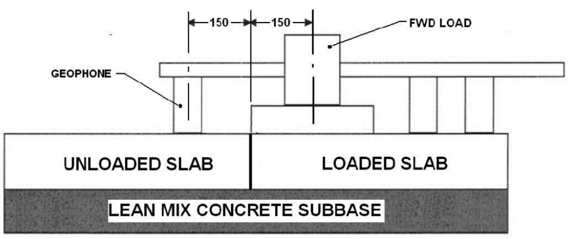
Note that different load transfer values may be obtained depending on which side of the joint is loaded, so both sides of the joint should be tested, and the lowest value used.

Figure 4.24: Typical deflection testing locations for PCP and JRCP



Source: Federal Highways Administration (2008).

Figure 4.25: FWD configuration at pavement joints



Source: Wright (2009).

As load transfer is often lowest in the outer wheelpath of the most heavily trafficked lane, joint load transfer should be measured at this location.

To detect voids under slabs, corner deflections are measured using the FWD at three load levels (25 kN, 40 kN and 55 kN) to establish the load-deflection response for each test location.

On clear, sunny days, testing should be conducted before midday to minimise the possibility of joint lock-up at the time of testing. On cool, overcast days, deflections may be performed throughout the day.

There are no standards governing the location and quantity of joints to be tested; this generally depends on the project scope and the results of a visual survey.

5. Pavement Composition and Subgrade Characterisation

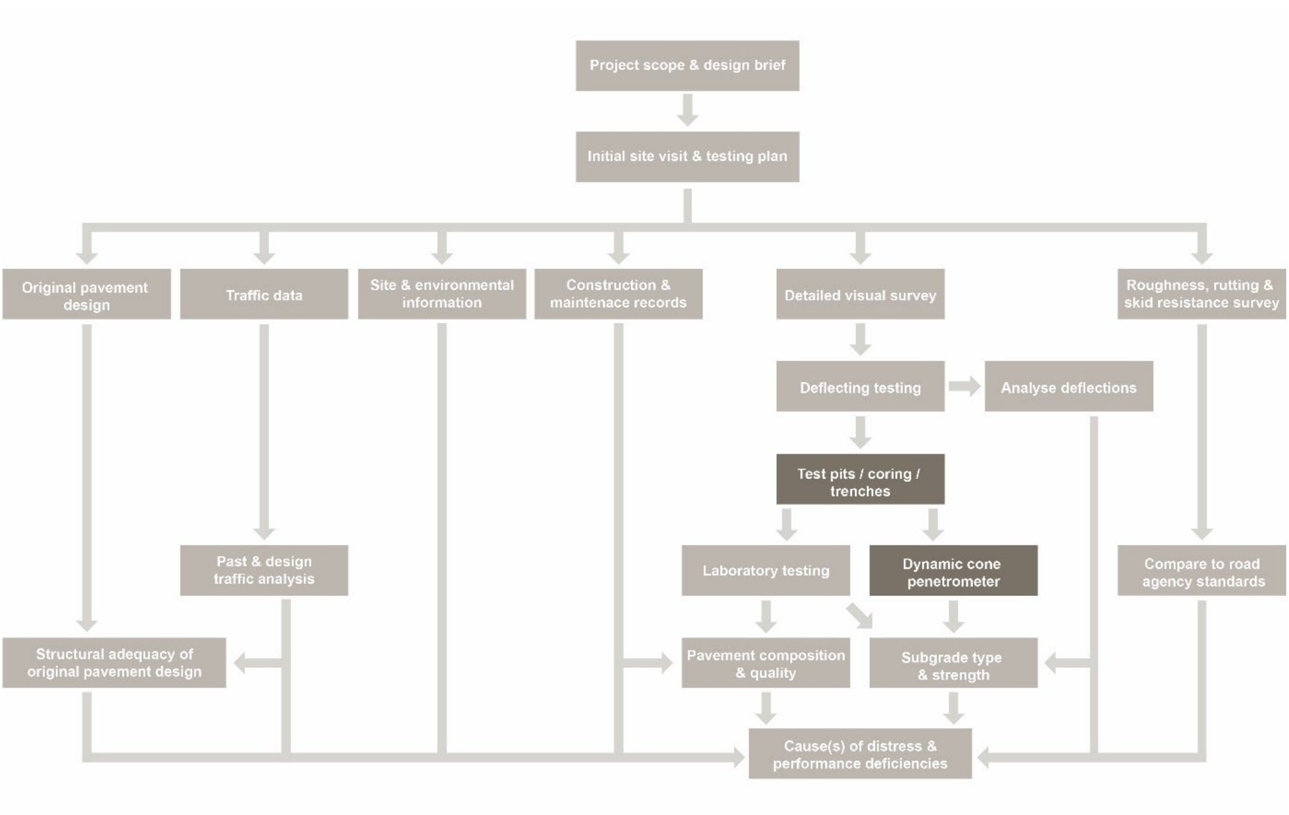
5.1 Introduction

Information about the quality and thickness of the pavement materials is essential for a thorough understanding of the causes of pavement distress (Figure 5.1). An assessment of the material types, within and beneath the pavement, including their extent and condition, may be required for a full appreciation of pavement performance. Measurement of the in situ condition of pavement materials such as density and moisture content is often useful where distress at the interface between different materials is observed. Such distress may be indicative of either compaction difficulties during previous stages of construction or the result of differential permeabilities or an ineffective drainage system. The measurement of subgrade moisture contents and their variations along the pavement also serves as a guide to the performance and effectiveness of the pavement's moisture control system.

Knowledge of the existing pavement composition is useful in resolving anomalies between the visual condition and the load/deflection response, in determining reasons for the nature of particular pavement performance and, ultimately, in making decisions about appropriate rehabilitation treatments. In some cases, pavement composition data is obtained for inclusion in tender documents for the construction of rehabilitation treatments.

As pavement investigations are a high-cost activity, it is important to correctly locate test sites to maximise the information obtained. Deflection results, combined with the visual condition of the pavement, may be used to identify pavement areas representing a range of moduli and conditions, which can then be the target of subsequent investigation. In this process it is often useful to compare areas of sound and distressed pavement to determine whether variations in the pavement composition (e.g. layer thickness, condition of pavement materials) have contributed towards or caused the difference in performance.

Figure 5.1: Design steps discussed in Section 5



5.2 Coring of Bound Materials

For pavements with asphalt surfacings or bound cemented materials, the pavement investigation may also include coring of bound layers, as this is a more cost-effective means of estimating layer thicknesses and sampling bound materials for laboratory testing. Coring is also useful in determining the depth of cracking below the surface and to investigate asphalt stripping. However, when cemented materials are covered by thick asphalt layers, caution is advised in using coring results to evaluate the extent of micro-cracking due to the uncertainty of knowing where to core when cracks are not visible on the pavement surface.

Coring may be undertaken to assist in explaining variations in pavement condition or surface deflections. As a consequence, the coring sites are best selected after both the visual and deflection surveys have been completed.

Dynamic cone penetrometer (DCP) tests (Figure 5.2) may be undertaken in the core holes to provide a measure of in situ subgrade strength. In such cases, and when moisture-induced distress is being investigated, it is recommended dry ice be used to cool the core barrel rather than water (Figure 5.3).

Figure 5.2: Dynamic cone penetrometer

Source: Australian Road Research Board.

Figure 5.3: Coring of bound materials using compressed air and dry ice

Source: Australian Road Research Board.

5.3 Pavement Pits and Trenches

Where records of the pavement configuration are unavailable or considered unreliable, pit or trench sampling will enable the profile to be observed and representative samples to be taken. Pits enable the determination of:

- type, thickness, quality and condition (density and moisture content) of each pavement material
- sampling of pavement materials and subgrade for laboratory testing
- layers in which rutting and/or cracking is significant
- delamination between layers
- in situ subgrade strength, by use of DCP, static cone penetrometer or in situ California Bearing Ratio (CBR) tests.

For pavements with substantial rutting, trenches across the full lane width or coring in and between ruts may also be required to establish the extent of deformation of individual pavement layers and the subgrade (Figure 5.4). If the pavement has been widened, trenching will also enable an assessment of material differences between lanes.

The efficiency of the investigation may be better served by collecting a greater number or quantity of samples or conducting extra in situ testing than may initially appear necessary. This is because the cost and time involved in re-establishing sampling and testing resources at the site to obtain additional information can be prohibitive. The retention of excess samples of material following evaluation testing is also prudent, as it will permit additional testing to be conducted if required.

The testing undertaken on sampled materials is largely based on the material specification requirements of the road agency (see Section 6.6.1 and Section 6.6.2). At times, however, it may prove more informative to carry out testing of fundamental material qualities (strength and permeability) than the usual indicator tests (moisture content, binder content, grading and Atterberg Limits) on which the specification may be based.

Where in situ stabilisation of the existing pavement is an option, materials may need to be sampled from the test pits for use in laboratory mix design of the stabilisation treatment. If data from the visual survey, roughness testing and other sources suggest the subgrade may be expansive (refer to Table 6.8), the subgrade material should be sampled to confirm this.

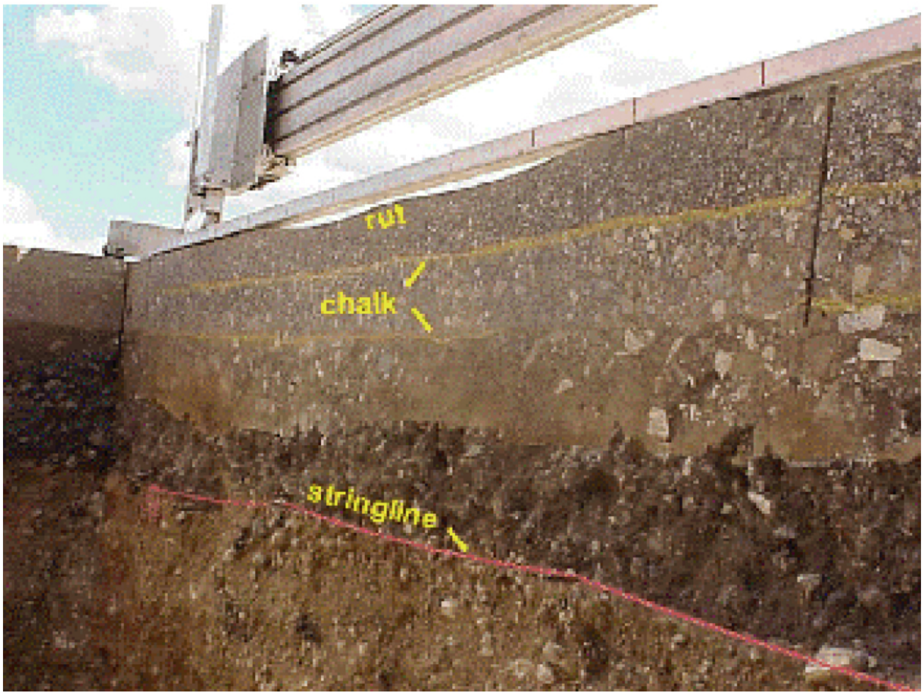
Measurement of the in situ condition of pavement materials such as density and moisture content is often useful where distress at the interface between different materials is observed. Such distress may be indicative of either compaction difficulties during construction or the result of differential permeabilities or an ineffective drainage system. The measurement of subgrade moisture contents and their variations along the pavement also serve as a guide to the performance and effectiveness of the pavement's moisture control system.

Before the test pit is excavated, the surface condition should be recorded and photographed. At sites where there is fine surface cracking it may be necessary to wet and partially dry the surface to highlight the cracking. Where there is rutting, placing a straight edge on the surface can make the extent of the rutting more apparent in the photograph.

Where pits are excavated in thick asphalt or concrete pavements, it is usually necessary to use a pavement saw to cut the surface. To reduce the influence of the sawing cutting water on the moisture in the underlying layers, the amount used should be kept to a minimum. In excavating the material from the test pit, it is common to carefully excavate and sample each pavement layer. Samples for moisture content determination need to be stored in an air tight container to reduce evaporation. The condition of each pavement layer should be noted as it may provide valuable information on the cause of distress. For example, for granular material, observations of whether the moisture content is noticeably higher in the top or bottom may provide insight into the source of excessive moisture.

After the test pit has been excavated, the layer interfaces are identified to enable measurement of layer thicknesses. In the event that a trench has been excavated across the lane, the variation in thickness within and between wheelpaths provides a measure of the load-induced rutting of each pavement layer (Figure 5.4).

Figure 5.4: Example of pavement investigation in a trench



Source: Texas Department of Transportation (2018).

Information obtained in the field excavation should be recorded on a borehole log sheet or similar.

A detailed description of pavement investigation procedures is provided in VicRoads Technical Bulletin 40 *Pavement Investigation: Guide to Field Inspection and Testing* (VicRoads 1995a).

5.4 In situ CBR from DCP Testing

One of the principal objectives of subgrade evaluation is to determine subgrade design CBR values 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 homogeneous section identified in need of a structural treatment.

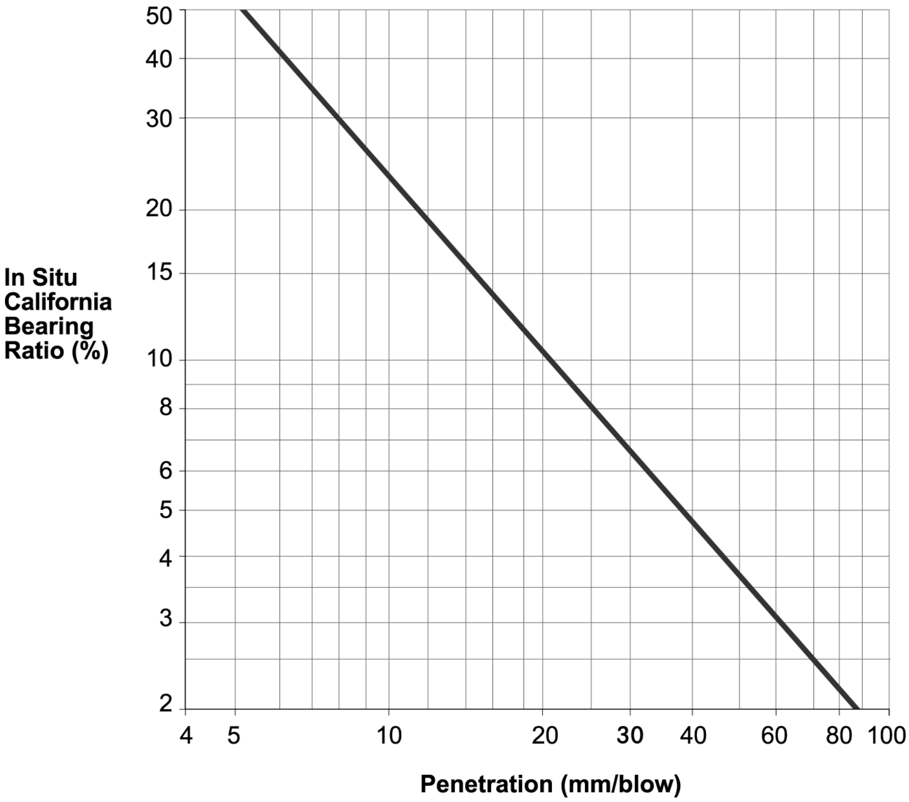
In the event that DCP testing of the subgrade has been undertaken, the data may be utilised in estimating the subgrade design CBR for use in the design of structural treatments. Such DCP testing is most applicable to situations where the support values from the in situ subgrade soil conditions are expected to be similar to those after pavement rehabilitation.

Particularly for large projects, it is recommended that the CBR results estimated from the penetration values be checked against a remoulded laboratory CBR test. There can be significant variation in results depending on the subgrade moisture conditions at the time of testing. This is why the subgrade moisture content is required to translate field measurements to appropriate design values. If the CBR results have not been measured in the most critical moisture condition, adjustment of the results for seasonal moisture variations may be required.

DCP tests should be carried out in accordance with AS 1289.6.1.3-1998 or NZS 4402.6.5.2:1988. The DCP test should be restricted to fine-grained subgrades to avoid misleading results due to the influence of large particles. The CBR can be determined from the results of dynamic cone penetrometer testing using Figure 5.5. 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, such as Mulholland (1984), Schofield (1986) and Smith and Pratt (1983).

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

Figure 5.5: Correlation between Dynamic Cone Penetration and CBR for fine-grained cohesive soils



Penetration readings are commonly measured to a depth of at least 1 m below the top of the subgrade or until sufficiently high values are measured compared to the overlying subgrade.

In utilising the in situ CBR in the determination of a subgrade CBR value, consideration needs to be given to variation in penetration values with depth. For the majority of soil types, the best correlation with the in situ CBR test is achieved when the DCP mm/blow is calculated from a weighted average of blows/50 mm for the first three 50 mm intervals using weightings of 0.7, 0.2 and 0.1 for each interval (NZ Transport Agency 2018). Therefore, the weighted DCP results from the first 150 mm below the CBR test depth is used to determine the design CBR. Below this depth, soil properties do not significantly affect results.

However, this methodology should be tempered with the Austroads (2018a) advice that evaluation of the actual support provided to the pavement structure by the subgrade can be complicated by the strength variations that often occur with depth. It is essential that the potential effects of any weak layers below the design subgrade level are considered in the pavement design process, particularly for low-strength materials occurring to depths of about one metre. Where strength decreases with depth, the subgrade may be sub-layered for the purposes of the structural design of flexible pavements and when calculating the effective subgrade strength for rigid pavement designs. For subgrade strengths that are constant or improve with depth, the support at the design subgrade level governs the pavement design.

Section 10.7.2 describes the use of field-estimated CBR values in the design of treatments for flexible pavements, while Section 11 and Section 12.3 describe the use in the design of concrete overlays.

6. Causes and Modes of Distress

6.1 Introduction

In the design of any rehabilitation treatment, the first and most important phase is the identification of the mode or modes of distress evident in the pavement and their associated root cause or causes. A root cause analysis integrates and interprets the various failure mechanisms, the existing pavement structure, and the traffic history to determine the root cause or causes of the pavement distress. If the root cause of distress is incorrectly assessed, then it is very unlikely that the selected rehabilitation treatment will be effective, regardless of the quality of its design and construction.

In undertaking this part of the rehabilitation design, it is necessary to analyse the collected data in search of meaningful correlations between:

- the visual condition and the test data of the existing pavement
- the various factors that are known to affect the performance of pavements.

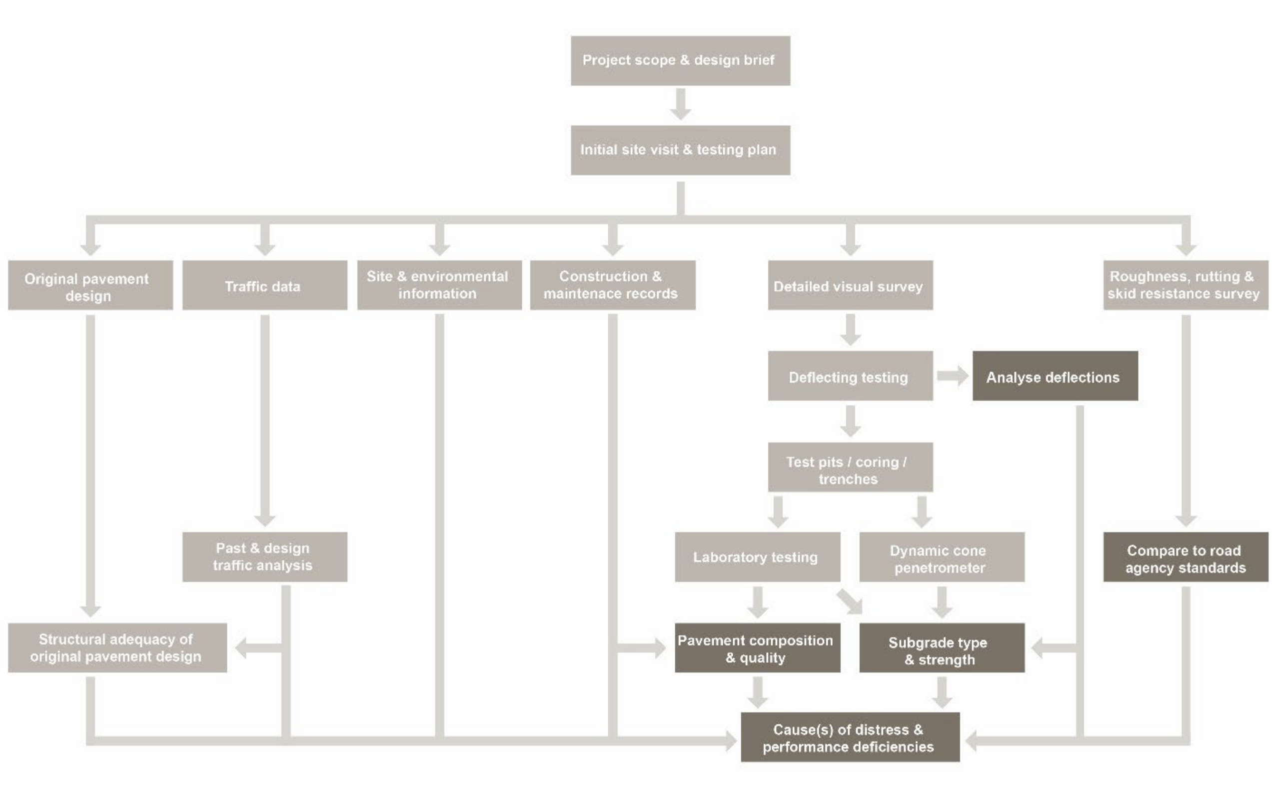
The identification of the modes and causes of pavement distress should usually involve:

- the definition of the location, nature and severity of the distress
- assessment of the root cause of the distress, including the importance of load and environmental factors on pavement performance.

As engineering judgement is crucial to this process, the relationships between the defects and their causes and the required rehabilitation treatments are presented in a non-prescriptive manner (refer to Appendix A). In addition, the repeatability and reproducibility of pavement condition measurements need consideration.

Figure 6.1 illustrates this critical step in the design process, including considering road agency standards in identifying performance deficiencies.

Figure 6.1: Design steps discussed in Section 6



6.2 Classification of the Causes of Distress

The causes of pavement distress may be broadly classified as either external (that is, causes linked to the environment in which the pavement is operating) or internal (which are causes intrinsic to the structure and composition of the pavement). The principal external causes are:

- climate (temperature, rainfall, UV radiation)
- imposed loadings (traffic intensity, vehicle mass, axle configuration, tyre pressure)
- adverse moisture regime (excessive moisture, seasonal moisture variation)
- inadequate or unstable foundation (subgrade strength, subgrade expansion and contraction).

A feature common to each of these causes is their dependence on time, hence, in assessing the influence of these causes on any observed distress, the age of the pavement and its components should be considered.

In addition, there are several internal causes linked to the structure and composition of the pavement, including:

- inadequate material quality and condition (strength, particle gradation, plasticity, density, moisture level)
- inadequate material thickness.

These internal causes may be built into the pavement at construction as a result of inadequate specifications and/or poor construction processes or practices or may develop over time. In the latter case, they are usually linked to one or more of the external causes (e.g. pavement material becoming inadequate due to breakdown under the imposed loadings).

In assessing pavement distress, it is useful to identify the *initial* mode of distress and its cause, because these will determine the most appropriate form of rehabilitation. For example, two pavements may both exhibit severe rutting and cracking; in one, cracking occurred first and allowed moisture into the pavement, which was consequently weakened and subsequently rutted, whereas the other pavement initially rutted and then the deformed surface cracked. This knowledge may result in the treatment of these two pavements being quite different. It is also important to assess whether the distress is a manifestation of a deficiency or deficiencies in the overall pavement or only in the surfacing.

6.3 Distress Modes

There is an extensive range of distress modes, which may occur in a pavement, depending on the type of pavement, the environment in which it operates and its structure and composition. Appendix A provides a series of photographs of various distress modes with descriptions of their principal characteristics and a list of possible causes.

It is important that personnel involved in the management, maintenance and rehabilitation of pavements are conversant with this information because it is critical that they have a common understanding of the various modes of distress and use consistent terminology in reporting the visual manifestation of distress. Inconsistent reporting of a visual pavement condition can lead to unnecessary expenditure either as a result of undertaking inappropriate work or failing to implement essential rehabilitation at the most appropriate time.

6.4 Evaluation of Pavement Condition Data

6.4.1 Visual Condition

The type, severity and extent of visual distress provide the most immediate and direct indicator of both the mode or modes of distress and associated causes.

Evaluation of visual condition should include:

- identification of patterns in the type and distribution of distress
- links between types of distress and other recorded features e.g. cut/fill boundaries, drainage and vegetation
- links between distress and other data recorded, e.g. deflections, construction history and geology
- links between distress and traffic loading, e.g. where distress is related to road grade, points of entry and exit of heavy vehicles.

In evaluating a pavement, it is important to determine the adequacy of the surface and subsurface drainage systems in preventing or controlling the infiltration of moisture and to determine the extent to which pavement performance relies on the integrity of the drainage systems.

Appendix A lists treatment options for flexible pavements by distress type as characterised by visual defects.

6.4.2 Roughness

The initial step in evaluating the roughness data is to compare the measured values against acceptable values set by the road agency, which may vary with the design speed and the function of the road.

For pavements with measured roughness below the appropriate investigatory level for the road class, the past rate of increase of roughness with time/loading repetitions is a useful indicator of when, in terms of time or loading repetitions, the relevant investigatory levels might be reached.

Numerous pavement defects may contribute to the increase in pavement roughness. These include:

Flexible pavements

- depressions
- ruts
- potholes
- patches
- corrugations
- shoving
- delamination, debonding
- stripping
- cracking

Rigid pavements

- stepping/faulting
- rocking
- pumping
- spalling
- patches

Note that roughness can develop from both load and non-load (e.g. material volume changes associated with moisture changes) sources. Identifying the cause of the roughness can be critical with respect to selecting an appropriate rehabilitation treatment.

Visual assessment of the possible causes of roughness can be useful in deciding rehabilitation treatments. Roughness may be due to load-associated factors, environmental factors or deficiencies in construction, maintenance and restoration of service trenches. Where the causes are not load-associated, a limited rehabilitation treatment, designed to solely restore pavement evenness could be the most appropriate choice, e.g. cold planing and a thin overlay, or for a sprayed seal surfaced granular pavement, scarifying and reshaping with or without additional granular material.

When considering serviceability, roughness versus age profile curves may be used to estimate a possible remaining service life of the pavement.

6.4.3 Rutting and Shape Loss

Evaluation of the transverse profile data derived from either straight edge or automated measuring devices is also a relative assessment involving comparison with desirable levels of rutting established by road agencies. Table 6.1 sets out typical indicative investigation levels for rutting on various classes of existing road.

Table 6.1: Indicative investigation levels of rutting

Road function	Percentage of road length with rut depth(1) exceeding 20 mm
Freeways and other high-class facilities	10
Highways and main roads (100 km/h)	10
Highways and main roads (less than 80 km/h)	20
Other local roads (sealed)	30

1 Measured with a 1.2 metre straight edge.

In contrast to the roughness levels, however, these rutting levels are principally designed to ensure the safety of road users and need to be considered in conjunction with other factors affecting safety (e.g. climate, road geometry, traffic speed and proximity to traffic control devices). Therefore, evaluation of the rutting data, and of the need for pavement rehabilitation to correct rutting where the levels exceed these investigation levels, is dependent on the pavement location to which the data relates. For instance, rutting of a section of pavement approaching a signalised intersection in a wet environment is likely to have a greater influence on road safety than the same level of rutting on a rural road in a dry environment and, hence, more in need of a rehabilitation treatment.

To assist in identifying the cause of rutting, the existence or absence of associated shoving is an important attribute. Where ruts are wide and there is little or no evidence of shoving, they are more likely related to deformation at depth e.g. at the subgrade level as a result of insufficient pavement strength. Sometimes compaction of the pavement under traffic may also contribute to the deformation.

Where rutting is associated with shoving, it is indicative of:

- inadequate shear strength of the surfacing or base material
- poor bonding between the upper pavement layers
- lack of containment of the pavement edge.

If the cause of shoving is not immediately evident, excavation across the lane should enable identification of which layer has shoved and the sampling of materials for laboratory testing. In the case of asphalt-surfaced pavements, asphalt coring within and between ruts can be used to evaluate the extent to which the asphalt has deformed and provide samples for testing the quality of the asphalt.

Inadequate pavement strength is the result of pavement layers being too thin or of insufficient quality to distribute the applied load sufficiently to avoid overstressing lower layers in the pavement or the subgrade. To assess whether rutting is due to inadequate pavement strength, it is useful to plot measured pavement deflections at various chainages against measured rut depth at the time of deflection testing. The higher the correlation of rut depth and deflection the more likely the rutting is due to inadequate pavement strength. If rut depth does not correlate with pavement deflection and there is little or no shoving, the most likely cause is densification of the pavement layers under traffic early in the life of the pavement.

On expansive subgrades, excessive rutting may be due to the relative rise or ‘heave of the shoulders’ exaggerating the appearance of the rut.

6.4.4 Skid Resistance

Skid resistance data, like roughness, is most often used to confirm the existence of a potential problem and not to diagnose a problem. Skid resistance measurements may provide a functional justification for treating the section.

The initial step in evaluating the skid resistance data is to compare the measured values against indicative investigatory levels. Table 6.2, indicative investigatory skid resistance levels, as measured by SCRIM, provides an example of how Sideways Force Coefficient (SFC) values can be used. The values given in Table 6.2 are indicative only and may not be appropriate in all circumstances as discussed in *Guide to Asset Management Technical Information Part 15: Technical Supplements* (Austroads 2018b).

Table 6.2 also contains guidelines on differential friction levels. These levels have been developed from experience in Australia and New Zealand where SCRIM testing is undertaken in both wheelpaths and are considered to be practical levels to reduce risks associated with a braking vehicle rotating due to transverse variation in skid resistance. VicRoads (2018a, 2018b) provides a useful explanation of skid resistance investigatory levels.

Table 6.2: Examples of investigatory skid resistance levels

Site category	Site description	Investigatory levels of SFC_{50} (At 50 km/h or equivalent – local format)						
		30	35	40	45	50	55	60
		Investigatory levels of SFC_{50} (At 50 km/h or equivalent – typical international practice)						
		0.30	0.35	0.40	0.45	0.50	0.55	0.60
1#	• Signalised intersections • Pedestrian/school crossings • Railway level crossings • Roundabout and approaches			INVESTIGATION IS				
2	• Curves with tight radius ≤ 250 m • Gradients ≥ 5% and ≥ 50 m long • Freeway, highway; on and off ramps			ADVISED				
3#	• Intersections							
4	• Manoeuvre-free areas of undivided roads							
5	• Manoeuvre-free areas of divided roads							
6@	• Curves with radius ≤ 100 m		INVESTIGATION IS					
7#	• Roundabout and approaches		ADVISED					

Key to Thresholds below which Investigation is Advised

Roads with more than 2500 vehicles per lane per day

Roads with less than 2500 vehicles per lane per day

Notes:

a. # – Indicates Investigatory Level for Site Categories 1, 3 and 7 are based on the minimum of the four-point rolling average skid resistance from 50 m before to 20 m past the feature, or for 50 m approaching a roundabout.
b. Investigatory levels for Site Categories 2, 4, 5 and 6 are based on the minimum of the four-point rolling average skid resistance for each 100 m section.
c. The difference in Sideways Force Coefficient values between wheelpaths (Differential Friction Levels) should be less than 0.10 (or 10 for local format) where the speed limit is greater than 60 km/h; or less than 0.20 (or 20 for local format) where the speed limit is 60 km/h or less.
d. Curves with radius > 250 m and road sections with a gradient < 5% are to be considered as either Site Category 4 or 5.
e. @ – applicable only if the curve cannot be travelled at 50 km/h.

Source: Based on VicRoads (2018a).

Investigatory levels for site categories one and three are generally based on the minimum of the four-point rolling average skid resistance for the section from 50 m before to 20 m past the feature, or for 50 m approaching a roundabout.

There are numerous factors which contribute to a reduction in skid resistance or an increase in skidding accidents. These can be categorised as either surface texture defects or contributory shape defects as given in Table 6.3. These factors need to be considered during the investigation as they may influence the selection of a rehabilitation treatment.

Table 6.3: Factors contributing to reduced skid resistance

Texture defects	Shape defects
Flushing Stripping Polishing Aggregate shape or alignment Contaminants on road	Depressions Corrugations Crossfall

The initial step in evaluating the texture depth data is to compare the measured values against acceptable values set by the road agency, which may vary with the design speed and the function of the road.

6.4.5 Surface Texture

If surface texture deficiencies are identified, then the visual condition data may assist in identifying the cause(s) (refer to Appendix A).

The deficiency may also be related to the surfacing type being used. A guide to the typical relative texture depths of new surfaces is provided in Table 6.4 and Table 6.5. It should be noted that the texture depth of aged surfaces may be significantly influenced by traffic.

Table 6.4: Typical relative surface texture depths of new bituminous surfacings

Surfacing	Relative texture depth
Sprayed seals, 10 mm or larger	Greatest
Open-graded asphalt	
Ultra-thin asphalt	
Sprayed bituminous seals, 7 mm	
Stone mastic asphalt	
Microsurfacing	
Dense-graded asphalt	
Fine gap-graded asphalt	Least

Source: Austroads (2009c).

Table 6.5: Typical relative surface texture depths of new concrete surfacings

Concrete surface finish	Relative texture depth
Transverse tined	Greatest
Longitudinal hessian drag	Least

Source: Austroads (2009c).

6.5 Surface Deflections

6.5.1 Flexible Pavements

The structural adequacy of an existing thin bituminous surfaced granular pavement in terms of resistance to permanent deformation may be assessed by comparing the characteristic deflection to the design deflection (Figure 9.2). The design deflection is determined from the design traffic loading over the design period. If the characteristic deflection exceeds the design deflection, the pavement needs strengthening to inhibit deformation under anticipated future traffic. Section 9.2 describes the process to calculate the characteristic deflection.

For flexible pavements with bound layers (e.g. asphalt), the mechanistic-empirical procedure (MEP) for strengthening design (Section 10) may be utilised to assess structural adequacy.

Deflection data can provide significant information about the state of a pavement. For instance:

- Very high local deflections (more than 1.5 mm) may indicate weak subgrade conditions.
- High values of curvature function (D_0-D_{200}) may indicate low modulus in the upper pavement layers, or a pavement with cracked surfacing. For granular pavements with thin bituminous surfaces, the curvature function is likely to be 25% to 35% of the maximum deflection. Values higher than about 35% possibly indicate low modulus of the granular basecourse.
- Substantially different deflections and curvature functions between left and right wheelpaths may indicate the presence of a pavement widening, patches, water ingress from the adjacent verge or seasonal moisture effects.
- A high deflection peak near a pavement edge may be caused by poor local drainage such as a blocked subsurface drain.
- A series of high deflection peaks at a similar location in both wheelpaths may indicate a poorly backfilled culvert or service trench or a poorly-drained junction between two pavement types.
- A generally low but extremely variable deflection pattern may indicate an old, failing, bound pavement which may be cracked or poorly patched.
- A substantial (> 0.15 mm) positive residual Benkelman Beam deflection implies a weak pavement, probably due to poor compaction. A negative residual deflection may indicate shearing within the pavement but is more commonly associated with pavements incorporating cemented layers where the beam supports are within the deflection bowl. A well-defined area of low deflections with high positive residual Benkelman Beam deflections may indicate unstable pavement material.

As discussed in Section 6.4.3, by plotting the severity of rutting against maximum deflection it is possible to assess whether rutting is related to an inadequate structural capacity or densification or shear deformation of pavement layers.

Plots of maximum deflection against chainage, together with pavement composition data can assist in identifying changes in pavement composition.

6.5.2 Rigid Pavements

The evaluation of deflection data to determine the load transfer at joints involves the determination of the load transfer efficiency. Table 6.6 provides guidance in interpretation of load transfer across transverse contraction joints (NCHRP 2004). If complete load transfer exists, the ratio will be 100%, and if no load transfer exists, the ratio will be 0%. Poor load transfer may cause large increases in slab stresses and deflections, resulting in slab break-up and loss of serviceability.

Load transfer restoration should be considered for all transverse joints and cracks that exhibit measured deflection load transfer between 0% and 50%. This applies to jointed concrete pavements with or without existing asphalt surfacing.

Table 6.6: Load transfer efficiency rating

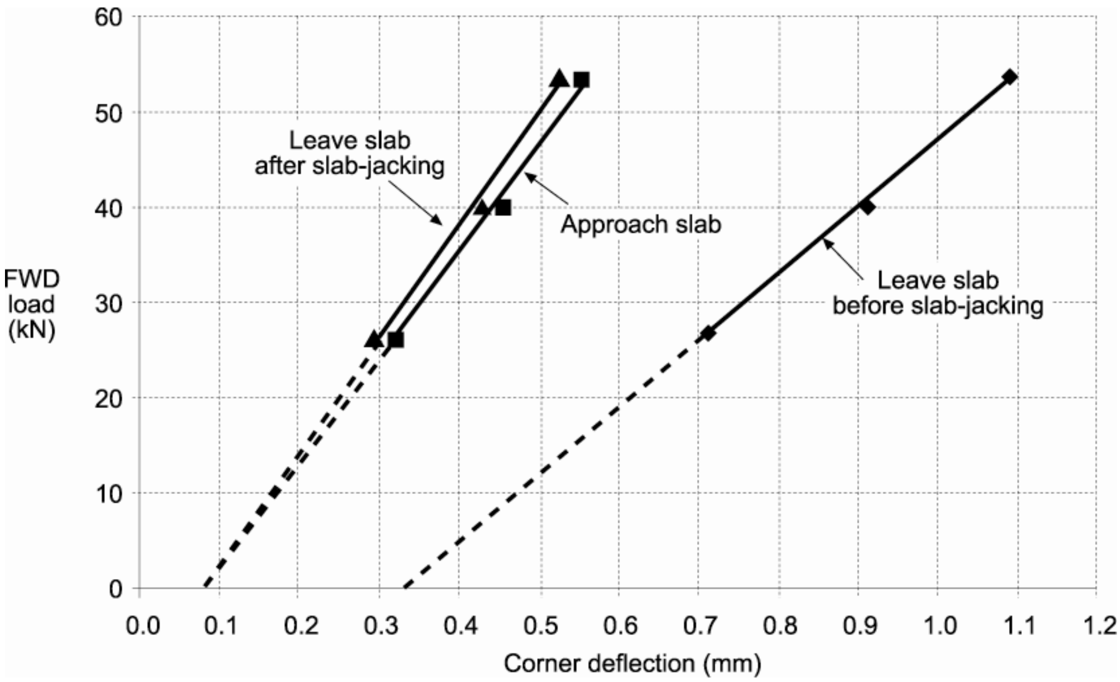
Load transfer class	Load transfer efficiency (%)
Excellent	90–100
Good	75–89
Fair	50–74
Poor	25–49
Very poor	0–24

Source: Adapted from NCHRP (2004).

Loss of support resulting in large deflections and stresses in the slabs is associated with substantial distress in jointed concrete pavements, including faulting, corner breaks, diagonal cracks and finally complete break-up of the slab. For continuously reinforced concrete pavements, the loss of support beneath a joint is a serious structural problem and leads rapidly to edge punch-outs.

To detect voids under slabs, corner deflections are measured using the FWD at three load levels (25 kN, 40 kN and 55 kN) to establish the load-deflection response for each test location (Figure 6.2).

Figure 6.2: Illustration of slab void detection



Source: American Association of State Highway and Transportation Officials (AASHTO) (1993).

Typically, locations with no voids have responses that cross the deflection axis near the origin (i.e. deflections at zero load, less than or equal to 0.05 mm).

A load-deflection response, which crosses the axis at points further removed from the origin, is indicative of a void beneath the slab.

Using this procedure, the percentage of joints with underlying voids may be computed and, thus, indicate the number of joints which will need treatment to restore structural adequacy.

6.6 Pavement Composition and Material Quality

6.6.1 Flexible Pavements

The different types of materials used in the construction of flexible pavements each exhibit characteristic modes of distress often linked to a deficiency or deficiencies in the quality of the material. The characteristic modes of distress and their possible causes are listed in Appendix A.

The interpretation of pavement distress modes will be greatly assisted by knowledge of the materials in the pavement and the characteristic distress modes associated with each of those materials.

Asphalt

In addition to the commonly examined characteristics of binder content, air voids and grading, the evaluation of distress in asphalt may need to address mix modulus, permeability, binder viscosity, aggregate degradation, stripping and delamination between asphalt layers. The characteristics of asphalt and their influence on its performance are discussed in detail in Part 4B of the *Guide to Pavement Technology* (Austroads 2014). Table 6.7 lists the expected service lives of different types of asphalt when used as a pavement surfacing.

Deformation of asphalt is a distress mode generally associated with inappropriate mix selection, mix components or mix design (excessive binder content) but may also be caused by inadequate compaction at construction (high air voids) or over-compaction of the mix under traffic (low air voids). The cracking and ravelling distress modes are also caused by inadequate mix design (insufficient binder or inadequate allowance for binder absorption into aggregate) but are often the result of oxidation/ageing of the binder and/or an underlying weak pavement.

Table 6.7: Typical service lives of surfacings

Surfacing type	Expected average service life[1] of treatments (years)	Comment on service lives
Sprayed seals 5 and 7 mm	5 to 7	Traffic volumes very important; initial treatments may have lower lives while reseal treatments on low trafficked roads can have long lives
Sprayed seals 10 mm and larger	8 to 15	Traffic volumes very important, also climate
Double application seals	8 to 15	Traffic volumes very important, also climate
Open-graded asphalt[2]	7 to 10 (standard binder) 10 to 15 (modified binder)	Traffic volumes very important, also climate
Thin open-graded asphalt[2]	7 to 10 (standard binder) 8 to 12 (modified binder)	Can be significantly influenced by turning traffic
Dense-graded asphalt	8 to 20	Traffic volumes important, also climate
Stone mastic asphalt, coarse gap-graded asphalt	10 to 20	Traffic volumes important, also climate
Fine gap-graded asphalt	15 to 25	Generally applicable to light traffic only
Slurry/microsurfacing	5 to 10	Traffic volumes and climate are important
Cape seal	8 to 15	Traffic volumes important, also climate
Concrete	30 to 40	Maintenance issues in concrete normally related to construction problems, joints and cracking

1 The service lives in this table are for average conditions and assume that the pavements are structurally sound. Service conditions which affect the expected life include:
** Traffic volume. High traffic volumes and high stress areas where there are braking and turning traffic will tend to give 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.*

2 Clogging or reduction in voids of an open-graded asphalt and thin open-graded asphalt may influence effective life.

Source: Adapted from Austroads (2009c).

Sprayed seal surfacing

The performance of sprayed bituminous seals depends principally on the rates at which the aggregate wears (or polishes) under traffic and the binder oxidises. The typical life of a sprayed seal surfacing is 5–15 years, although in some very stable climates, seal lives of up to 25 years are possible.

Flushing and stripping are often the result of inappropriate binder application rate either because of poor design or poor spraying practice. Flushing, however, can also be the result of sealing over an already flushed surface, sealing over patches, which have not had sufficient time to strengthen (routine maintenance should precede a reseal by at least two months), or of using excessive cutter in the binder. Stone penetration into the granular base is also a common source of flushing, hence flushing may be indicative of a base with inadequate strength, compaction and dry-back.

Controlling the moisture content of pavements during construction reduces the risk of damage to the pavement. Austroads Pavement Reference Group (APRG) Technical Note 13 (APRG 2003a) provides advice related to specifying, constructing and testing pavements.

Stripping can result from the use of dusty or poorly pre-coated aggregate, poor aggregate spread rate, insufficient cutter in the binder in cool or cold conditions, too much cutter in hot conditions or the onset of rain soon after sealing. Heavy traffic, if not anticipated at the design stage, can cause the aggregate to be punched into the underlying pavement or to be pulled from the pavement (stripping), particularly in turning or braking zones. Stripping can also occur where there has been excessive penetration of the binder into an underlying granular base course, which is particularly absorbent.

Cracking may be the result of some deficiency in the underlying pavement (shrinkage cracks in bound or plastic gravel bases, rutting in weak bases) or oxidation of the sealing binder.

Granular materials

As granular materials rely on aggregate interlock for their strength, their performance is dependent on the durability of the individual stone particles. Granular materials meeting road agency specifications generally have a service life exceeding 20 years. The quality and quantity of fines in the granular material also influence performance.

Characteristic modes of distress in granular materials typically include:

- rutting
- cracking in the overlying surfacing
- potholing
- roughness increases due to volume changes, either shear (load) or moisture-induced swelling or shrinkage.

All these distress modes are the result of poor-quality material, a high degree of saturation or poor compaction or a combination of these factors. Rutting can also result from high stresses induced by heavy traffic at braking and turning areas.

Commonly measured properties of granular materials sampled from test pits include in situ density and moisture content, plasticity and grading.

The characteristics of unbound granular materials and their influence on its performance are discussed in detail in Part 4A of the *Guide to Pavement Technology* (Austroads 2008c).

Stabilised materials

Stabilised materials are categorised as either modified or bound materials depending on the type and concentration of the binder (refer to Part 2 (Austroads 2018a) and Part 4D (Austroads 2019) of the *Guide to Pavement Technology*). Bound stabilised materials are those that set after mixing and compaction to exhibit significant tensile capacity. Bitumen emulsion-stabilised materials are typically defined as a modified material and foamed bitumen stabilised materials as bound after initial curing.

The life of these materials is heavily dependent on the pavement structure and traffic loading. Typically, they are designed for 20 or more years and may be used as a subbase or base layers (refer Austroads 2018a).

Modified granular materials are granular materials to which small quantities of stabilising agents 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 [cemented] material).

Evaluation of the construction quality of stabilised materials typically includes unconfined compressive strength (UCS) testing or modulus testing of cores extracted from the pavement.

6.6.2 Rigid Pavements

The causes and modes of distress of concrete base and subbase are discussed below.

Concrete base

Concrete base distress typically includes:

- cracking
- joint stepping or faulting
- scaling
- joint spalling
- joint sealant distress.

Cracking usually reflects a lack of thickness, strength or compaction in the base or poor joint layout but may be induced by construction practices such as the insertion of tie bars. Cracks are also often caused by shrinkage during the curing process or by thermal movements resulting from temperature differences through the base.

Dislocation on joints can result from erosion of the underlying subbase or from differential settlements in the subgrade. Consequently, the erosion resistance of the subbase material is an important consideration, particularly for heavily trafficked roads.

Scaling can also be caused by poor compaction or inadequate curing but may also be the result of using excessive retarder, excessive hand finishing or slurrying of the surface or delaying the placement of concrete over an existing slurried surface.

Deterioration in joint sealants will inevitably occur over time, so age is an important consideration when assessing sealant condition. Other factors include poor selection of the type of sealant or poor design of the sealant dimensions.

Due to their influence on several distress modes, the strength and level of compaction of concrete bases are key factors to be monitored in the design and construction of rigid pavements. While flexural strength is a fundamental engineering parameter for thickness design purposes (Austroads 2018a), it is often converted to an ‘equivalent’ compressive strength for quality assurance purposes because of the relative simplicity of compressive testing.

Consequently, field cores are commonly taken for compressive strength and relative density testing where low strength or low concrete compaction is suspected to be the source of pavement distress. The strength test results are compared to the strength levels specified in the works, with appropriate allowances for curing in the road-bed, and the shape and size of the cores. The density test results are divided by the reference density obtained on cylinders from the same lot at the time of construction and the ratios compared with the compaction levels specified in the works.

For details of quality requirements for concrete bases, see Austroads (2017b, 2018a).

Subbases

Subbases under concrete bases may consist of:

- lean-mix concrete
- bound materials – cemented materials and dense graded asphalt
- unbound granular materials, used for lightly trafficked pavements.

Therefore, the modes of distress in subbase materials depend on the nature of the material. The different distress modes for unbound and bound subbases are as previously described for granular and bound stabilised materials, respectively.

Evaluation of the quality of unbound granular materials typically includes moisture, particle size distribution, plasticity and compaction.

Evaluation of the quality of stabilised materials typically includes compressive strength and compaction testing.

For details of quality requirements for subbases, refer to Austroads (2017b, 2018a).

6.7 Subgrade Classification and Strength

The performance of a subgrade is measured by its ability to resist rutting, and, hence, depends on the stress level to which it is subjected. In turn, this is determined by the thickness and/or moduli of the overlying pavement layers and the intensity and loading of traffic passing over it. The provision of good formation drainage, uniform compaction and moisture content, are important to the durability of subgrades.

Subgrade distress is typically manifested as a loss of shape or rutting, but other distress modes include:

- cracking
- heaving and/or settlement.

Rutting reflects either an inadequate thickness or quality of the overlying pavement materials, which may be the result of inadequate design (e.g. overestimate of subgrade strength), poor construction or weakening of the subgrade during service, due predominantly to an ingress of moisture. Longitudinal cracking is commonly associated with expansive subgrades and relatively uniform edge heaving may also occur. Table 6.8 provides guidance on the expansive nature of subgrades.

Table 6.8: Guide to classification of expansive soils

Expansive nature	Liquid limit (%)	Plasticity index	PI x % < 0.425 mm	Potential swell (%)*
Very high	> 70	> 45	> 3200	> 5.0
High	> 70	> 45	2200–3200	2.5–5.0
Moderate	50–70	25–45	1200–2200	0.5–2.5
Low	< 50	< 25	< 1200	< 0.5

* *Swell at Optimum Moisture Content and 98% Maximum Dry Density using standard compactive effort; four-day soak. Based on 4.5 kg surcharge.*

Source: Austroads (2018a).

Commonly undertaken tests to evaluate subgrades include in situ strength, moisture, plasticity and grading.

6.8 Structural Adequacy of Original Design

Test pit data provides information about the thickness of each layer, the quality of materials and the strength of the subgrade. By using this data, together with the original design data (Section 3.2), the allowable traffic loading of the pavement can be estimated using the procedures for the design of new pavements (Austroads 2018a). This evaluation enables an assessment to be made as to whether the inadequacy of the original pavement design has contributed to the observed distress (Appendix A).

7. Selection of Treatments for Flexible Pavements

7.1 Introduction

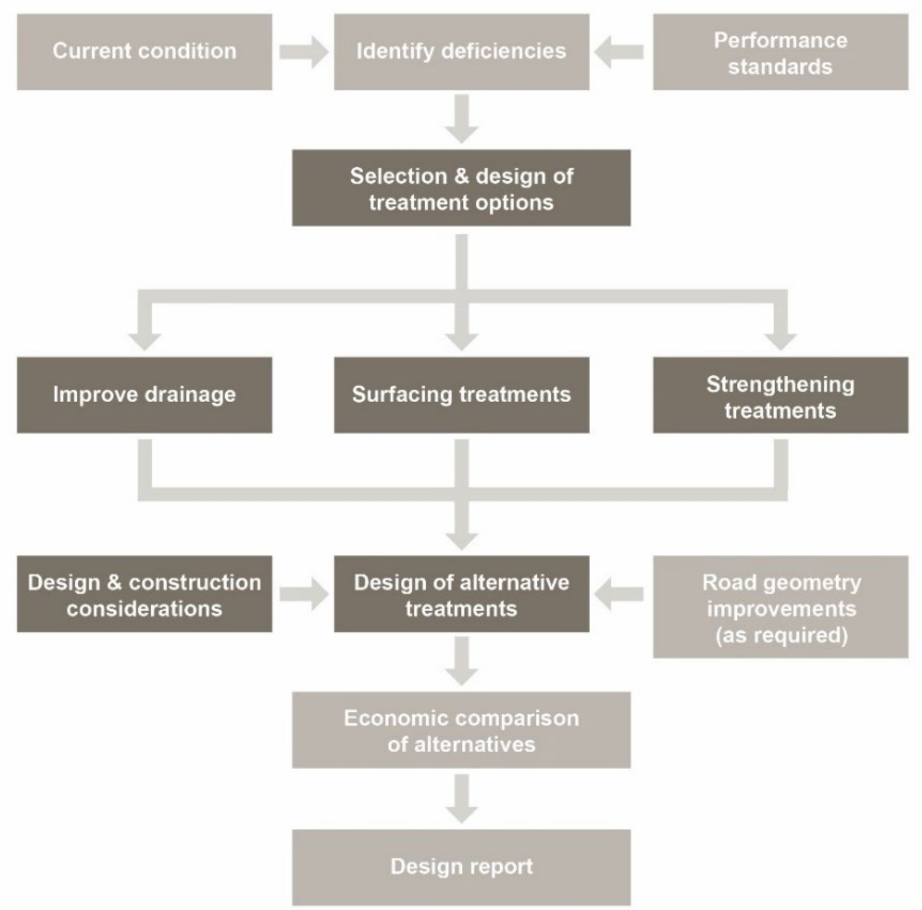
When an existing pavement no longer provides an acceptable level of service for the prevailing traffic and/or environmental conditions or when the current level of service is not deemed to be adequate for anticipated changes in those conditions, then some form of treatment is required.

Section 3, Section 4 and Section 5 of this Part discuss the methods of investigation to measure and collate data on the existing pavement condition and the process for evaluating this data to establish the structural and functional condition of the pavement.

As discussed in Section 6, one important reason for data collation and investigating the current pavement condition is to identify the causes and modes of distress and performance deficiencies.

This section provides descriptions of the rehabilitation treatments that may be used to address deficiencies in the pavement condition and highlights some of the design and construction issues associated with each of the treatments (Figure 7.1).

Figure 7.1: Design steps discussed in Section 7



7.2 Overview of Treatments Options

In the design of any rehabilitation treatment, the first and most important phase is the identification of the mode or modes of distress evident in the pavement and their associated cause or causes (Section 6). If the mode or cause of distress is incorrectly assessed, then it is very unlikely that the selected rehabilitation treatment will be effective, regardless of the quality of its design and construction. By comparing the current condition to road agency standards, the performance deficiencies can be identified. An appropriate rehabilitation treatment is one that addresses the cause of the pavement distress and deterioration and is effective in both repairing it and inhibiting its reoccurrence.

A considerable amount of analysis and engineering judgement is required when determining treatment options.

If a flexible pavement needs strengthening, the appropriate treatments include:

- structural (granular, asphalt or concrete) overlays
- in situ or plant-mixed stabilisation
- major patching
- reconstruction.

The thickness design of structural granular and asphalt overlays and stabilisation treatments are discussed in Section 9 and Section 10. Section 11 describes the procedure to design concrete overlays.

If the smoothness or shape of the pavement needs to be improved for reasons of serviceability only and not for structural reasons, the possible treatments include:

- removal and replacement of the existing surfacing and, possibly, some of the underlying pavement material
- asphalt surfacing, with or without underlying regulating or shape-correcting layers; typically, a single layer of asphalt achieves a 30% to 50% reduction in roughness
- for sprayed seal granular pavements, scarifying and reshaping with or without the addition of granular base material and resealing
- slurry surfacing
- combination of slurry surfacing and asphalt to improve smoothness and/or shape and a sprayed seal surface
- asphalt surfacing placed in conjunction with thick asphalt strengthening layers
- asphalt or sprayed seal surfacing in combination with in situ recycling of the existing pavement materials
- asphalt or sprayed seal surfacing in combination with granular overlays
- reconstruction of pavement.

If skid resistance needs to be improved, options include:

- asphalt surfacing, specifically open-graded asphalt (OGA) and stone mastic asphalt (SMA)
- sprayed seals
- slurry surfacing.

A more detailed discussion of treatments to improve surface characteristics including skid resistance, road-tyre interaction noise and surface spray is provided in Part 3 of the Guide (Austroads 2009c). Brief descriptions of the surface treatments mentioned above are given in Section 7.4.

It is emphasised that with any of these treatments, a check is required to ensure the pavement has sufficient structural capacity (refer to Section 9 and Section 10) to support the treatment under the anticipated traffic loading.

Consideration also needs to be given to the effect of design and construction constraints (refer to Section 7.7).

7.3 Treatments to Improve Drainage

7.3.1 General

Water on the road can cause problems for vehicles including aquaplaning, reduction in visibility due to spray, reduced availability of the roadway for vehicles and loss of stability for vehicles in floodways. Pedestrians and cyclists require that spray and water splashing from passing vehicles are minimised, velocities in gutters and channels are not excessive and that there are safe inlets to underground or outlet drainage facilities.

Where drainage of the pavement is impeded such that safety or the integrity of the pavement structure is compromised, swift corrective action is required. Rehabilitation of drainage systems may involve reshaping of the pavement surface by cold planing and reinstatement or by a corrective asphalt overlay to provide adequate pavement crossfalls. With sprayed seal granular pavements, scarifying and reshaping with or without additional granular base material is an option.

Side drainage defects, such as the blocking of entry pits by debris or a local collapse, can easily be repaired, but if left unattended, they can lead to major safety problems and/or weakening of the pavement structure.

In considering the design of any rehabilitation treatment, drainage and moisture control are of fundamental importance due to their effect on pavement condition and performance. Under well-controlled moisture conditions, most pavement structures and materials – even those which, in theory, might be barely adequate for the anticipated traffic – will perform quite adequately. These same structures and materials, however, can exhibit rapid deterioration if the pavements are subject to moisture build-up. Where poor drainage or moisture control is identified as contributing to the condition of the existing pavement, rehabilitation treatments must include steps to correct problems associated with both surface and subsurface drainage systems. Shoulder sealing is effective in inhibiting moisture ingress to the pavement and subgrade.

Table 7.1 lists some of the treatment options for deficiencies in drainage systems, which may affect moisture conditions in or beneath the pavement. These treatments, which can be categorised in terms of whether they relate to the surface or subsurface drainage systems, are detailed in the following sections.

Table 7.1: Drainage improvements

Condition	Treatment options
Shallow and/or silting table drains	De-silt or deepen drains
Blocked subsurface drainage	Remove the blockage or install a new bypass drain
Moisture ingress from elevated shoulders and medians	Install subsurface drainage at the edge of pavement closest to the point of moisture ingress Seal shoulders and place impermeable material in median
Impermeable aggregate in subsurface drainage layers	Remove and replace with free-draining material
Blockage of kerbed drain inlets	Remove the blockage – where the blockage is the result of an asphalt overlay, locally cold planing of the overlay Raise kerb and channel
Impermeable shoulders caused by boxed construction	Install subsurface drains at the pavement edge and through the impermeable shoulders
Water ponding in hollows in the pavement surface	Correct pavement shape
Infiltration from cuttings	Install subsurface and surface interceptor drains at the base of the cut slope Provide a drainage blanket under pavement
Infiltration through cracks and other surface discontinuities	Seal pavement
Accumulation of moisture in sag curves	Install subsurface drains at the low point and transverse subsoil drains on the grade leading to the low point
Moisture accumulation at changes of pavement type or thickness	Install subsurface drains along the junction to connect with edge drains

Note that where pavement widening is undertaken, consideration needs to be given to the compatibility of the materials proposed for the widening, in terms of both permeability and thickness, and those in the existing pavement. Where the materials used in the widening are not compatible with those in the existing pavement, moisture may accumulate at the junction of the materials causing surface deformation, cracking and potholing. In these cases, the installation of subsurface drains to provide a pathway for moisture to flow from the junction to the edge drains at the limits of the widened pavement provides an effective solution.

7.3.2 Surface Drainage System

A poorly functioning surface drainage system is typically evidenced by the accumulation of water on the pavement, shoulder or the adjacent verge. The causes, as outlined in Table 7.1, can be relatively mundane – for example, blockage of inlet pits or stormwater drains, undulations in the pavement surface – and have correspondingly straightforward treatments, e.g. removal of the blockage or overlay of the pavement to correct the surface shape, which do not require detailed discussion. Other causes can be more complex (such as sheet flow from adjacent land, inadequate capacity of the drainage system, flooding in local watercourses) and may require more complex and substantial treatments, although some may still be relatively simply treated such as de-silting table drains to restore their capacity or repairing catch drains.

The more complex treatments include the installation of interceptor drains to pick up inflow from adjacent lands, replacing existing stormwater drainage to provide additional capacity or raising the grade line of the road to bring it above the flood level. The need for and extent of any such work would depend on the frequency and severity at which water ponds on the surface. Austroads *Guide to Road Design Part 5: Drainage Design* (Austroads 2018e, 2018f, 2018g) provides information on the selection and design of effective surface drainage systems.

7.3.3 Subsurface Drainage System

In rehabilitation work, particularly in the wetter areas of Australasia and where roads are subject to heavy traffic, it may be necessary to provide subsurface drainage systems for the purposes of draining the subgrade and pavement materials and to intercept ground water before it reaches the pavement structure. In doing this, consideration must be given to the permeabilities and moisture characteristics of the pavement and subgrade materials, as the relative permeabilities within the road structure are a major factor in determining the preferred seepage paths and, hence, the effectiveness of the drainage system. For instance, installation of subsurface drains in fine-grained soils may be of little use in improving bearing value due to the low permeability of these materials.

Pavement deflection measurements and the condition of the existing pavement are useful in identifying where subsurface drainage may be required.

To function effectively, drains must always have a consistent grade control with a fall of, normally, not less than 0.5%, to allow efficient removal of water. Undulating drainage will not function correctly with accumulation of water in low sections.

Drainage is often installed in pavements after construction of the pavement structure in order to avoid damage to the drainage pipe (or other medium) by heavy construction plant. Temporary drainage may be required to prevent waterlogging and softening of pavement layers. Typical spacings of discharge points are 60–100 m.

Some different approaches to the control of seepage caused by surface infiltration and by capillary action from an underlying water table are illustrated in Figure 7.2 to Figure 7.5. As illustrated, longitudinal subsurface drains should be located within the pavement area, particularly where there is adjacent concrete kerb and channel, to ensure continuity of flow between the pavement layers and the subsurface drain.

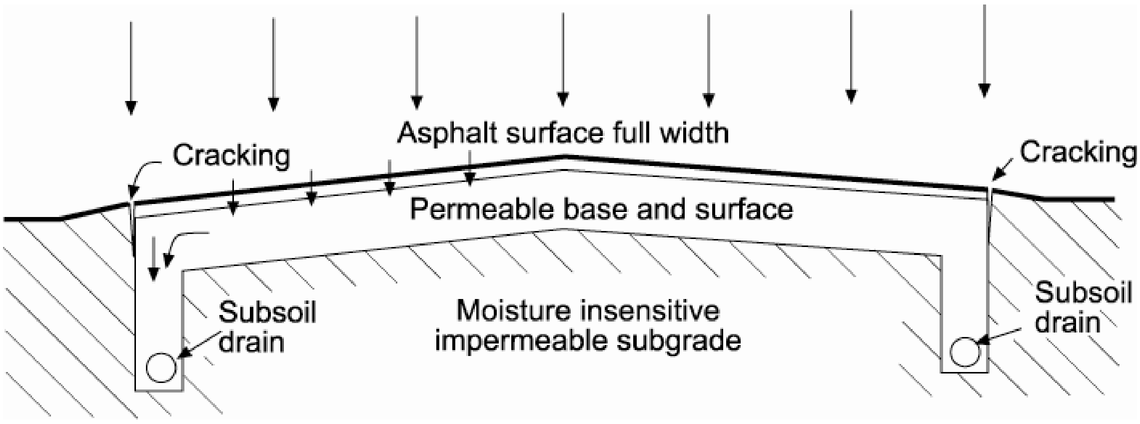
While subsurface drains are usually located along the outer edge of the pavement (Figure 7.2) in some cases, such as road widening or deep asphalt patching, drainage measures may be required within the trafficked area of the pavement to prevent excessive moisture increases.

Older pavements often have ineffective subsurface drainage. Hence, new drains may be required to remove or intercept moisture accessing the pavement.

The outlets of all existing subsurface drains must be checked and cleared of any blockages and treated so as to ensure they remain clear. If not already installed, consideration should be given to the installation of vermin guards. An example of such a guard is provided in VicRoads *Standard Drawings for Roadworks – Standard Drawing SD 1631* (VicRoads 1995b).

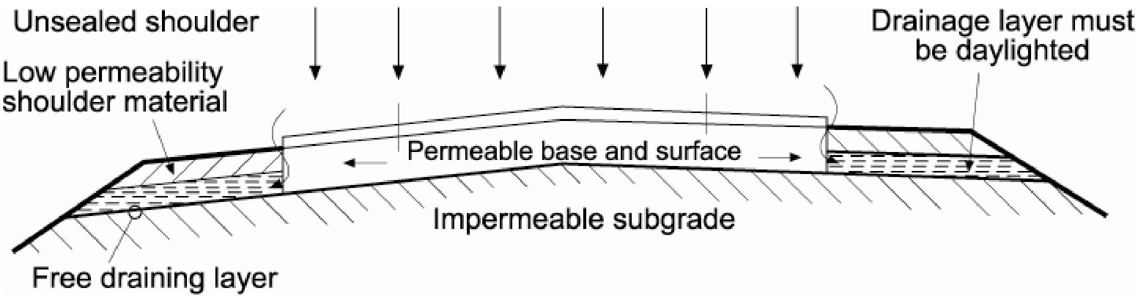
The selection and design of subsurface drainage systems are discussed in Part 10 of the *Guide to Pavement Technology Part 10: Subsurface Drainage* (Austroads 2009d).

Figure 7.2: Drainage for surface infiltration sealed shoulder



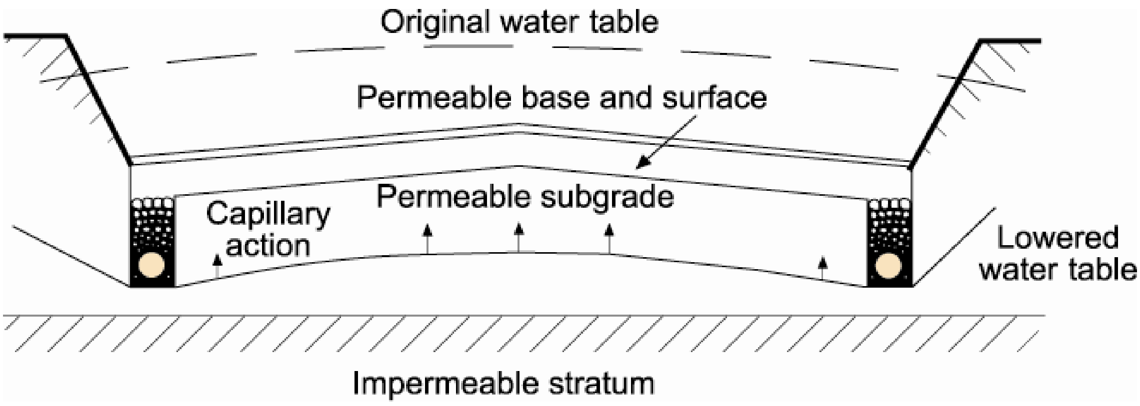
Source: Gerke (1987).

Figure 7.3: Drainage for surface infiltration unsealed shoulder



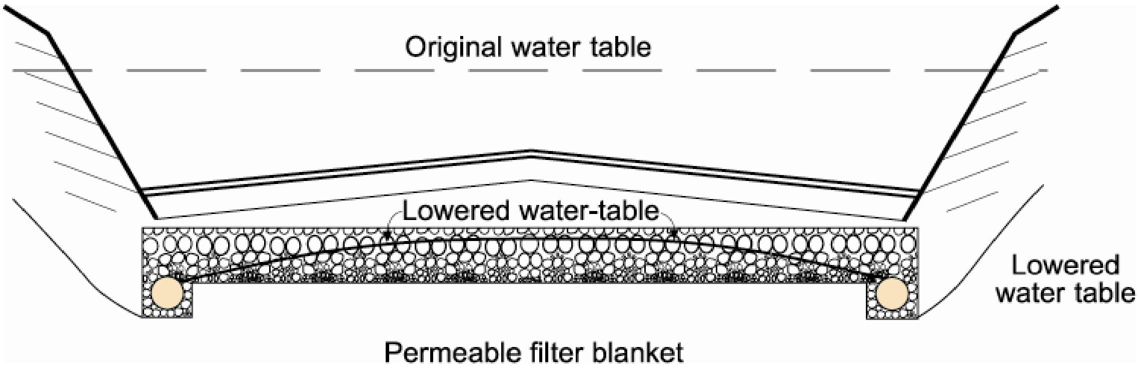
Source: Gerke (1987).

Figure 7.4: Drainage trenches to lower water table



Source: Gerke (1987).

Figure 7.5: Horizontal filter blanket to lower water table



Source: Gerke (1987).

7.3.4 Filter Layers

Whether the drainage system is a vertical trench or a horizontal blanket, it is common practice that either a granular filter or a synthetic geotextile filter fabric is an essential element of the drain.

Where granular filter layers are used, the grading of the filter material should meet the following requirements:

- It should be fine enough to prevent substantial migration of fine particles from the adjacent soil, which could contaminate and clog the drainage layer. This is typically controlled by limiting the ratio of D_{15} of the filter material to the D_{85} of the soil, where D_{15} and D_{85} are the sieve sizes through which 15% and 85% of particles, respectively, pass.
- It should be coarse enough to be more permeable than the adjacent soil so that it does not cause excess pore pressure to build up in the soil and desirably possess minimal capillarity.
- Filter and backfill materials must be carefully but adequately placed and compacted to avoid contamination and resist applied loads without excessive settlement (Gerke 1987). Where placed as a horizontal subbase layer, the filter material should be observed for rutting during compaction and under subsequent loading by construction traffic.

In recent years, granular filters have been largely replaced by artificial fabric filters, known as geotextiles. These materials are often more convenient to install, particularly where the pavement is being constructed over or within wet and/or soft soils and can operate successfully between fine-grained soils and relatively coarse-grained drainage material. Where geotextile filters are used, the material and class of geotextile should be checked against design requirements, specifically the Equivalent Opening Size should be compatible with the adjacent soil because fabric filters can become clogged in the same manner as granular filters. It is also desirable that the geotextile be placed against a smooth, fine-grained surface as sharp rock protrusions greater than 19 mm can damage the geotextile. If sharp protrusions cannot be avoided, then a heavier grade of geotextile should be used.

Geotextiles and filters are discussed in detail in Part 4G and Part 10 of the Guide (Austroads 2009e; Austroads 2009d).

7.3.5 Types of Pavement Drains

There are a number of different types of edge drain, most of which use one or more geosynthetic materials, generally as a filter medium. Subsurface drainage materials include: slotted and unslotted PVC subsoil pipe, pipe sock, filter material (aggregate), geotextile filters and drainage cell systems (or prefabricated geocomposite edge drains [PGEDs]).

Pipe drains

Three commonly used pipe-based systems are:

- perforated pipe underdrains (PPUDs), which consist of a perforated pipe backfilled with gravel, having no filter (geotextile or sand)
- geotextile-wrapped underdrains (GWUDs), which consist of a perforated plastic pipe backfilled with gravel and then a geotextile filter wrapped around the gravel
- geotextile-socked perforated pipes (GSPPs), which consist of a perforated pipe with a geotextile filter surrounding it, i.e. the pipe is 'socked' usually with sand as the backfill material.

Geocomposite edge drains

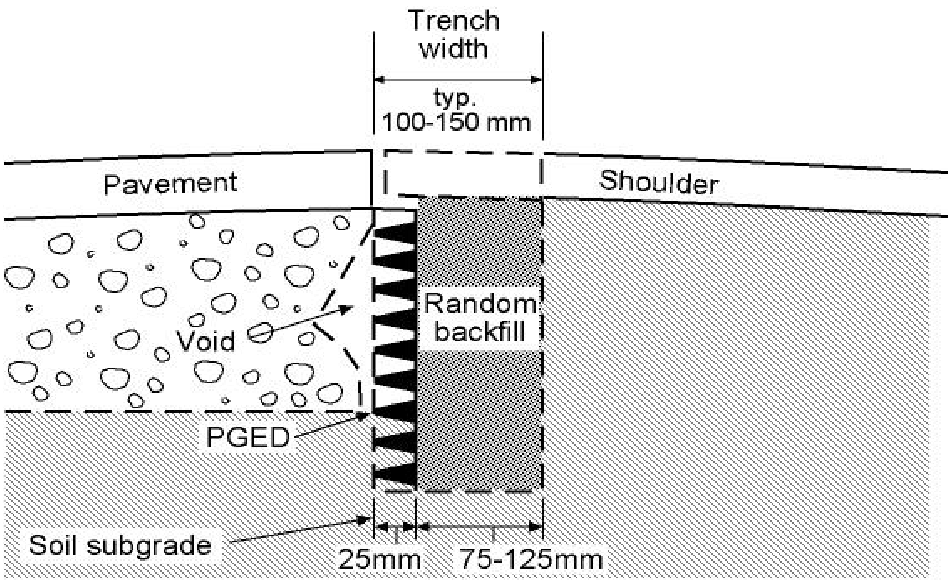
Apart from pipe drains, the fourth common type of edge drainage structure is the prefabricated geocomposite drain (PGED).

A PGED consists of a polymer core (often of an 'egg box' structure) enveloped by a geotextile filter, which is assembled in a factory and installed in the field as a completely manufactured unit. As these drains can be installed in a narrower trench or slot than the pipe-based systems, they can be more cost effective, particularly in retrofit situations.

However, PGEDs may clog due to intrusion of fines and to buckling during or following construction. Because it is almost impossible to clean a geocomposite once it becomes clogged, the presence of erodible fines and the potential for migration of the same should be investigated before a geocomposite (or any) edge drainage system is selected. As with other applications, the geotextile filter must be carefully selected to ensure that it is compatible with such conditions.

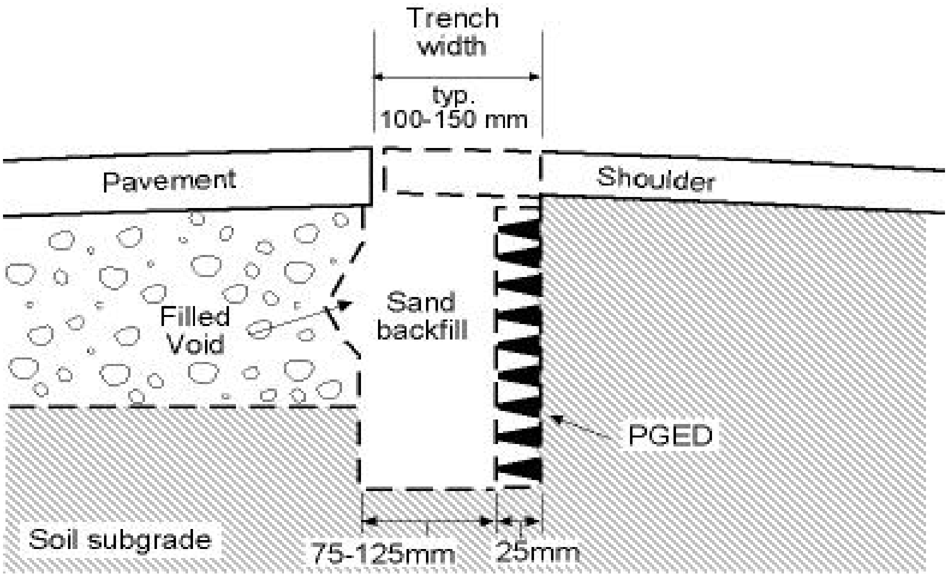
Another problem associated with buckling in geocomposites is the development of voids in granular pavement layers, resulting from localised cave-ins brought about by the loss of support for the trench wall from the geocomposite. Installation of a dense, sand backfill between the pavement and the drain, with the drain placed against the shoulder side of the trench, as shown in Figure 7.6 and Figure 7.7, has been found to reduce clogging and buckling. Adequate compaction of the trench back-fill material without damaging the drain is also a key performance factor. In loaded areas of the pavement, no-fines concrete is commonly used to provide a more deformation-resistant but permeable back-fill material.

Figure 7.6: Existing location of prefabricated geocomposite edge drains according to current placement methods



Source: Koerner et al. (1994).

Figure 7.7: Recommended location of prefabricated geocomposite edge drains according to current placement methods



Source: Koerner et al. (1994).

7.3.6 Design and Construction Issues

While there are numerous approaches to treating deficiencies in pavements caused by inadequate drainage, a feature common to most of them is the need for a considerable level of care during construction. Some of the important factors of which construction personnel should be aware include the following:

- A good pavement starts with a good foundation. A stable, smooth and free-draining platform is required, not only for construction of the pavement, but also to promote run-off of surface water resulting from wet weather during construction.
- There is potential for localised depressions resulting from poor subgrade preparation or the subsequent loading by construction equipment, particularly if soft areas exist in the subgrade, to pond water below the pavement surface and result in a loss of foundation support.
- Maintaining the quality of pavement materials, specifically their grading and plasticity, during construction as well as considering the potential for serious premature pavement distress that can result from contamination of permeable material by fines are important factors. Care is required to protect the permeable materials from contamination by fines from such sources as dirty equipment, adjacent backfilling operations or erosion sedimentation (Christopher & McGuffey,1997).
- There is significant risk that excessive compaction over already installed subsurface drainage, particularly edge drains, can cause distortion of and damage to the drains.
- Placement of unstabilised permeable base materials requires close control of the material grading and attention to any activities that might cause segregation.
- There is a need to avoid unnecessary movement of vehicles and plant on the placed material and minimise the speed of traffic and turning movements on the material to limit the risk of distortion and rutting in the finished surface.
- It is important to inspect and test the pavement drainage system to ensure its proper operation toward the end of construction.

Further details about construction and maintenance of subsurface drains are provided in Part 7 (Austroads 2009f) and Part 8 (Austroads 2009g) of the Guide, respectively.

7.4 Treatments for Surfacing Distress

7.4.1 Introduction

For flexible pavements with surfacing distress, this section describes the following treatments:

- a wide range of sprayed bituminous sealing treatments (Section 7.4.2)
- surface enrichment, rejuvenation, joint and crack sealing (Section 7.4.3)
- asphalt overlays and inlays after milling (Section 7.4.4)
- recycling of asphalt (Section 7.4.5).

7.4.2 Sprayed Seals

A number of options exist for the treatment of surface distress using sprayed seals. These options will not add to the overall strength of the pavement (except for an asphalt overlay which can add a measure of strength) but may slow the rate of deterioration through improving the pavement's resistance to water ingress or through preventing further break-up of the surface.

Table 7.2 provides a useful introduction to the application and performance of these treatments which are described in more detail below.

Table 7.2: Effect of sprayed seal, slurry surfacing and combined resurfacing treatments on existing surfacing characteristics

Property requiring improvement	Sprayed bituminous seal treatments				Slurry surfacing		Combined treatments	
	Surface enrichment	Single application sprayed seal (single/single)	Multiple application sprayed seal	Geotextile reinforced sprayed seal	Microsurfacing	Slurry seal	Correction or regulation course plus SAM/SAMI(2)	Correction or regulation course plus SAM/SAMI with asphalt surface
Bitumen ageing/ oxidation	Delays further oxidation							
Roughness	No effect				Some improvement, more with multiple layers		Good	Very good
Waterproofing properties	Reasonable	Good	Very good	Excellent	Minor improvement		Excellent	
Skid resistance	May reduce	Excellent			Fine texture good at low speeds but may reduce at high speeds		Excellent	As for asphalt
Structural strength	No effect					Minimal to no effect	Minimal but depends on thickness of asphalt layers	
Robustness (relating to sharp turning traffic)	No effect	Poor, but improved with modified binders	Some improvement over single coat seals due to interlocking of aggregate		Moderate		More robust if double application used	As for asphalt
Water spray reduction	No effect	May achieve some improvement depending on aggregate size			Minimal effect		Good	As for asphalt
Permeability of surface	Some reduction	Low			Moderate to high		Low	
Flexibility	No effect	Remains the same as for existing surface			Poor		Good	
Shape correction ability	No effect				Some improvement more with multiple layers		Good	Very good
Surface reflection cracking	Little effect	Good	Very good	Excellent	Poor		Excellent	Excellent
Likely life of treatment(1)	2 to 5 years	5 to 15 years	8 to 15 years	8 to 15 years	5 to 10 years		5 to 10 years	5 to 12 years

1 Depends on the condition of the existing surface and the structural condition of the pavement.

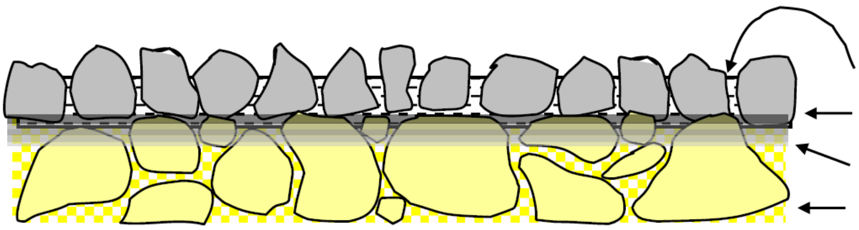
2 SAM is a strain alleviating membrane, SAMI is a strain alleviating membrane interlayer

Source: Austroads (2009c).

Sprayed bituminous seals

A sprayed seal consists of a thin coating of binder sprayed onto the surface of the underlying pavement and into which a layer of single-sized cover aggregate is spread and rolled (Figure 7.8). Used extensively on both lightly and heavily trafficked roads, a seal provides a hard-wearing waterproofing layer with good surface texture and skid resistance, which contributes significantly to the overall performance of the pavement.

Figure 7.8: Single/single seal



Source: Austroads (2018c).

All sealing works are defined as one of three broad categories:

1. initial treatment (an application of a prime or initial seal to a prepared basecourse)
2. secondary treatment (an application of a sprayed bituminous treatment on an initial treatment)
3. retreatment (an application of a sprayed bituminous treatment on an existing bituminous surfacing).

Resealing is the application of a seal to an existing bituminous surface to maintain or restore the integrity of the existing surface, restore surface texture and improve skid resistance.

While a seal or reseal commonly involves only one application of aggregate and binder, multiple applications of binder and aggregate may be used under some circumstances, such as pavements subject to heavy traffic and curves with adverse crossfall. The aggregate size should be adjusted according to the proposed use and texture of the existing surface – common sizes are 7 mm, 10 mm and 14 mm. The larger sizes provide a more robust surface but require higher applications of bitumen and hence involve greater cost. The binder used in a seal is typically cutback bitumen, although considerable use is now being made of polymer modified bitumen to provide a stronger, more resilient surface. In hotter regions, straight bitumen can be used in preference to a cutback bitumen while in the temperate regions there is some use of bitumen emulsion binder. For bitumen emulsions, the issue of 'breaking' (separation of bitumen and water) and 'curing' (evaporation of water) times must be considered. While breaking may occur relatively quickly, curing takes from 3 to 24 hours to reach a stage where the binder has sufficient strength to retain the aggregate under free-flowing traffic, i.e. without traffic control measures in place.

Types of sprayed bituminous seals include:

- prime
- initial seal
- single and multiple application seals and reseals
- sand seal
- surface enrichment/rejuvenation
- strain alleviating membrane (SAM)
- strain alleviating membrane interlayers (SAMI)
- geotextile reinforced seals (GRS)
- fibre reinforced seals (FRS)

For further details, refer to Part 3 and Part 4K of the Guide (Austroads 2009c, Austroads 2018c).

Appropriate uses

- Extend the service life of an existing sealed flexible pavement by reducing the surface ingress of air and water and, thus, protecting the underlying pavement and subgrade from water damage and slowing the rate of oxidation in the existing bituminous surface.
- Provide a wearing surface resistant to abrasion by vehicles.
- Correct surface deficiencies, such as block cracking or surface ravelling on pavements where deflections and curvatures are less than the design values and surface defects are due to oxidation and hardening of the binder.
- Restore skid resistance diminished due to polishing of the aggregate or loss of texture.
- Arrest further deterioration until an overlay or other rehabilitation measures are taken.

Inappropriate uses

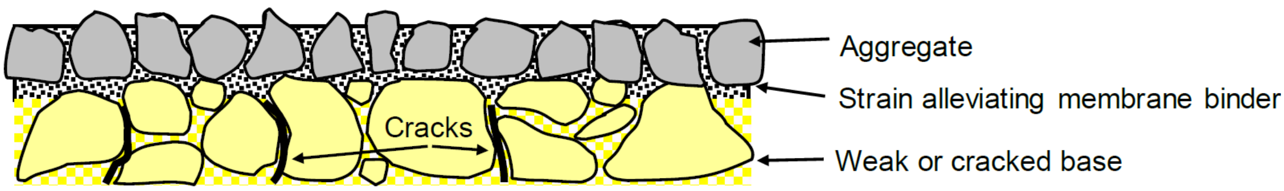
- Pavements requiring surface shape correction (e.g. to rectify rutting and roughness) or strengthening.
- Areas with turning or braking traffic, where a surface with high shear resistance is required.
- A sprayed seal with larger stone sizes (greater than 10 mm) in areas where traffic noise is not acceptable.

Strain alleviating membrane

A strain alleviating membrane (SAM) seal is a sprayed seal surfacing designed to eliminate or minimise reflective cracking in a distressed pavement by the use of a highly modified polymer modified binder (including crumb rubber).

The concept of a SAM seal (Figure 7.9) is to provide a relatively thick membrane of a robust binder that absorbs movement from a weak or cracked underlying layer. SAMs are generally not effective if used with smaller aggregates as the binder layer is too thin to effectively absorb the strain. The use of larger aggregates (10 mm or larger) is recommended.

Figure 7.9: Strain alleviating membrane



Source: Austroads (2018c).

Appropriate uses

- Inhibit reflective cracking in surfacings placed on cracked pavements or cement-stabilised bases.
- Minimise the loss of fines from underlying layers in pavements where pumping is occurring.
- Act as a holding seal prior to treatment by rehabilitation/reconstruction.

Inappropriate uses

- Beneath surfacings subjected to significant shear forces (e.g. small-radius roundabouts).
- As a long-term treatment for severely fatigued cement treated bases where the strength of the overall pavement is inadequate for the anticipated traffic loading.

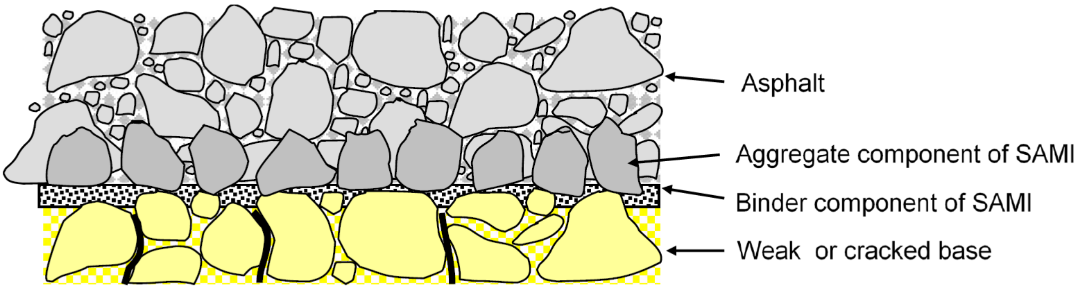
Further details are contained in Part 3 and Part 4K of the Guide (Austroads 2016; Austroads 2018c) and VicRoads Technical Notes No. 14 and 48 (VicRoads 2008; VicRoads 2006).

Strain alleviating membrane interlayer

A Strain Alleviating Membrane Interlayer (SAMI), as with a SAM, alleviates mechanical strains that occur in a road pavement; however, SAMIs are placed as an interlayer beneath asphalt layers (Figure 7.10). They are not intended to be used as a permanent wearing course and should be covered by asphalt within a few days.

SAMIs should only use an aggregate of size 10 mm or larger, applied at a light spread rate that is sufficient to carry construction vehicles to place the asphalt layer. The binder in a SAMI is usually heavier in application rate and more heavily modified than a SAM binder.

Figure 7.10: Strain alleviating membrane interlayer



Source: Austroads (2018c).

Appropriate uses

- Inhibit reflective cracking in surfacings placed on cracked pavements or cement-stabilised bases.
- Minimise the loss of fines from underlying layers in pavements where pumping is occurring.
- Waterproof thin asphalt-surfaced granular pavements lacking sufficient structural support to prevent premature cracking of a new asphalt overlay.

Inappropriate uses

- Beneath surfacings subjected to significant shear forces (e.g. small-radius roundabouts).
- As a long-term treatment for severely fatigued cement-stabilised bases where the strength of the overall pavement is inadequate for the imposed loading.

Further details are contained in Part 3 and Part 4K of the Guide (Austroads 2016; Austroads 2018c) and VicRoads Technical Notes No. 14 and 48 (VicRoads 2008; VicRoads 2006).

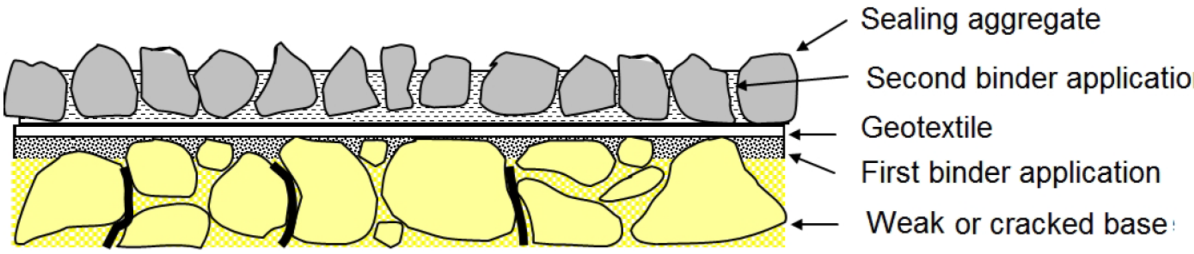
Geotextile reinforced seal

Geotextile reinforced seals (GRS) are produced by spraying a layer of bitumen onto a pavement (bond coat), then covering this bitumen with a layer of geotextile and lightly rolling to hold the geotextile in place. The application rate of the bond coat needs to be sufficient to ensure that the geotextile is fully impregnated with bitumen but not so heavy that there is free bitumen on the surface of the geotextile. Excessive bond coat application may result in pick-up by plant and vehicles used in the construction of the overlying sprayed seal or asphalt treatments. An added benefit of the heavier bond coat is a more substantial waterproofing membrane.

GRS can be used to provide more robust waterproofing, and as a SAM or SAMI treatment, and may be considered the most effective technique when treating badly cracked and distressed bound and unbound pavements. A double/double seal is typically applied over the geotextile (Figure 7.11) if it is intended to be a SAM wearing course, with single/single seal generally only used for SAMI applications.

GRS are more sensitive to weather conditions during and several weeks after construction, and as such they should be programmed to allow trafficking in warm weather.

Figure 7.11: Geotextile reinforced seal



Source: Austroads (2018c).

Geotextiles are produced using one or sometimes two types of polymer in the form of long filaments. They are either woven or needle-punched into sheets, typically 3.5 m to 3.7 m wide, and delivered in rolls. Non-woven needle-punched fabrics are preferred over woven fabrics as they have more uniform elongation, better resistance to tearing and superior bitumen/fabric adhesion. Further guidance on geotextiles is provided in Austroads (2009e).

Geotextiles are now widely used in bituminous surfacings and other pavement applications because of the benefits they provide, which include:

- delaying the onset of reflective cracking (e.g. for new sprayed seals on cement-treated base pavements)
- reducing the loss of fines due to pumping
- reinforcing asphalt, sprayed seals and reseals (e.g. geotextile reinforced seal on clay pavements)
- relieving tensile strains at the bottom of surface course layers.

Appropriate uses

- Inhibit reflective cracking in surfacings placed on cracked pavements or cement-stabilised bases.
- Minimise the loss of fines from underlying layers in pavements where pumping is occurring.
- Waterproof thin asphalt-surfaced granular pavements lacking sufficient structural support to prevent premature cracking of a new asphalt overlay.

Inappropriate uses

- Beneath surfacings subjected to significant shear forces (e.g. small-radius roundabouts, steep grades).
- Where the underlying pavement is unsound or poorly drained.
- As a long-term treatment for severely-fatigued cement-treated bases where the strength of the overall pavement is inadequate for the imposed loading.

Fibre reinforced seal (FRS)

A fibre reinforced seal (FRS) uses an emulsion polymer modified binder and chopped glass fibre as reinforcement. The process uses a purpose-built sprayer which, in a single pass:

- sprays binder onto the pavement
- cuts the required amount of fibre glass to length, generally 60 mm, and blows this onto the first layer of binder
- sprays a second layer over the cut fibres.

The bitumen and fibre layers are immediately covered with an aggregate which is locked into place using an aggregate scatter coat.

FRS can be used as a SAM or a SAMI and can be expected to have enhanced performance compared to treatments that use with polymer modified binder (PMB) alone.

Microsurfacing

Microsurfacing is a form of slurry surfacing, in which the conventional bitumen emulsion used in a slurry seal is replaced by a polymer modified emulsion and a larger aggregate size can be used. The stronger binder and larger aggregate provide a more robust mix, which is capable of being spread in variably thick layers for rut filling and correction courses. A polymer modified cationic, rapid-setting emulsion slurry is preferred for heavily trafficked areas, because the rapid curing characteristics of the binder normally allow the road to be reopened to traffic within 30–45 minutes of laying.

Microsurfacing is able to address a number of pavement maintenance requirements that cannot be readily achieved by sprayed seals or asphalt, e.g. minor shape correction while matching into existing levels. Larger sized mixes may be designed to achieve shape correction in excess of 20 mm in multiple layers. The mixes can usually be applied with prior milling or tack coating.

Microsurfacings do not provide any structural strength. Where used on pavements with high deflections, a microsurfacing will crack early in its life.

Higher performance attributes can be derived through the inclusion of additives such as fibres or emerging technologies. These additives may improve flexibility or strength of material in the applied microsurfacing and are generally proprietary.

A microsurfacing should not be considered as a treatment to prevent crack reflection and is likely to reflect existing cracks within months of placement. If it is used on a cracked pavement, it is suggested that another treatment, such as SAM seals or GRS, be placed first to mitigate crack reflection. A microsurfacing can then be placed to provide the benefits described above.

Various nominal sizes of microsurfacing are specified based on the nominal largest aggregate size. The nominal aggregate sizes typically used in Australasia are as follows for typical applications (the numerical designation refers to millimetres):

- Size 4 and size 5 microsurfacing are in common use in Australia for local government residential resurfacing works, airfield and shared pathways.
- Size 7 microsurfacing is predominately used by state road agencies for shape and correction courses, or as a final wearing course for lower speed roads.
- Size 10 microsurfacing is used for rut or shape correction or on sites where higher final texture depth is required.

Table 7.3 provides further guidance for rut filling and correction.

Table 7.3: Rut filling and correction

Nominal size	Sizes 4 & 5	Size 7	Size 10
Void filling (e.g. cape seal)	✓		
For rutting 10–15 mm deep	✓	✓	
For rutting 15–25 mm deep		✓(see note below)	✓
For rutting 25–40 mm deep		✓(see note below)	✓(see note below)

Note: Application in multiple layers is suggested where the depth of a rut exceeds 2–2.5 times the nominal size of the mix.

Source: Austroads (2018d).

The Austroads *Guidelines and Specifications for Microsurfacing* (Austroads 2018d) details uses and applications of microsurfacing. Mix design, plant, field application, sampling and testing are all included along with a model specification.

Appropriate uses

- As a non-structural wearing course on stiff, strong pavements, i.e. pavements exhibiting low deflections and curvatures.
- Where a surface texture similar to that of dense-graded asphalt is desired at a lower cost.
- Where loose aggregate would be a problem (pedestrian areas, bicycle paths).
- To improve low speed skid resistance (microtexture).
- To reduce road noise compared to a sprayed seal.
- To correct minor surface irregularities (Table 7.3).
- To resurface a stripped/worn seal, or an oxidised/ravelled asphalt surface.

Inappropriate uses

- On pavements with inadequate structural capacity (i.e. high deflections) or where resistance to reflective cracking is required.
- On moderately and severely rutted pavements with rut depths in excess of 15 mm and where subject to heavy traffic.
- On surfaces, which have been rejuvenated or on primerseals incorporating a cutback binder, where those treatments were completed within the previous 12 months.

7.4.3 Holding Actions

Surface enrichment and rejuvenation

Surface enrichment of a sprayed seal surface involves the spraying of a light application of a low viscosity grade of bituminous material (cutback bitumen or bitumen emulsion) or foamed bitumen onto the surface so that it runs into the voids of the existing surfacing. This treatment increases the amount of binder in the layer, but care must be taken to ensure that adequate surface texture remains. This treatment extends the life of the surfacing by ensuring the retention of the existing cover aggregate. Surface enrichment is generally applicable to low traffic sites such as shoulders and rest areas and may also assist in waterproofing the surface.

A rejuvenating treatment is the application of a proprietary rejuvenating agent, usually in the form of an emulsion. Rejuvenation is used to replace the lost oils and resins in oxidised bitumen. Rejuvenation materials have a lower viscosity than the bitumen materials used in surface enrichment. They are particularly applicable to asphalt pavements for reducing permeability and delaying the onset of ravelling through ageing and oxidation of bitumen binders.

Enrichment and rejuvenation treatments are normally only used on roads in essentially good condition apart from aged binder where there is a risk of aggregate loss. They are generally only practical where traffic volumes are low, and traffic can be diverted onto another lane or the road shoulders. Traffic should not be allowed onto the treated surface until the binder has cured sufficiently to avoid pick-up. In some cases, a light coating of sand or grit can be used to reduce the time before trafficking.

Surface enrichment and rejuvenation can result in reduced skid resistance through a residue of surface binder. Traffic speed restrictions should remain in place until this residue has worn off and the skid resistance levels rise to acceptable levels.

Appropriate uses

- To provide an economical method of extending the life of a sprayed seal where there are sufficient surface texture voids to accommodate the enrichment binder.
- To restore the aggregate retention properties of the binder in sprayed seals which are exhibiting minor aggregate loss or asphalts exhibiting a loss of fine aggregate.
- To restore binder that has deteriorated due to oxidation and hardening before the aggregate has worn to the stage where a reseal is necessary.

Inappropriate uses

- May be inappropriate to use emulsion on heavily trafficked roads due to the time required for the bitumen emulsion to fully break.
- May be unsuitable for use on impermeable seals.

Joint and crack sealing

Hot-poured, rubberised bitumen products have been used for many years for crack filling. More recent advances with polymer modified products have led to their acceptance as premium crack filler materials. In this application, flexibility is critical, as the material must be able to move to accommodate the movement associated with the opening and closing of the crack without undue distress. This requires the correct balance between adhesion of the filler to the walls of the crack and cohesion of the filler to be maintained. Elastomeric polymer modified fillers have been found to fulfil these requirements.

While the process can successfully seal cracks, it does not eliminate them. A study by Transport SA (1999) has shown that the effectiveness of fillers is limited to a few years after which the cracks will require refilling.

On expansive subgrades subject to frequent volume changes, a rubber sealant may be the best option.

Additional details are provided in APRG/Australian Asphalt Pavement Association (AAPA) Work Tips Nos. 8 and 9 (APRG/AAPA 1988 and 2010).

Appropriate uses

- As a temporary treatment to prevent infiltration of water into the pavement.
- Sealing of widely spaced block cracking and longitudinal or transverse cracking prior to reseal or overlay.
- Inhibiting the loss of pavement materials through pumping.

Inappropriate uses

- Areas with extensive crocodile cracking.
- Closely spaced block cracks, unless care is taken to remove excess sealant from the pavement surface – excess sealant on the surface can create small ponds in wet weather which can increase the risk of aquaplaning.
- Overbanding is not suitable for areas where skid resistance may be required in lane changing movements by motor cyclists. In such areas, routing and sealing of cracks is the preferred method of crack sealing.

7.4.4 Asphalt Work

Introduction

In this section various surfacing treatments using asphalt are described. These treatments include asphalt overlays and inlays using various types of asphalt.

Table 7.4 provides a useful introduction to these surfacing treatments. Note that these asphalt treatments may be used in conjunction with SAM, SAMI and GRS seal treatments described in Section 7.4.2.

Table 7.4: Effect of asphalt resurfacing treatment in existing characteristics

Parameter requiring improvement	Asphalt treatment				
	Dense-graded asphalt (DGA)	Fine gap-graded asphalt (FGGA)	Stone mastic asphalt (SMA)	Open-graded asphalt (OGA)	Thin open-graded asphalt (TOGA)
Bitumen ageing/oxidation(1)	Covers oxidised surface			Covers oxidised surface, requires a seal or heavy tack coat on existing surface to minimise moisture infiltration into the pavement	
Roughness	All asphalt treatments reduce surface roughness. Improvements to the shape of existing surfaces may require additional use of correction/regulation layers (generally using dense-graded asphalt)				
Waterproofing properties	Good if compacted adequately and the layer is thick enough			Surfacing is permeable but usually combined with heavy tack coat or seal for waterproofing	
Skid resistance	Good at low speeds; reduces as speed increases	Suitable for low speeds only	Good, particularly for high speed freeways		
Structural strength	Improves strength depending on layer thickness	Some minor improvement but normally a surfacing only	Improves strength depending on layer thickness	Minimal	None
Robustness/shear resistance (relating to sharp turning traffic)	Excellent	Fair	Excellent	Generally fair to poor. Improves when PMBs are used	
Water spray	Poor	Very poor	Good	Excellent	Very good
Permeability of surface	Low	Low	Low	The surface is designed to be permeable	
Flexibility (strain tolerance)	Relatively stiff but influenced by binder type	Greater flexibility than dense graded mixes	Relatively flexible	Flexible	
Surface reflection cracking	Limited ability to resist reflection cracking in thin layers		Some ability to resist reflection cracking	Fair ability to resist reflection cracking	Some ability to resist reflection cracking
Likely life of treatment	8 to 20 years	15 to 25 years	10 to 20 years	7 to 15 years	7 to 12 years

1 Hot-in-place recycling can also be an option for aged asphalt surfaces.

Source: Austroads (2009c).

Asphalt overlay

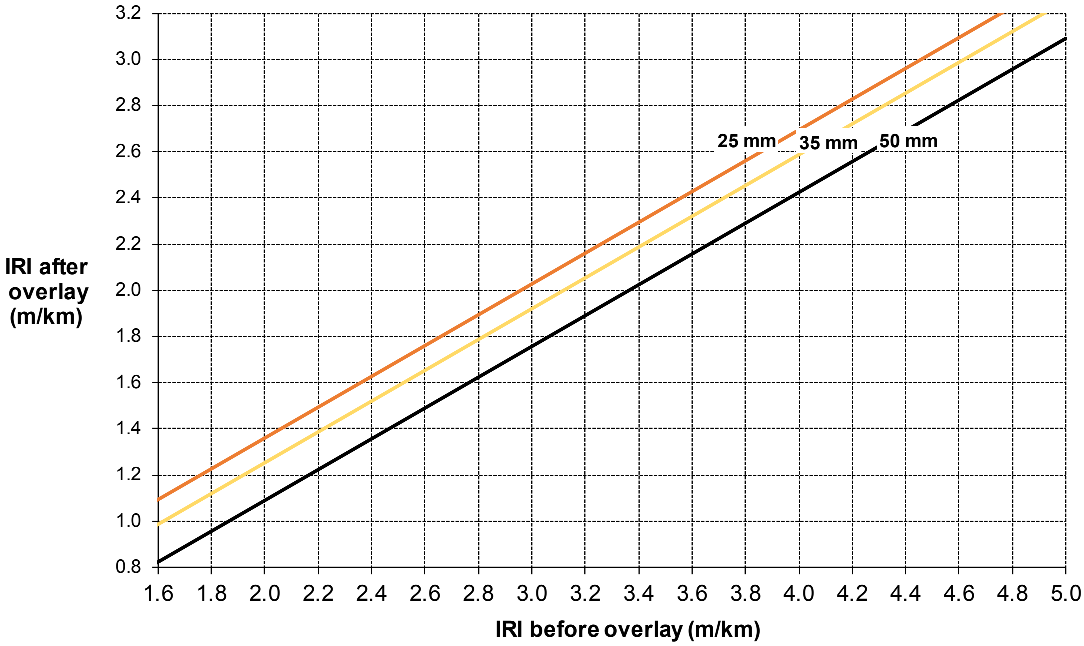
An asphalt overlay is an application of a layer of asphalt to an existing pavement surface. While overlays are commonly used to strengthen distressed pavements, they are also placed to remedy surface deficiencies such as shape, texture or general deterioration, e.g. roughness. Asphalt overlays are also used for pavements where sprayed seals are not desirable due to high traffic volumes, a predominance of turning or braking traffic, high road/tyre noise or loose aggregate.

Prior to placement of an overlay, initial treatments to address other forms of distress (e.g. crack sealing, patching), cold planing of the existing surface to improve shape or ensure a match with adjacent fixed levels and adjustment to manholes and drainage structures and patching, may be required. In addition, preparation for an asphalt overlay will generally require repairs to any localised base failures, construction of pavement widenings (if any) and, if significant improvement in roughness is required, construction of a correction course.

In respect of general surface deterioration, performance relationships in road agency asset management systems may be used to assess the thickness of overlay required to reduce the existing roughness to the desired level. Figure 7.12 is an example of one such relationship.

It is inappropriate, however, to design and implement an overlay project without a proper survey of the existing pavement conditions. If an overlay was to be constructed on a weak or very flexible pavement then any improvement in surface condition could be compromised by premature structural distress of the overlay. For this reason, it is recommended that any proposed asphalt overlay be analysed to ensure sufficient structural capacity exists (Section 10).

Figure 7.12: Example of the effect of asphalt overlay thickness on roughness



Source: Based on VicRoads (1998).

An appropriately designed and constructed structural asphalt overlay can be expected to last up to 10–20 years provided support conditions do not deteriorate. The in-service performance of asphalt layers depends on the traffic, environmental and drainage conditions.

Appropriate uses

On pavements which exhibit any of the following:

- inadequate structural capacity for current or expected future traffic loading
- excessive traffic noise
- unacceptable roughness
- unacceptable structural distress
- unacceptable level of safety (skid resistance).

Inappropriate uses

- Over unstable bases/subbases e.g. substandard material or excessive moisture.
- Where the finished pavement surface is required to match the levels of adjacent surfaces such as kerbs, structures etc, unless it is combined with cold planing.
- For light-duty non-structural asphalt overlays, refer to APRG Technical Note No. 4 (APRG 1997).

Dense-graded asphalt

Dense-graded asphalt (DGA) is the most commonly used asphalt type.

DGA mixes have a continuous distribution of aggregate size and filler (i.e. evenly distributed from coarse to fine) and a low design air voids generally in the range of 3 to 7%.

Table 7.5 indicates commonly used mix sizes and their typical uses.

Table 7.5: Selection of dense-graded asphalt mixes

Nominal size (mm)	Typical layer thickness (mm)	Typical use
7	20 to 30	Commonly used for surfacing residential streets and pedestrian areas where thin layers and fine surface texture is required
10	25 to 40	General purpose wearing course mix suitable for both light and moderate traffic applications
14	35 to 55	Wearing course mix for heavier traffic applications; also, intermediate course to suit layer thickness
20	50 to 80	General purpose base and intermediate course mix for wide range of uses

Source: Adapted from Austroads (2009c and 2014).

On more heavily trafficked pavements it is important that the asphalt does not flush, deform or fatigue under the action of traffic. Resistance to flushing and deformation is improved through the use of coarser gradings and stiffer binders. Polymer modified binders can be used to enhance both rutting resistance and fatigue properties (Table 10.4).

Further information is available in Austroads (2009c and 2014).

Stone mastic asphalt

Stone mastic asphalt (SMA) is a gap-graded mix with a high proportion of coarse aggregate. It provides an interlocking stone-on-stone skeleton that resists permanent deformation. The coarse aggregate skeleton is filled with a mastic of binder, filler and fine aggregate.

SMA has a high filler and bitumen content. The coarse aggregate skeleton must be able to contain all the mastic binder while maintaining the stone-to-stone contact essential for rut resistance. Too much mastic will result in flushing, bleeding and loss of pavement shear resistance. Too little mastic will result in high air voids, increased permeability and reduced pavement durability.

Small percentages, typically 0.3% by mass, of mineral or cellulose fibres are commonly used to minimise the risk of binder drain down during transport and placing. Polymer modified binders (PMBs) may also be used to further reduce risk of drain down and bleeding under severe performance conditions or to enhance the flexural performance of the mix. SMA is inherently more flexible and provides better fatigue life than a dense-graded asphalt due to the higher binder content.

Although the air void content is similar to a dense-graded asphalt, a compacted SMA mix has a surface texture appearance approaching that of open-graded asphalt and provides noise attenuation and surface texture properties between those of dense-graded asphalt and open-graded asphalt. The lower air voids, when compared with open-graded asphalt, make SMA a more durable mix.

Generally, SMA is used as a surfacing material in nominal size 7 mm, 10 mm and 14 mm mixes in Australia, although the use of 14 mm nominal size has diminished in line with European trends to smaller size SMA mixes. The smaller sizes provide lower surface noise levels, reduced risk of segregation and enables reduced layer thicknesses while retaining a well-textured surface.

A sprayed seal is provided under the SMA where there is concern about the ingress of surface water to the pavement.

Appropriate uses

- Strengthen roads and/or improve shape.
- Provide a more robust wearing course for roads subject to heavy traffic and high stress areas (e.g. intersections).
- Provide enhanced texture depth.
- Provide a more flexible and durable wearing course for light duty pavements (e.g. minor residential streets, cycleways, footpaths, etc.).

Inappropriate use

- Over unstable bases/subbases i.e. substandard material or where there are drainage problems.
- Further information is provided in Austroads (2014).

Open-graded asphalt

Open-graded asphalt (OGA) surfacing is recognised as having superior noise-reducing and spray-reducing properties compared to other types of surfacing, as well as reducing the risk of aquaplaning and affording good adhesion at high speeds. For these reasons, it is used as a surfacing on urban highways and freeways and heavily trafficked arterials with abutting noise-sensitive properties.

OGA is a coarse, gap-graded asphalt mix commonly manufactured with a maximum size aggregate of 10 mm or 14 mm (occasionally 20 mm) and with typically 18–23% air voids after compaction. Layer thicknesses in Australia and New Zealand generally range from 25–40 mm.

As the service life of an OGA made using conventional Class 170 bitumen is about five to eight years, it is now common to use a PMB to achieve a longer life and to enhance structural integrity and stone retention. In particular, PMBs have been found to be effective in delaying the premature closing up of the stone matrix under traffic with the commensurate loss of noise and spray reduction properties. Where the pavement is sound and provides a stiff support for the OGA, a polymer modified binder OGA may last up to 12 years or more.

An important consideration when incorporating an OGA surfacing is the egress of water from the layer; it is essential that the layer drains freely, otherwise it will flood and its effectiveness in lowering the risk of aquaplaning and reducing spray and noise generation will be significantly compromised. In urban roads with concrete kerb and channel the practice is to construct the underlying pavement to match the level of the channel and then place the OGA so that it will drain into the channel. On roads with sealed shoulders, it is preferable to construct the shoulder pavement flush with the adjacent road pavement and then place the OGA as an overlay. The lateral extent of the OGA layer is limited so that the edge of the layer drains onto the surface of the shoulder pavement. The concern with both practices is that they result in a sudden change in level at the edge of the OGA, which can present a hazard to cyclists and pedestrians.

The drainage capacity of an OGA surfacing can also be adversely affected by patches or trench reinstatements, which are typically constructed with dense-graded asphalt. The use of dense-graded asphalt for these applications is a result of the relatively small areas involved, which require the asphalt to be placed by hand, and hand working of OGA is not recommended. Water flowing within the OGA layer will pond at the upstream side of these patches due to the lower permeability of dense-graded asphalt and the effectiveness of the surfacing will be reduced.

Another important consideration is the progressive clogging of the voids with detritus, such as airborne dust as well as the grime and contaminants carried on vehicle tyres. In relatively dry climates, clogging due to airborne dust is a particular problem, on which cleaning with high-pressure sprays have had only limited effect. Some equipment developments (Milne 2001) might provide a more effective solution to this problem.

Since an OGA layer is not waterproof, it is prudent when laying it on a pervious surface, such as a larger aggregate size asphalt mix (e.g. size 20 mm, size 28 mm), an existing cracked asphalt layer or a layer of granular material, to incorporate a waterproofing layer beneath the OGA. One option is to apply a SAMI seal (without cutter) beneath the OGA (for night work it may not be possible to spray the modified binder effectively in the lower ambient temperatures). Where a SAMI is used, some problems may be encountered with later maintenance or rehabilitation when attempting to mill and resheet with OGA without also having to replace the SAMI. An alternative is to surface the pavement with a smaller size, dense-graded asphalt (e.g. size 10, size 14) initially and preferably allow it to be subjected to traffic for a period, to close up the surface voids, prior to placing the OGA.

Use of OGA on concrete pavements

In many cases, it is necessary to place OGA on concrete pavements and bridge decks. In such cases it is common practice to:

- clean the surface and spray a very light (highly cutback) prime
- apply a SAMI
- place a 25 mm thickness of size 10 mm dense-graded asphalt followed by an OGA wearing course.

The dense-graded asphalt layer beneath the OGA is a sacrificial layer, required to protect the underlying concrete from damage when the OGA is milled and replaced at the end of its service life. In this case, a milling machine with fine milling drum is recommended.

Where OGA is to be applied on a concrete surface, special attention is needed to ensure proper adhesion between the OGA and the concrete surface. This includes the use of a special curing compound (for new concrete surfaces only) and a special primer (for old concrete surfaces) to eliminate the action of the dust and to provide adequate bonding between the concrete surface and the subsequent layer of bituminous surfacing. A SAMI may also be used, in addition to the prime, to aid bonding of asphalt.

While there are numerous approaches to treating a concrete surface prior to placing the OGA, the appropriate treatments should include:

- hessian drag finish on concrete (no transverse texturing is required)
- the curing membrane must be either bitumen emulsion or hydrocarbon/bitumen emulsion blend, compatible with the subsequent bituminous surfacing
- a quick-drying prime (QDP) at 0.04 to 0.06 L/m^2 on a concrete surface that does not have the necessary curing membrane
- a 10 mm seal designed as a reseal midway between a normal seal and a SAM seal, using a surface texture allowance of +0.2 L/m^2
- a minimum of 25 mm of dense-graded asphalt, followed by a minimum 30 mm of a size 10 mm OGA to allow future replacement of open graded asphalt without reducing structural concrete thickness.

Refer to Part 4B of the Guide (Austroads 2014) and VicRoads Technical Note No. 4 (VicRoads 2004).

Appropriate uses

- On major urban roads to achieve reduction in noise and spray generation.
- On freeways to reduce noise and spray generation and to reduce the risk of aquaplaning and afford good adhesion at high speeds.

Inappropriate uses

- On tight curvilinear alignments where, with frequent crossfall/superelevation transitions, the capacity of the OGA to drain freely in wet weather can be adversely affected with consequential effects on vehicle handling.
- At or near intersections where heavy braking and turning of vehicles impose shear forces on the pavement and oil leaks from stationary vehicles can cause softening of the binder.
- On porous (i.e. size 20 mm or greater) asphalt bases without an underlying waterproof layer.

Ultra-thin asphalt surfacings

The term ultra-thin asphalt (UTA) is generally applicable to any asphalt surfacing designed to be placed in layers with thicknesses of 20 mm or less. In practice, the terms are taken as associated with particular mix types that have been specifically designed as thin surface retreatments to restore surface characteristics with a minimum thickness of asphalt.

These types of asphalt mixes originated in Europe, generally as proprietary products, and are usually variations of OGA or SMA mix types. Most of these mixes are designed to provide good texture depth and low noise characteristics for use on major roads.

UTA surfacings have been developed as a means of restoring surface characteristics of otherwise sound pavements using the shape correction and surface texture characteristics of asphalt while minimising the thickness of the surfacing. These thin, flexible surfacings are more economical and slightly more tolerant of surface deflections than thicker asphalt layers and do not significantly raise levels relative to adjoining surfaces.

Most of these UTA mixes are permeable and can lack the independent integrity associated with thicker layers.

Thin OGA mix types generally comprise a more graded product than standard OGA, as well as a PMB to ensure adequate resistance to heavy traffic shearing forces. They also require a heavy application of a bitumen emulsion PMB tack coat for waterproofing and adequate bond to the underlying surface which, in turn, involves the use of a modified asphalt paver to place the binder layer immediately ahead of the asphalt. The surface characteristics of thin OGA are similar to normal OGA but without the same level of water spray reduction due to the reduced porosity.

Appropriate uses

- For minor shape correction of structurally sound pavements.
- On cracked pavements in combination with a geotextile SAMI treatment.
- To obtain a moderate reduction in road-tyre noise and spray generation.

Inappropriate uses

- Areas subject to significant shear forces (e.g. small-radius roundabouts).
- On pavements requiring strengthening.
- Further details are available in Austroads (2009c).

Cold planing

Cold planing (milling) is a cost-efficient process of accurately and rapidly removing a specified thickness of an irregular pavement surface caused by distress such as rutting, shoving, surface corrugation or surface cracking. Most commonly, it is used to remove asphalt surfacing, but modern equipment can now mill heavily bound cementitious layers and plain concrete, although the millings from these materials tend to be much finer and not as suited to recycling as millings from asphalt layers.

The cold planing process is normally carried out with a self-propelled machine, equipped with a horizontal drum on which are mounted an array of cutting teeth which rotate on their mountings to equalise the wear across each tooth. Moving on wheels or crawler tracks with a variable forward speed, the machine passes over the layer to be removed and the drum is lowered to pulverise the material to the selected depth, which can be up to 300 mm with heavy-duty machines. A levelling system governs the cold planing depth with reference to a standard height such as a kerb. Manual depth control is also possible. The even nature of the vertical milled edges allows for flush cold planing and ensures a good bond between new and old material. Refer to Austroads (2009f) for additional information.

The milled material is loaded by conveyor to a truck and transported off site. In the case of asphalt millings, these are commonly transported to the mixing plant, where they can be reprocessed into new asphalt mixes.

Cold planing can be used in conjunction with other repair techniques, such as the application of an asphalt overlay. In this application, the process provides an alternative to a correction course by removing the high points on the surface so that a more uniform overlay thickness can be achieved and, hence, a better level of surface finish in terms of shape and ride quality.

It can also be used to remove asphalt adjacent to kerb and channel so that the subsequent overlay can be matched to the level of the channel. In this application, it is normal to vary the depth of planing from a maximum equal to the required thickness of the overlay, at the face of the channel, to zero at some distance from the channel. That distance should be at least 1 m if a reasonable thickness of overlay is to be maintained across the pavement. This is particularly important in residential streets where modern garbage collection vehicles follow a very channelised path that results in the outer wheel being within 1 m of the kerb. If a narrower width of planing is adopted, then the overlay thickness at a point coinciding with the outer wheelpath can be reduced considerably.

The other major design decision to be made is the depth to which the existing pavement material will be planed, which is dependent upon the purpose. If the purpose is to improve skid resistance or drainage, the removal depth should be minimised. If the operation is performed in conjunction with an overlay, the depth of cold planing should be based on the soundness of the existing asphalt and required final depth and surface level of pavement as determined by the design process.

As noted previously, the presence of a geotextile in the pavement can cause problems as the fabric may clog the cutting teeth or become entangled around the cutting drum.

Refer to APRG/AAPA Work Tip No. 5 (APRG/AAPA 1997).

Appropriate uses

- Restore surface shape.
- Remove distressed surface material.
- In conjunction with an asphalt overlay, prevent loss of kerb and channel capacity.
- Improve surface drainage.

Inappropriate use

- As a stand-alone treatment where there is a risk of ravelling or delamination of the profiled surface.

7.4.5 Recycling of Asphalt

Plant mixed recycling

This process firstly involves the removal of asphalt surfacing material from a pavement by means of either cold planing or heating and planing. After removal, the reclaimed asphalt is reheated and mixed with virgin aggregate and binder and/or a rejuvenating agent at the batch plant to achieve the specified requirements for the new mix. It may then be replaced on the same pavement or elsewhere using a conventional paving operation.

Hot in-place asphalt recycling (HIPAR)

The HIPAR process is an in situ surface rehabilitation process undertaken using a train of equipment, which, in a single pass:

- heats and mills the surface
- mixes the millings with additional aggregate, binder and rejuvenating agent, as necessary, to form a new mix
- lays and compacts the mix back onto the pavement.

The process can be used to recycle asphalt to a depth of about 50 mm or can be used to increase the overall thickness of asphalt by reworking part of the existing surface and adding sufficient additional material to enable replacing a thicker layer.

The benefits of the HIPAR process over conventional mill and paving operations are cost savings, conservation of resources and reduced traffic delays as a result of the one pass operation. The heating operation, however, consumes significant quantities of gas. The success of the process is very dependent on the quality of information available about the existing pavement, including the properties and thickness of the existing asphalt and the presence of moisture in the pavement.

Therefore, it is essential that a detailed and extensive investigation of the existing asphalt be made as part of the selection and design of this treatment. The success of the process also depends on the prevailing weather conditions, which to achieve optimum results should be hot and dry. The process should not commence unless the air temperature exceeds 10 °C and is rising.

The properties of the existing asphalt (binder type and quality, aggregate grading, bulk density, air voids) as well as any rejuvenating agent used also influence the material properties of the recycled asphalt mix. Rejuvenating agents are usually light, high penetration oils of low viscosity, which are usually proprietary products but may also consist of blends of bitumen, emulsion or modified bitumen.

Appropriate uses

- Rehabilitation of substantial lengths of major roads which are structurally sound and free of drainage problems.
- Restoration of surface shape, texture and skid resistance.
- Rejuvenation of oxidised binders in asphalt by the addition of a rejuvenating agent and/or new binder.
- Rehabilitation of asphalt layers that have problems caused by poor mix design or poor construction practices.
- Addition of a thin layer of asphalt to a pavement by the addition of new materials.

Inappropriate uses

- Rehabilitation of light-duty pavements which may be damaged by the loads imposed by the recycling train and narrow roads and roads having small radius bends within which the train cannot be easily manoeuvred.
- Treating reflective cracking in asphalt overlying a bound cementitious layer.
- On asphalt containing tars, high contents of rubber and some types of PMBs or geotextiles, where there is extensive use of crack sealants or where the asphalt has been treated with a sprayed seal.

For further details refer to Part 4E of the Guide (Austroads 2009h).

Cold in-place asphalt recycling

Cold in-place asphalt recycling is a process undertaken at ambient temperatures in which part or all of an existing asphalt pavement is milled and the millings mixed with a rejuvenating/stabilising agent in a pugmill before being relayed and compacted on the pavement. The plant used for the process can be a single recycler unit or a recycling train, which travels on the roadway and provides a homogeneous process throughout the cold planing and mixing stages. The depth of milling is limited to a maximum of 100 mm.

The rejuvenating agents used are proprietary agents, which act to restore binder viscosity or bitumen emulsion or a combination of both. Where water is introduced with the rejuvenating agent (e.g. bitumen emulsion) it is critical to the subsequent performance of the material that the moisture be eliminated during the compaction and curing stages.

The recycled material exhibits physical properties which lie between hot mix asphalt and bituminous stabilised material (Austroads 2009h). It is generally applicable to roads with low to moderate traffic volumes. For optimum performance, it is important that the material exhibits strong adhesion. Where an emulsion is used, there will be a curing period as the emulsion breaks during which the material will gain strength. Therefore, it is recommended that the material not be opened to traffic for at least 1.5 to 2 hours to enable the material to gain sufficient strength to withstand trafficking.

Most cold mixes have relatively open grading and high air voids in order to facilitate the drainage of water during the curing period. For this reason, they are susceptible to ravelling and should be sealed or overlaid with asphalt within a few weeks of construction.

As with HIPAR, the success of the process is dependent on the quality of information about the properties and thickness of the existing materials, so a thorough investigation should be undertaken prior to the selection of the treatment.

For further details refer to Part 4E of the Guide (Austroads 2009h).

Appropriate use

- Rehabilitation of lightly to moderately trafficked roads.

Inappropriate use

- On pavements that require significant strengthening, if not followed by a structural overlay.
- To achieve a high-quality finish and a high ride-quality level. (Note: improvements in the performance of emulsions and developments with rejuvenating agents could lead to its use on roads with much higher traffic loads.)

Diamond grinding

While diamond grinding is usually specified for concrete pavements, it may also be applicable for aged asphalt pavements surface where the binder is sufficiently hard to not allow the narrow and short fins to collapse. The default blade spacing of 2.5 mm detailed in Roads and Maritime Services NSW specification R93 (Roads and Maritime 2014) is applicable for dense-graded asphalt. There is no experience in Australia for the application of diamond grinding on SMA and OGA asphalt surfacing.

7.5 Treatments for Strengthening Pavements

7.5.1 Introduction

For pavements identified as needing strengthening, this section describes the following treatments:

- heavy patching
- asphalt overlay
- granular overlay
- concrete overlay
- chemical and mechanical stabilisation of pavement layers and subgrade.

An empirical method to determine the thicknesses of granular overlays is presented in Section 9, whilst a mechanistic-empirical procedure of determining the thicknesses of all the above strengthening options is presented in Section 10.

7.5.2 Heavy Patching

Heavy patching, in contrast to light-duty maintenance patching, is the process of carrying out deep or full-depth repairs of localised, severely distressed sections of pavement. The depth of patching usually exceeds 75 mm. The most common locations for such repairs are the slow lane of heavily trafficked roads and at signalised intersections and roundabouts where axle and shear loads are heaviest. When relatively permeable pavements are patched, subsurface drains may need to be constructed around heavy patches to prevent the accumulation of excessive moisture.

Heavy patching is a common rehabilitation option in busy urban settings, particularly where permitted lane closure times are often limited, and work is frequently carried out at night. Patching is used with asphalt overlays to provide a long-term treatment; however, many heavy patching treatments tend to be sub-optimal in terms of structural design and efficiency but are used to temporarily reinstate serviceability.

For pavement sections subject to heavier loadings and/or shear stresses, increased resistance to rutting and fatigue cracking may be obtained by considering the various types of polymer modified binders available (refer to Austroads 2009c and 2014).

General surfacing dense-graded asphalt has a maximum size of either 10 mm or 14 mm to provide the best waterproofing for underlying layers. Larger nominal size mixes (20 mm and over) tend to be much more porous and hence, will not waterproof the surface. They will, however, have better structural capacity and be more cost-effective in lower layers. The normal convention for the ratio of layer thickness to nominal mix size applies equally to heavy patching (i.e. thickness 2.5 to 4 times nominal mix size).

Appropriate uses

- Localised severe distress or failures where full reconstruction is not economically viable.
- Where asphalt surfacing fails by rutting or shoving and/or has undergone extensive cracking leading to surface break up.
- As an emergency treatment to sustain a pavement through cold and wetter months.
- As an alternative to strengthening where level constraints prevent a structural overlay.

Inappropriate use

- Pavements with OGA surfacings are generally not suitable for patch repairs as the drainage properties tend to be disrupted at joints. An OGA may, however, be added to the whole pavement surface as a surface course at a later date.

7.5.3 Asphalt Overlay

In addition to the use of asphalt as a surfacings treatment, asphalt overlays are used to strengthen pavements.

Section 10 provides guidance on procedures to calculate the thickness of structural asphalt overlays.

7.5.4 Granular Overlay

Granular overlays are used to improve both the serviceability and structural capacity of a pavement.

Construction of a granular overlay or resheet is a means of enhancing the ride quality and performance and extending the life of roads generally subject to lower traffic levels. Granular overlays are placed over granular pavements to improve pavement strength, to inhibit excessive rutting of the pavement due to subgrade deformation and/or existing pavement material breakdown or to improve pavement shape. Generally, a granular overlay is placed when the serviceability of an existing pavement has become intolerable for the road user.

Material for the overlay should comply with the appropriate specification for pavement base material. Often, in rural areas particularly, where local knowledge and experience or availability of materials and cost considerations prevail, granular overlays are placed as nominal 100 mm to 150 mm layers without a formal design process. For best results, however, it is advisable to determine the thickness of granular overlays based upon deflection data (Section 9) and/or Figure 8.4 and Figure 12.2 of Austroads (2018a).

Prior to placement of the granular material on the existing sealed surface it may be necessary to scarify the seal to avoid trapping moisture at the base of the overlay. The need to scarify the seal depends on the environment (e.g. arid or wet), the shape of the existing surface and the permeability of the various pavement layers. Localised patching, reshaping and drainage improvements may also be required.

Two methods of thickness design of granular overlays for flexible pavements are provided:

- Section 9 describes the empirical method based on measured maximum deflections.
- Section 10 describes the mechanistic-empirical procedure, which may include use of measured deflection bowls.

Appropriate use

- Where there is no restriction on level control from roadside fixtures, overhead structures or to maintain access to adjacent properties.

Inappropriate use

- May be difficult to place under traffic.

7.5.5 Concrete Overlay

Concrete overlays are used to improve both the serviceability and significantly improve the structural capacity of a pavement.

Section 11 provides guidance on procedures to calculate the thickness of structural concrete overlays. The subbase under the overlay needs to comply with the minimum requirement for the design of new rigid pavements (Austroads 2018a). Critical issues in this regard are:

- whether the existing pavement needs be milled to remove surface distress or for shape correction
- the uniformity of support to the overlay considering the variability of pavement deflections
- identifying local areas of weak pavement that need patching to provide more uniform support to the overlay
- the fatigue damage to any existing bound materials due to past traffic
- the suitability of any existing bound materials to form the subbase or part thereof.

Appropriate use

- Where there is no restriction on level control from roadside fixtures, overhead structures or to maintain access to adjacent properties.

Inappropriate use

- May be difficult to place under traffic.

7.5.6 In situ Stabilisation of Granular Pavements

General

Stabilisation is the process of improving a material to achieve a long-term increase in its load bearing properties. A flexible pavement is a layered system in which each of the layers fulfils a specific function in providing the long-term support of traffic loading. To significantly change the properties of one layer without consideration of the likely effect or compatibility with other layers can result in failure of the process. The properties of each pavement material and the subgrade, as well as the moisture regime in and around the pavement, must be considered.

Stabilisation by modification of a material is preferred when the quality of a material, not its thickness, is deficient. Modified granular materials are granular materials to which small amounts of stabilising binders 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 [cemented] material). Modified granular materials are considered to behave as unbound granular materials, i.e. they do not develop tensile strain under load (Austroads 2018a).

Stabilising the base will not solve the problem of a weak subgrade but can help reduce moisture ingress through the surface or shoulders by lowering the permeability of the base.

The main methods of stabilisation used are described in Table 7.6, which also gives broad guidance to selecting a stabilisation type.

Table 7.6: Guide to selecting a method of stabilisation

Particle size	More than 25% passing 75 µm sieve			Less than 25% passing 75 µm sieve		
Plasticity index (PI)	**PI ≤ 10**	**10 < PI < 20**	**PI ≥ 20**	**PI ≤ 6 & PI x %passing 75 µm ≤ 60**	**PI ≤ 10**	**PI > 10**
Binder type						
Cement and cementitious blends(1,3)	Usually suitable	Doubtful	Usually unsuitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually unsuitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful	Usually unsuitable	Usually suitable	Usually suitable	Usually unsuitable
Bitumen/ lime blends	Usually suitable	Doubtful	Usually unsuitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually unsuitable	Usually unsuitable	Usually suitable	Usually suitable	Doubtful
Dry powder polymers	Usually suitable	Usually suitable	Usually unsuitable	Usually suitable	Usually suitable	Usually unsuitable
Other proprietary chemical products(2)	Usually unsuitable	Usually suitable	Usually suitable	Usually unsuitable	Doubtful	Usually suitable

1 *The use of some chemical binders as a supplementary addition can extend the effectiveness of cementitious binders in finer soils and soils with higher plasticity.*

2 *Should be taken as a broad guideline only. Refer to trade literature for further information.*

3 *TMR uses triple blend and have a method based on % passing 0.425 mm sieve and linear shrinkage (Volker & Hill 2016).*

Source: Austroads (2019a).

The mix design of stabilised pavement materials is detailed in Austroads (2019a).

7.5.7 Granular (Mechanical) Stabilisation

Granular stabilisation is the process of blending various construction materials to produce one material which has the desired properties. The most common form of granular stabilisation is the addition of coarser material to a finer material to achieve an improved particle size distribution (PSD) and plasticity. Combining different materials in this manner should be based upon careful calculation and laboratory or field blending tests to establish blending ratios.

Appropriate uses

- Improving the performance of granular materials with moderate or high plasticity and/or poor grading in sealed or unsealed pavements.
- Improving granular material in shoulder pavements to enable the application of a seal.

Inappropriate uses

- In pavements where the increased permeability of the stabilised material is incompatible with that of abutting materials and hence, likely to contribute to subsequent moisture-induced distress.
- In pavements overlying wet or weak subgrades.

7.5.8 Cement and Cementitious Stabilisation

Cement and cementitious stabilisation bonds together particles in both a physical and chemical reaction. It can be effectively applied to most soil types although limitations occur with high plasticity soils.

The term ‘cementitious’ is used to cover a range of binders which contain a pozzolanic additive which is a siliceous or alumino siliceous material, that in finely divided form and in the presence of moisture, chemically reacts at ordinary room temperatures with calcium hydroxide released by the hydration of Portland cement or lime to form compounds possessing cementitious products. Pozzolanic additives include fly ash and iron and steel slags, which may be combined with lime or cement to form cementitious binders. Most cementitious stabilisation work in Australia and New Zealand is carried out by the in situ process.

Cementitious materials can be used as an aid to construction where the addition of a small amount (about 1–2%) may provide some cohesion in a material otherwise lacking in plastic fines or increase the Optimum Moisture Content to allow for a wider moisture range for compaction.

The use of various cementitious additives can be cost-effective. These additives include ground granulated blast furnace slag, fly ash and hydrated lime, which prolong setting times and reduce the propensity for shrinkage cracking in the stabilised material. The latter property can be useful in reducing reflective cracking in thin asphalt surfacing constructed over a stabilised base.

For lightly trafficked roads, the depth of stabilisation is generally limited to between 150 mm and 250 mm.

For highway pavements carrying higher levels of traffic, pavement recycling in a single layer exceeding 300 mm has been used through the development of ‘deep lift’ recyclers, specialised binder spreaders, slow-setting binders and high-performance compaction equipment. In this application, a minimum binder content of 4–5% is often used.

The ability to stabilise to this depth, combined with the ability to handle high cementitious binder contents, has enabled the use of single, thick, heavily bound base layers of significant tensile strength, thereby making in situ stabilisation an appropriate treatment on more heavily trafficked roads. In so doing, it eliminates the alternative processes of removing the upper layers of the pavement to allow stabilisation of the lower layers and then replacing and stabilising the upper layers or a total reconstruction involving two layers of imported cement-treated material. In both cases, the structural capacity of the pavement is highly dependent on achieving and maintaining a strong bond between two thinner cemented layers.

Deep lift stabilisation is suitable over most subgrade types, although compaction of a thick layer over a weak subgrade is difficult. Subgrades with a CBR greater than 5% at the time of construction should enable satisfactory compaction of the stabilised base. If the CBR is less than 5% at the time of construction, corrective action to improve the subgrade will be found to be both necessary and cost-effective.

Cementitiously stabilised pavements may develop shrinkage cracking, which can impact on pavement and surfacing performance. The likelihood and severity of shrinkage cracking usually increases with increasing binder content and will also be affected by construction practices.

The need to prevent the reflection of shrinkage cracking through to the pavement surfacing will depend on the likely impact of the cracking on pavement performance, e.g. loss of waterproofing, and the acceptance of undertaking future maintenance, e.g. crack sealing, resurfacing, etc. For high traffic loadings a thick granular or asphalt overlay may be provided to inhibit the reflection of the cracking through to the surface. On low-volume roads a conventional sprayed seal may suffice. Numerous other surfacing options including strain alleviating membrane seals, asphalt with/without modified binder combined with strain alleviating membrane interlayers may also be adequate. Refer to Austroads (2009c) for further details on surfacing options.

Guidance on the mix design, including selection of binder type and quantity is provided in Austroads (2019a). In addition, useful information about work practices is contained in road agency and industry technical notes.

Consideration needs to be given to the required curing process and time for the selected stabilisation treatment as this may affect traffic management requirements.

Section 10 provides guidance on the thickness design of stabilisation treatments.

Appropriate uses

- On granular pavements exhibiting load-related distress due to the quality of the pavement materials and where better-quality materials are not readily available.
- On soils other than highly organic soils, high plasticity soils or well-graded gravels.

Inappropriate uses

- On pavements where the risk associated with shrinkage cracking and consequential reflective cracking in thin asphalt layers is unacceptable.
- On pavements that require significant strengthening, the use of modified cementitious material, rather than a bound material, may not provide adequate strengthening.
- On pavements with weak subgrades less than CBR 5% at the time of construction, deep lift stabilisation greater than 300 mm in thickness is not appropriate due to the lack of subgrade support.

7.5.9 Lime Stabilisation

Hydrated lime ($Ca(OH)_2$) is formed by mixing quicklime (CaO) and water (H_2O). Quicklime is the most common form of 'lime' used in the stabilisation of soils, and hydrated lime is used when blending cement, slag and fly ash to form a cementitious binder.

Lime is particularly effective in heavy clays where the lime reacts with the clay minerals or other pozzolanic components in the soil to form a tough, water-insoluble gel of calcium silicates and calcium aluminates, which binds the clay particles together. Thus, lime stabilisation involves a combined process of bonding and waterproofing.

This has an immediate effect on the soil structure producing a more friable material with lower plasticity and density and, in most types of clay, with a significant and immediate strength gain. In wet clays, the change in plasticity increases the optimum moisture content, which has the effect of drying out the clay as an aid to construction or as preparation for further treatment, such as cement stabilisation. The required lime content increases with the clay content of the soil and its plasticity. In selecting the lime content used for stabilisation, cognisance needs to be taken of the resulting of lime demand testing. An initial estimate of the required lime content can be obtained by mixing samples of the soil with water and different proportions of lime to find the minimum quantity of lime required to give a solution with a pH of 12.4 (Austroads 2019).

Additional CBR or UCS testing may be undertaken to confirm that a reaction with lime will occur and result in significant and consistent strength gains, particularly if the design of the pavement is to be based on these assumptions. Testing requirements are discussed in more detail in Austroads (2019).

As discussed for cement and cementitious-stabilised bases, the surfacings options of lime-stabilised granular bases may need to consider the risk of reflective cracking.

Section 10 provides guidance on the thickness design of stabilisation treatments.

Appropriate use

- In clay and clayey soils to reduce plasticity and thereby decrease moisture susceptibility.
- In clay and clayey soils to improve long-term subgrade strength and pavement support.

Inappropriate use

- In highly organic soils and sandy soils which contain no clay.

7.5.10 Bitumen Stabilisation

In contrast to cementitious binders, which usually react chemically with the host material, bitumen acts simply as an adhesive binding agent, which sticks soil particles together and the bitumen provides a degree of waterproofing to the soil. The benefits of bitumen stabilisation are an increase in rut-resistance over the unbound granular materials, quick construction at lower cost than reconstruction and providing a durable and less moisture-sensitive pavement material, which can be trafficked immediately.

As bitumen is a semi-solid material at ambient temperature, it either needs to be modified or heated to reduce its viscosity sufficiently to enable its ready dispersion into the soil. Bitumen used in stabilisation may be modified by emulsification, in which the bitumen is combined with water and an emulsifying agent in a process, which produces a suspension of fine droplets of bitumen in water. When the emulsion 'breaks' the bitumen is deposited on the host material. The usual form of hot bitumen used in stabilisation is foamed bitumen in which hot bitumen is mixed with a small quantity of water. When the hot bitumen comes into contact with cold water, the mixture expands to a minimum requirement of 10 times its original volume and forms a fine mist or foam.

Using bitumen emulsion

For stabilisation using bitumen emulsion, only slow-setting anionic emulsions are suitable and are normally added at a rate of 2–3% by mass of residual bitumen, although lower rates (0.5–1%) can be effective in well-graded materials in dry climates. The emulsion can also be used in combination with other binders such as lime or cement.

Using foamed bitumen

Foamed bitumen is a hot bituminous binder that has been temporarily converted from a liquid state to a foamed state by addition of a small amount of water (2–3% of the bitumen mass). In the foamed state bitumen can be mixed with aggregates at ambient temperatures and in situ moisture contents. The bitumen foam coats the fine fraction of the treated aggregate, creating a mastic that binds the larger particles of the aggregate skeleton. A foaming agent may be needed to ensure the bitumen foaming properties are acceptable.

In Australia, the foamed bitumen content added to the aggregates normally range from 2.5 to 3.5% and hydrated lime is normally added (between 1% and 2%) as a secondary binder to improve strength of the mix and dispersion of bitumen through the aggregate. Cementitious binders are used in place of lime in some areas.

In New Zealand, foamed bitumen contents are commonly slightly lower (2.5–3.0%) and 1% cement is used as the secondary binder, although lime may be used for higher plasticity materials.

The strength/modulus of foamed bitumen stabilised mixes is derived from:

- friction between the aggregate particles
- viscosity of the bituminous binder under operating conditions
- cohesion within the mass resulting from the binder itself, and the adhesion between the bituminous and hydraulic binders and the aggregate.

Similar to other stabilising binders, foamed bitumen stabilisation can be undertaken in situ or plant mixed. The foamed bitumen is incorporated into the recycling drum or into the plant where it wets and coats the surface of the fine fraction particles to form a flexible bound pavement material. The mixing of the foamed bitumen and host granular material is critical to the success of the process because the bitumen is in its foamed state for only a very short period and coating of the particles must be achieved within this period.

Section 10 provides guidance on the thickness design of stabilisation treatments.

Where the design traffic exceeds 10^7 ESA, a minimum surfacing of 30–40 mm hot mix asphalt layer is recommended. For lower traffic loadings, either a sprayed seal or hot mix asphalt can be used.

Appropriate uses

- Low-plastic granular materials where cohesion is low, and/or a waterproofing action is needed.
- Pavements where the surfacing exhibits flushing or fattiness and where conventional reseals or thin asphalt overlays would not be effective treatments.
- Weak bases overlying strong subgrades.

Inappropriate uses

- Material containing higher proportions of clay and silt fines (> 25%) and/or having moderate to high plasticity (PI > 20).
- Material that has insufficient fines (less than 5% clay and silt fines).

Austroads (2019a) provides guidance on foamed bitumen stabilisation mix design.

7.5.11 Other Chemical Stabilising Binders

There are a number of generic chemical stabilising binders that have been utilised mostly in a dust suppressant role but also in pavement strengthening. Brief descriptions of some of these products are given below.

Organic, non-bituminous binders, lignosulphonates

The major source of these products is the wood pulping industry. Sodium, calcium and ammonium lignosulphonates are derived from the 'sulphite' pulping process and tall oil pitch and pine tar are by-products of the 'kraft' pulping process. They act in soils to bind particles together and in clays to act as a dispersant to make the clay less plastic and after compaction provide a denser, stronger pavement.

They are most effective in dry (temperate and non-humid) climates on moderately to highly plastic soils, although the minimum plasticity index of the soil for greatest effectiveness varies with the product. The level of binder in the soil increases with repeated applications although this is offset to some degree by their tendency to be washed out by heavy rain. These binders are best applied by using a stabiliser and mixing the binder thoroughly into the pavement material.

Electro-chemical stabilisers, sulphonated petroleum, enzymes

These are mainly derived from sulphonated petroleum products and are highly ionic. They work by expelling adsorbed water from the soil structure, which decreases air voids and hence, increases compaction. A more cohesive 'tighter' unsealed road surface can be obtained. Maximum strength of the pavement material may not be attained until up to 20 days following application. The effectiveness of these stabilisers can be very material-dependent, particularly with respect to the clay mineralogy.

Dry powdered polymers, PVC, PVA

Dry powdered polymers (DPP) act to preserve the strength of moisture-susceptible gravels by a process of 'external and 'internal' waterproofing involving the formation of a hydrophobic soil matrix and preferential coating of clay and silt fines. The soil matrix has a reduced permeability and so limits the ingress of moisture (external waterproofing), while the coating of the fines reduces the softening and lubricating effect of any moisture that does enter the pavement (internal waterproofing). As they are hydrophobic, they can reduce the moisture sensitivity of a gravel pavement material without stiffening the pavement. A disadvantage, however, is the dusting, which occurs because the binder cannot be effectively wetted.

DPPs should not be confused with water-soluble polymers sometimes used to enhance cementitious reactions. The products can be very expensive and the selection of proprietary products of this type requires careful consideration and investigation to ensure that the binder is appropriate to the specific pavement problems.

DPPs may require a carrier such as fly ash for effective introduction into the pavement material. They have been reported to be very effective on sandy soils in dry climates. For example, sulphonated hydrocarbons, because they thermally bond to a carrier such as fly ash, can be finely distributed through gravel soils.

Refer to AustStab Technical Note 3b (AustStab 2007) and APRG Technical Note 14 (APRG 2003b).

Appropriate uses

- Unsealed roads and shoulders as a dust palliative.
- Materials in salt-prone areas.
- Stabilisation of poor-quality or wet subgrades.

Inappropriate uses

- Pavements requiring strengthening due to the poor quality of pavement materials.
- Heavy rainfall areas due to the susceptibility for some binders to be leached from the soil.

7.6 Treatments for Pavements on Expansive Subgrades

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 6.8. A number of strategies to minimise volume changes in highly expansive soils are listed in Austroads (2018a).

7.7 Design and Construction Considerations

7.7.1 Community Attitudes

In the current social environment, there is usually an expectation that rehabilitation works will consider and minimise the impact of the works on local residents, businesses and road users. The attitude of these communities to the work can significantly influence its scale and timing. Particular issues which need to be addressed include:

- nuisance effects such as dust, noise, smell
- environmental impact such as removal of trees for side tracks, soil erosion, production of smoke and other effluent
- degree of inconvenience to local residents, businesses and road users – length of delays, use of detours or temporary side tracks, access restrictions
- the scheduling of the work – time of year, time of day, time required for completion of the work.

7.7.2 Grade Line Restrictions

Limitations on modifying the grade line of the road can have a major impact on the choice of treatment types. In urban areas the need to match the levels of adjacent surfaces such as kerb and channel and intersecting roads and to maintain access to adjacent properties may limit treatment options. While the need to match levels of intersecting roads and maintaining property access are still important in rural areas, they may be less restrictive than in urban areas. In rural areas, a more common limitation is imposed by the need to avoid floodway afflux problems.

While these limitations may preclude the use of conventional overlay treatments, there is also the option of raising the level of adjacent surfaces to enable an overlay to be placed while maintaining adequate drainage and property access. This decision should be adopted with caution not only because of the additional expense, but also because of the possible restriction on the choice of future treatments. Therefore, in most cases, such level limitations necessitate the use of alternative treatments such as:

- milling and replacing the existing surfacing/pavement layers
- milling adjacent to the junction with the existing surface prior to overlaying
- thin surfacings, e.g. ultra-thin asphalt
- polymer modified binder asphalt overlays
- in situ stabilisation of existing materials
- reconstruction.

7.7.3 Depth Restrictions Due to Services

Particularly for urban pavements, existing services (including electricity, gas, water, sewerage and telecommunications) may limit the rehabilitation treatment. In selecting treatment options, consideration may need to be given to the minimum cover requirements to inhibit damage during construction.

7.7.4 Road Geometry

In any rehabilitation design, the effects of the possible treatment or treatments, whether beneficial or detrimental, on the present road geometry (i.e. vertical and horizontal alignment, cross-sectional shape) should be considered. For instance, pavement widenings may provide additional benefit by enabling increased horizontal radii of curvature on sharp bends, while varying an overlay thickness across the pavement section to accommodate variable pavement strength can reduce a steep crossfall.

Conversely, a pavement widening, which is to be constructed without a subsequent overlay can have a detrimental effect on surface drainage if the crossfall adopted on the widening is flatter than that on the adjacent pavement. Equally, a widening might involve increasing an embankment width by a small but unmanageable amount.

Similarly, the provision of a thick overlay may involve additional costs for raising the level on the shoulder and adjacent verges and relocating guardrails and drains. These additional costs need to be considered when evaluating treatment options.

Guidance on road geometry is provided in Austroads *Guide to Road Design Part 3: Geometric Design* (Austroads 2016).

7.7.5 New Pavement Abutting an Existing Pavement

When designing rehabilitation treatments that involve the construction of a new pavement abutting an existing pavement, the relative strengths, permeabilities and thicknesses of adjacent materials need careful consideration.

New pavement abutting an existing pavement should have a structural capacity equal to or better than the existing pavement. The joint between the existing and new pavement structure should not be in the wheelpath (see Section 7.7.6).

If upgrading of the existing pavement by widening is being considered, consideration needs to be given as to whether it is cost-effective to rehabilitate the existing pavement to the same structural capacity as the new widening. Although this is desirable in terms of future maintenance, in some projects it may not be the most cost-effective strategy considering whole-of-life costs.

Other factors to be considered include surface drainage, sub-soil drain installation or relocation, matching existing crossfall and smooth transition of horizontal alignment and joint seals. The minimum width of the pavement widening should be sufficient to ensure access for mechanical compaction equipment within the treated area.

Where the surface level of the new abutting pavement is required to match an existing pavement, the material should be placed in such a manner as to provide a smooth riding surface across the junction. In some cases, it may be necessary to remove a sufficient amount of existing pavement material to enable a smooth, transitional surface to be constructed across the junction.

Where it is necessary to taper the thickness of a layer to provide a smooth transition with an existing surface, the layer should end at a chase cut or be cold planed into the existing asphalt. The chase should have a minimum depth not less than twice the nominal aggregate size of the mix being placed, and with a minimum width of 400 mm. Where necessary, removal of coarse particles from a layer of tapering can be achieved using hand raking.

Where the thickness of the layer tapers to less than twice the nominal size of the mix, the area upon which material of such thickness is to be placed should be tack-coated uniformly. Further details of asphalt joints may be found in Part 4B of the Guide (Austroads 2014).

7.7.6 Pavement Jointing Considerations

The structural competency of the pavement at longitudinal construction joints is generally not as sound as in other areas. This may be due to:

- reduced compaction arising from bridging of the joint by the roller during construction
- lack of aggregate interlock
- segregation of coarse aggregate.

As a result, pavements tend to be weaker and more permeable at longitudinal construction joints. Load-induced deformation of granular materials and shrinkage cracking of cemented materials can occur along these discontinuities.

To reduce the risk of premature distress, construction joints should not be located in wheelpaths.

7.7.7 Shoulder Sealing

The safety and pavement performance of roads can be significantly enhanced by sealing the shoulders, and hence, consideration should be given to incorporating shoulder sealing as part of any rehabilitation treatment on such roads. The heavier the traffic, the greater the user benefits derived from sealing shoulders.

Since sealing the surface tends to make the shoulder more attractive for vehicular traffic, pavement materials need to be of sufficient quality and thickness to carry the occasional vehicle loading. The method to determine the design traffic loading for the shoulder design can vary between road agencies but should include local observations wherever practicable.

Irrespective of traffic volumes, in high rainfall areas it is good practice to seal out to and over the lip of the shoulder on the high side of superelevated curves. This is needed to seal a common entry point of moisture into the base material. Limitations occur when road maintenance funds and adequate formation are not available.

Note that repositioning of safety barriers may be necessary as a result of shoulder sealing.

7.7.8 Staged Construction

The adopted rehabilitation strategy may include a series of treatments staged over a period of years, with the aim of providing a serviceable pavement for the selected analysis period.

A staged approach to the rehabilitation may be adopted for a number of reasons including:

- Funding and economic considerations:
  - the economic optimisation of alternative rehabilitation strategies
  - the need to apportion limited funds between projects of equivalent priority in order to achieve at least some minimum standard
  - the achievement of greater user benefits by improving the serviceability of a greater length of pavement for the same outlay
  - taking advantage of the time-value of money (i.e. the present value of the cost of a future stage is discounted).
- Programming or scheduling constraints:
  - the impending availability of an alternative route or reduction in traffic, e.g. opening of a bypass
  - the aggregation of a number of projects requiring similar treatment to justify the establishment of specialist plant
  - the efficient allocation of available resources.

- Pavement performance considerations:
  - the surfacing life may be significantly less than the pavement structural life
  - allowing for the progressive correction of loss of shape, particularly for pavements on expansive subgrades
  - allowing an upgrading of the drainage system to improve the strength and modulus of a pavement prior to retesting the deflections for a possible overlay
  - allowing the subgrade heave in a deep cutting to occur prior to placing the final pavement surface
  - increasing the structural capacity of the pavement to match the growth in traffic
  - improving safety and serviceability by limited treatments solely to correct rutting and roughness
  - allowing settlement of soft foundations to occur prior to placing the final pavement surface
  - in situ stabilisation of an existing pavement to provide a limited service life initially and ultimately a uniform subbase to a granular overlay at some time in the future.

Care should be exercised when adopting staged construction strategies because:

- deferral or cancellation of the later stages may lead to the need for unexpected remedial works and dramatically alter the economics of the original strategy
- the construction of later stages may interfere with earlier works such as drainage outlets, batter stability, etc.

In areas where distress is likely to be due to environmental factors, rather than load-related factors, consideration may be given to restoring evenness as an initial stage of rehabilitation.

7.7.9 Construction Under Traffic

Whereas new construction work is typically carried out clear of traffic, this is rarely the case for rehabilitation works, which almost exclusively occur on in-service pavements.

A range of options may be available for traffic management during the works. These include:

- detouring traffic for the duration of the project
- side tracks for traffic only while work is in progress
- carrying out the work under restricted traffic flow
- closing the site to traffic.

In planning any traffic management scheme, cognisance should be taken of the requirements of Australian Standards, New Zealand Standards or road agency codes of practice for traffic management.

Selection of the most appropriate option requires consideration of:

- inconvenience to road users
- restrictions on hours of work, e.g. at night only
- public safety, both for vehicular and pedestrian traffic
- provision of a safe working environment
- cost of traffic provisions
- beneficial effects of traffic on the work, e.g. identification of soft or weak spots prior to sealing and wheel rolling of the surface
- detrimental effects of traffic on the work, e.g. ravelling or scouring of unsealed gravel surfaces and smearing of clay or road grime on interim asphalt surfaces.

7.7.10 Risk, Design Sensitivity, Construction Tolerances and Degree of Control

Because of the many factors which must be evaluated in the design of rehabilitation treatments and which influence performance to varying degrees, there is no absolute certainty that the desired performance will be achieved.

In any given application, the range of possible rehabilitation treatments will exhibit differing sensitivity to variations in a range of design parameters or construction factors. Factors, which may be difficult to quantify or control but can significantly affect the quality or subsequent performance of the works include:

- variability of the existing pavement materials and subgrade
- variability in the thickness of the existing pavement or a critical layer within the pavement, especially in relation to in situ treatments such as hot in-place asphalt recycling and pavement stabilisation
- pavement drainage or protection of the subgrade from infiltration
- location of joints and transitions between different treatments
- operator skills with specialised plant and equipment
- variability of application rates and mix consistencies
- changing climatic conditions such as rainfall and temperature.

The relative sensitivities of each of the possible rehabilitation treatments to these factors need to be considered when comparing alternative treatments. If a design is particularly sensitive to one or more of these factors, it may be preferable either to allow for some degree of over-design, adjust the estimating rates, reschedule the work or, in the extreme, exclude the treatment from further consideration.

The risk of poor performance can be controlled, to some extent, in the selection of design parameters and the degree of conservatism in the values adopted for each of these parameters. For the empirical design of granular overlays (Section 9), a simple method of achieving a more conservative thickness design is to adopt a value for the design traffic over the design period higher than that which is anticipated. It is suggested that the use of a value of up to two times the anticipated traffic may be warranted for projects with a higher-than-normal risk of poor performance. For the design of other structural treatments, the risk depends on the reliability selected for the project.

It is appropriate that the acceptable level of risk varies with the function of the road for which the treatment is being designed, such that the level of acceptable risk diminishes with the importance of the road and the scale of the project. For some projects, a higher risk of premature treatment distress is accepted than would be considered appropriate for new construction. For instance, improving pavement support in specific locations may be foregone for economic reasons, and this may increase the risk of premature distress.

7.7.11 Availability of Plant, Personnel and Material

Certain rehabilitation treatments are most efficiently and effectively constructed or applied using specialised items of plant operated by skilled staff. Examples include hot in-place asphalt recycling, cold planing and stabilisation. When use of these techniques is being evaluated, the following factors should be considered:

- plant availability
- personnel with appropriate skills, knowledge and experience
- plant establishment costs relative to the project size
- availability of process control testing facilities
- proximity of similar projects which may utilise the same plant
- scheduling plant usage with the work program.

The viability of certain treatments can also depend on the availability of special materials. These may range from manufactured products such as customised geotextiles or polymer modified binders, which may take some time to supply, to standard granular material, which may first have to be located, tested and then transported over some distance to the site.

8. Treatments for Rigid Pavements

8.1 Introduction

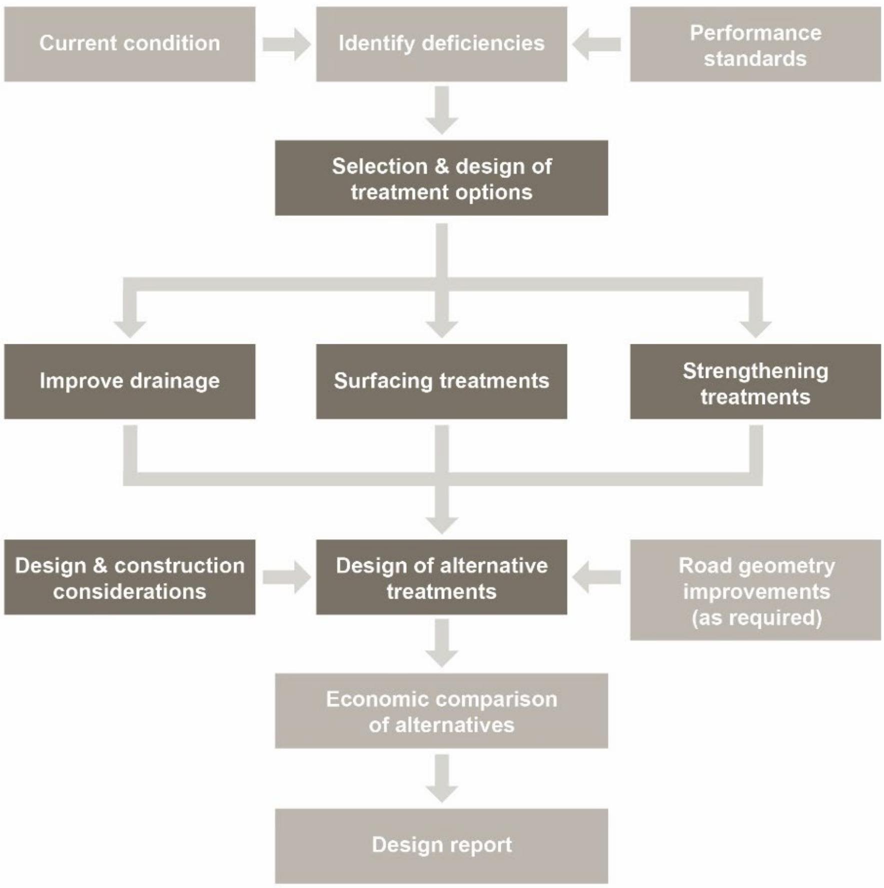
When an existing pavement no longer provides an acceptable level of service for the prevailing traffic and/or environmental conditions or when the current level of service is not deemed to be adequate for anticipated changes in those conditions, then some form of treatment is required.

Section 3, Section 4 and Section 5 of the Part discuss the methods of investigation to measure and collate data on the existing pavement condition and the process for evaluating this data to establish the structural and functional condition of the pavement.

As discussed in Section 6, one important reason for data collation and investigating the current pavement condition is to identify the causes and modes of distress and identify deficiencies.

This section provides descriptions of the rehabilitation treatments that may be used to address deficiencies in the pavement condition of a rigid pavement and highlights some of the design and construction issues associated with each of the treatments (Figure 8.1).

Figure 8.1: Design steps discussed in Section 8



8.2 Overview of Treatments Options

In the design of any rehabilitation treatment, the first and most important phase is the identification of the mode or modes of distress evident in the pavement and their associated cause or causes (Section 6). If the mode or cause of distress is incorrectly assessed, then it is very unlikely that the selected rehabilitation treatment will be effective, regardless of the quality of its design and construction. By comparing the current condition to road agency standards, the performance deficiencies can be identified. An appropriate rehabilitation treatment is one that addresses the cause of the pavement distress and deterioration and is effective in both repairing it and inhibiting its reoccurrence.

A considerable amount of analysis and engineering judgement is required when determining treatment options.

The rehabilitation options for rigid pavements by distress type as characterised by the visual defects are listed in Appendix A. Treatment options for rigid pavements can be categorised under four broad headings:

- treatments for drainage (Section 8.3)
- treatments for surfacing distress (Section 8.4)
- treatments for joint distress (Section 8.5)
- treatments for structural distress (Section 8.6).

This section provides broad guidance on these treatments, while more detailed design information is found in the following Roads and Maritime documents:

- *Standard Pavement Surface Drainage Details: Volume 5: Rigid Pavement Details* (Roads and Maritime 2012)
- *Pavement Standard Drawings: Rigid Pavement: Volume MP – Plain Concrete Pavements* (Roads and Maritime 2015a)
- *Pavement Standard Drawings: Rigid Pavements: Volume MJ – Jointed Reinforced Concrete Pavements* (Roads and Maritime 2015b)
- Pavement Standard Drawings: Rigid Pavements: Volume MC – Continuously Reinforced Concrete Pavements (Roads and Maritime 2015c).

8.3 Treatments to Improve Drainage

Further to the discussion in Section 7.3, several drainage issues that need to be considered in rehabilitation of rigid pavements are as follows:

- Some rigid pavements, which were constructed before current design standards existed, do not have subsurface drainage along the pavement edge. As a consequence, excess water can accumulate in the base and subbase. For plain concrete pavements, this can lead to joint sealants being ejected under the action of truck loadings. In such cases, the cost-effectiveness of retro-fitting edge drains needs consideration. It is important that the subsurface drain be designed and constructed to drain water that may accumulate at the interface between the concrete base and the subbase. Roads and Maritime *Standard Pavement Surface Drainage Details, Volume 5: Rigid Pavement Details* (Roads and Maritime 2012) may be useful in designing the treatment.
- In addition, some subsurface edge drains use sand as the filter material. Over time, soil may accumulate in the sand. The resulting clogging of the filter materials adversely affects the performance of such drains. In such cases, the cost-effectiveness of replacing the filter material needs consideration.

8.4 Treatments for Surfacing Distress

8.4.1 General

The common surface distress types are:

- scaling and rutting
- loss of surface texture
- loss of surface shape
- plastic shrinkage or pre-hardening cracking.

Scaling and rutting can be treated by bonded concrete topping, grinding or profiling or by overlaying with asphalt. The choice of the treatment depends on the size of the area to be treated, the age and condition of the surrounding pavement and the traffic and environmental conditions. Where an asphalt overlay is proposed to treat deficiencies in a concrete pavement, consideration needs to be given to the means of minimising the potential for reflective cracking in the overlay.

Loss of surface texture, which may lead to a loss of skid resistance, can be restored by transverse grooving or surface grinding. While grooving is generally more effective on higher-speed roads (> 80 km/h), grinding is effective on lower-speed roads provided the original surface was textured by brushing or hessian dragging. As the loss of texture is generally of concern only if it adversely impacts on the safety of road users, the decision to undertake any treatment should reflect consideration of all the factors affecting road safety.

Localised surface irregularities, often in the form of high spots resulting from poor construction practice or small steps (< 10 mm) at joints, can be removed by grinding/profiling. Impact tools are not allowed because of the potential for damage to the otherwise sound pavement. Where steps at joints are evident, it may be indicative of a more serious underlying problem (pumping and erosion, etc.) in which case, any surface treatment is likely to be temporary.

More extensive loss of surface shape may also be corrected using asphalt overlays.

Typical plastic shrinkage cracks comprise discrete cracks of less than 500 mm length each and depth less than 50% of the slab thickness, which do not intersect a formed edge. It is essential that engineering judgement is exercised in selection of treatment for plastic shrinkage cracks. Cumulative crack length of 1 m in 25 mm^2 of area may be treated by application of low viscosity penetrating epoxy resin. The epoxy resin should not extend laterally by more than 15 mm beyond the edge of the crack nor completely fill the tining.

8.4.2 Bonded Concrete Topping

This technique is used to repair surface defects in concrete slabs. It involves partial-depth patching to a maximum depth of about 30 mm and, as such, is shallower than most spall repairs. Figure 8.2 illustrates such a repair to a concrete pavement; note the repairs extend a minimum of 50 mm beyond the extent of the defect.

Figure 8.3 shows a thin bonded topping, which was applied to correct slumping of a slip formed edge. Its early failure appears to be related to unsatisfactory construction techniques such as poor surface preparation, inadequate curing of the patch material, and possibly the use of an unsuitable repair material. Roads and Maritime experience is that thin epoxy repairs commonly delaminate under traffic and that careful cutting to square the edges of the repair, scrabbling of the base and sides and curing the concrete repair material are all essential to achieving a satisfactory patch.

Figure 8.2: Examples of thin bonded repairs

Sawing along each side to provide vertical edges

Acute corners must be avoided by squaring all joints and junctions as shown

Repaired area after 25 years

By comparison, similar repairs carried out using inferior products and/or methods have shown a high maintenance demand

Source: Roads and Traffic Authority (RTA) (2000).

Figure 8.3: An unsuccessful thin bonded topping

8.4.3 Grinding/profiling

Grinding/profiling is a process undertaken with purpose-built equipment which uses diamond saw blades, gang-mounted on a cutting head. There are typically 150 to 200 blades per metre width of cutting head depending on the hardness of the aggregate in the concrete.

Grinding/profiling is a cutting process that is quite distinct from cold planing, which is an impact or chipping process. Cold planing typically creates a surface, which is rough and relatively noisy (Figure 8.4). By contrast, grinding creates a fine 'corduroy' surface texture, which is quieter and smoother, and provides improved skid resistance (Figure 8.5). The process is routinely used overseas to achieve one or more of these outcomes.

Some reports indicate that a smoother ride quality may be achieved by grinding against the direction of traffic flow.

Grinders/profilers require water to cool the cutting head. The water, the grinding slurry and any broken chips should be removed from the pavement surface by washing with high pressure water or vacuum processing. If left on the pavement, this material discolours the pavement and can re-set to mar the ride quality of the pavement. It can also fill joint recesses and damage seals under traffic.

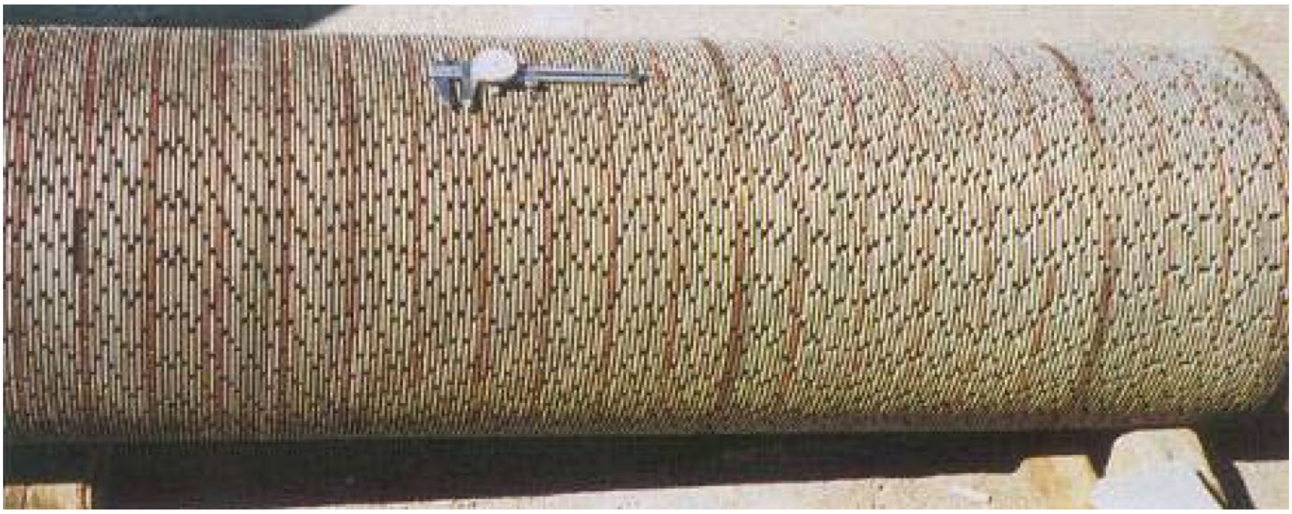
Grinding may have to be followed by transverse grooving in order to restore water paths for high-speed traffic.

Figure 8.4: Joint damage and harsh surface texture caused by cold planing

Figure 8.5: The surface of a bridge deck which has been longitudinally ground and transversely grooved

Cold planing is not an acceptable alternative to grinding/profiling and should not be used on exposed concrete pavements (i.e. without an asphalt surfacing). Cold planing causes unacceptable damage to joints (as shown in Figure 8.4) which will create an ongoing maintenance demand. The resulting ride quality may also be unsatisfactory. Modern diamond grinding machines can achieve significant improvements in surface smoothness and ride quality as well as reduced tyre noise (Figure 8.6).

Figure 8.6: Diamond grinding head for concrete surface texture correction



8.5 Treatments for Joint Distress

8.5.1 General

Principal forms of joint distress include:

- loss or degradation of the joint sealant
- joint spalling.

While there is currently debate over the need to seal joints in plain concrete pavements and some evidence that unsealed joints perform satisfactorily, it is advisable to maintain the integrity of the joint sealant. The risk in not maintaining the sealant is that:

- detritus such as stones, sand, etc. can accumulate in the joint causing high local stresses during expansion of the pavement and consequential spalling along the joint
- water ingress at joints will soften the subbase and subgrade and will promote pumping of fines and a loss of slab support.

The most effective method of repairing or replacing seals, will vary considerably depending on the type and age of the existing sealant, traffic conditions, and subbase and subgrade types.

Spall repairs (see Section 8.5.3) will normally be carried out with the intention of restoring the integrity of a joint, reducing water ingress, or restoring the integrity and ride quality of the running surface. In assessing the need for treatment, consideration should be given not only to the current state of spalling but to the rate and extent of deterioration, together with the likely consequences.

The options for treating spalls include:

- sawing and resealing
- routing and sealing (where resawing is unsuitable)
- patching.

Details of these treatments are provided in Roads and Maritime standard drawings (Roads and Maritime 2015a, 2015b, 2015c).

8.5.2 Joint Seal Replacement

In replacing existing seals, the most efficient method is to resaw the existing joints in order to produce clean faces (for optimum bond) with a minimum of arris spalling.

It is important to check the design joint width for each situation, because it is possible that the original (existing) joints are too narrow, in which case the new (resawn) joint may also be too narrow, and the silicone will be stretched beyond its design limit. This appears to be most common in isolation joints and will often explain (at least in part) the reason for premature seal failure.

If the existing joint widths are inadequate, then the resawing width should not be less than that designed in accordance with road agency standards.

Where the existing joint width is adequate, the resawn joint will be marginally wider than required. While the resawing width must be sufficient to provide clean new faces, any extra width should be minimised, for reasons of cost and ride quality.

International experience indicates that transverse joints begin to influence ride quality (i.e. become noticeable to motorists) at a width of about 14 mm. Therefore, if an existing joint is 9 mm wide and the width is increased by 2 mm with each reseal, ride quality will start to be adversely affected on the third reseal.

Note that the seal design method is different for tied joints and untied joints. Seals in tied joints are subjected to much lower movements and, hence, can be placed at lesser widths. For the same reason, larger deviations are tolerated on the width-to-depth ratio of the sealant.

Wherever possible, the sealant manufacturer's recommendations should be followed, and every effort must be made to ensure that specifications and guidelines agree with the intent of those recommendations. Where conflict arises, specialist advice should be sought.

Every effort must be made to prevent the ingress of sawing slurry into existing induced cracks. Cracks at contraction joints will typically be about 1 mm to 1.5 mm wide during winter. Slurry, which settles into the bottom of such narrow cracks, will be difficult to wash out, and its presence will obstruct closure of the joint in the subsequent summer. In severe cases, this can lead to the following problems:

- increased pressure on terminal anchors, possibly resulting in their rotation
- compression failures
- pavement 'blow-ups' – in some overseas countries it has been found that this problem is very often associated with the accumulation of incompressibles in the bottom of joints that, in hot weather, cause a (bottom) eccentric compressive force which results in an upward thrust of the joint.

These risks can be minimised if resawing is carried out during the warmer months. At other times, the use of a temporary sealant (such as a spline) in the bottom of the existing saw cut and increased care is required to minimise ingress of slurry.

Details of joint seal replacements are provided in Roads and Maritime standard drawings (Roads and Maritime 2015a, 2015b, 2015c).

8.5.3 Joint Spall Repairs

Spalls are characterised by the disintegration of concrete along the joint arrises as shown in Figure 8.7, Figure 8.8 and Figure 8.9.

Spall repairs will normally be carried out in order to restore the integrity of the joint, to arrest further deterioration, to reduce water ingress and to restore the running surface.

However, treatments such as resawing or routing of joints are also useful for minimising future spalling and/or arresting the progress of existing spalling by relieving the concentrated arris stresses which are typically the causes of spalling.

Details of joint spall repairs are provided in Roads and Maritime standard drawings (Roads and Maritime 2015a, 2015b, 2015c).

Figure 8.7: Spalling at an inclined ribbon in a longitudinal joint

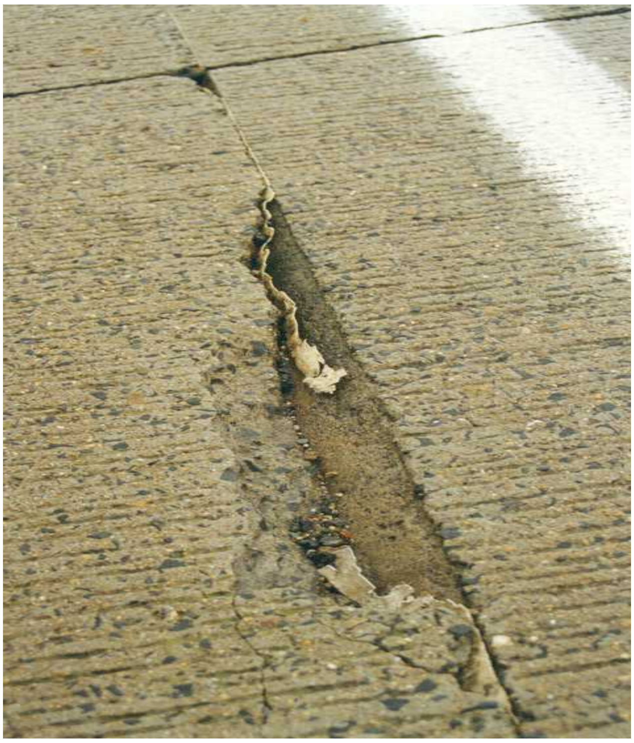


Figure 8.8: Spalling at an inclined and depressed ribbon in a longitudinal joint

Figure 8.9: Illustrations of spalling and spall repairs

Spalling at a longitudinal tied joint

Single-sided spall repair

8.6 Treatments for Structural Distress

8.6.1 General

Structural distress is typically classified as that which reduces the load-carrying capacity of the pavement. Its most common manifestation is full-depth cracking sufficiently wide to reduce or preclude load transfer across the crack. A discussion on the definition and causes of structural cracking is provided in Appendix A.

The treatment of structural distress involves relatively large-scale and complex repair methods including:

- slab undersealing (Section 8.6.2)
- cross-stitching (Section 8.6.3)
- full or part-slab replacement (Section 8.6.4).
- asphalt overlay (Section 8.6.5)
- concrete overlay (Section 8.6.6)
- slab fracture with overlay (Section 8.6.7)

The applicability of several of these methods is summarised and discussed in Table 8.1.

Table 8.1: Treatments for structural cracking

Method	Comments
Full slab replacement	Recommended for slabs with: • low-strength concrete and/or • multiple cracking and/or • cracking plus substantial joint distress or • a need for subbase repairs(1) Detailed guidelines are provided in Roads and Maritime (2015a, 2015b, 2015c).
Part-slab replacement	Recommended for slabs with: • localised cracking and/or • single crack plus localised joint distress or • a need for subbase repairs(1) Detailed guidelines are provided in Roads and Maritime (2015a, 2015b, 2015c).
Cross-stitching	Detailed guidelines are provided in in Roads and Maritime (2015a, 2015b, 2015c). Effectiveness is greatly increased if stitching is completed before the crack (or joint) opens beyond about 0.5 mm(2). Routing and sealing may be required depending on the width of the crack(3).
Corner slab replacement	Detailed guidelines are provided in Roads and Maritime (2015a, 2015b, 2015c).
Asphalt overlay	Refer to Section 8.6.5
Concrete overlay	Refer to Section 8.6.6
Slab fracture with overlay	Refer to Section 8.6.7

1 This could apply where the cracking was caused by subbase erosion or differential settlement.

2 In cases where stitching is viable, it would be very desirable to complete it before the crack becomes a full structural crack, i.e. before it grows full-length between joints and/or edges.

3 The purpose of routing and sealing is typically to minimise water ingress to underlying layers and to protect tie bars (or stitch-bars) from corrosion.

8.6.2 Slab Undersealing

This treatment stabilises and underseals concrete pavement slabs by pumping a grout mixture through holes drilled in the pavement into voids beneath the slabs. The purpose of grout injection is to stabilise the slab by filling these voids.

The grout should have high early strength properties – particularly when the pavement will be subjected to trafficking within several hours of initial set – and should be non-shrink to ensure that the voids are completely filled after the grout has set.

'Mud-jacking' is a form of undersealing that involves raising the slab level by applying the grout through holes drilled in a grid pattern. It uses a grout, which commonly consists of fly ash with approximately 6% cement.

Appropriate uses

- For areas that are experiencing loss of support of the slab. The treatment should be performed as soon as loss of support is noted and before the voids become so large that they cause pavement failure.
- To reduce deflections, particularly differential deflections at joints, and thus reduce future pumping, slab cracking and reflective cracking through asphalt overlays.
- To improve load transfer capacity of the slab joints and reduce high deflection levels.

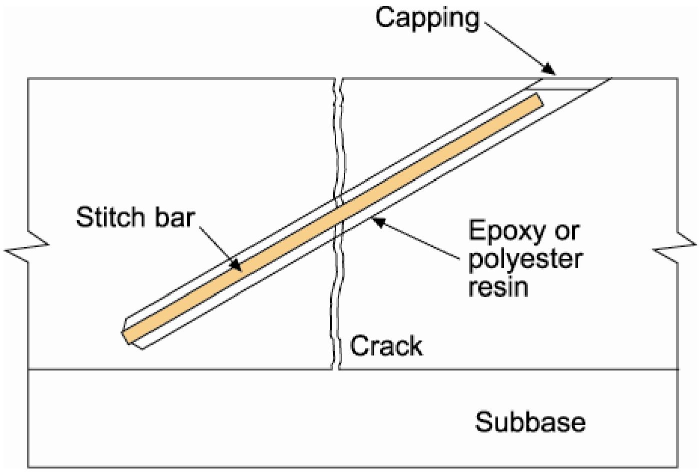
Inappropriate use

- Where slab undersealing does not correct depressions, increase the design structural capacity, or eliminate existing faulting caused by moisture and temperature variations in the slab and traffic loading on the slab.

8.6.3 Slab Cross-stitching

Slab stitching is a process to retain aggregate interlock across joints/cracks, thereby maximising shear load transfer, at joints or cracks, which are considered to have the potential to open under environmental effects and traffic loading. It involves drilling a 30° angle hole from one side of the crack (joint) to the other and inserting a nominal 300 mm to 400 mm length of reinforcing bar (normally Y12) so that it is at least 25 mm below the surface of the pavement. The bar is then fixed in place using epoxy grout as the 'stitch' (Figure 8.10). Its effectiveness relies on it being completed before the crack or joint has opened sufficiently to compromise the shear load transfer.

Figure 8.10: Typical section of a stitched crack



The process is applicable to joints which have been inadequately tied (or where ties have corroded) and for tying kerbs to the pavement.

Spacing of stitches will vary between 400 mm and 1 m depending upon the slab thickness and relief edge distance. The epoxy fixing operation is best carried out during warmer weather or during the middle of the day when the crack will be most tightly closed.

Early stitching will:

- maximise load transfer between slabs hence significantly reduce the chances of secondary slab cracking
- keep out incompressible material and water hence avoid the need for routing and sealing.

Appropriate use

- Pavements where cracks or tied joints which are not normally intended to open, have opened (or are likely to open) due to absence or inadequate performance of tie bars.

Inappropriate uses

Stitching is unlikely to be effective where:

- the slab is extensively cracked
- the concrete has low compressive strength (< 20–25 MPa)
- where base thickness is less than 180 mm.

In doubtful cases, the decision to stitch (in preference to alternatives such as slab replacement) should take into consideration factors such as:

- concrete quality (e.g. is cracking, spalling surface abrasion evident)
- traffic intensity
- previous success under similar conditions.

Further details of slab stitching are given in Roads and Maritime (2015a, 2015b, 2015c).

8.6.4 Full-depth Concrete Patching

Full-depth concrete repairs restore the ability of a transverse crack or joint to transfer loading between slabs and to minimise vertical deflection. The treatment involves the careful removal of distressed concrete pavement by full-depth saw cuts across the slab and along any longitudinal or transverse joints bounding the repair. Full-depth repairs should be equivalent to the main slab in all respects. Regardless of whether the main slab is reinforced or not, it is advisable to reinforce the repair.

Load transfer at the transverse joint is the most critical design feature affecting the performance of the full-depth patch. This is achieved by proper installation of dowel bars of sufficient size and number. This method provides the necessary load transfer across the joints and allows full-depth sawing which minimises the damage to the adjacent slabs. Reliance upon aggregate interlock alone has proven to be particularly unreliable in heavy traffic situations due to spalling, corner breaks etc.

On multi-lane highways, the condition of the adjacent lanes should be reviewed concurrently with the lane being marked for repair. If distress areas in adjacent lanes are similar, it is desirable to align patch boundaries to avoid small offsets, wherever possible, to maintain continuity.

Appropriate uses

- Jointed concrete pavements typically require far more patching at the joints than mid-slab. Depending on severity, the following distress types, which occur at or near transverse joints may require full-depth patching: blow-up, corner break, spalling of concrete, joint load transfer system deterioration, patch/adjacent slab deterioration, inadequate dowel design, misalignment and corrosion or enlargement of the dowel socket under heavy traffic, patch/adjacent slab deterioration.

Inappropriate uses

- Where the distress is limited to the top portion of the slab (thin bonded surface repairs should be considered).
- Where testing indicates the presence of voids beneath the slab (slab undersealing should be considered).
- Where there is the presence of subsurface water or surface water infiltration (drainage improvement or joint and crack sealing should be considered).
- Where testing indicates texture problems (transverse grooving or mechanical roughening).
- Where there are load transfer problems (consider load transfer restoration).
- Where pavement assessment indicates limited service life (structural overlay needed).

Further details are available in Roads and Maritime (2015a, 2015b, 2015c).

8.6.5 Asphalt Overlays

Asphalt overlays may be used to correct surface condition and as strengthening treatments. There are two broad categories of treatment depending on the condition of the concrete pavement prior to overlay:

- asphalt overlays placed on concrete pavements that have received fractured slab treatment as described in Section 8.6.7
- asphalt overlays placed on existing intact concrete pavements.

In relation to the latter case, consideration needs to be given to:

- the need to pre-treat the concrete pavement prior to the overlay
- reflective cracking from the underlying joints and cracks.

Pre-treatments may include using diamond grinding to correct surface condition (Section 8.4.3), joint repairs (Section 8.5), and structural treatments such as slab undersealing (Section 8.6.2) and slab replacements (Section 8.6.4). The asphalt overlay thickness is then designed (Section 12.2) considering these pre-treatments.

8.6.6 Concrete Overlays

Concrete overlay may be used to remedy functional and structural deficiencies of existing concrete pavement.

There are two broad categories of treatment depending on the condition of the concrete pavement prior to overlay:

- concrete overlays placed on concrete pavements that have received fractured slab treatment as described in Section 8.6.7.
- concrete overlays placed on existing intact concrete pavements.

An unbonded concrete overlay over an intact concrete pavement is designed in the same way as a new rigid pavement, using the existing concrete base as a subbase. If the existing concrete base is stable, it is assumed to provide equivalent support to the concrete overlay as would a new lean-mix concrete subbase of the same thickness.

To debond the concrete overlay from the existing rigid pavement and to correct the surface shape, a minimum thickness of 25 mm dense-graded asphalt or geotextile fabric is usually required on the existing pavement.

Section 12.3 describes the thickness design process.

8.6.7 Slab Fracturing Techniques with Overlay

Crack and seating and rubblisation are two types of slab fracturing slab techniques that can be used to inhibit reflective cracking in asphalt overlays of concrete pavements:

- Cracking and seating of concrete pavements is the process of cracking the pavement into approximately 300 mm to 600 mm square sections and then rolling it to push the cracked slabs down onto the underlying pavement.
- Rubblisation pulverises the concrete until it resembles a coarse granular material (Figure 8.11, Figure 8.12). An asphalt overlay is then applied over the fractured and seated slabs to strengthen the pavement.

Figure 8.11: Concrete pavement after being rubblised using a multi-headed breaker

Source : Ceylan et al. (2005).

Figure 8.12: Rubblised pavement surface after rolling

Source: Ceylan et al. (2005).

The intent of the process is to create concrete pieces that are sufficiently small to reduce horizontal slab movement to a point where thermal stresses which contribute to reflective cracking will be greatly reduced, yet still retain some aggregate interlock between pieces. This should ensure that the majority of the original structural strength of concrete pavement is retained. Seating of the broken slabs after cracking is intended to re-establish support between the base and the slab where voids may have existed.

Crack pattern and the sizes of the concrete pieces are the two most important design parameters. Continuous longitudinal cracks tend to reflect through asphalt overlays and should be avoided while the smaller the cracked pieces, the less is the potential for reflective cracking, but the greater is the reduction in structural strength.

The severity of deterioration in the pavement is the principal determinant for selecting this treatment. The assessment of the severity of deterioration should be based on the number of cracked slabs, the degree of load transfer at joints, the condition of joints, the level of subgrade support and presence of voids, pumping or rocking slabs. AASHTO (2015) includes the following severity levels for consideration of rubblisation option:

- extensive distress, structurally inadequate, or
- more than 20% of joints need repair, or
- structural cracking to more than 20% of the area, or
- more than 20% of the area has been previously patched.

Although an expensive process, when concrete patching, joint repair, undersealing, grinding etc. are no longer economically feasible then slab fracturing becomes a viable alternative.

Rubblisation of the concrete results in a layer with significant permeability. Consideration needs to be given to the drainage of this rubble.

Prior to the overlay placement, a levelling course may be needed to restore shape.

In relation to the thickness of asphalt overlays, Khazanovich et al. (2012) discuss various methods.

Appropriate use

- For a concrete pavement with a substantial number of faulted cracks, a significant loss of load transfer with associated faulting at joints or cracks, shearing of longitudinal tie bars, or any combination of these distress manifestations.

Inappropriate uses

- In locations close to culverts, underground ducts, and drainage pipes.
- Where the process would result in an unacceptable reduction in the overall structural strength of the concrete slabs the determination of which depends on:
  - the characteristics of the original pavement
  - the size of the cracked concrete pieces
  - the construction equipment and methods used
  - the existing subgrade or base support.
- Where the subbase is not sufficiently firm to prevent continuing vertical movement between the concrete fragments.

8.7 Treatments for Pavements on Expansive Subgrades

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 6.8. A number of strategies to minimise volume changes in highly expansive soils are discussed in Austroads (2018a).

8.8 Design and Construction Considerations

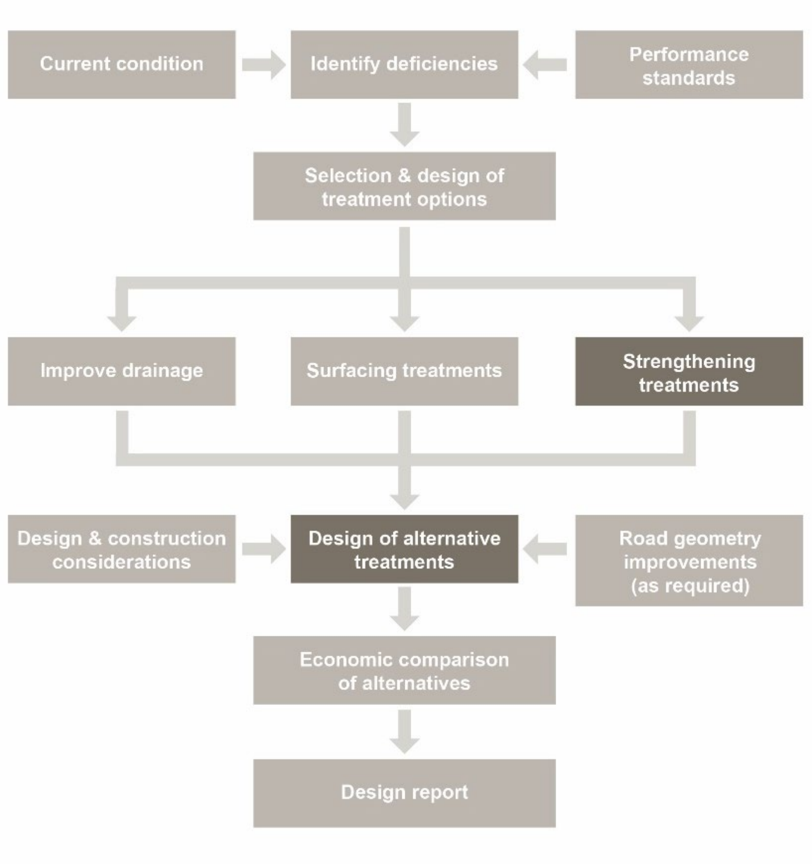
In relation to the design and construction considerations for rigid pavements treatment, these are similar to those discussed for flexible pavements in Section 7.7.

9. Empirical Design of Granular Overlays for Flexible Pavements

9.1 Introduction

This section describes procedures for determining the thickness of granular overlays on granular pavements to rectify structural deficiencies of an existing pavement (Figure 9.1). As the design method does not consider fatigue cracking, it assumes a thin (< 40 mm) bituminous surfacing is provided after the overlay.

Figure 9.1: Design steps discussed in Section 9



Such treatments may often be placed on pavements for other reasons that relate to the rectification of the functional characteristics of pavements, such as shape, ride quality and surface competency. The structural adequacy of these treatments also needs to be considered.

The design procedure is based on empirically-derived design deflections, which provide the allowable design traffic in terms of rutting and shape loss of these pavements. In addition, a design chart is provided to estimate the thickness of granular overlays required to reduce the measured deflections to the design deflections. Granular overlay requirements for design traffic loadings up to 10^8 ESA can be derived using these design charts.

This design chart procedure assesses the overlay requirements for permanent deformation on the pavement surface using the measured maximum deflection. It does not make any provision for a limitation on the allowable traffic loading considering fatigue of any bound layer (e.g. an asphalt surfacing). If flexible pavements include bound materials, the mechanistic-empirical procedure is used (Section 10).

The design procedures have been developed to utilise maximum deflections (D_0) measured by the following testing devices:

- 80 kN single axle with dual wheels (tyre inflation pressure 550 kPa) as measured by a Benkelman Beam
- 80 kN single axle with dual wheels (tyre inflation pressure 750 kPa) as measured by a Deflectograph
- 50 kN dual wheel load (tyre inflation pressure 760 kPa) as measured by a Traffic Speed Deflectometer (TSD)
- Falling Weight Deflectometer (FWD) loading with a plate diameter of 300 mm and an applied load of 40 kN (plate contact stress of 566 kPa).

Details of specific deflection measurement requirements are provided in Section 4.8.

Layer thickness derived from the procedures should be considered to be minimum requirements and do not consider construction tolerance.

The assumptions used in the development of these procedures are detailed in Austroads (2008a). The methodology only applies to pavements that are rehabilitated to quality standards specified by Austroads Member Agencies.

Marginal or non-standard materials should only be used as the overlay material after consideration of:

- the documented performance history of the proposed material
- costs relative to standard materials
- the design traffic loading
- the climate at the site
- the moisture sensitivity of the subgrade
- the quality and uniformity of the materials as shown by laboratory testing
- consequences of poor performance
- suitability and cost-effectiveness of mechanical or chemical stabilisation.

Non-standard materials commonly have lower moduli than standard granular materials, so greater thickness is required to reduce the stresses/strains on the subgrade. However, it is not always possible to obtain equivalent performance by using thicker layers of non-standard materials. The use of the non-standard materials may result in inferior performance due to deformation within the pavement layers under traffic loading leading to rutting and early pavement distress. Hence, the performance history of the proposed material is an important consideration.

9.2 Characteristic Deflections

9.2.1 General

The overlay design procedures are based on characteristic deflections calculated from the measured maximum deflection (D_0), summarised as follows:

- deflections are adjusted for seasonal moisture variations
- as deflections may be measured by Benkelman Beam, Deflectograph, FWD or TSD, they need to be standardised using Table 9.2
- the adjusted deflections are then divided into homogeneous sub-sections
- characteristic values of deflection are calculated for each sub-section.

Hence, the characteristic deflections for a pavement sub-section, by definition, include all the appropriate seasonal and standardisation adjustments.

9.2.2 Adjustment of Deflections to Account for Seasonal Moisture Variations

Due to seasonal moisture variations, deflections may need to be adjusted to represent the pavement in its weakest condition. In some localities, increases in deflection during or soon after wet seasons are common due to higher moisture contents within the pavement layers or subgrade.

However, the seasonal effects of moisture usually cannot be defined by a straightforward rainfall/deflection relationship. Adequate consideration should be given to the main causes of changes in the moisture regime, which include surface infiltration, permeability, moisture susceptibility of the pavement layers and subgrade, site evaporation, and drainage conditions. The evaporation and drainage conditions limit the quantity of water available to infiltrate the subgrade, while the permeability controls the rate of infiltration, and consequently, the time lag between significant rainfall and the effects on deflection.

Deflection measurements should be undertaken when the subgrade is in its weakest state. When testing occurs at other periods in the year the deflection data should be corrected to simulate this condition. The magnitude of seasonal correction factors that need to be applied to deflections will depend on a number of influences, such as:

- subgrade type
- climate
- proximity to the groundwater table
- extent and effectiveness of the drainage system.

If data on the seasonal variation of deflections is available for regional areas and this could be used for other roads where similar conditions apply.

Table 9.1 provides moisture correction factors for deflections and curvatures where better information is not available. These factors have been derived for pavements with unsealed shoulders and clay subgrades with fair drainage conditions. Caution should be exercised in applying them to all situations as permeable subgrades (e.g. silts) may have significantly greater changes than indicated in Table 9.1. Maximum deflections are adjusted by multiplying the measured values by the appropriate moisture correction factors.

Table 9.1: Maximum deflection correction factors for seasonal moisture

Winter and spring rain (temperate climates)		Summer rain (tropical and sub-tropical climates)	
Month when deflections are measured			
January to April(1)	May to December	June to December(1)	January to May
1.3	1.0	1.3	1.0

1 If the watertable is less than 3 m below the surface the correction factor is 1.0.

9.2.3 Standardisation of Deflections

As the maximum deflections of a pavement test site measured by Benkelman Beam, Deflectograph, FWD and TSD differ, it is necessary to standardise the measured values.

In terms of maximum deflections, the design deflections (Figure 9.2) are based on values measured using a Benkelman Beam (Austroads 2008a). Hence, the values measured with a Deflectograph, FWD and TSD need to be converted to equivalent Benkelman Beam values. As the standardisation factors required for these conversions vary with pavement composition and subgrade strength, the most accurate factors are those obtained by paired field measurements. In the event practitioners elect not to undertake such correlation studies, presumptive standardisation factors are provided in Table 9.2. Note that the TSD factors are based on a measurement speed of 40–80 km/h, relate to pavement deflections estimated by the area under the deflection slope curve method (Section 4.9.2) and are applicable deflections of thin bituminous surfaced granular pavements.

Table 9.2: Deflection standardisation factors

Deflection measurement device	Deflection standardisation factor
Deflectograph, 80 kN single axle with dual tyres	1.2
TSD, 50 kN dual tyres	1.2
Falling Weight Deflectometer, 40 kN load	1.1

When deflections are measured using FWD the applied contact stress may differ the values of 566 kPa associated with a 40 kN load. In such cases, the deflections are linearly adjusted to values associated with a contact stress of 566 kPa (Equation 3) before being standardised using the factor in Table 9.2.

$$D_{Ref} = D_x \left(\frac{\sigma_{ref}}{\sigma_x}\right) \quad (3)$$

where

D_{Ref} = deflection at contact stress of reference stress (mm)

D_x = deflection at applied plate contact stress (mm)

σ_{ref} = reference plate contact stress, 566 kPa (kPa)

σ_x = applied plate contact stress during measurement (kPa)

Similarly, for the TSD the reference dual-tyre load is 50 kN. The measured dynamic dual-tyre load may vary from 50 kN. In such cases, the deflections are linearly adjusted from the load during deflection measurement to the reference load using Equation 4 before being standardised using the factor in Table 9.2:

$$D_{ref} = D_x \left(\frac{L_{ref}}{L_x}\right) \quad (4)$$

where

D_{ref} = deflection at the dual-tyre load (mm)

D_x = deflection at applied dual-tyre load (mm)

L_{ref} = reference dual-tyre load, 50 kN

L_x = applied dual-tyre during measurement (kN)

9.2.4 Adjustment of Maximum Deflections to Account for the Testing Temperature

The empirical design method is used for assessing the overlay requirements of thin asphalt-surfaced granular pavements. In such cases the temperature of thin surfacing during deflection testing does not significantly affect the measured maximum deflections.

However, the procedures described in Section 9.2.5 to identify homogeneous sections may also be used for pavements with thick asphalt layers as described in Section 10.4. For such pavements, any significant variations in pavement temperature during the deflection measurements needs to be considered in the identification of homogeneous sub-sections. In such cases, the deflections need to firstly be adjusted from the pavement temperature at the time of testing to the in-service conditions.

The in-service pavement temperature at a site is characterised by the weighted mean annual pavement temperature (WMAPT). WMAPTs have been determined for selected locations throughout Australia and New Zealand (Appendix B) so that for any particular site the WMAPT of the nearest or most appropriate location should be adopted, or calculated using the method given in Appendix B.

Appendix C describes the process to adjust the maximum deflections.

9.2.5 Selection of Homogeneous Sections

The structural capacity of a pavement that requires an overlay typically varies both longitudinally and transversely over the project alignment. Therefore, to design cost-effective overlays, it is usually necessary to divide the project into sub-sections that have relatively uniform strength, which is indicated by the measured deflections. However, varying the overlay thickness for short sections may be an impractical rehabilitation strategy and the pre-treatment of localised areas of high deflection is generally more economic.

Examination of deflection trends and site conditions in conjunction with other relevant data can assist in identifying uniform sub-sections. Information that is often useful includes:

- subgrade type and variability
- drainage
- groundwater conditions
- topography
- construction and maintenance history, including pavement age
- pavement composition (material types, quality and layer thickness)
- extent, type and severity of pavement distress
- design traffic.

Poor selection of homogeneous sub-sections can result in excessive overlay requirements. It is generally recommended that sub-sections should exceed 100 m in length and should be considered homogeneous if the deflection values have a coefficient of variation CV (i.e. standard deviation divided by mean) of 0.25 or less. If this value exceeds 0.25, the sub-section may need to be further subdivided. The cumulative difference approach (Appendix D) may be useful in the identification of homogeneous sub-sections.

9.2.6 Calculation of Characteristic Deflections

For the design of flexible overlays on flexible pavements, a characteristic deflection is assigned to each sub-section for evaluation purposes. These values are determined after the seasonal and standardisation adjustments have been made to the individual test measurements. However, an equivalent outcome would also result if the adjustments were applied only to characteristic deflections rather than individual test site data, provided the same adjustments apply to all data in the sub-section.

The characteristic deflection (CD) for a homogeneous sub-section of pavement is defined as value exceeded by only 10% of the measured values. Where the number of deflection measurements in a homogeneous sub-section is less than 10, the maximum value is used as the CD. Otherwise, the CD is equal to the average deflection (μ) plus a factor f times the standard deviation (SD) (Equation 5) assuming the maximum deflections follow a normal distribution.

$$CD = \mu + f \times SD \qquad (5)$$

where

f is selected by the designer to provide a 10% probability of the characteristic value not being exceeded by an individual value

The recommended values of 'f' in Table 9.3 are given for guidance only. Alternative procedures may be used if the distribution of deflections is not consistent with a normal distribution.

In summary, the CDs are the fully adjusted values of deflection that is a useful index for the structural adequacy of a particular section of pavement.

Table 9.3: Recommended values for '*f*'

Number of deflection measurements *	*f* *
10	1.38
12	1.36
14	1.35
16	1.34
19	1.33
24	1.32
≥ 30	1.31

** After identifying areas to be patched/reconstructed.*

9.3 Design Periods and Traffic Loading

For granular overlays, the design traffic loading is expressed in terms of equivalent standard axles (ESA) as described in Section 7 of Austroads (2018a). Typically, design periods of 10–20 years are used for granular overlays.

While the design procedure includes a general consideration of the usual variability associated with materials and the construction process, there will always be a risk that the pavement will reach the end of its service life before the design period has elapsed. This risk is attributed to, among other things, the uncertainty associated with predictions of the traffic volume and the magnitude of axle group loads over the design period, the inaccuracies associated with the use of average values in estimations of material properties, layer thicknesses etc. and the variability in these parameters likely to occur within a project.

Situations arise where the designer or asset manager may wish to reduce this risk. A simple method of achieving this is to adopt a design traffic loading higher than that which is anticipated. Increasing the expected traffic loading by a factor of up to two may be warranted for some projects.

9.4 Design Deflections

The pavement deflection performance criteria developed by Austroads for the design of granular overlays are only applicable to flexible pavements that do not include asphalt and/or cemented materials. Where these materials are present, the structural performance of the flexible pavement is unable to be accurately determined by deflections within a simplified design chart procedure. The mechanistic-empirical procedures (refer to Section 10) are used to design strengthening treatments on pavements containing asphalt and/or cemented materials.

For the design of granular overlays on granular pavements, the principal criterion is the limitation of permanent deformation (e.g. rutting) on the pavement surface. Satisfactory pavement performance in terms of permanent deformation under the design traffic should be achieved by limiting the CD on opening to traffic to a value no greater than the design deflection. By limiting the CD in this manner, permanent deformation should be kept to an acceptable level.

The Austroads relationship between design deflection and design traffic loading predicted using Equation 6 and Equation 7 is shown in Figure 9.2 and may be used for unbound granular pavements with thin bituminous surfacing. The design deflections are for use with maximum deflection under an 80 kN single axle load as measured by Benkelman Beam (Section 4.9.2).

Design traffic less than 10^6 ESA:

$$D = 3.666 - 0.422 \log N \qquad 6$$

Design traffic of 10^6 ESA or more:

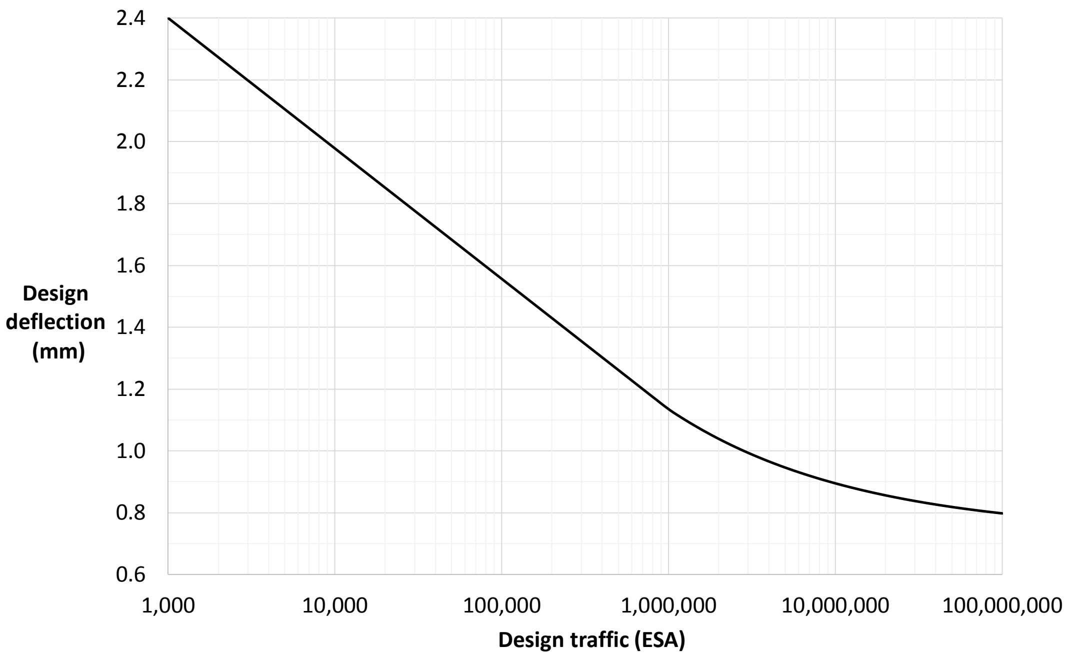
$$D = 0.731 + 91.202N^{-0.3924} \quad (7)$$

where

D = design deflection (mm)

N = design traffic loading (ESA)

Figure 9.2: Design deflections to limit permanent deformation



9.5 Determination of Granular Overlay Thickness

Granular overlays are usually placed on existing granular pavements and are typically surfaced with a bituminous seal or a thin (< 40 mm) layer of asphalt. These existing surfacings are commonly removed prior to the placement of the granular overlay. After construction of the overlay, a new thin bituminous surfacing is provided.

If the existing pavement has a thin bituminous seal within 300 mm of the finished surface level, it is desirable in wetter climates to remove or break the seal to ensure that water is not trapped in the overlay.

The CD is compared to the design deflection obtained from Figure 9.2. If the CD is less than or equal to the design deflection, an overlay is not required to strengthen the pavement. Otherwise, Figure 9.3 is used to obtain the overlay thickness to reduce the CD to the design deflection.

The design chart given in Figure 9.3 is based on available field data that indicates a 6% reduction in deflection for each 25 mm of granular overlay thickness where no other works are undertaken. If poor drainage has contributed to the condition of the existing pavement, then greater reductions in deflections may result from significant drainage improvements.

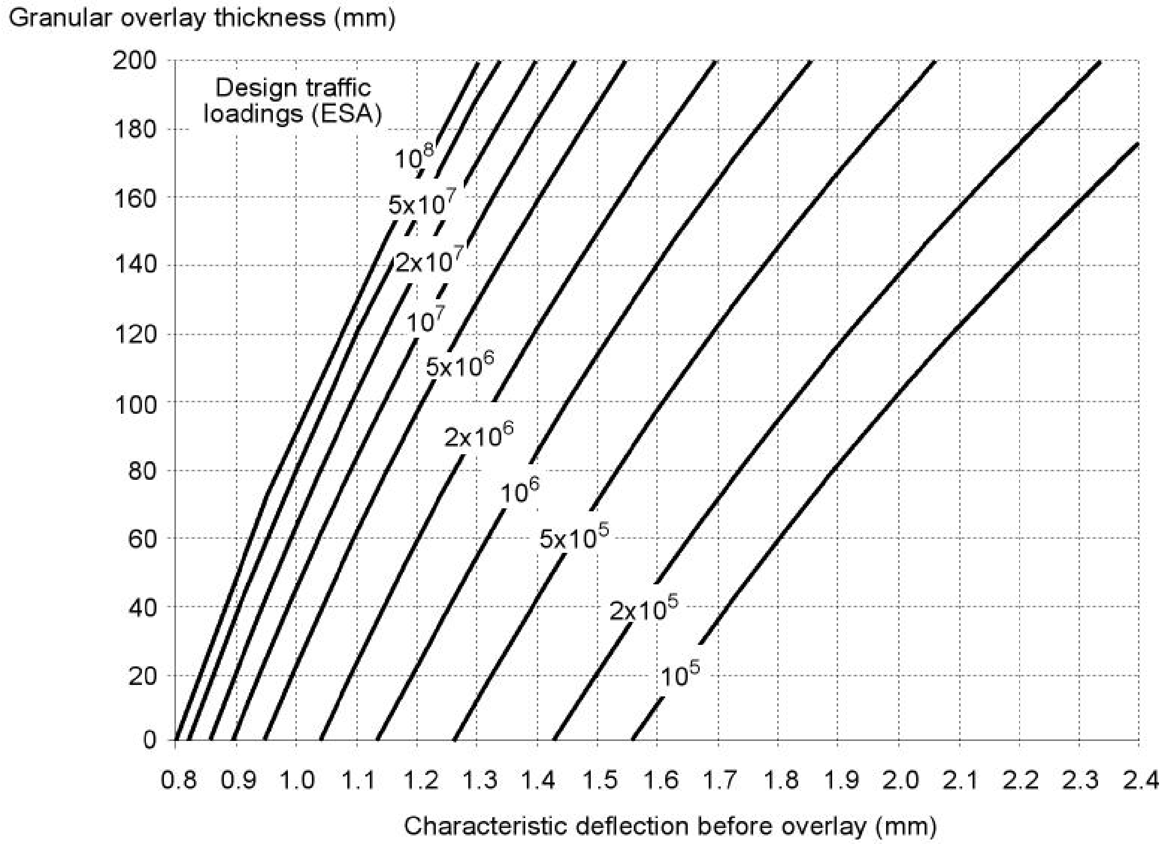
The calculated overlay thicknesses are based on the use of granular materials typical of those used by the Austroads member agencies and described in Austroads (2018a) and Austroads (2008c).

These design procedures are detailed in Table 9.4 and Figure 9.4 in flow chart form. A calculation worksheet is provided in Appendix E.

Table 9.4: Procedure for granular overlays

Step	Activity
1	Calculate the design traffic loading in ESA
2	Measure the pavement deflections
3	Adjust the measured maximum deflections for seasonal moisture variations (refer to Section 9.2) and standardised to Benkelman Beam values by multiplying by the factors in Table 9.2
4	Divide the test length into homogeneous sub-sections and calculate the characteristic deflection (CD) for each sub-section
5	Using the design traffic loading (Step 1), obtain the design deflection from Figure 9.2 or Equation 6 and Equation 7
6	Compare the CD to the design deflection. If the CD is less than or equal to the design deflection, an overlay is not required. Otherwise, proceed to Step 7
7	Using the CD (Step 4) and Figure 9.3, estimate the overlay thickness required for the design traffic loading

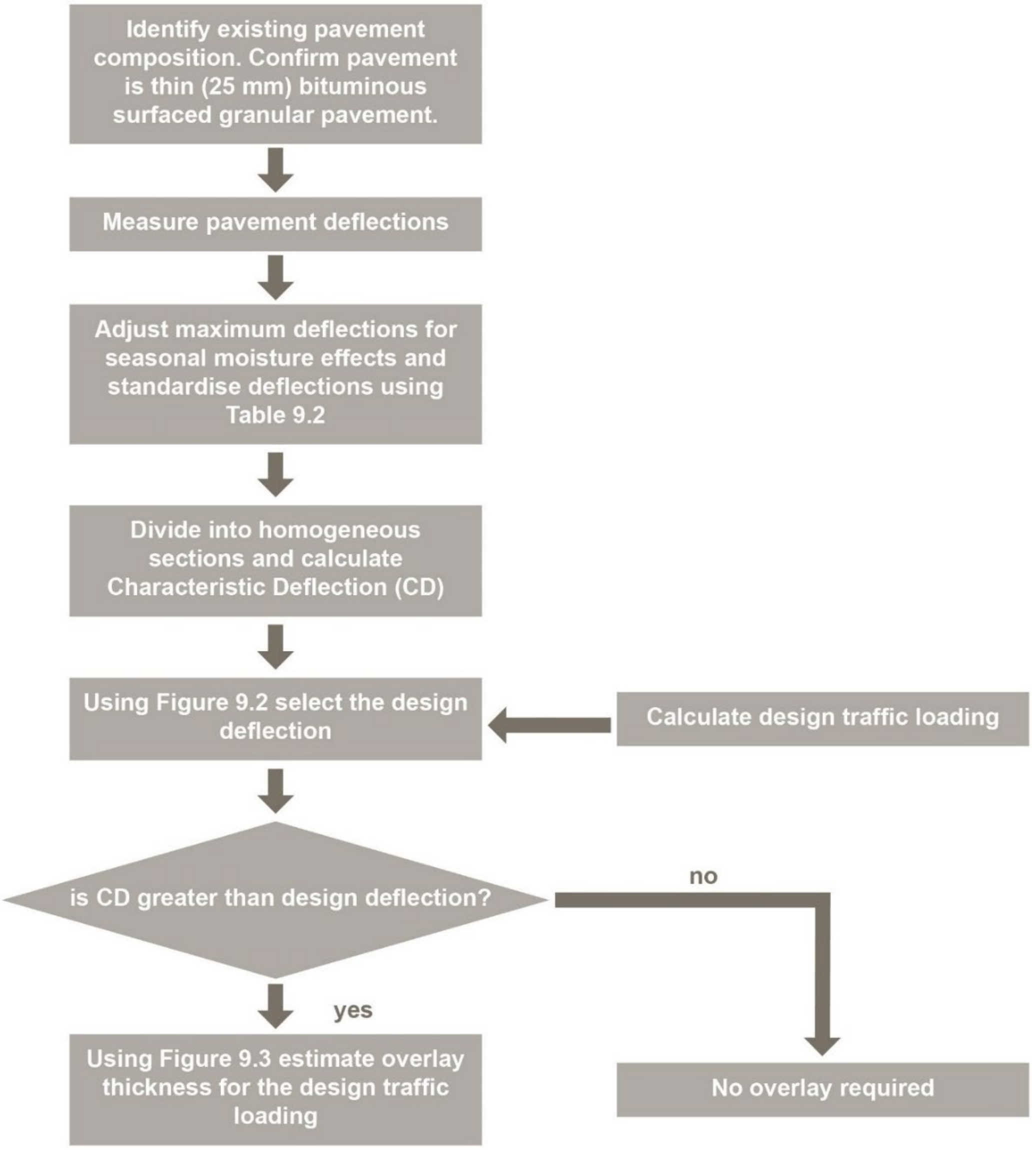
Figure 9.3: Granular overlay design charts



Design example

An example of use of the design procedure is provided in Appendix F.

Figure 9.4: Flow chart for granular overlays on thin bituminous surfaced granular pavements without bound materials



10. Mechanistic-empirical Procedure of Designing Strengthening Treatments for Flexible Pavements

10.1 Introduction

This section describes procedures for determining the design thicknesses of structural treatments constructed to rectify distress and structural deficiencies of an existing flexible pavement (Figure 10.1).

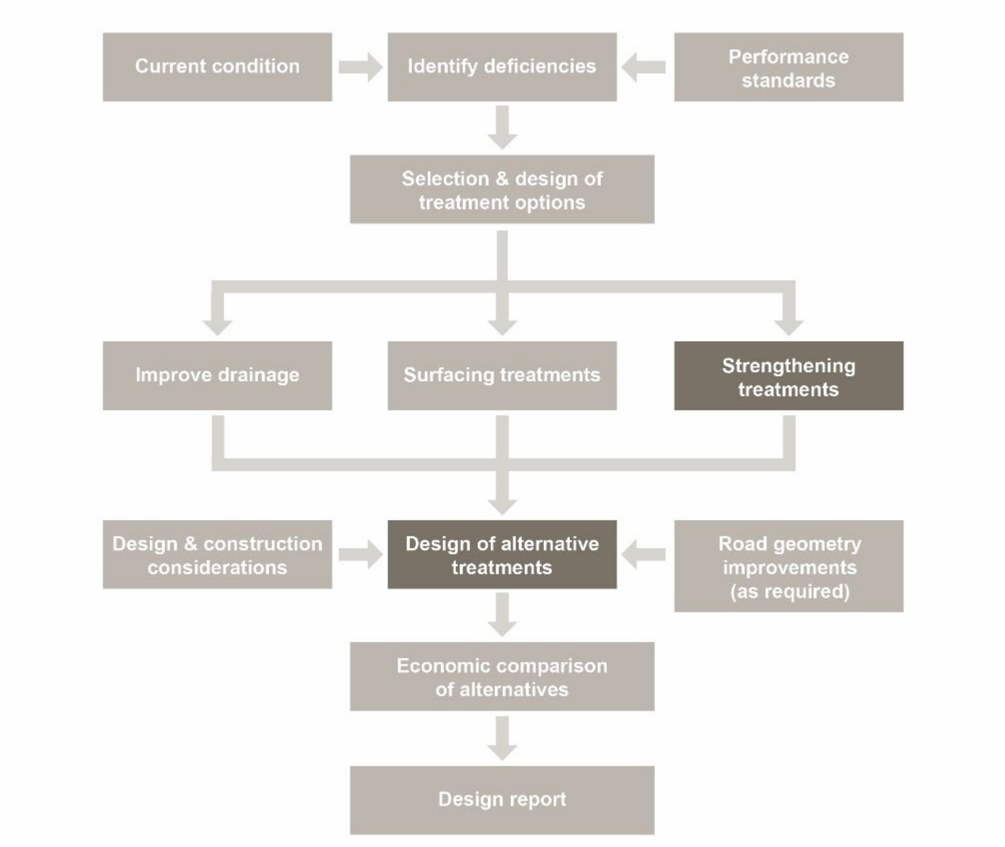
Such treatments may often be placed on pavements for other reasons that relate to the rectification of the functional characteristics of pavements, such as shape, ride quality and surface competency. The structural adequacy of these treatments also needs to be considered.

This section details a mechanistic-empirical procedure (MEP) applicable to all types of structural treatment for flexible pavements other than the use of concrete overlays and inlays (see Section 11). The design process is similar to that used to design new pavements (Austroads 2018a), except that the design moduli of the existing pavement materials and subgrade are estimated. Deflection testing may be used to derive these design moduli and to select homogeneous sub-sections along the length of the existing pavement.

The MEP provides the designer with the capability of designing a broad range of pavement strengthening treatments, including asphalt overlays, inlays, major patching and stabilisation of pavement layers and subgrade.

Layer thicknesses derived from the MEP should be considered to be minimum requirements and take no account of construction tolerance. This is particularly critical for bound pavement thicknesses.

Figure 10.1: Design steps discussed in Section 10



10.2 Mechanistic-empirical Procedure

The MEP for the design of strengthening treatments is, in principle, identical to the mechanistic-empirical procedure for the design of new pavements (Austroads 2018a), except that there is an initial phase in which the properties of the materials in the existing pavement are determined.

Asphalt overlays and inlays, foamed bitumen stabilisation, cement stabilisation and lean-mix concrete (LMC) are designed to limit fatigue cracking in each treatment layer and permanent deformation of the pavement. Granular overlays with sprayed bituminous surfacings are designed only to limit permanent deformation of granular layers and subgrade.

Bound materials (asphalt, cemented materials, LMC, foamed bitumen stabilised materials) within the existing pavement could be expected to have little or remaining fatigue life when a structural treatment is being designed. Hence, the design method described in Section 10.10 does not address the strengthening requirements that inhibit fatigue cracking of the existing bound layers. Strengthening treatments are designed to limit fatigue cracking in the bound treatment materials and permanent deformation of the treated pavement.

As there can be a significant risk of cracking from the existing bound materials reflecting through an overlying treatment, careful consideration of this distress mode is required as reflective cracking rather than fatigue cracking may limit the service life of the treatment. To reduce the risk of reflective cracking in the design of new pavements (Austroads 2018a), a minimum cover equivalent to 175 mm of sound asphalt over cemented material or LMC is provided. As described in Austroads (2018a), granular materials can be used as cover either solely or in conjunction with asphalt. In the design of rehabilitation treatments consideration should be given as to whether the treatment will inhibit reflective cracking to the required extent over the design period. If the risk of reflective cracking within the design period is acceptable, the post-cracking life of bound materials may be considered.

Alternatively, if it is anticipated that reflective cracking will limit the life of the treatment, the design traffic used in assessing structural adequacy in terms of fatigue and permanent deformation may be reduced to consider this reflective cracking life. For instance, consider a trial treatment involving surface milling, placement of geotextile reinforced seal and a polymer modified stone mastic asphalt. Past experience in the vicinity of the project indicates that this treatment lasts for 10 years before reflective cracking necessitates retreatment. In this case, the MEP is used to check if the allowable traffic loadings in relation to fatigue of the inlay and pavement permanent deformation exceed the design traffic over 10 years. The whole-of-life costs of this treatment may then be compared to structural treatments designed to outlast traffic over design periods of 20 years or more, the service life of which are not being limited by reflective cracking.

This MEP is intended to be used to design strengthening treatments for pavements that have been in-service a number of years. The MEP must not be used to assess the structural adequacy of pavements within three years of opening to traffic. Newly-constructed pavements can take years for the pavement and subgrade to reach stable, equilibrium conditions with the local environment. As such, the use of design moduli based on in situ testing can be misleading. For newly-constructed pavements, the procedures for the design of new pavements (Austroads 2018a) are used to determine structural adequacy.

The MEP requires the design moduli for existing pavement layers and the subgrade to be estimated as accurately as possible. The methods described in Section 10.4 and Section 10.5 can be used for this purpose.

10.3 Design Periods and Traffic Loadings

The design traffic loading is expressed in terms of cumulative number of repetitions of axle loads of each axle group type as described in Section 7 of Austroads (2018a). Typically, design periods of 10–20 years are used for strengthening treatments. With certain combinations of overlay thickness and existing pavement characteristics, the onset of fatigue cracking may occur well before the permanent deformation criterion is exceeded. In such cases, a shorter design period may be employed for fatigue than deformation if this results in lower whole-of-life costs. The rationale is that as permanent deformation occurs deeper in the pavement than fatigue cracking, which is limited to the overlay itself, it is more difficult and expensive to rectify. There is also a practical issue about the normal service life expectancy of asphalt surfacings, which for dense-graded asphalt averages between 8 and 20 years (refer Table 6.7).

As described in Section 10.2, for a given project, the design period and associated traffic loading may vary between treatment options, in which case whole-of-life costs are used to evaluate the cost-effectiveness of the options.

10.4 Selection of Homogeneous Sub-sections

The structural capacity of a pavement that requires rehabilitation typically varies both longitudinally and transversely over the project alignment. Therefore, to design cost-effective structural treatments, it is usually necessary to divide the project into sub-sections that have relatively uniform strength, which is indicated by the measured deflections. This process is described in Section 9.2.

Deflection data can also be used to identify sites for other pavement investigations or to directly calculate moduli of the existing pavement (refer Section 10.5).

For pavements with total asphalt thicknesses more than 25 mm, the maximum deflections need to be adjusted from the measurement temperature to the WMAPT prior to the selection of the homogeneous sub-sections (Section 9.2.5). Appendix C provides the temperature adjustment method.

10.5 Back-calculation of Moduli from Measured Deflections

10.5.1 Introduction

The moduli of the existing pavement layers and subgrade may be determined from the measured FWD surface deflections and layer material properties in a process known as back-calculation. Back-calculation is a methodical trial and error approach used to determine the combination of layer moduli such that the predicted deflection bowl best matches the measured deflection bowl.

The FWD pavement surface deflection bowls are measured with a plate diameter of 300 mm and an applied load selected according to the stiffness of the pavement. A larger diameter plate (450 mm) is available for testing on the top of earthworks materials. Details of specific deflection measurement requirements are provided in Section 4.9.

Numerous back-calculation routines are available for estimating layer moduli. In selecting an analysis routine, it should be noted that the rehabilitation treatment design method is based on mechanistic-empirical procedures similar to those used for the design of new pavements (Austroads 2018a). The allowable traffic loadings are determined from the critical strain responses calculated from the linear elastic model (e.g. CIRCLY, Mincad Systems 2009). The accuracy of the analysis is likely to be greater if the linear elastic model used to back-calculate the existing pavement and subgrade moduli is the same as used in the design of new pavements (Austroads 2018a).

An Austroads algorithm for back-calculation of moduli is described in Austroads (in press), which is an enhancement of the ARRB back-calculation computer program EFROMD2 (Elastic properties FROM Deflections) software (Vuong 1991). This process automatically iterates the layer moduli of an input pavement configuration that best represents the existing pavement structure and subgrade. However, it is also possible to manually iterate a design solution using a linear elastic model directly. This Austroads algorithm requires the back-calculation of granular and subgrade anisotropic moduli consistent with the design of new pavements (Austroads 2018a). In the event that the back-calculation program used is limited to isotropic modulus values, the isotropic moduli for granular and subgrade materials need to be converted to anisotropic values such that the horizontal modulus (E_h) is half the vertical value (E_v). The vertical modulus of a layer may be approximated by multiplying the back-calculated isotropic modulus by a factor of 1.1 and the associated horizontal modulus ($E_h = E_v/2$).

In the selection of design moduli of unbound and modified granular materials (Section 10.7.4), the total thickness is divided into five sub-layers and a modulus ratio (R) is calculated from the ratio of the modulus of the top sublayer and the subgrade modulus. The ratio R then enables the modulus of the other granular sublayers to be calculated. If the back-calculation method allows the option of modelling the total granular thickness consistent with this design modulus method this simplifies the granular modulus characterisation. In this option, the back-calculation method may only iterate the modulus of the top granular sublayer, and the other sublayers are derived using R. However, this option is not recommended if the treatment involves partial removal of granular materials. In this case, calculation of base and subbase moduli may be more appropriate from which the equivalent moduli (Equation 11) after treatment can be calculated.

Similarly, some back-calculation methods provide an option for five sub-layers characterisation of each selected subgrade/lime-stabilised subgrade layer.

In relation to the modulus seed values to commence the iteration process, the values selected for the subgrade layers are particularly important. Consideration needs to be given to all available information including DCP results as to whether the subgrade moduli are increasing or decreasing with depth. In this regard the calculation of the composite modulus (Appendix G) at each measured deflection offset may be of use. The composite modulus at measured deflection offsets exceeding the depth from the pavement surface to the top of the subgrade is indicative of the variation in subgrade moduli with depth. For instance, if the composite modulus increases with depth, consideration could be given to constraining the back-calculated subgrade moduli in the same manner.

The back-calculation of the modulus is undertaken on measured FWD deflections without correction for temperature and seasonal moisture contents. Consideration of these influences is undertaken in estimating the design modulus (Section 10.7).

10.5.2 Selection of Deflection Bowls for Modulus Back-calculation

The existing pavement and subgrade moduli relevant for the design of rehabilitation treatments are those with deflection bowls close to the CD values for each homogeneous sub-section (Section 10.4).

A useful adjunct to this procedure is to analyse those deflection bowls close to the CD to identify bowls of similar shape using Statistical Cluster Analysis (SCA) as used by Queensland Department of Transport and Main Roads (2015). Following these bowl groups, the average deflection bowl within each group is calculated and used in the modulus back-calculation. This has the advantage of reducing random errors in bowl measurements.

The practice of back-calculating the modulus at each deflection test site and then calculating characteristic moduli for each layer is not recommended. When deflection bowls are predicted using moduli so determined, the predicted deflections are commonly well in excess of the measured values. Hence this modulus selection process tends to underestimate the structural adequacy of the pavement.

In the event that Deflectograph or TSD deflections have been measured, it is useful to analyse this data before undertaking FWD testing. The analysis involves determination of the homogeneous sub-sections and calculation of the CD for each sub-section. FWD deflection testing is most useful at sites with Deflectograph or TSD deflections close to the CD value as these back-calculated moduli are used in designing strengthening treatments.

For projects with more uniform characteristics where only FWD deflection bowls are measured, the lengths of homogeneous sub-sections are commonly longer. Again, the first step is to identify sites at which the measured FWD deflections are close to the CD of each homogeneous sub-section: these are the sites for back-calculation of moduli.

At selected FWD sites, asphalt coring/pits may be excavated to determine the existing asphalt and cemented materials' thicknesses for use in the back-calculation model. If pits are excavated, then pavement layers may be assessed for material type, quality and the layer thicknesses for use in the modulus back-calculation. In selecting the sites for pits, including distressed and sound areas may assist in determining the cause of the distress. The SCA process may also be useful in site selection.

At coring/pit sites, the in situ subgrade CBR may be estimated using DCP testing (refer to Section 5.4). These estimates of in situ CBR provide a valuable check on the back-calculated subgrade moduli.

10.5.3 Pavement and Subgrade Configuration

In terms of the pavement layers and subgrade layers used in the back-calculation model, the following guidance is provided:

- The total number of pavement layers and subgrade layers to be iterated in the model should not exceed the number of geophones used to measure the bowl.
- There is insufficient information in the measured FWD deflection bowls to back-calculate the modulus of layers 75 mm or less in thickness. Accordingly, for thin asphalt surfacings on granular pavements, the modulus of the thin asphalt layer should be set to a fixed value in the back-calculation model rather than iterated. The modulus value used is selected considering the modulus used in the design of new pavements (Austroads 2018a), adjusted for current in situ condition and age.
- If the total thickness of asphalt layers is 75 mm to 150 mm, a single asphalt modulus should be back-calculated representing the composite value. If the total thickness exceeds 150 mm, the model may enable the upper and lower asphalt modulus values to be back-calculated.
- If the total thickness of unbound and modified granular materials is 150 mm or less, the single modulus should be back-calculated representing the composite value. If the total thickness of granular materials is 150 mm to 500 mm, the model may enable the modulus of two layers, commonly the base and subbase, to be back-calculated separately. If the total thickness of granular materials exceeds 500 mm, the model may enable back-calculation of three granular layers, commonly base, upper subbase and lower subbase. As mentioned in Section 10.5.1, some back-calculation methods enable the total thickness of unbound and modified granular materials to be sub-layered consistent with the method used to derive design moduli (Section 10.7.4). In this case, the above guidance on number of layers does not apply.
- If the pavement includes selected subgrade materials or lime-stabilised subgrade, a single composite modulus value is commonly back-calculated. However, if the total thickness of these materials exceeds 300 mm, the model may provide for back-calculation of upper and lower modulus values. As mentioned in Section 10.5.1, some back-calculation methods enable the total thickness of each selected subgrade or lime stabilised subgrade to be sub-layered consistent with the method used to derive design moduli (Section 10.7.3). In this case, the above guidance on number of layers does not apply.
- The in situ subgrade needs to be sub-layered in the model. Where the top of the subgrade is within 500 mm of the surface, the preferred subgrade characterisation in the model is three sub-layers: upper subgrade layer 300 mm thick, intermediate subgrade layer 500 mm thick and lower subgrade of semi-finite thickness. If the top of the in situ subgrade is more than 500 mm below the surface, the preferred subgrade characterisation in the model is two sub-layers: upper subgrade layer 300 mm thick and lower subgrade of semi-infinite thickness. As described in Appendix G, calculation of the composite modulus may be useful in selecting the initial subgrade moduli to commence the back-calculation.
- Note that in the back-calculation model, the assigned values of Poisson's ratio and degree of anisotropy need to be consistent with those adopted for the design of new pavements (Austroads 2018a). In particular, unbound granular materials and subgrade layers should have a degree of anisotropy of two.

- In the event that the back-calculation software used is limited to isotropic modulus estimation, the isotropic modulus values calculated for subgrade, selected subgrade material and granular moduli need to be corrected to anisotropic values for determination of design moduli (Section 10.7). For each material, the vertical modulus may be approximated by multiplying the back-calculated isotropic modulus by a factor of 1.1 and the associated horizontal modulus (Eh = Ev/2).

10.6 Selection of Trial Treatment

10.6.1 General

Section 7.5 describes strengthening treatments for flexible pavements. The trial treatments selected for evaluation need to consider the objective of the rehabilitation treatment and the causes of current distress (Section 6).

The design process involves specifying the pavement materials to be used including existing materials, the thickness of each material and the relative positions of these materials in the pavement.

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

If an asphalt overlay/inlay is being designed, the trial thickness needs to consider:

- the minimum thickness required to correct pavement roughness, rutting and surface drainage/crossfall
- the proposed milling depth (if any) required to restore pavement shape, remove substandard material or surface cracking
- whether the service life of the overlay/inlay will be limited by reflective cracking (Section 10.2).

10.6.2 Cementitious Stabilisation of Pavement Layers

For lightly trafficked roads the thickness of the stabilised layer is generally between 150 mm and 250 mm.

For pavements carrying high levels of traffic (e.g. freeways and highways), pavement stabilisation in a single layer exceeding 300 mm is possible through the use of 'deep-lift' recyclers, specialised binder spreaders, slow-setting binders and high-performance compaction equipment. However, not all road agencies allow the thickness to exceed 300 mm in a single layer. Furthermore, the maximum thickness may be limited by the type or availability of plant, subgrade/pavement support conditions and environmental constraints (e.g. vibration limits on buildings in close proximity to the work). A minimum binder content of 4–5% (by dry mass) may be used on some roads with high traffic volumes where a bound cementitious layer is being investigated.

For single stabilised layers greater than 200 mm thick, consideration needs to be given to the lower density of the material in the lower portion of the layer (Moffatt et al. 1998). If a significantly lower density is anticipated in the lower portion, the stabilised layer may be sub-layered in the mechanistic model with layer design moduli and flexural strength reflecting the in-service densities.

Where a higher binder content is used with deep-lift in situ stabilisation, say 4–5%, the layer becomes more susceptible to shrinkage cracking which can be reflected through overlying layers and may impact on pavement and surfacing performance.

The need to prevent the reflection of shrinkage cracking through to the pavement surfacing will depend on the likely impact of the cracking on pavement performance, e.g. loss of waterproofing, and the acceptance of undertaking future maintenance e.g. crack sealing, resurfacing etc. For high traffic loadings, a thick asphalt overlay e.g. 175 mm or greater, may be provided to inhibit the reflection of the cracking through to the surface. On low-volume roads surfacing options that may be adequate include strain-alleviating membrane seals, asphalt with/without modified binder combined with strain-alleviating membrane interlayers. Refer to Part 3 of the Guide (Austroads 2009c) for further details on surfacing options.

10.6.3 Foamed Bitumen Stabilisation of Pavement Layers

Where design traffic exceeds 10^7 ESA, a minimum of 30–40 mm thickness of asphalt surfacing is recommended. For lower traffic loadings either a sprayed bituminous seal or asphalt surfacing can be used.

10.7 Procedures for Elastic Characterisation

10.7.1 Introduction

The MEP requires a reasonable knowledge of the structural composition and condition of the existing pavement layers, often provided by pavement investigation and/or good historical construction and maintenance records. From this information, the material type and thickness of each pavement layer is established and the in situ subgrade CBR estimated for each homogeneous sub-section. As discussed in Section 5.1, pavement deflection measurements combined with the visual condition of the pavement may be used to identify pavement investigation sites.

The test data from the weaker areas of pavement in each homogeneous subsection are usually of most relevance. The representative moduli of the subgrade and unbound granular materials need to be estimated using the design procedures similar to those used for new pavements (Austroads 2018a). This occurs after adjusting the measured subgrade CBR for any seasonal effects or proposed remedial treatments i.e. drainage improvements, provision of sealed shoulders, etc. that will change the moisture regime or the support conditions.

10.7.2 Subgrade

Despite the back-calculation process and the DCP testing providing information about the variation in subgrade modulus with depth, consistent with the design of new pavements (Austroads 2018a) the subgrade is characterised using a single design modulus of semi-infinite depth. Such subgrade design moduli reflect the modulus of the upper subgrade as this commonly has the most influence in the support provided to the pavement layers. In the event that the subgrade moduli decrease with depth below the top of subgrade, consideration should be given to using the modulus from the upper weakest portion of subgrade to derive the subgrade modulus with the overlying subgrade materials being characterised as a selected subgrade material.

The subgrade design modulus is determined considering:

- results of in situ CBR testing (e.g. dynamic cone penetrometer) adjusted for possible seasonal moisture variations
- results of laboratory CBR testing of field samples
- moduli back-calculated for the top of the subgrade from measured FWD surface deflections adjusted for possible seasonal moisture variations
- suggested maximum design modulus values in Table 10.1 for use when laboratory CBR results are not available
- limiting the subgrade design modulus to a maximum of 150 MPa (consistent with Austroads 2018a).

Representative values of Poisson’s ratio are:

- 0.45 for cohesive materials
- 0.35 for non-cohesive materials.

Table 10.1: Suggested maximum subgrade design modulus values

Description of subgrade material		Maximum vertical design modulus (MPa)(1,2)	
Material	**Unified soil classification**	**Excellent to good drainage**	**Fair to poor drainage**
Highly plastic clay	CH	100	50
Silt	ML	80	40
Silty-clay, sandy-clay	CL	100	70
Sand	SW, SP	150	150

1 *Table 5.4 of Austroads (2018a) provides typical presumptive subgrade design CBR values. The maximum modulus values in Table 10.1 have been derived using twice these CBR values to allow for non-typical subgrades and equilibrium moistures.*

2 *Maximum modulus for use when laboratory CBR results are not available.*

The moisture content of the subgrade at the time of deflection testing or in situ CBR testing may differ from typical in-service moistures in the treatment design period, due to both seasonal moisture changes and any drainage improvements. Hence, in selecting the subgrade design moduli, it may be necessary to adjust these field-estimated subgrade design moduli or design CBR for any significant expected moisture effects. 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 design modulus used relates to the value at the design moisture content. In considering suitable design moisture contents, it is useful to keep in mind moisture contents and soaking conditions that are considered to be similar to in-service values in the design of new pavements (Austroads 2018a) as shown in Table 10.2.

Table 10.2: 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

Source: Austroads (2018a).

As discussed in Section 10.5, the subgrade is sub-layered into three layers in the back-calculation process to assist the model to match the measured deflections. This differs from the design procedure of new pavements where a single, more conservative, semi-infinite subgrade is commonly adopted. The subsequent MEP design process is based on procedures for the design of new pavements, and uses a single, semi-infinite subgrade layer. It is recommended that the back-calculated moduli for the top (300 mm) layer of subgrade be used to derive the semi-infinite support condition if the back-calculated subgrade modulus increases with depth.

10.7.3 Selected Subgrade and Lime-stabilised Subgrade

Section 8.2.2 of Austroads (2018a) describes the elastic characterisation of selected subgrade material and lime-stabilised subgrade materials for use in the mechanistic-empirical design of new flexible pavements. This method forms the basis of procedures described below. These thickness design procedures may be conservative for lime-stabilised materials for which the lime content is determined using Method A of Austroads (2019a). Queensland Department of Transport and Main Roads (2019) have a structural design method for use with lime-stabilised materials designed using Method A.

For both selected subgrade and lime-stabilised materials, a maximum design modulus of 150 MPa is normally adopted. In addition, Table 10.1 provides suggested maximum design moduli for use when laboratory CBR results are not available. Representative values of Poisson's ratio are:

- 0.45 for cohesive materials
- 0.35 for non-cohesive materials.

Each selected subgrade material and each lime-stabilised material is divided into five sub-layers as required for the design of new pavements. The design moduli used consider the influence of the modulus of the underlying material on the maximum modulus that the material can develop.

In the event that back-calculated moduli are not used to derive the design moduli, the design CBR of each material is determined using the following properties at the deflection measurement sites that have measured maximum deflections close to the CD of the homogeneous section:

- results of in situ CBR testing (e.g. DCP) adjusted for possible seasonal moisture variations
- results of laboratory CBR testing of field samples
- suggested maximum design modulus values in Table 10.1.

Where the in situ CBR values vary with depth in a material, the design CBR is generally based on the value representative of the upper weakest portion.

The moisture content of the selected subgrade materials and lime-stabilised subgrade materials at the time of deflection testing or in situ CBR testing may differ from typical in-service moistures in the treatment design period, due to both seasonal moisture changes and any drainage improvements. Hence, in selecting the design moduli, it may be necessary to adjust these field estimated design moduli or design CBR for any significant expected moisture effects. In considering suitable design moisture contents for selected subgrade materials, it is useful to keep in mind the moisture contents and soaking conditions considered similar to in-service values in the design of new pavements (Austroads 2018a) as shown in Table 10.2.

Utilising these design CBR values and following the procedures in Section 8.2.2 of Austroads (2018a), each selected subgrade material and lime-stabilised material is divided into five sub-layers. A check is then made as to whether the modulus assigned to each sub-layer is reasonable considering the design CBR of the material in the sub-layer.

When back-calculated moduli are considered in the determination of design moduli, further modifications are required to the procedures in Section 8.2.2 of Austroads (2018a). As mentioned in Section 10.5, the pavement model commonly provides for back-calculation of the single composite modulus value for each material. However, if the total thickness of a material exceeds 300 mm, the model may provide for back-calculation of upper and lower modulus values. In such cases, the design moduli shall be based on the average of the upper and lower modulus values.

Note that the measured deflections are not adjusted for seasonal influences prior to the back-calculation of moduli. However, the influence of these factors needs to be considered in determining the design moduli.

Where the back-calculated moduli of each selected subgrade and lime-stabilised material exceeds the back-calculated modulus of the underlying material (e.g. subgrade design modulus), each material is subdivided into sub-layers according to the following guidelines:

1. Determine the design CBR of the material using the in situ CBR, laboratory CBR results and Table 10.1 suggested maximum values as described above.
2. Divide the thickness of each selected subgrade and lime-stabilised subgrade material into five equi-thick sub-layers.
3. The vertical modulus (E_v) of the top sub-layer of each selected subgrade and each stabilised 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 subgrade material or lime-stabilised subgrade) determined using Equation 8.

$$E_{V \text{ top sub-layer}} = E_{V \text{ underlying material}} \times 2^{(\text{thickness of each selected subgrade or stabilised subgrade layer}/150)} \qquad 8$$

4. The ratio of moduli of adjacent sub-layers of each material is given by Equation 9.

$$R = \left[\frac{E_{V \text{ material top sub-layer}}}{E_{V \text{ underlying material}}}\right]^{\frac{1}{5}} \qquad 9$$

5. Using the ratio R, the vertical modulus of each sub-layer of each material is then calculated from the modulus of the adjacent underlying sub-layer, beginning with the in situ subgrade, the modulus of which is known.
6. If the middle sub-layer modulus of a material calculated in Step 5 is less than or equal to the relevant back-calculated value for each material, proceed to Step 9. Otherwise, proceed to Step 7.
7. Using the back-calculated moduli for each selected subgrade and/or lime-stabilised material, calculate the ratio R of the moduli of adjacent sub-layers such that the back-calculated modulus is assigned to the middle of the five sub-layers, using Equation 10.

$$R = \left[\frac{E_{V\ back-calculated}}{E_{V \text{ underlying material}}}\right]^{\frac{1}{3}} \qquad 10$$

8. Using the revised R value determined in Step 7, the modulus of each sub-layer is then calculated from the modulus of the adjacent underlying sub-layer.
9. For all selected subgrade and lime-stabilised subgrade materials, the other elastic parameters required for each sub-layer is then calculated from the following relationships:

$E_H = 0.5E_V$ (refer to Section 5.6 of Austroads 2018a)

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

Note that if the back-calculated moduli of any selected subgrade or lime-stabilised material is less than the design modulus of the underlying layer (Section 10.7.2), the material is not divided into five sub-layers but assigned a single design modulus for the entire thickness.

10.7.4 Unbound and Modified Granular Materials

Section 8.2.3 of Austroads (2018a) describes the elastic characterisation of granular materials for use in the mechanistic-empirical design of new flexible pavements. This forms the basis of the procedures described below. Note that if granular materials in the existing pavement are to be removed as part of the rehabilitation treatment, this change in thickness and material type needs to be considered in the elastic characterisation.

An important aspect of the granular materials’ elastic characterisation is the quality of the granular material. The modulus of the top sub-layer of granular material is limited by the material quality as described in Section 6.2.3 and Section 8.2.3 of Austroads (2018a). The results of the pavement investigation may be used to classify the granular material consistent with Austroads (2018a): namely, high standard crushed rock base, normal standard crushed rock base, gravel base and subbase with a laboratory-soaked CBR greater than 30% or lower subbase quality. This classification limits the maximum modulus a material can develop as described in Section 6.2.3 of Austroads (2018a).

In the event that back-calculated moduli are not used to derive the design moduli, the total thickness of all granular layers is divided into five sub-layers and modulus assigned consistent with the method used for the design of new pavements (Section 8.2.3 of Austroads 2018a). A check is then made as to whether the modulus assigned to each sub-layer is reasonable considering the granular material quality.

Back-calculation of layer moduli from measured FWD deflection bowls (refer Section 10.5) may also provide useful information about the layer moduli of unbound and modified granular materials. In utilising back-calculated granular moduli in design moduli determination, the following need consideration:

- The back-calculated granular moduli at the deflection measurement sites that have measured maximum deflections close to the CD are most relevant. These weaker areas of pavement govern the life of the rehabilitation treatment.
- The moisture contents of granular materials and the subgrade at the time of deflection testing may differ from typical in-service moistures in the design period, due to both seasonal moisture changes and any drainage improvements. Hence, in selecting the design moduli, it may be necessary to adjust the back-calculated moduli for any significant expected moisture effects.
- The moduli of granular materials have been observed to be stress-dependent, i.e. the moduli of granular materials increase with increasing stress levels. The applied load stress during the FWD testing of the existing pavement at the field temperature may differ from stresses in the strengthened pavement under in-service axle loads at the WMAPT. The difference in stress condition between testing and in-service may also be minimised by the judicious choice of FWD test load. If an adjustment of granular moduli for stress dependency is required, it may be necessary to use a linear elastic model to calculate the difference in applied stress and use laboratory triaxial test data (e.g. Vuong, Potter & Kadar 1988) to adjust the back-calculated moduli for the change in applied stress. These adjustments may have a significant effect on the design moduli of granular base materials.

Where appropriate, the elastic characterisation is based on dividing the total thickness of all granular layers into five sub-layers as described in Austroads (2018a), except where the granular layers are placed directly onto a bound cemented material or a lean-mix concrete subbase. The sub-layer moduli are influenced by the total thickness and quality of the granular material, modulus of the material underlying the granular materials and the thickness and modulus of the overlying bound materials. This new pavement model assumes the granular design moduli exceed those of the underlying subgrade, selected subgrade material or lime-stabilised subgrade. In addition, this model assumes the granular modulus decreases with depth.

However, the back-calculated granular moduli may not be consistent with this new pavement design model. For instance, when moduli are back-calculated for both the upper and lower halves of the total granular thickness, the lower modulus may exceed the upper value which is inconsistent with the elastic characterisation used in the new pavement model. In such cases, the design moduli will generally be based on the upper layer back-calculated moduli.

In addition, if the back-calculated granular moduli are lower than the design moduli of the underlying subgrade, selected subgrade material or lime-stabilised subgrade, it is not possible to define an elastic characterisation consistent with Austroads (2018a). In such cases, the granular layer is not divided into five sub-layers but assigned a single design modulus for the entire thickness.

After consideration of these issues on the back-calculated moduli and where granular sub-layering is required, the granular moduli should be determined as follows:

1. When back-calculated moduli have been determined for more than one layer, the equivalent modulus for the total thickness of all granular materials is calculated using Equation 11 after adjusting the back-calculated moduli for possible seasonal moisture variations.

$$E_e = \left[\frac{\sum_i h_i {E_i}^{0.333}}{T}\right]^3 \qquad 11$$

where

E_e = equivalent modulus of granular material (MPa)

E_i = back-calculated modulus of granular layer *i* (MPa)

h_i = thickness of granular layer *i* (mm)

T = total thickness of all granular materials (mm)

2. Divide the total thickness of all unbound granular materials into five equi-thick sub-layers.
3. The vertical modulus (E_v) of the top sub-layer is the minimum of the value indicated in Section 6.2.3 of Austroads (2018a) considering the thickness and modulus of the overlying bound materials and that determined using Equation 12.

$$E_{V\ \text{top granular sub-layer}} = E_{V\ \text{underlying material}} \times 2^{(\text{total granular thickness}/125)} \qquad 12$$

   Where there is more than one type of granular material, the thickness used in Equation 12 is the total thickness of all granular materials.
4. The ratio of moduli of adjacent sub-layers is given by Equation 13.

$$R = \left[\frac{E_{V\ \text{top granular sub-layer}}}{E_{V\ \text{underlying material}}}\right]^{\frac{1}{5}} \qquad 13$$

5. The vertical modulus of each sub-layer may then be calculated from the modulus of the adjacent underlying sub-layer, beginning with the in situ subgrade, the modulus of which is known.
6. If the middle sub-layer of the five sub-layers (Step 5) has vertical moduli equal or less than the relevant back-calculated value for the material (Step 1), proceed to Step 9. Otherwise, proceed to Step 7.
7. Using the back-calculated equivalent granular modulus (E_e) calculated in Step 1 and using Equation 14, calculate the ratio of the moduli of adjacent sub-layers such that the back-calculated modulus is assigned to the middle of the five sub-layers.

$$R = \left[\frac{E_{V\ back-calculated}}{E_{V\ \text{underlying material}}}\right]^{\frac{1}{3}} \qquad 14$$

8. Using the revised R value determined in Step 7, the vertical modulus of each sub-layer is then calculated from the modulus of the adjacent underlying sub-layer.
9. Check that the vertical modulus calculated for each sub-layer does not exceed the maximum modulus that the granular material in the sub-layer can develop due to its intrinsic characteristics (Section 6.2.2 and 6.2.3 of Austroads (2018a)). If this condition is not met, adjust the R value to meet this requirement.
10. For all granular materials, the other elastic parameters required for each sub-layer is calculated from the following relationship:

$$E_H = 0.5E_V \text{ (refer Section 6.2 of Austroads 2018a)}$$
$$f = E_V/(1 + \nu_V)$$

For granular materials placed directly onto a bound cemented material or LMC subbase, no sub-layering is required. In this case the design modulus is the minimum of the equivalent back-calculated modulus for the material (Step 1) and the value indicated in Section 6.2.3 of Austroads (2018a) considering the quality of the granular materials and thickness and modulus of overlying bound materials.

Note that the measured deflections are not adjusted for seasonal moisture influences or temperature effects prior to back-calculation of moduli. However, the influence of these factors needs to be considered in determining the granular design moduli.

Modified granular pavement materials are granular materials to which small amounts of stabilising binders have been added to improve modulus or to correct other deficiencies in properties (e.g. reducing plasticity) without causing a significant increase in the tensile capacity (i.e. producing a bound material). Modified granular materials have a maximum 28-day unconfined compressive strength (UCS) of 1 MPa, when tested without soaking (Austroads 2019). Modified granular materials are considered to behave as unbound granular materials (i.e. they do not develop significant tensile strain under load). In practice this condition can be difficult to ensure unless the stabilised material is reworked after most of the binding action has occurred.

Where a treatment is designed to include a modified granular material, the elastic characterisation of the modified material is similar to that described above for unbound granular materials, except that a maximum design moduli of 1000 MPa may be used for the top sub-layer.

For some projects, the existing pavement materials need to be milled to restore pavement shape, or to remove substandard material or surface cracking. In such cases, the proposed milling depth may need to be considered when determining the design moduli of granular materials.

10.7.5 Asphalt

In relation to the elastic characterisation of asphalts used in the treatment layers, these should be in accordance with the procedures described in Section 6 of Austroads (2018a).

In terms of the use of back-calculated moduli or moduli of excavated cores to derive design moduli for existing asphalt, the design values should be limited to not exceed the values used for the design of new asphalt as described in Austroads (2018a). Furthermore, in determining the design modulus of the existing asphalt, consideration needs to be given to the effect of further fatigue damage to the existing layers during the rehabilitation treatment design period.

To estimate existing asphalt design moduli from back-calculated moduli, it is necessary to firstly adjust the back-calculated moduli from the measurement temperature to the in-service temperature (WMAPT). The method of modulus temperature adjustment varies with severity of cracking during the deflection measurements (Equation 15).

$$E_{BC\_WMAPT} = E_{BC\_T}e^{(k\ (WMAPT-T))} \quad (15)$$

where

E_{BC_WMAPT}	=	back-calculated asphalt modulus at WMAPT (°C)
E_{BC_T}	=	back-calculated asphalt modulus at pavement temperature during measurement, T (°C)
k	=	–0.08 when the asphalt is sound –0.05 when the asphalt is moderately cracked –0.02 when the asphalt is crocodile cracked
WMAPT		weighted mean annual pavement temperature (°C)
T		asphalt temperature when the deflections were measured (°C)

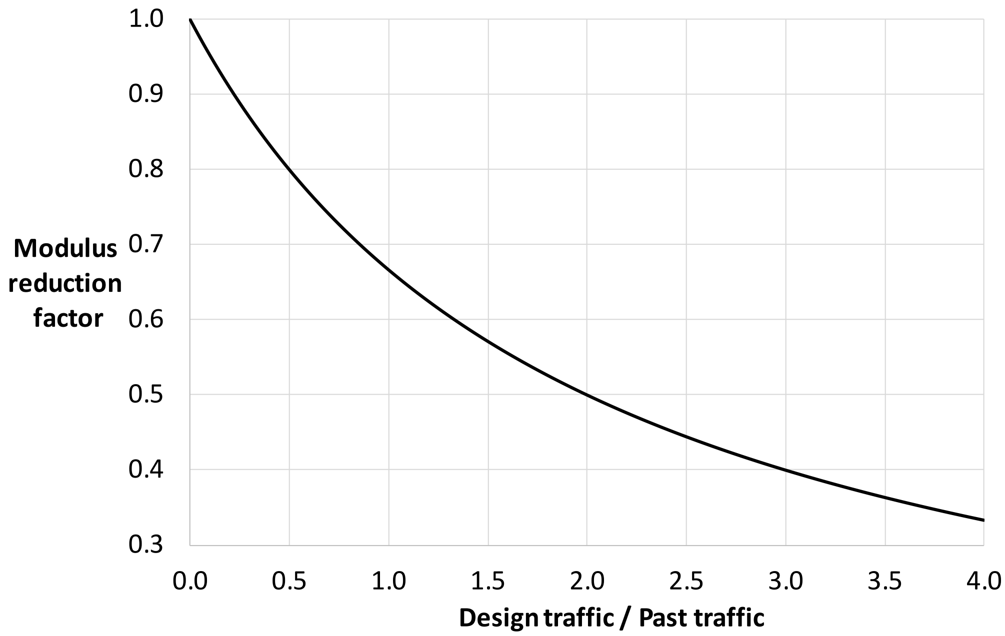
Furthermore, in determining the design modulus of the existing asphalt for which there is evidence of fatigue cracking, consideration needs to be given to the effect of further fatigue damage to the existing asphalt during the rehabilitation treatment design period. Figure 10.2 shows the modulus reduction factor (MRF) to allow for future damage to the existing cracked asphalt calculated using Equation 16. The adjusted moduli are determined by multiplying the temperature-corrected back-calculated modulus by the MRF. The degree to which the modulus changes depends on the ratio of the treatment design traffic (Section 10.3) to the past traffic loading since the last treatment (Appendix I), expressed in terms of ESA.

$$\text{MRF} = \frac{1}{(1+TR/2))} \quad (16)$$

where

MRF	=	modulus reduction factor
TR	=	traffic ratio, design traffic for the treatment in ESA divided by the past traffic in ESA (Appendix I)

Figure 10.2: Modulus reduction of existing bound layers during treatment design period



Accordingly, the following procedure is used to calculate the design modulus of existing asphalt for use in Section 10.10:

- If the temperature-adjusted back-calculated modulus is greater than the asphalt modulus used in the design of new pavements (Austroads 2018a) for the project WMAPT and heavy vehicle design speed, the design modulus of the existing asphalt is calculated as follows:
  - If the existing asphalt is not fatigue-cracked, the design modulus of the existing asphalt is the design modulus for new pavements determined in accordance with the procedures described in Section 6 of Austroads (2018a) for the project WMAPT and heavy vehicle design speed.
  - If the existing asphalt is fatigue-cracked, the design modulus of the existing asphalt is the minimum of the design modulus for new pavements (Austroads 2018a) and the temperature-adjusted back-calculated modulus multiplied by the MRF (Equation 16) calculated using the traffic ratio.
- If the temperature-adjusted back-calculated modulus is less than or equal to the asphalt modulus used in the design of new pavements (Austroads 2018a) and is greater than the presumptive cracked moduli calculated using Equation 17 (Figure 10.3), the design modulus of the existing asphalt is calculated as follows:
  - If the existing asphalt is not fatigue-cracked, the design modulus of the existing asphalt is the temperature-adjusted back-calculated modulus.
  - If the existing asphalt is fatigue-cracked, the design modulus of the existing asphalt is the maximum of the presumptive cracked moduli calculated using Equation 17 and temperature-adjusted back-calculated modulus multiplied by the MRF (Equation 16) calculated using the traffic ratio.
- If the temperature-adjusted back-calculated modulus is less than or equal to the presumptive cracked moduli calculated using Equation 17, the temperature-adjusted back-calculated modulus is used as the design modulus of the existing asphalt.

In the absence of measured data, a Poisson's ratio of 0.40 may be used for asphalt regardless of the age or extent of cracking.

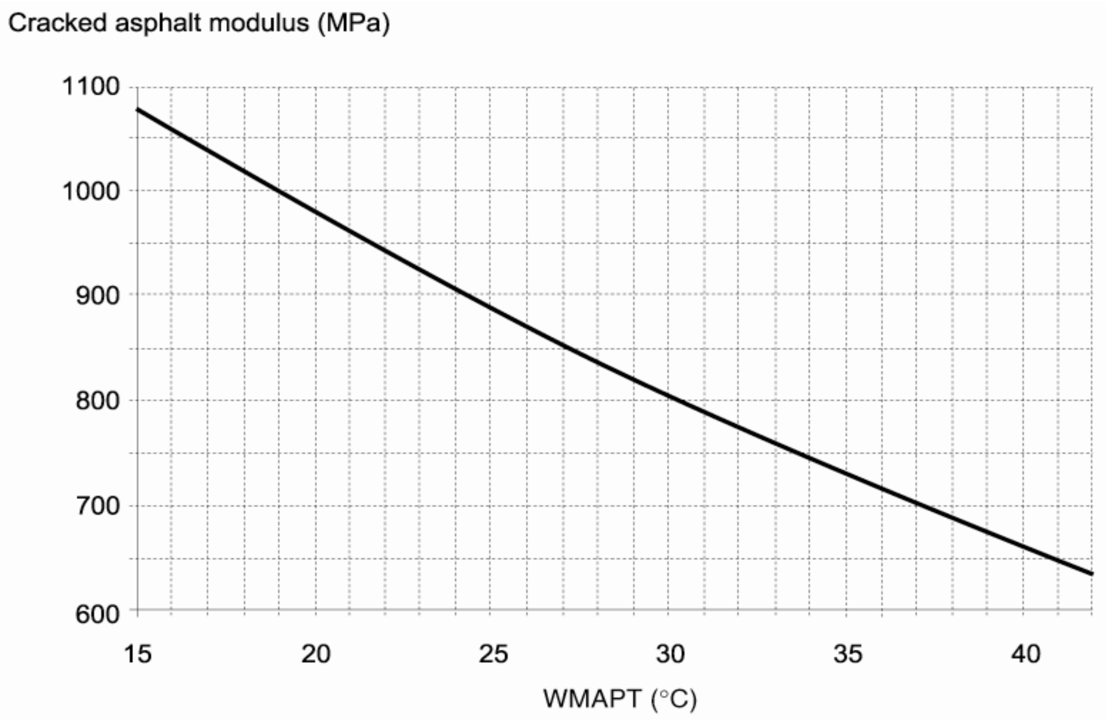
$$E_{AC} = 1450\ e^{-0.02\text{WMAPT}} \quad (17)$$

where

E_{AC} = presumptive design modulus for cracked dense-graded asphalt (MPa)

WMAPT = Weighted mean annual pavement temperature (°C)

Figure 10.3: Presumptive design moduli for existing dense-graded asphalt layers



Similarly, to estimate existing asphalt design moduli from indirect tensile moduli of excavated asphalt cores, it is necessary to firstly adjust the core moduli from the measurement temperature (T) to the project in-service temperature (WMAPT) and from the measurement loading rate (40 ms) to the heavy vehicle design speed following the procedure in Austroads (2018a) except that Equation 18 should be used to calculate the temperature adjustment factor.

$$\frac{\text{Field modulus at WMAPT}}{\text{Laboratory modulus at test temperature (T)}} = e^{(\text{k(WMAPT}-\text{T))}} \quad (18)$$

where

k = –0.08 when the asphalt is sound
–0.05 when the asphalt is moderately cracked
–0.02 when the asphalt has crocodile cracked

As discussed above for back-calculated moduli, in determining the design modulus of the existing cracked asphalt, consideration needs to be given to the effect of further fatigue damage to the existing asphalt during the rehabilitation treatment design period. As a consequence, the corrected core modulus may overestimate the effective modulus during the design period. Accordingly, the procedure described above to calculate the design modulus of asphalt from back-calculated modulus also applies to existing asphalt moduli estimated from core testing.

10.7.6 Cemented Material and Lean-mix Concrete

In relation to the elastic characterisation of cemented materials and lean-mix concretes (LMC) used in the treatment layers, these should be in accordance with the procedures described in Section 6 of Austroads (2018a).

In terms of the use of back-calculated moduli to derive design moduli for existing cemented materials and LMC, the back-calculated values should be limited to not exceed the values used for the design of new cemented materials and LMC as described in Austroads (2018a). Furthermore, in determining the design modulus of an existing cracked cemented material or LMC layer, consideration needs to be given to the effect of further fatigue damage to the existing layers during the rehabilitation treatment design period.

Consistent with the process described for asphalt (Section 10.7.5), Figure 10.2 shows the modulus reduction factor to allow for future load-induced damage to the existing cracked cemented materials and LMC calculated using Equation 16. The moduli adjusted for future fatigue damage are determined by multiplying the back-calculated modulus by the modulus reduction factor.

Accordingly, the following procedure is used to calculate the design modulus of existing cemented materials or LMC layer:

- If the back-calculated modulus is greater than the modulus used in the design of new pavements as described in Section 6 of Austroads (2018a), the design modulus of the existing material is calculated as follows:
  - If the existing layer is not fatigue-cracked, the design modulus of the existing material is the design modulus for new pavements (Austroads 2018a).
  - If the existing layer is fatigue-cracked, the design modulus of the existing material is the minimum of the design modulus for new pavements (Austroads 2018a) and the back-calculated modulus multiplied by the MRF (Equation 16) calculated using the traffic ratio.
- If the back-calculated modulus is less than or equal to the modulus used in the design of new pavements (Austroads 2018a) and greater than the presumptive cracked moduli in Table 10.3, the design modulus of the existing material is calculated as follows:
  - If an existing cemented material or LMC layer is not fatigue-cracked, the design modulus of the existing material is the back-calculated modulus.
  - If an existing cemented material or LMC layer is fatigue-cracked, the design modulus of the existing material is the maximum of the presumptive cracked moduli as provided in Table 10.3 and the back-calculated modulus multiplied by the MRF calculated from Equation 16.
- If the back-calculated modulus is less than or equal to the presumptive cracked moduli as provided in Table 10.3, the back-calculated modulus is used as the vertical design modulus of the existing material.

In the absence of measured data, Poisson's ratio values in Table 10.3 may be used for cracked cemented material and LMC.

Table 10.3: Presumptive elastic characterisation of cracked cemented material and LMC

Cracked material	Vertical modulus (MPa)	E_V/E_H	Poisson's ratio	Sub-layered
Cemented material	500[(1)]	2.0	0.35	No
Lean-rolled concrete (cracked by normal traffic)	500	2.0	0.35	Yes
Lean-rolled concrete (cracked by construction traffic)	350	2.0	0.35	Yes
Lean-screeded concrete	700	1.0	0.20	No

1 This value applies for cemented materials for which a design modulus of 2500 MPa or more is used in design of new pavement layers. For lower design moduli, the presumptive cracked modulus is 1/5th of the design modulus.

Source: Austroads (2018a).

10.7.7 Foamed Bitumen Stabilised Material

The elastic characterisation of newly constructed foamed bitumen stabilised (FBS) materials is described in Appendix H.

In terms of the use of back-calculated moduli to estimate FBS materials' design moduli, it is necessary to firstly adjust the back-calculated moduli from the measurement temperature to the in-service temperature. The back-calculated modulus is multiplied by modulus temperature adjustment calculated using Equation 15 with a k value of –0.025.

Consistent with the process described for asphalt (Section 10.7.5), Figure 10.2 shows the modulus reduction factor to allow for future load-induced damage to the existing cracked FBS calculated using Equation 16. The moduli adjusted for future fatigue damage are determined by multiplying the back-calculated modulus by the modulus reduction factor.

The design moduli for existing FBS materials is similar to that described in Section 10.7.5 for asphalt:

- If the temperature-adjusted back-calculated modulus is greater than the FBS modulus used in the design of treatment layers (Appendix H) for the project WMAPT and heavy vehicle design speed, the design modulus of the existing FBS is calculated as follows:
  - If the existing FBS is not fatigue-cracked, the design modulus of the existing FBS is the design modulus of a treatment layer (Appendix H) for the project WMAPT and heavy vehicle design speed.
  - If the existing FBS is fatigue-cracked, the design modulus of the existing FBS is the minimum of the design modulus for a treatment layer and the temperature-adjusted back-calculated modulus multiplied by the MRF (Equation 16) calculated using the traffic ratio.
- If the temperature-adjusted back-calculated modulus is less than or equal to the FBS modulus used in the design of treatment layers (Appendix H) and is greater than 500 MPa, the design modulus of the existing FBS is calculated as follows:
  - If the existing FBS is not fatigue-cracked, the design modulus is the temperature-adjusted back-calculated modulus.
  - If the existing FBS is fatigue-cracked, the design modulus of the existing FBS is the maximum of the presumptive cracked FBS modulus of 500 MPa and temperature-adjusted back-calculated modulus multiplied by the MRF (Equation 16) calculated using the traffic ratio.
- If the temperature-adjusted back-calculated modulus is less than or equal to 500 MPa, the temperature-adjusted back-calculated modulus is used as the design modulus.

In the absence of measured data, a Poisson's ratio of 0.40 may be used for FBS materials regardless of the age or extent of cracking.

10.8 Procedures for Determining Critical Strains

The procedure to calculate critical strains are similar to that of Section 8.2 of Austroads (2018a). When the treatment is an asphalt, cemented material or LMC layer, the tensile critical strains at the bottom of such new bound materials is calculated.

The new pavement design procedures (Austroads 2018a) do not describe the procedure for the thickness design of new pavements with FBS materials. In this Part, the critical strains to determine the allowable traffic loading in relation to fatigue of foamed bitumen stabilised materials are the maximum horizontal tensile strains at the bottom of the layer. These critical strains are calculated following the same procedure as used to predict asphalt critical strains as described in Austroads (2018a).

10.9 Performance Relationships

Performance relationships are required to predict the allowable number of load repetitions from the critical strains. The following relationships are used in this Part:

- To predict the allowable number of repetitions in terms of permanent deformation, the subgrade strain criteria given in Equation 3 of Austroads (2018a) is used.
- To predict the allowable number of load repetitions in terms of asphalt fatigue, Equation 25 of Austroads (2018a) is commonly used for mixes with conventional binders. For asphalt mixes using non-conventional binders, the allowable repetitions predicted using Equation 25 may be multiplied by the relevant presumptive factor listed in Table 10.4. These factors only apply to asphalt overlays and inlays less than 50 mm thick. In addition, Section 6.5.11 of Austroads (2018a) provides guidance on the use of laboratory fatigue testing data.
- To predict the allowable number of load repetitions in terms of FBS materials fatigue, Equation 19 is used, which is based on the asphalt fatigue relationship of Austroads (2018a). Procedures have yet to be developed to design FBS layers to a desired project reliability level.
- To predict the allowable number of load repetitions in terms of cemented materials fatigue, Section 6.4.6 of Austroads (2018a) provides three methods of determining an in-service fatigue relationship.
- To predict the allowable number of repetitions in terms of LMC fatigue, Equation 27 of Austroads (2018a) is used.

Table 10.4: Presumptive allowable traffic loading adjustment factors for asphalt wearing course with non-conventional binders

Austroads binder grade	Allowable traffic loading adjustment factor
Multigrade 1000/320	1.0
A30P	1.0
A35P	1.5
A20E	2.0
A25E	2.0
A15E	2.5
A10E	3.0

$$N = \left[\frac{6918(0.856 \times V_b + 1.08)}{E^{0.36}\mu\varepsilon}\right]^5 \qquad 19$$

where

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

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

V_b = volume of binder in the FBS material (%), commonly 7%

E = FBS design modulus (MPa)

10.10 Treatment Design

10.10.1 Introduction

The procedure described in this section is applicable to the design of flexible treatments on flexible pavements.

This procedure is well-suited for the structural design of treatments for pavements without existing bound pavement materials (e.g. asphalt, cemented material). For example, for the design of asphalt overlays on sprayed seal-surfaced unbound granular pavements.

For an existing pavement with bound materials, the procedures described in this section takes no account of the remaining fatigue characteristics of the existing bound materials.

As discussed in Section 10.2, where treatments are placed over existing materials, consideration needs to be given to the risk that reflective cracking may limit the service life of the pavement. In some cases, it may be appropriate to modify the design period and associated design traffic loading used in the structural treatment design to align with anticipated service life as limited by reflective cracking. The potential for reflective cracking to dominate service life needs careful consideration throughout the design process.

10.10.2 Design Method

The procedures to calculate allowable load repetitions of a trial treatment are the same as described in Table 8.3 of Austroads (2018a), with the performance relationships described in Section 10.9.

For lightly trafficked roads (DESA < 10^5), load-induced asphalt fatigue cracking is uncommon (Austroads 2018a). For such roads, it is not necessary to design treatments to inhibit asphalt fatigue.

Examples of use of the design procedure are provided in:

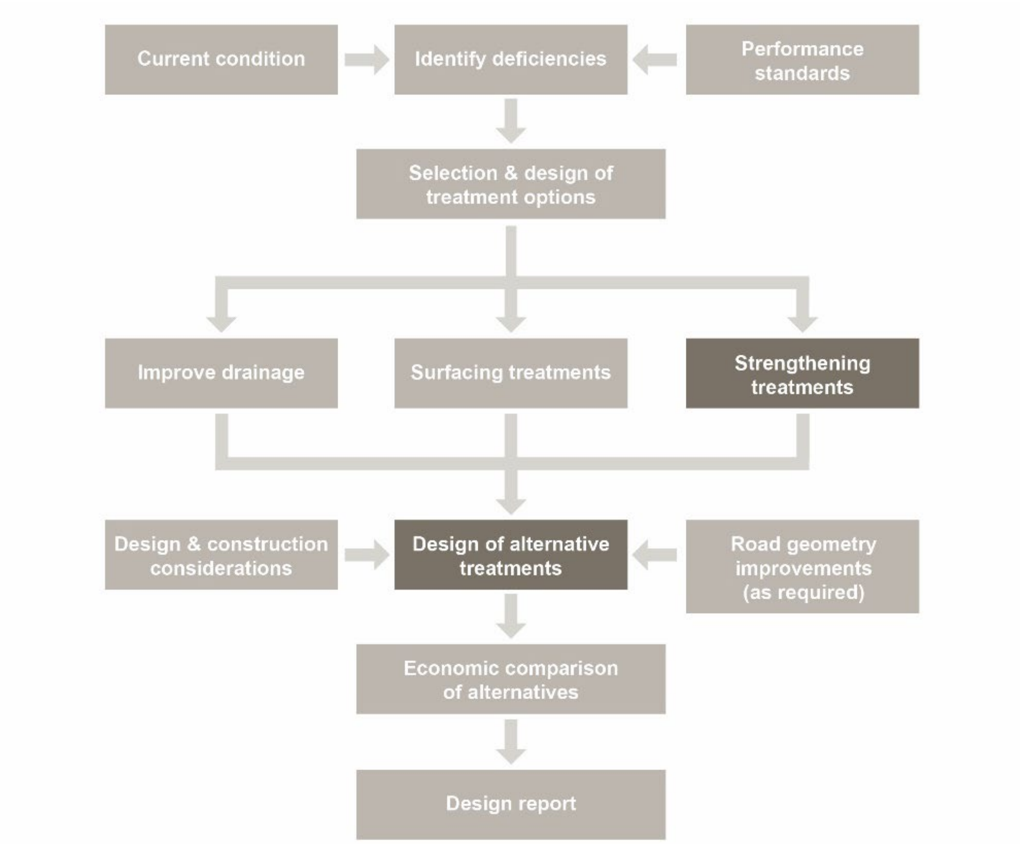
- Appendix J: asphalt inlay
- Appendix K : cement-stabilised base
- Appendix L : granular overlay considering subgrade lime stabilisation
- Appendix M : modified granular base
- Appendix N : foamed bitumen stabilisation of a granular pavement.

11. Concrete Overlays on Flexible Pavements

This section provides guidance on the design of concrete overlays on flexible pavements using the existing pavement as a subbase where appropriate. Concrete overlays can be used to rectify structural deficiencies of an existing pavement (Section 7.5.5, Figure 11.1).

Concrete overlays are designed using the procedures for the design of new concrete pavements as described in Section 9 of Austroads (2018a). Depending on the composition and properties of the existing pavement and the type of concrete overlay to be placed on it, the existing pavement may require modification or additional material to provide an adequate subbase for the concrete overlay.

Figure 11.1: Design steps discussed in Section 11



The types of concrete pavements that can be designed as overlays are the same as for new pavements (refer to Section 9.2 of Austroads (2018a)). In addition, concrete overlays may be designed with or without tied or integral concrete shoulders as discussed in Section 9.3.5 of Austroads (2018a). The design traffic loading is expressed in terms of (refer to Section 7 of Austroads 2018a):

- cumulative number of heavy vehicle axle groups (HVAG)
- the distribution of axle group loads by type and load.

The subbase under the concrete overlay needs to meet the requirements of Section 9.2.2 of Austroads (2018a). The surface layer of the existing pavement may form all or part of the minimum subbase requirements, as indicated in Table 9.1 of Austroads (2018a), where it includes:

- sound asphalt with a minimum asphalt design modulus of 2000 MPa at the WMAPT and design speed (based on indirect tensile testing of cores extracted from the existing pavement and procedures in Section 6.5.5 of Austroads (2018a).
- materials that can be stabilised to meet the requirements of Section 9.2.2 of Austroads (2018a).

Subject to the requirements of Section 9.2.2 of Austroads (2018a), the existing pavement may be improved to meet the recommended thickness for a bound subbase, by either:

- cementitious stabilisation of the existing pavement material (where suitable) to the required depth
- adding a correction layer of dense-graded asphalt to an existing asphalt surface (e.g. where the existing surface includes 100 mm of sound dense-graded asphalt, the addition of a further 50 mm would constitute a 150 mm bound subbase)
- constructing a new subbase in accordance with Section 9.2.2 of Austroads (2018a).

In the design of concrete overlays, the existing pavement materials below any existing sound bound materials (e.g. asphalt) are considered to be part of the subgrade. The equivalent subgrade design CBR of any existing cracked asphalt, unbound granular materials, selected subgrade layers and the in situ subgrade materials, is calculated using the following procedure:

- As the strength of a pavement/subgrade varies from site to site, it may be necessary to divide the test section into sub-sections, having relatively uniform equivalent subgrade design CBRs. For such situations, surface deflection testing may be used to identify homogeneous sub-sections as described in Section 9.2.
- For each homogeneous sub-section, the in situ CBR of each earthworks material may be estimated using the penetration measured using dynamic cone penetrometer (DCP) and Figure 5.2. The design CBR of each material is selected after considering the effects of any differences between the moisture content of materials during testing and moisture contents likely to occur during the design period.
- Existing granular material of base, upper subbase and lower subbase qualities may be assumed to have a design CBR of 80%, 30% and 15%, respectively.
- Cracked asphalt is assumed to have a design CBR of 80%.
- The equivalent subgrade strength (CBR_m) is given by the following formula in Equation 20.

$$CBR_m = \left[\frac{\sum_i h_i \, CBR_i^{0.333}}{\sum_i h_i} \right]^3 \qquad 20$$

where

CBR_i = CBR value material *i*

h_i = thickness of material *i*, in metres

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

The following conditions apply to the use of Equation 20:

- Layers of thickness less than 0.2 m are combined with an adjacent layer. The lower CBR value is adopted for the combined layer.
- The equation assumes that higher CBR materials will be used in the upper layers and is not applicable where weaker layers are located in the upper part of the subgrade.
- Filter layers are not included in the calculation.
- The maximum CBR from the use of this formula is 15%.

Using the equivalent subgrade strength, the effective CBR is determined in accordance with Figure 9.1 of Austroads (2018a) to enable concrete overlay thickness to be determined.

An example of use of the design procedure is provided in Appendix O.

12. Thickness Design of Structural Treatments for Rigid Pavements

12.1 Introduction

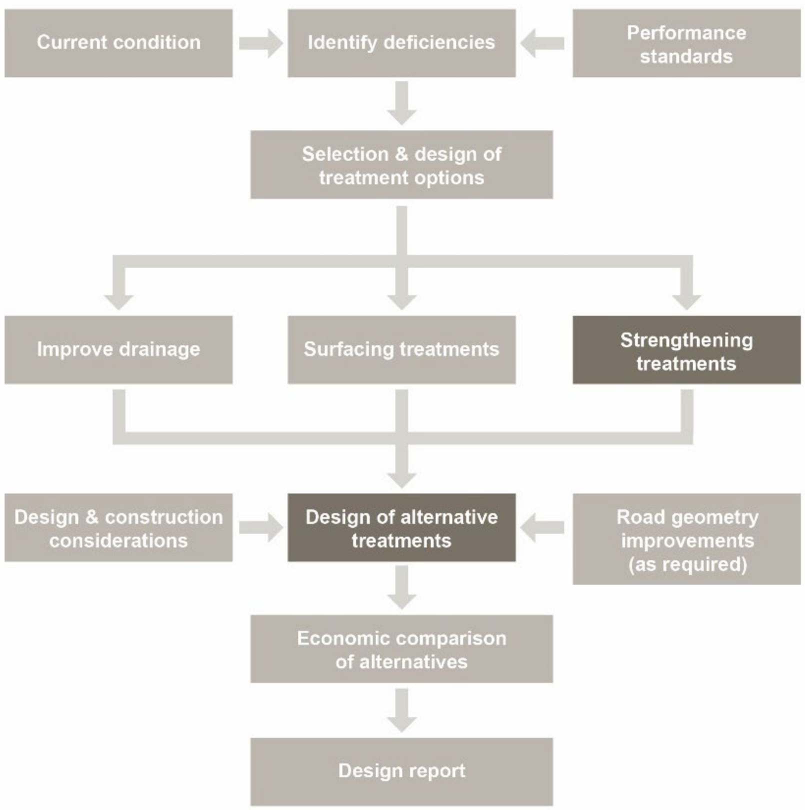
As described in Section 8.6.5 and Section 8.6.6, asphalt and concrete overlays can be used to rectify functional and structural deficiencies.

The scope of each of these procedures may be summarised as follows:

- Section 12.2 provides procedures to inhibit reflective cracking of asphalt overlays on concrete pavements. Deflection testing across joints and cracks is used to assess the overlay thickness required to protect against load-induced reflective cracking.
- Section 12.3 details the method to design concrete overlays on concrete pavements. The overlay is designed in the same way as a new concrete pavement.

All design procedures also utilise information about the pavement configuration (i.e. structure and composition) derived from historical records or pavement investigation (Figure 12.1).

Figure 12.1: Design steps discussed in Section 12



12.2 Asphalt Overlays on Rigid Pavements

The predominant distress mode for asphalt overlays on rigid pavements is the development of reflection cracks in the asphalt directly above cracks and joints in the underlying rigid pavement.

The following procedure provides a basis for determining an overlay thickness sufficient to delay the development of these reflective cracks to an acceptable level by:

- limiting the mean deflections on both sides of the joint/crack
- limiting the differential deflection across joints and cracks (difference between deflections measured on the two sides of the joint/crack caused by a load placed on one side)
- minimum cover requirements.

The required overlay thickness is the greater of either the thickness to limit the effect of load-induced movements or the thickness to limit the effect of thermally induced movements.

FWD deflection testing of existing jointed rigid pavements is used to assess joint load transfer and to identify the presence of voids under slabs as discussed in Section 4.10. If slab undersealing is to be undertaken, deflections measured after the undersealing should be used in the design of the structural overlay.

The recommended overlay thickness to protect against reflective cracking caused by load-induced differential movements across joints and cracks is the thickness required to reduce (Asphalt Institute 2000):

- jointed concrete pavement:
  - the mean of the measured deflections each side of a joint/crack to a tolerable level of 0.57 mm
  - the measured differential deflection ($d_u – d_l$), of a joint/crack to tolerable levels of 0.08 mm
- continuously reinforced concrete pavement:
  - the mean of the measured deflections each side of a joint/crack to a tolerable level of 0.44 mm.

These tolerable levels are based on FWD deflection using a contact stress of 566 kPa (40 kN load).

Deflection reductions are calculated on the basis of a 1% reduction per 5 mm of asphalt overlay thickness.

If the thickness to limit load-induced movements is excessive, the slab must either be supported by restoring the load transfer at joints by retrofitting dowels or using an under-grouting treatment or be transformed into smaller segments using the crack and seat or rubblising processes discussed in Section 8.6.7. Once these works are complete, a new deflection survey is necessary to determine the asphalt overlay thickness requirements.

The minimum cover requirements to limit both load-induced and thermally induced movements, is best established based on relevant field performance data in the vicinity of the project. An indication of the minimum cover requirements needed to inhibit reflective cracking is provided by the 175 mm minimum asphalt cover requirement adopted for the design of new pavements with bound cemented materials (Austroads 2018a). The Asphalt Institute (2000) also provides guidance on minimum cover requirements based on slab length and the temperature environment.

If the estimated overlay thickness is considered to be excessive, alternative measures to inhibit reflective cracking may be considered such as:

- retrofitting dowels or slab-jacking to reduce load-induced movements at joints/cracks
- saw cutting the overlay above the joints and sealing to control where reflective cracking occurs
- overbanding of the joints/cracks prior to overlaying
- placement of a SAMI prior to overlaying and the use of polymer modified binders in the asphalt.

An example of use of the design procedure is provided in Appendix P.

12.3 Concrete Overlays on Rigid Pavements

Unbonded concrete overlay on rigid pavements may be appropriate for concrete pavements in poor condition, including severe cracking or material-related distress provided the pavement is stable and provides uniform support to the overlay. Unstable areas need to be repaired prior to overlay.

An unbonded concrete overlay on rigid pavement is designed in the same way as a new rigid pavement, using the existing concrete base as a subbase. If the existing concrete base is stable, it is assumed to provide equivalent support to the concrete overlay as would a new lean-mix concrete subbase of the same thickness.

To debond the concrete overlay from the existing rigid pavement, a minimum thickness of 25 mm dense-graded asphalt or geotextile fabric is usually required on the existing pavement.

The types of rigid pavements that can be designed as overlays are the same as for new pavements (refer to Section 9.2 of Austroads 2018a). In addition, these concrete overlays may be designed with or without tied or integral concrete shoulders as discussed in Section 9.3.5 of Austroads (2018a).

For concrete overlays, thicknesses are calculated using the design procedures for new rigid pavements as detailed in Section 9 of Austroads (2018a). The traffic loading is expressed in terms of:

- cumulative number of heavy vehicle axle groups (HVAG)
- the distribution of axle group loads by type and load as defined in Section 7 of Austroads (2018a).

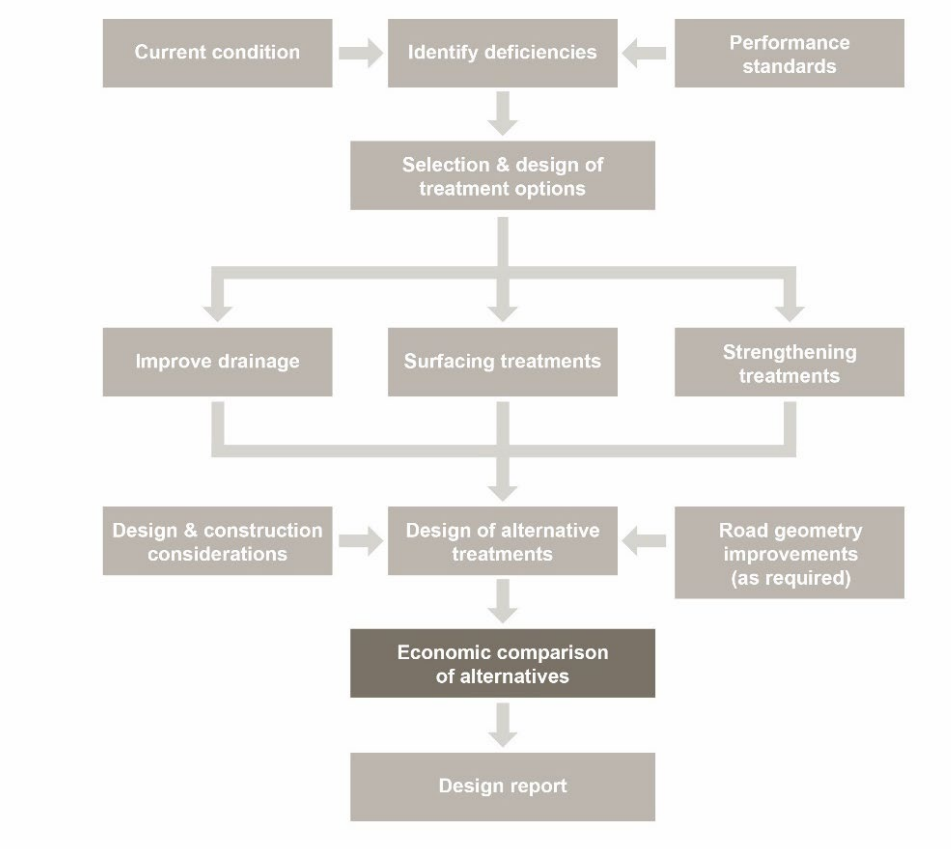
13. Economic Comparison of Alternative Treatments

13.1 Introduction

Section 7 and Section 8 discuss the attributes and limitations of rehabilitation treatments from a technical perspective and how these influence the identification of appropriate treatments for particular applications. An equally important consideration in the evaluation of alternative rehabilitation treatments is the relative costs and benefits of the treatments. This comparison is commonly made using life cycle costing (LCC) or whole-of-life of costing (WHOLC) techniques, which use simplified economic models to calculate the current value of costs occurring over the expected service lives of the treatments.

This section discusses the key parameters and the models used in the economic comparison and Appendix Q provides a worked example (Figure 13.1).

Figure 13.1: Design steps discussed in Section 13



The purpose for an economic comparison is to evaluate alternative treatments primarily according to the criterion of minimum total (whole-of-life) cost, giving due consideration also to the safety and service provided to road users and others who may be affected by the road or its construction. In particular, other criteria which may need to be considered are:

- maintenance requirements
- the scale of the project
- the requirement to construct under traffic
- noise and spray effects
- the potential for differential settlement over the road alignment.

In view of the simplicity of the models, designers are advised to consider carefully the results of any economic comparison of rehabilitation options and not to rely on it as the sole determinant of the most appropriate option.

It should be noted that staged construction approaches require interventions to be undertaken at the required time and, if a structural treatment, when the traffic loading exceeds a design value. There is always a risk works will not be undertaken at the required time and/or funds may not be available at the required time. This risk needs to be considered when comparing alternative treatments.

More detailed advice is provided in the *Austroads Project Evaluation Compendium* (Austroads 2001).

13.2 Method for Economic Comparison

There are two economic models that may be used for comparison of alternative rehabilitation treatments. 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. The other model is the Equivalent Annualised Cash Flow (EACF) method.

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 at a selected evaluation year can be calculated using Equation 21.

$$PWOC = C + \sum_i M_i (1+r)^{-x_i} - S(1+r)^{-z} \qquad 21$$

where

PWOC	=	present worth of costs at selected evaluation year
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 date of opening the road to traffic 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 the present worth of costs, the principal parameters are:

- initial rehabilitation cost
- subsequent costs associated with the regime of maintenance and rehabilitation activities, including routine and periodic maintenance and structural rehabilitation, appropriate to each treatment
- salvage value of the pavement at the end of the analysis period
- evaluation year, the year to which future costs are discounted
- 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 a range of traffic growths and investment periods are evaluated to gauge the 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.

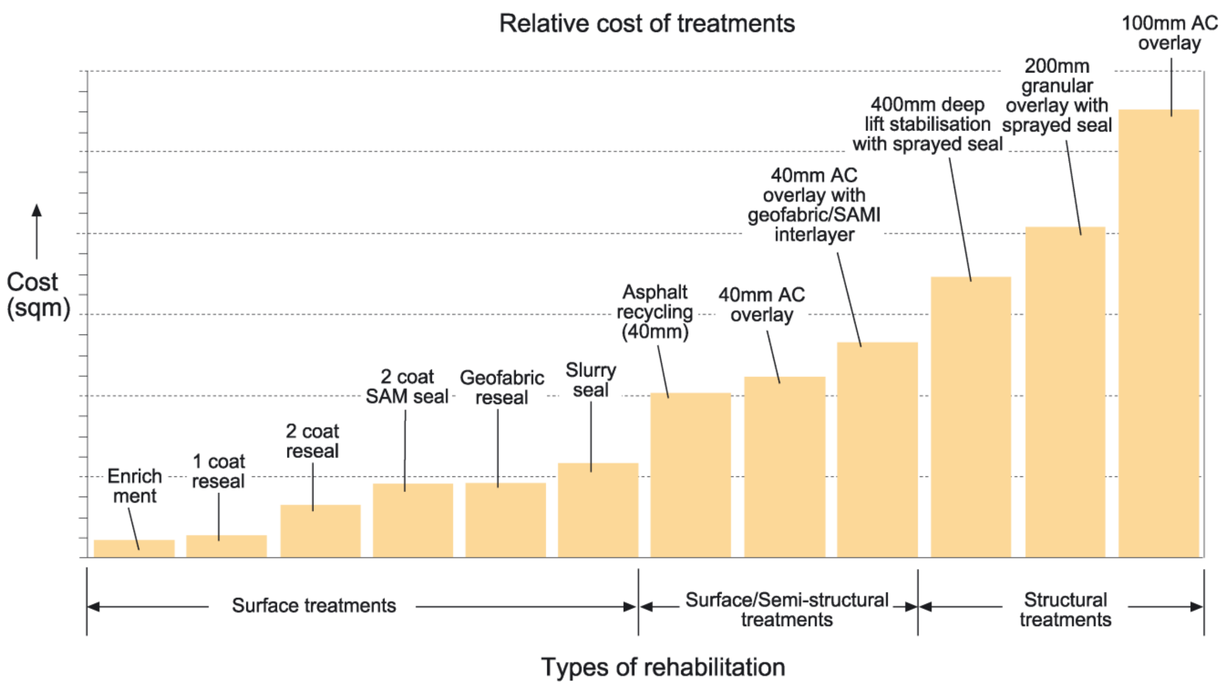
The economic model also may not account for substantial differences in the social, political or environmental impacts of future maintenance and rehabilitation activities associated with alternative treatments. For instance, one alternative may require reconstruction involving a total road closure at 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.

13.3 Economic Parameters

13.3.1 Initial Rehabilitation Costs

Unit costs for alternative rehabilitation treatments will vary widely depending on the locality and scale of the project, the availability of suitable natural and processed materials and material standards. Most road agencies will have developed a database of unit costs over the years, and these can be used to build up costs for a particular project. Figure 13.2 provides useful guidance of relative costs between treatments. It can be used to extend current knowledge of the local cost of two or three of the treatments shown and should enable a reasonable estimate for the likely local costs for all the treatments listed.

Figure 13.2: Relative costs of rehabilitation treatments



Source: Adapted from Ramanujam, private communication (1998).

There are, however, several other less obvious costs, which warrant consideration. For example, some alternatives may require more excavation or more fill (e.g. where surface levels are fixed by external constraints), may require more extensive relocation of services, or require more shoulder material. Significant saving in shoulder material and reduced interference with services may be achieved, for equivalent performance, for example, by using a full-depth asphalt or cemented pavements which are thinner than an unbound granular pavement. Consequently, comparing the cost per square metre of the pavement alone is often misleading. A more realistic comparative cost may be the cost per kilometre for the pavement rehabilitation including ancillary works such as shoulder improvements and relocation of services or the total project cost (including all overheads). The latter is likely to be appropriate where the overheads, and other non-productive costs, which are not necessarily included in the pavement unit costs, vary with the type of treatment.

Such costs include:

- **Provision for traffic:** Alternatives which take longer to build or require more complex or extensive traffic deviation works usually incur higher traffic control costs. In some cases, however, the cost of relocating traffic onto a detour or side track rather than constructing under traffic can shorten the construction period and hence, reduce overall project costs.
- **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, less moisture-sensitive and avoid the cost of reworking.
- **Establishment costs:** These include costs associated with the setting up and transport of plant, etc. These costs vary between alternatives.
- **Supervision costs:** Supervision costs are time-dependent: alternatives which are slower to construct tend to cost more to supervise.

There are significant economies of scale in some paving operations for larger projects, particularly for supply and laying of materials. In many cases, there may not be 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.

While the economic comparison focuses more on WHOLC, the impact of high initial cost alternatives on the overall state of the road network should be considered. It may not be efficient to adopt a high initial cost treatment even if that has a favourable WHOLC, where doing so would compromise the capacity to improve other links in the network and raise the overall standard of the network. Conversely, especially where major expenditure is undertaken, a network subject to short repeat interventions may deteriorate within the available road funding.

13.3.2 Subsequent Maintenance/Rehabilitation Costs

The nature and extent of future maintenance is dependent on pavement type and rehabilitation treatment. For example, routine maintenance costs of rigid pavements are generally less than those of sprayed seal surfaced, unbound granular 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 treatments. In many cases, however, there may be insufficient information to reliably consider future maintenance and strengthening costs of various alternatives, although these details are becoming available (e.g. Bennett & Moffatt 1995; Porter & Tinni 1993).

Road user costs due to maintenance activities 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 treatment where there are differences in:

- the level and frequency of maintenance activities
- the duration of construction delays
- the levels of traffic safety during construction
- 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 various treatment options is known, road user costs should be included.

Typical service lives of surfacings treatments given in Table 6.7 may assist in estimating future maintenance costs during the analysis period.

13.3.3 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 pavement 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 13.2), careful consideration should be given to the discount rate used.

Despite the challenges, some guidance can be given as to how to evaluate salvage value:

- For a project where the predicted condition of the pavement at the end of the analysis period is such that the base layer could serve as the subbase layer for the subsequent next project, then the salvage value is equal to the cost in current dollars (say, year 2019) for construction in say, year 2039 of a pavement to subbase level (less any costs for tidying up the works, scarification, compaction, drainage renovation etc.), discounted to the evaluation year (2019).
- 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 for the subsequent next project, then the salvage value is equal to the cost in current dollars (say, year 2019) for construction in say, year 2039 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 2019). 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 (RUC) in the evaluation, then the estimated RUC 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 RUC.

13.3.4 Real Discount Rate

The real discount rate must be selected to express future expenditure in terms of present values. 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 (Commonwealth of Australia 2016) use a discount rate of 7%. This rate is expressed in real terms, i.e. it excludes inflation. However, other jurisdictions may require the use of other rates for other purposes. In most cases, it is desirable to carry out a sensitivity analysis with discount rates of 4% and 10%, which is also a requirement of the Commonwealth of Australia.

13.3.5 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 rehabilitation treatments and should not be less than the longest design period of the alternative treatments.

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.

13.4 Road User Costs

Road user costs, which include vehicle operating costs, travel time costs and vehicle accident costs, are the most complex of the costs considered in a life cycle cost analysis, as they cannot be assigned easily.

To estimate the reduction in road user costs of each alternative treatment, the road condition, such as roughness, needs to be estimated for each year of the analysis period.

The vehicle operating costs for rehabilitation alternatives may be excluded from the analysis, if they are essentially similar for alternatives and 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), HDM-4 (Kerali 2000) and Thoresen and Roper (1996), or from the Australian Bureau of Statistics.

13.5 Predicting Performance of Treatments

A key element in the comparison of alternative treatments is the prediction of the performance of the treatments throughout the analysis period.

The structural performance of treatments is estimated using the procedures described in Section 9 to Section 12.

In terms of pavement surfacings, typical service lives are shown in Table 6.7. The service lives in this table are for average conditions. 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 in-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 Table 6.7 for each of the competing alternatives. In addition, models for the predicted performance form the core of asset management systems, which in turn, provide a basis for asset management and budget forecasting.

The extent to which actual performance will agree with predicted performance depends (inter alia) on the following:

- the accuracy of the performance prediction model inherent in the design process
- prediction of traffic loading
- prediction of a value of subgrade strength which is representative over the analysis period
- appropriate characterisation of pavement materials
- the extent to which actual environmental effects have been allowed for in the design
- construction quality
- the extent to which the maintenance and rehabilitation strategy is followed in practice.

Further, the level of dependence on these aspects will differ for each of the alternatives being considered. Hence, a separate assessment of these aspects is needed for each alternative. Commonly, the last aspect in the above list is the most critical and most difficult to assess.

13.6 Example of Whole-of-Life Costing of Alternatives

Appendix Q gives an example of the use of whole-of-life costing to compare alternative rehabilitation treatments.

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Appendix A Identification, Causes and Treatment of Visual Distress

A.1 Flexible Pavements

The principal modes of pavement distress of flexible pavements are as follows:

- deformation
  - rutting (Appendix A.1.1)
  - shoving (plastic flow) (Appendix A.1.2)
  - depression and heave (Appendix A.1.3)
  - corrugation (Appendix A.1.4)
- cracking
  - block cracking (Appendix A.1.5)
  - crocodile cracking (Appendix A.1.6)
  - transverse cracking (Appendix A.1.7)
  - diagonal cracking (Appendix A.1.8)
  - meandering cracking (Appendix A.1.9)
  - crescent-shaped cracking (Appendix A.1.10)
  - longitudinal cracking (Appendix A.1.11)
  - edge defects (Appendix A.1.12 and Appendix A.1.13)
- delamination (Appendix A.1.14)
- stripping
  - sprayed seal (Appendix A.1.15)
  - asphalt (Appendix A.1.16)
- ravelling (Appendix A.1.17)
- flushing (Appendix A.1.18)
- polishing (Appendix A.1.19)
- potholing (Appendix A.1.20)
- failed patches (Appendix A.1.21).

Some of these distress types are shown in Figure A 1, Figure A 2 and Figure A 3 below.

These defects are described below, together with a photograph and a listing of the likely causes of the particular defect and possible treatments.

The causes of these defects can be categorised into three groupings:

- structural
- environmental
- construction quality.

In many cases, there may be more than one apparent cause which may make it difficult to identify the primary cause of the defect.

Figure A 1: Deformation defects in flexible pavements

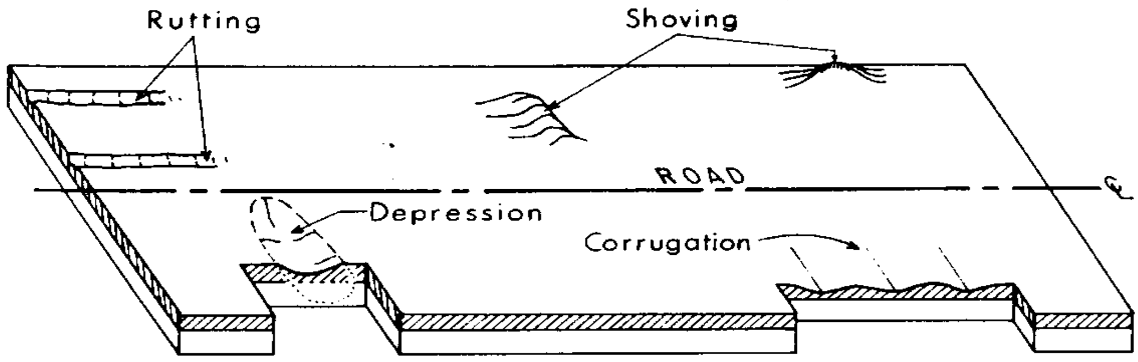


Figure A 2: Cracking of flexible pavements

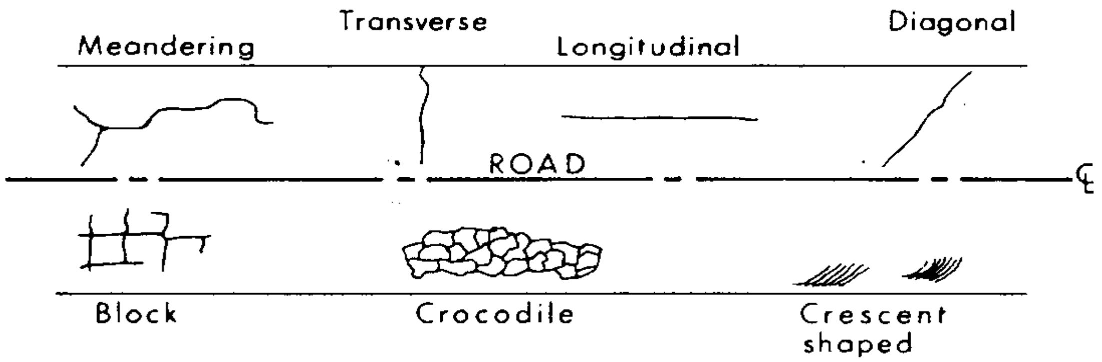
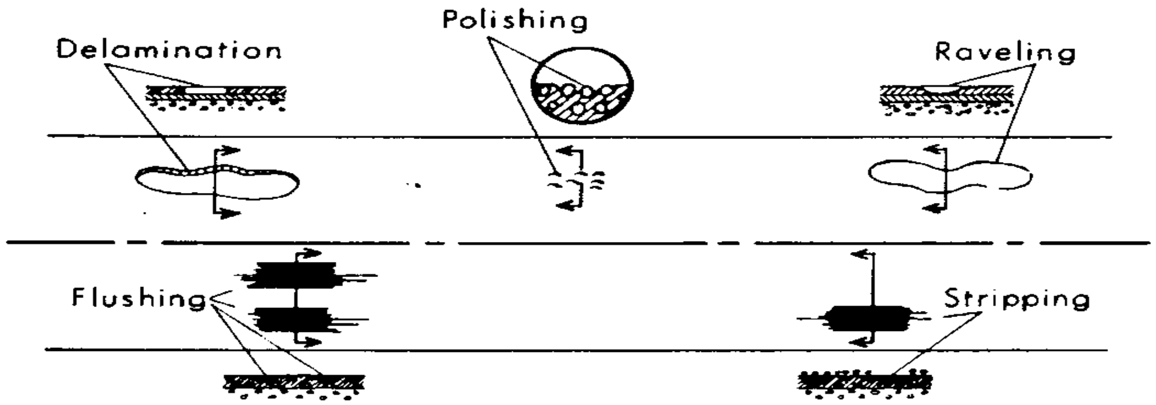


Figure A 3: Surface distress of flexible pavements



A.1.1 Deformation – Rutting (DR)

Description:
Longitudinal deformation in a wheelpath. Result of densification of pavement layers, including subgrade or plastic shear deformation of upper layers. Bound lower layers may not be affected. Length to width ratio – determined using a straight edge laid on the high points – normally greater than four to one. May occur in one or both wheelpaths of a lane but mostly in the outer wheelpath nearest to the pavement edge.

Causes:
- ingress of water through the pavement surfacing or road edges into base, subbase and subgrade
- structural overloading of the pavement and/or inadequate pavement thickness (exacerbated, in asphalt pavements, by high pavement temperatures)
- inadequate quality of pavement materials
- poor quality construction control, particularly compaction and drainage
- pavement at terminal condition.

Non-structural treatments:
- in-place asphalt recycling
- cold planing to remove high points
- microsurfacing
- thin asphalt surfacings (non-structural)
- ultra-thin overlays – provided rut depth ≤ 2.5 x mix size
- rip, reshape and reseal – appropriate for granular pavements.

Structural treatments:
- drainage improvements
- asphalt overlay or granular resheet
- deep lift asphalt
- partial reconstruction and asphalt overlay
- in situ stabilisation
- heavy patching
- total reconstruction.

A.1.2 Deformation – Shoving, Plastic Flow (DS)

Description:

Shoving: Bulging and horizontal deformation of the road surface – generally occurs in areas of high shear stress.

Plastic flow: Deformation in asphalt of asphalt surfaces.

Causes:

- lack of containment at pavement edge combined with swelling of moisture-susceptible pavement material and/or repeated passage of heavy vehicles on relatively narrow sealed formations
- inadequate pavement thickness
- inadequate quality of pavement materials e.g. asphalt mixes with poor aggregate interlock combined with turning/accelerating traffic; or poor binder/aggregate adhesion (stripping)
- inadequate compaction of surfacing or base material
- localised softening of asphalt binder due to fuel/oil spillage, or temperature-susceptible binder or excess binder content in asphalt
- lack of bond between pavement layers
- moisture in pavement and/or subgrade.

Non-structural treatments:

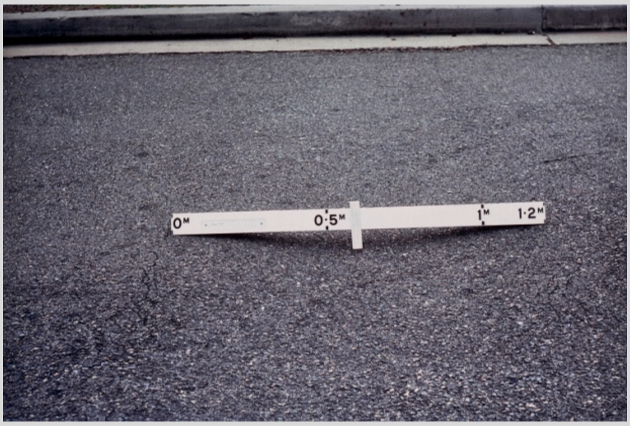
- in-place asphalt recycling
- cold planing of unsound material and replacement with adequate material
- where due to deficiency of unsealed shoulder, resheet shoulder.

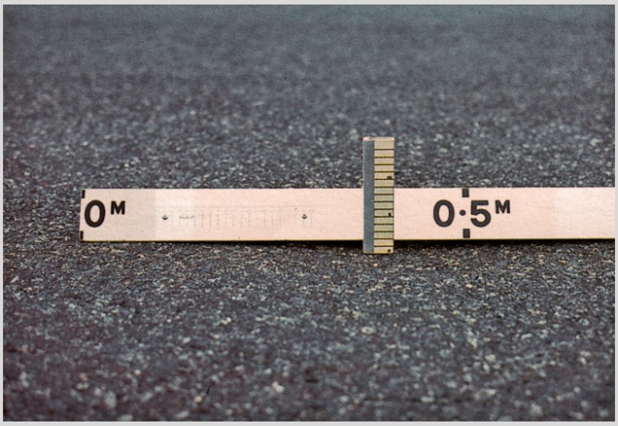
Structural treatments:

- drainage improvements
- asphalt or granular overlays
- partial reconstruction and overlay
- in situ stabilisation
- heavy patching
- reconstruction.

A.1.3 Deformation – Depression and Heave (DD)

Description:
Irregular depressions and bulges in the pavement surface.





Causes:

- moisture movement, especially in expansive clay subgrades
- inadequate drainage
- inadequate compaction, particularly of base material
- inadequate quality of pavement materials
- settlement of trench backfill due to poor compaction or softening caused by leakage from services.

Non-structural treatments:

- regulate with asphalt or cold overlay
- reduce moisture ingress by sealing shoulders.

Structural treatments:

- drainage improvements
- asphalt or granular overlays
- partial reconstruct and overlay
- in situ stabilisation
- heavy patching
- reconstruction.

A.1.4 Deformation – Corrugations (DC)

Description:
Transverse undulations in the pavement surface or base, most commonly associated with spray seal or unsealed pavements but can occur in thin asphalt surfacing.

Wavelengths of undulations can range between 0.3 and 2 metres.

Causes:
- local failure in the pavement
- inadequate material quality, e.g. inability of asphalt surfacing to resist heavy vehicle loading
- defective works practice such as irregular compaction energy input
- poor bonding between thin asphalt surfacing and concrete base.

Treatments:
- remove and replace base and reseal or overlay
- in situ stabilisation
- cold plane and overlay asphalt-surfaced pavements.

A.1.5 Cracking – Block Cracking (CB)

Description:
Interconnected cracks forming a series of blocks approximately rectangular in shape. Typically distributed over a large area of pavement.


Causes:
- reflection from underlying joints
- shrinkage or fatigue in an underlying bound (cemented) or macadam layer
- inadequate slab thickness
- ageing and hardening of bituminous surfacing.

Treatments:
- crack filling
- SAM seals, reinforced seals, ultra-thin overlays
- SAMI or geotextile seal plus asphalt overlay
- cold plane and overlay
- in situ asphalt recycling and overlay.

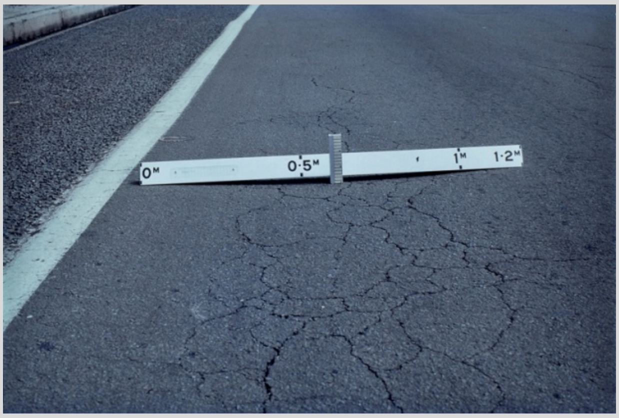
A.1.6 Cracking – Crocodile Cracking (Alligator Cracking, Crazing) (CR)

Description:

Interconnected or interlaced cracks forming a series of small polygons resembling a crocodile skin.

Crocodile cracking is often confined to the wheelpaths and may have a noticeable longitudinal grain.

The presence of crocodile cracking usually signifies that the surfacing has reached the end of its design life.



Causes:

- fatigue-induced structural cracking (high pavement curvature value)
- inadequate pavement thickness
- moisture in formation
- inadequate quality of pavement or surfacing materials (e.g. brittle or aged bitumen or low-strength aggregates)
- lack of compaction in asphalt or cementitious layers.

Non-structural treatments:

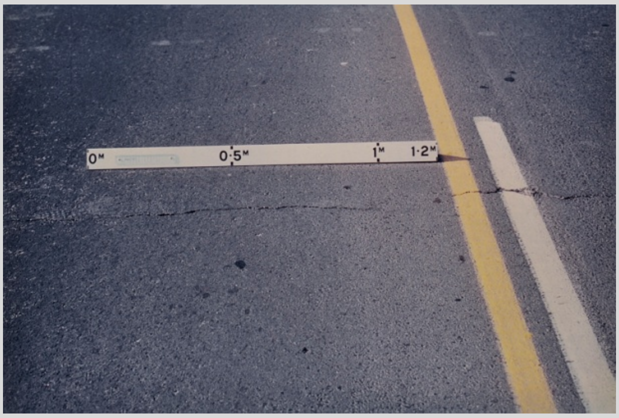
- SAM seals, reinforced seals, ultra-thin overlays
- SAMI or geotextile seal plus thin asphalt overlay
- cold planing and thin overlay
- in situ asphalt recycling.

Structural treatment:

- drainage improvements in combination with another treatment such as an overlay
- SAMI plus asphalt overlay
- in situ asphalt recycling plus overlay
- cold planing plus overlay
- in situ stabilisation
- heavy patching
- reconstruction.

A.1.7 Cracking – Transverse Cracking (CT)

Description:
An unconnected crack running across the pavement.

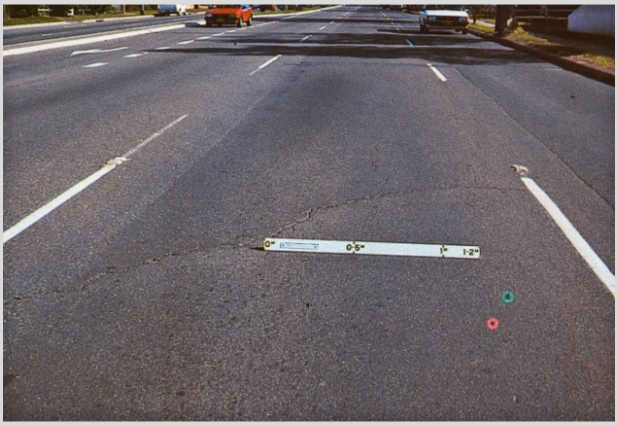



Causes:
- reflection of shrinkage crack or joint from an underlying cemented base
- construction joint or shrinkage crack in asphalt surfacing (shrinkage cracking in asphalt can be related to low ambient temperatures and/or bitumen hardening)
- structural failure of cement concrete base
- shrinkage of slab during curing as a result of sawing joints too late or excess slab length
- settlement associated with an underground service or a structure (bridge/culvert)
- intrusion of tree roots into the subgrade or pavement layers.

Treatments:
- crack seal
- SAM seals, reinforced seals, ultra-thin overlays
- SAMI or geotextile seal plus asphalt overlay
- cold planing and overlay
- in situ asphalt recycling.

A.1.8 Cracking – Diagonal Cracking (CD)

Description:
An unconnected crack running diagonally across the pavement.



Causes:

- age hardening of bitumen
- shrinkage of slab during curing as a result of sawing joints too late or excess slab length
- reflection of underlying joint
- settlement associated with an underground service or a structure (bridge/culvert)
- construction joints
- intrusion of tree roots.

Treatments:

- crack seal
- SAM seals, reinforced seals, ultra-thin overlays
- SAMI or geotextile seal plus asphalt overlay
- cold planing and overlay
- in situ asphalt recycling and overlay.

A.1.9 Cracking – Meandering Cracking (CM)

Description:
Unconnected irregular crack, varying in line and direction; which usually occurs singly.

Causes:

- moisture in formation
- reflection of shrinkage cracking from bound base
- in asphalt surfacings on concrete pavements, due to inadequate slab thickness, unstable slabs or slab settlement
- shrinkage in concrete slabs during curing as a result of delay in sawing joints or excessive slab lengths
- settlement associated with an underground service or a structure (bridge/culvert)
- intrusion of tree roots into subgrade or pavement layers.

Treatments:

- crack filling
- SAM seals, reinforced seals, ultra-thin overlays
- SAMI or geotextile seal plus asphalt overlay
- cold planing and overlay
- in situ asphalt recycling and overlay
- remove trees.

A.1.10 Cracking – Crescent-shaped Cracking (CC)

Description:
Crescent, or half-moon-shaped, cracks often occurring in closely spaced parallel groups and commonly associated with shoving.

Most commonly associated with asphalt surfacings.

Causes:
- poor bond between wearing course and underlying layers
- inadequate thickness of wearing course
- dragging of asphalt under paver screed during laying, especially at low temperatures
- high horizontal shear forces associated with turning and acceleration movements
- inadequate asphalt mix for the traffic conditions
- low modulus base layer.

Non-structural treatments:
- crack filling
- cold planing and overlay
- in situ asphalt recycling and overlay.

Structural treatments:
- overlay
- heavy patching
- reconstruction.

A.1.11 Cracking – Longitudinal Cracking (CL)

Description:

- Cracks which run longitudinally along the pavement.
- Can occur singly or as a series of parallel or echelon cracks.
- Some limited branching can occur.
- Longitudinal cracking is often the first type of cracking initiated in a wheelpath or rut.

Causes:

When occurring singly:

- reflection of joints or shrinkage cracks in underlying cemented base
- poorly constructed joint in asphalt surfacing
- displacement of joint at pavement widening
- reflection of joints associated with road widening.

When occurring as a series of near-parallel cracks:

- volume change of expansive clay subgrade
- cyclical weakening of the pavement edge
- differential settlement between cut and fill
- reflection of cracks in underlying cemented subbase.

Treatments:

- drainage improvements
- sealing shoulders
- crack filling
- cold planing and overlay
- heavy patching
- reconstruction.

A.1.12 Edge Break (EB)

Description:

Occurs along the unsupported edges of asphalt or sprayed seal surfaces where the surface of an unsealed shoulder is below the level of the adjacent pavement surface. Seal/shoulder interface is directly trafficked resulting in abrasion and shear failure of pavement edge (manifest as fretting). May occur locally or be continuous over a length of road. Frequently occurs on tight curves or where the edge of the pavement is vulnerable to tyre wear and attrition.

Causes:

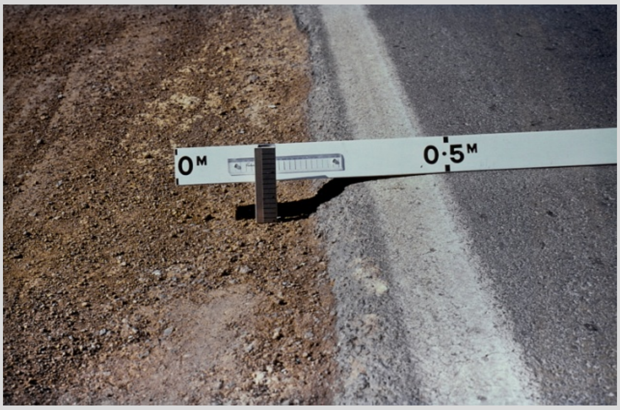
- an inadequate road alignment or sealed pavement width which causes vehicles to traffic the pavement edge
- omission of a shoulder resheet following pavement overlay
- erosion of shoulder by wind and/or water
- growth of vegetation at the edge of the surface seal.

Treatments:

- resheet shoulder
- bitumen stabilised shoulders
- seal shoulders
- local pavement widening.

A.1.13 Edge Drop-off (ED)

Description:
The vertical distance from the surface of the seal at the edge to the surface of the shoulder. Not usually considered a defect if the drop-off is less than 10 mm to 15 mm.



Causes:
- an inadequate road alignment or sealed pavement width which causes vehicles to traffic the pavement edge
- omission of a shoulder resheet following pavement overlay
- erosion of shoulder by wind and/or water
- growth of vegetation at the edge of the seal.

Treatments:
- resheet shoulder
- bitumen stabilised shoulders
- seal shoulders
- local pavement widening.

A.1.14 Delamination (SD)

Description:
Loss of a discrete section of wearing course layer.

A feature of delamination is that there is usually a clear delineation between the wearing course and the lower layer.

Causes:

- inadequate sweeping of granular base prior to priming
- inadequate prime or tack coat before placement of upper layers
- dislodging of fragments initiated by block cracking or crocodile cracking
- seepage of water through cracks in asphalt layer, resulting in breaking of bond between surface and lower layers
- weak loose layer immediately underlying a seal
- adhesion of surface binder to vehicle tyres
- inadequate thickness of asphalt layer.

Treatments:

- cold planing and overlay
- reseal.

A.1.15 Stripping of Sprayed Seals (SS)

Description:
The loss of aggregate from a sprayed seal leaving the binder exposed to direct tyre contact.

Causes:

- low binder application rate
- poor binder to stone adhesion due to dirty, dusty or wet aggregate exacerbated by lack of pre-coat on the aggregate
- age hardening (oxidation) or adsorption of binder
- incorrect blending of binder (cutter or flux oil content too high)
- stone deterioration
- inadequate rolling before opening of seal to traffic, particularly on curves and bends
- inappropriate stone size in reseal
- temperature susceptible bitumen.

Treatments:

- enrichment or rejuvenation of binder – only where aggregate loss is limited to a few stones
- reseal.

A.1.16 Stripping of Asphalt (SS)

Description:

The loss of bitumen and/or mineral aggregate or filler from an asphalt layer. The stripping could occur on the surface or within the layer.

Stripping within the layer may lead to the development of potholes.

Photographs show white fines that have been pumped to surface, an early indicator of stripping.

Causes:

- moisture entry through excessive voids or segregated or cracked surface
- low binder content
- age hardening (oxidation) or adsorption of binder
- incorrect mix design.

Treatments:

- depending on the extent of the stripping, the treatments may range from isolated repairs to complete removal and replacement of the affected asphalt layer.

A.1.17 Ravelling (or Fretting) (SR)

Description:
Progressive disintegration of pavement surface by loss of both binder and aggregate.

Causes:
- binder hardening and oxidation or damage by fuel
- inappropriate asphalt mix or poor mix design
- inadequate compaction and/or construction defects due to wet or cold weather, or the use of dirty, dusty or wet aggregate
- oil and fuel spillages.

Treatments:

- sprayed enrichment or rejuvenation
- cold overlay, slurry surfacing and microsurfacings
- asphalt overlay.

A.1.18 Flushing (or Bleeding, Fatty or Slick Surface, Black Spots) (SF)

Description:

An excess of binder on the surface of a pavement, which is liable to pick-up on tyres during hot weather. A potential safety concern because of loss of skid resistance.

Manifests as low texture depth and inadequate tyre to stone contact.

Causes:

- flushing in the underlying layer – i.e. a previous surfacing
- inappropriate asphalt type or inadequate mix design
- excessive rate of application of binder with respect to stone size in sprayed seals
- excessive prime or poor prime penetration into granular base causing excess binder in the surfacing
- in a seal or asphalt laid over a primer seal before volatiles in primer binder have evaporated
- penetration or punching of aggregate into an underlying layer e.g. low strength base
- oil and fuel spillages.

Treatments:

- reseal
- solvent treatment plus additional aggregate – for flushed sprayed seals
- cold planing, scrabbling and grooving – for fatty asphalt
- asphalt overlay.

A.1.19 Polishing (SP)

Description:

Smoothing and rounding of the upper surface of a sealing aggregate, usually occurs in the wheel tracks. Identified by relative appearance and feel of trafficked and untrafficked areas.

Polished areas will feel relatively smooth and will sometimes be noticeably shiny.

The degree of polishing cannot be quantified by observation.

Causes:

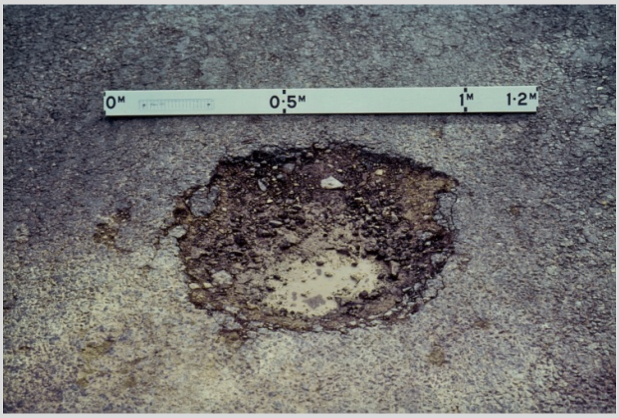
- inadequate resistance to polishing of surface aggregates, particularly by heavy traffic
- use of naturally smooth aggregates (e.g. water worn gravel).

Treatments:

- reseal
- cold overlay, slurry seal, microsurfacing
- ultra-thin overlay
- asphalt overlay.

A.1.20 Potholing (HO)

Description:
A steep-sided or bowl-shaped cavity extending into layers below the wearing course.



Causes:
- loss of surface material due to ravelling, stripping, cracking and/or delamination
- load-associated disintegration of the base
- ‘pick-up’ of bitumen wearing surface caused by binder adhesion to tyres
- moisture entry into a granular base through cracked surface course, resulting in localised softening
- poor quality base material.

Treatments:
- normally repaired by routine maintenance techniques
- reseal
- asphalt overlays.

A.1.21 Patches: Expedient Patch and Reconstructed Patch (PA)

Description:

A repaired section of pavement ranging in size from less than 1 m^2 to many linear metres of a half or even full pavement width.

It may or may not be associated with either a loss of serviceability (apart from a loss of appearance) or structural capacity.

Expedient patches will not normally be straight-sided and are often a temporary measure.

They may contribute to increased road roughness and subsequent further distress.

Example of expedient patch

Reconstructed patches are generally more permanent and will usually be straight sided.

Example of reconstructed patch

Causes:

- expedient patch: the repair of surfacing deficiencies (deformation or rutting, cracking, stripping, edge break etc.) without prior removal of the affected material
- reconstruction patch: ranges from correction of a pavement deficiency to the reinstatement of a service trench
- inadequate compaction may lead to further deformation and distress.

Treatments:

- A patch does not indicate the need for any further action but many patches, particularly within a short time period, indicate that there may be a systemic problem in the pavement that requires a more considered treatment. Consideration should be given to the reasons for the patching and a resurfacing appropriate to that type of defect.

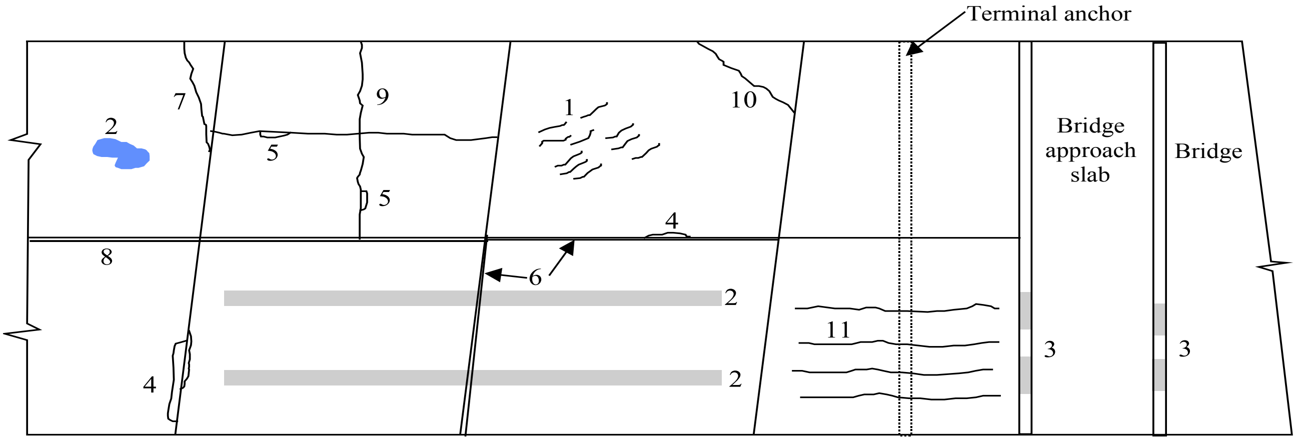
A.2 Concrete Pavements

The principal modes of pavement distress of plain concrete pavements are as follows:

- surface defects
- joint defects
- structural defects.

Typical distress types are shown in Figure A 4.

Figure A 4: Typical defects in plain concrete pavements



Category	Key	Brief Description
Surface	1	Plastic (pre-hardening) cracking
	2	Mortar and/or texture loss, scaling, potholing
Joint	3	Sealant failure
	4	Joint spalling
	5	Crack spalling
	6	Joint stepping
	7	Joint-related cracking
	8	Joint opening (of tied joints)
Structural	9	Mid-slab cracking (full depth and/or width)
	10	Corner cracking (full depth)
	11	Anchor cracking

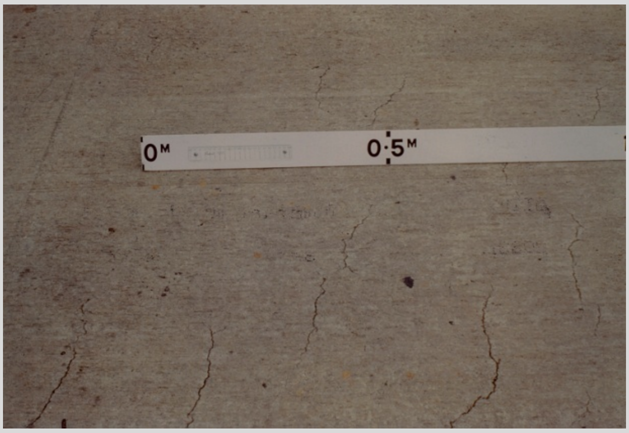
In assessing the condition of a concrete pavement, it is important to differentiate attributes which constitute abnormal behaviour and those which are to be expected as normal for that pavement type. Examples are:

- In CRCP and JRCP bases, limited transverse cracking is to be expected but, provided that other secondary distress is not evident (such as wide spalling) and that the cracks remain tight, the pavement should not require any treatment.
- At formed and tied longitudinal joints, which are designed to act as hinges, some minor spalling can be expected to occur and would not normally require treatment.
- Depending on the pavement type, some observed conditions may be cosmetic blemishes only. Whilst apparent to an inspector in close-up, they may be neither visible to, nor felt by, the road user. For this reason, it is important that users of this Part become familiar with the different types and characteristics of concrete pavement bases.

A.2.1 Plastic Shrinkage Cracking

Description:
In PCP pavements plastic shrinkage cracking usually has the following characteristics:

- *Frequency*: typically, at 50 mm to 500 mm spacing
- *Length*: typically, less than 500 mm
- *Orientation*: typically skewed, but sometimes also transverse or longitudinal
- *Location*: typically, within the slab, and rarely intersecting a joint or edge
- *Depth*: from 20 mm to 70 mm but can extend to the full depth of the slab.



Causes:
- paving in hot windy weather
- slight downhill movement on steeper crossfalls or gradients
- reflection of reinforcement pattern in JRCP pavements; indicative of settlement of the concrete during the latter part of the plastic phase when the mix has become too stiff to flow around the reinforcing bars.

Treatments for unreinforced concrete:
- The treatment is very much dependent on the activity, length, depth and orientation of cracks. If the cracking is deemed to be dormant (i.e. unlikely to grow), then a cosmetic treatment would suffice, although grit blasting will be required to remove curing compound and dust from the faces (cracking will often initiate before curing compound is applied, hence the compound is likely to penetrate the crack).

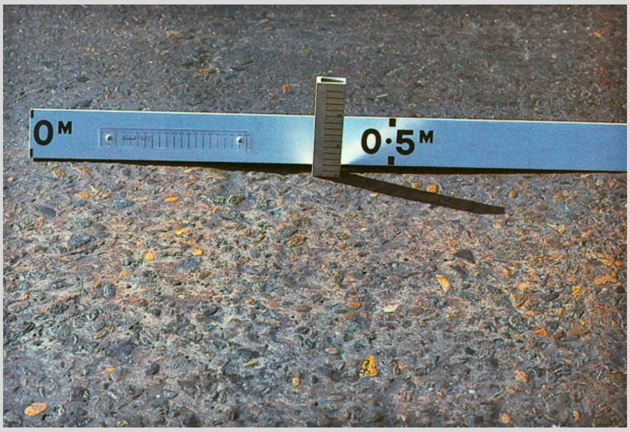
Treatments for Reinforced Concrete:
- Treatment should focus on protecting the reinforcement against corrosion. In order to bond a sealant, air or grit blasting would be required to remove curing compound and dust from the crack faces.

A.2.2 Scaling

Description:

Scaling describes the loss of a shallow layer or flaking of mortar from the surface and is typically less than 30 mm deep from the surface.

This may occur either in a localised area or over a wide surface area.



Causes:

When scaling occurs early in the life of the pavement, the cause may be low-quality concrete caused by factors such as:

- delamination within a mortar-rich layer just below the surface. This can result from the delayed addition of concrete over a slurried surface or excessive use of evaporation retarder
- excessive hand finishing, and/or slurrying of the surface by addition of water or evaporation retarder
- low cementitious content and/or high water – cement ratio
- inadequate curing and protection, or very cold or freezing conditions in the early days after concrete placement
- inadequate compaction (though this is more likely to result in structural failure or potholing).

Treatments:

Depending on the extent of distress, grinding or shot blasting may be required prior to re-texturing in order to restore a uniform surface profile.

A.2.3 Texture Loss (Mortar Loss and Rutting)

Description:
Loss of texture describes the reduction in macrotexture of a concrete pavement surface. It may be associated with scaling (mortar loss) or abrasion under traffic (rutting).

Macrotexture is commonly provided by tyning to depths between 0.4 mm and 1 mm.

Texture loss can have a pronounced adverse effect on the skid resistance on a concrete pavement. This can be exacerbated by ponding of water in the wheelpaths where the loss of texture is associated with rutting.

Causes:
- scaling
- abrasion due to traffic – confined to wheelpaths and typically a gradual process occurring over about 20 years
- excessive hand finishing, and/or slurrying of the surface by addition of water or evaporation retarder
- low cementitious content and/or high water – cement ratio
- inadequate curing and protection, or very cold or freezing conditions in the early days after concrete placement
- rain-affected surface mortar.

Treatments:
Depending on the extent of wheelpath wear, grinding (or ‘profiling’) or shot blasting may be required prior to re-texturing in order to restore a uniform transverse surface profile.

A.2.4 Potholing

Description:
Potholing is the loss of concrete to a depth and extent in excess of the largest aggregate size in the concrete.

Causes:

- presence of soft aggregates, clay balls or other foreign objects (e.g. mice) in the mix
- localised extremely poor-quality concrete
- localised extreme lack of compaction.

Treatments:

- remove and replace slab.

A.2.5 Sealant Distress

Description:
The repair and replacement of joint sealants is an accepted part of routine maintenance of a concrete pavement but a competent (and well-installed) seal is expected to have an effective life of around 15 years, and perhaps longer.

Causes:
- inappropriate selection of the sealant type
- poor design of the sealant dimensions
- installation contrary to manufacturer’s recommendations and/or design
- loss of adhesion between the sealant and its reservoir
- traffic damage, possibly due to insufficient recess and/or the presence of stones
- age deterioration of the sealant.

Treatments:
- remove and replace sealant.

A.2.6 Joint Spalling

Description:

Joint spalling describes the fracture or disintegration of the concrete along either a transverse or longitudinal joint edge (arris).

Spalling is categorised as minor, shallow or deep. Minor spalling is less than 50 mm wide while shallow spalling is greater than 50 mm wide but does not extend beyond the mid-depth of the slab. Deep spalling describes spalling to depths in excess of half the slab thickness.

Causes:

- arris rounding, where joint width is less than 30 mm
- weak concrete, mechanical damage including early saw cutting, ingress of incompressibles (all categories of spalling)
- inadequate concrete base depth to the top corrugation in formed corrugated longitudinal joints (shallow and deep spalling)
- dowel restraint due to misalignment, inadequate debonding of bent or burred dowels (deep spalling only).

Treatments:

- resealing, possibly widening of the joint by saw cutting to remove rounded area (for arris rounding)
- re-saw and reseal or rout and reseal the joint (for minor spalling)
- saw cut or rout the spalled area and patch with a thin bonded concrete (shallow spalling)
- slab replacement (deep spalling only).

A.2.7 Cracking at Contraction Joints

Description:
Cracking at joints with a characteristic Y shape.

Causes:
- accumulation of mortar in joint cavities.

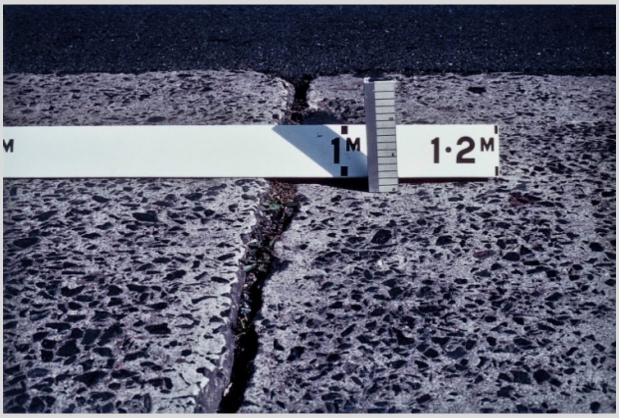
Treatments:
- remove mortar and if cracking does not develop, further treatment is not warranted.

A.2.8 Joint Stepping or Faulting

Description:

Joint stepping or faulting is a permanent vertical separation of the two slabs on either side of a joint. Faulting may often be accompanied by the pumping of fines from the subbase/subgrade.

This defect can occur at either a longitudinal or transverse joint.



Causes:

- inadequate subbase/subgrade support or differential support under adjacent slabs
- moisture movement, especially in expansive clay subgrades
- curling and warping of slabs (associated with temperature changes) and rocking of slabs.

Treatments:

- rectification of underlying cause(s) (moisture control, slab undersealing)
- cross-stitching or replacement of cracked slabs
- grinding or profiling to correct ride quality
- resealing of joints
- routing and sealing of cracks.

A.2.9 Slab Rocking

Description:
Rocking of a slab under the passage of traffic.

Rocking may often be accompanied by the pumping of fines from the subbase/subgrade, which may lead to slab failure.


Causes:
- inadequate subbase/subgrade support or differential support under adjacent slabs
- ingress of water and pumping of fines.

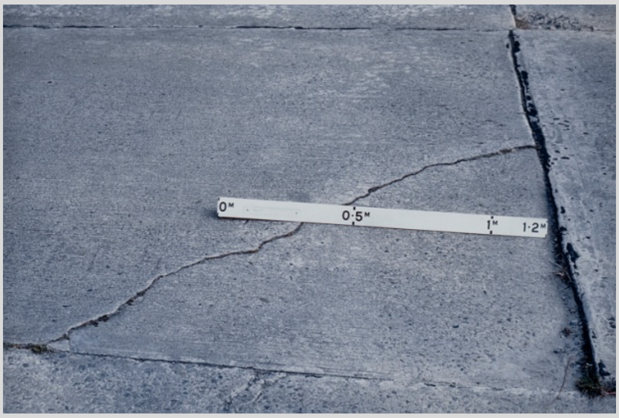
Non-structural treatments:
- reseal joints
- grind surface.

Structural treatments:
- rubblise/crack and seat concrete and then overlay
- slab undersealing and overlay
- reconstruction.

A.2.10 Structural Cracking

Description:
Structural cracking describes active (i.e. moving or growing) cracks, which extend through the full thickness of a concrete slab. The cracking typically falls within one of the following categories:

- longitudinal or transverse cracking over the full slab length or width
- skewed or corner cracking.



Causes:
- flexural fatigue cracking
- flexural fracture (a single very high load)
- low strength concrete
- voids around tie bar slots
- poor joint detailing e.g. mismatched joints, odd-shaped slabs, excessive length or width of slabs
- failure of a joint initiator
- late saw cutting
- reflection of cracking in subbase
- uneven subbase support.

Non-structural treatments:
- cross-stitching.

Structural treatments:
- slab replacement
- rubblise/crack and seat concrete and then overlay
- slab undersealing and overlay
- reconstruction.

A.2.11 Tie Bar Voids

Description:

Tie bar voids describe the introduction of air pockets around tie bars depressed into the slab during slipform paving, where recompaction of the concrete has not been adequate.

The voids are not normally detectable except when manifested by widened joints or structural cracking.

The opening of joints may reflect voids confined to the perimeter of the bar, which reduce the pull-out strength of the bar.

Structural cracking can reflect voids, which have initiated a weakness up to the surface of the slab.

Causes:

- inadequate compaction of the base after placement of the tie bar.

Treatments:

- cross-stitching – (voids confined to the perimeter of the bar)
- as for structural cracking – (voids forming a structural weakness up to the surface).

Appendix B Weighted Mean Annual Pavement Temperatures

The temperatures listed in Table B 1 can be used to select a Weighted Mean Annual Pavement Temperature (WMAPT, °C) for determining design moduli for asphalt and foamed bitumen stabilised materials and for adjusting measured deflections on flexible pavements from the measurement temperature to an in-service pavement temperature.

The following method was used to calculate the WMAPT at each site:

1. Obtain from the Bureau of Meteorology the monthly average daily maximum air temperature and the annual monthly daily minimum air temperature – www.bom.gov.au/climate/averages.
2. Calculate the monthly average air temperatures by averaging the maximum and minimum air temperatures.
3. Using Equation A1 and the monthly average air temperature (T_{air}), calculate the temperature weighting factors (WF) for each month.
4. For each site, average the 12 weighting factors obtained in Step 3.
5. Using average WF from Step 4 and Equation A2, estimate the weighted mean annual air temperature (WMAAT) for each site.
6. Using the WMAAT and Equation A3, estimate the WMAPT for each site.

Equation A1: Shell Weighting Factors (based on Chart W of the *Pavement Design Manual* (Shell 1978)):

$$WF = 10^{(-1.224+0.06508T_{air} - 0.000145{T_{air}}^2)} \qquad \text{A1}$$

Equation A2: WMAAT from average WF (based on Chart W of the *Pavement Design Manual* (Shell 1978):

$$WMAAT = 19.66 + 16.91 logWF + 0.3117(logWF)^2 \qquad \text{A2}$$

Equation A3: Estimating WMAPT from WMAAT (Chart RT of the *Pavement Design Manual* (Shell 1978, 100 mm asphalt):

$$WMAPT = -12.4 + \frac{6.32WMAAT}{\ln{(WMAAT)}} \qquad \text{A3}$$

Table B 1: Weighted Mean Annual Pavement Temperatures

Victoria	
Town	**WMAPT**
Bairnsdale	22
Ballarat	20
Benalla	26
Bendigo	24
Bright	22
Charlton	25
Dandenong	23
Dookie	25
Echuca	26
Frankston	23
Geelong	23
Horsham	24
Melbourne Region	24
Mildura	28
Nhill	24
Sale	23
Seymour	24
Swan Hill	27
Wangaratta	26
Warragul	22
Warrnambool	21
Wodonga	26
Yallourn	22

Tasmania	
Town	**WMAPT**
Burnie	20
Campbell Town	18
Devonport	20
Geeveston	18
Hobart	20
Launceston	20
New Norfolk	19
Queenstown	17
St Helens	20
Scottsdale	19
Swansea	20

South Australia	
Town	**WMAPT**
Adelaide	27
Bordertown	24
Ceduna	26
Keith	25
Murray Bridge	26
Port Augusta	30
Port Pirie	29
Renmark	28
Whyalla	29

Western Australia	
Town	**WMAPT**
Albany	24
Broome	40
Bunbury	26
Cape Leeuwin	26
Carnarvon	34
Dampier	40
Esperance	26
Eucla	27
Fremantle	28
Geraldton	31
Kalgoorlie	30
Kununurra	42
Manjimup	24
Meekatharra	36
Merredin	30
Morawa	32
Mt Magnet	35
Narrogin	26
Newman	39
Norseman	28
Northam	30
Ongerup	25
Paraburdoo	40
Perth	29
York	29

NSW and ACT			
Town	**WMAPT**	**Town**	**WMAPT**
Albury	26	Liverpool	28
Armidale	23	Merimbula	24
Bathurst	22	Mittagong	22
Bega	24	Molong	24
Bellingen	30	Moree	31
Blayney	19	Moruya	25
Bourke	33	Moss Vale	22
Braidwood	20	Mudgee	26
Broken Hill	30	Murrurundi	26
Byron Bay	31	Murwillumbah	31
Campbelltown	27	Narooma	24
Canberra	23	Narrabri	31
Casino	31	Narrandera	27
Cessnock	28	Newcastle	28
Cobar	31	Nowra	26
Coffs Harbour	29	Nyngan	31
Cooma	20	Orange	20
Coonabarabran	26	Parkes	28
Coonamble	31	Parramatta	28
Cowra	27	Port Macquarie	27
Deniliquin	27	Queanbeyan	23
Dubbo	29	Richmond	28
Finley	27	Singleton	29
Forbes	28	Sydney Region	28
Gilgandra	29	Tamworth	28
Glen Innes	22	Taree	28
Gosford	27	Tenterfield	24
Goulburn	22	Thredbo	13
Grafton	31	Tumut	23
Griffith	28	Wagga Wagga	26
Gundagai	26	Walgett	33
Hay	28	Warialda	29
Inverell	26	Wellington	28
Katoomba	20	Wentworth	29
Kempsey	29	Wilcannia	32
Kiama	27	Wollongong	27
Kiandra	12	Wyong	26
Lismore	30	Yass	24
Lithgow	20	Young	25

Queensland			
Town	**WMAPT**	**Town**	**WMAPT**
Ayr	35	Julia Creek	39
Baralaba	35	Kingaroy	29
Barcaldine	36	Longreach	37
Beaudesert	31	Mackay	34
Biloela	32	Maryborough	32
Birdsville	37	Miles	32
Blackall	36	Mitchell	32
Bollon	33	Monto	32
Boulia	38	Mt Isa	39
Bowen	36	Nambour	31
Brisbane Region	32	Normanton	40
Bundaberg	33	Palmerville	38
Cairns	37	Pittsworth	28
Caloundra	31	Quilpie	36
Camooweal	39	Richmond	38
Cardwell	36	Rockhampton	35
Charleville	34	Roma	33
Charters Towers	36	Southport	31
Clermont	35	St George	33
Cloncurry	39	St Lawrence	35
Cooktown	38	Stanthorpe	25
Cunnamulla	34	Surat	33
Dalby	30	Tambo	33
Emerald	35	Taroom	33
Gayndah	33	Thargomindah	35
Georgetown	38	Toowoomba	27
Gladstone	34	Townsville	37
Goondiwindi	32	Urandangi	38
Gympie	32	Warwick	28
Herberton	30	Weipa	39
Hughenden	37	Windorah	37
Ipswich	32	Winton	38
Isisford	36		

New Zealand	
Town	**WMAPT**
Auckland	23
Christchurch	19
Dunedin	18
Gisborne	21
Greymouth	19
Hamilton	21
Invercargill	17
Kaikoura	19
Masterton	20
Napier	21
New Plymouth	21
Nelson	21
Oamaru	18
Palmerston North	21
Queenstown	18
Rotorua	20
Taupo	19
Tauranga	21
Timaru	19

Northern Territory	
Town	**WMAPT**
Alice Springs	33
Barrow Creek	37
Daly Waters	40
Darwin	41
Katherine	40
Tennant Creek	39

Appendix C Adjustment of Deflections for Temperature

For flexible pavements with asphalt surfacings, the measured deflections vary with the pavement temperature during the deflection measurements. The greater the asphalt thickness, the more significant the variation.

For such pavements, the following procedures allow for the adjustment of the maximum deflection to a representative in-service temperature. The in-service pavement temperature at a site is characterised by the weighted mean annual pavement temperature (WMAPT). WMAPTs have been determined for selected locations throughout Australia and New Zealand (Appendix B) so that for any particular site, the WMAPT of the nearest or most appropriate location should be adopted or calculated using the method given in Appendix B.

For asphalt-surfaced granular pavements in which the surfacing thickness exceeds 25 mm, the deflections should first be corrected for seasonal moisture effects, and then corrected to the WMAPT using the following procedure:

Step 1 Determine the temperature factor f_T according to Equation A4.

$$f_T = \frac{\text{WMAPT for the site}}{\text{Measured pavement temperature at time of testing}} \quad \text{A4}$$

Step 2 Determine the temperature correction factors using Figure C 1 for deflectograph and Benkelman Beam and Figure C 2 for FWD and TSD. No temperature correction is required if the bituminous surfacing is less than 25 mm thick. Note that the TSD factors are based on measurement speed of 40–80 km/h and relate to pavement deflection estimated by the area under the deflection slope curve method (Section 4.9.2).

Step 3 Multiply the deflection by the corresponding deflection temperature correction factors.

Figure C 1: Temperature correction of deflectograph and Benkelman Beam deflections for various asphalt thicknesses

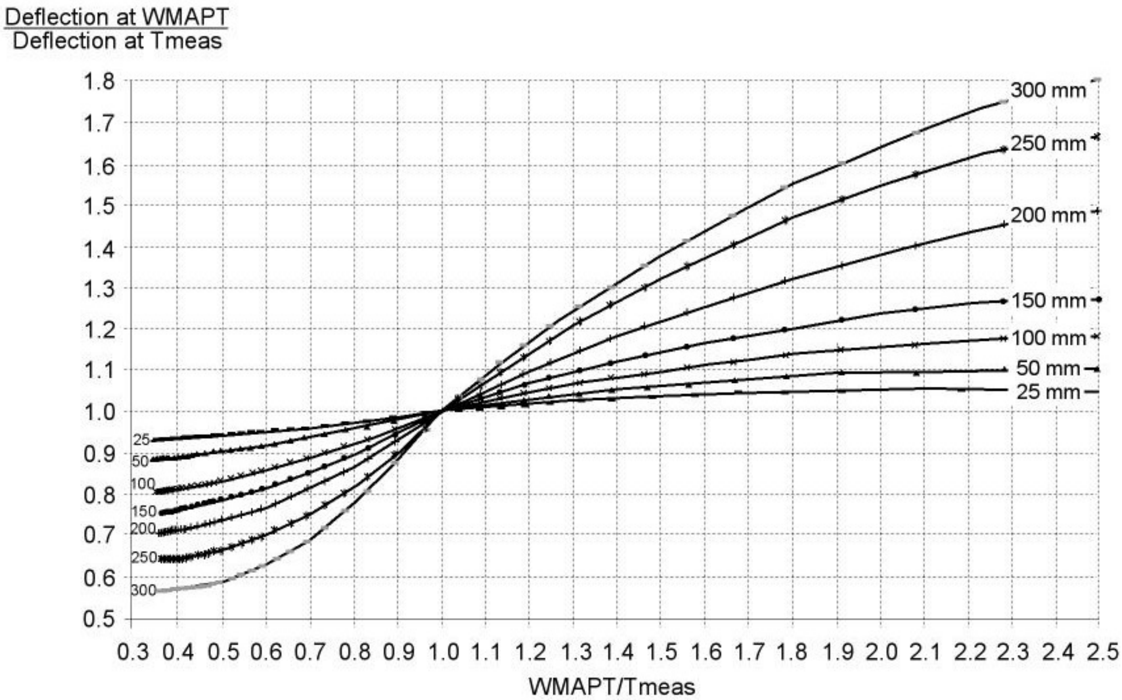
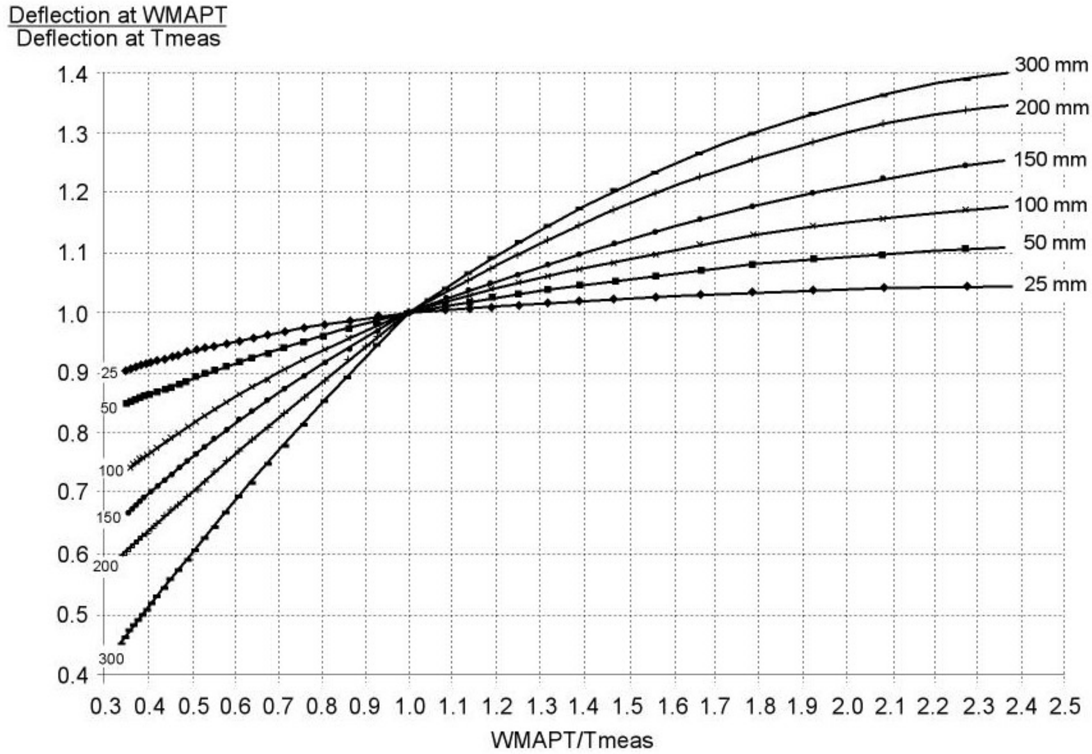


Figure C 2: Temperature correction of FWD and TSD deflections for various asphalt thicknesses



Appendix D Identifying Homogeneous Sub-sections Using the Cumulative Difference Approach

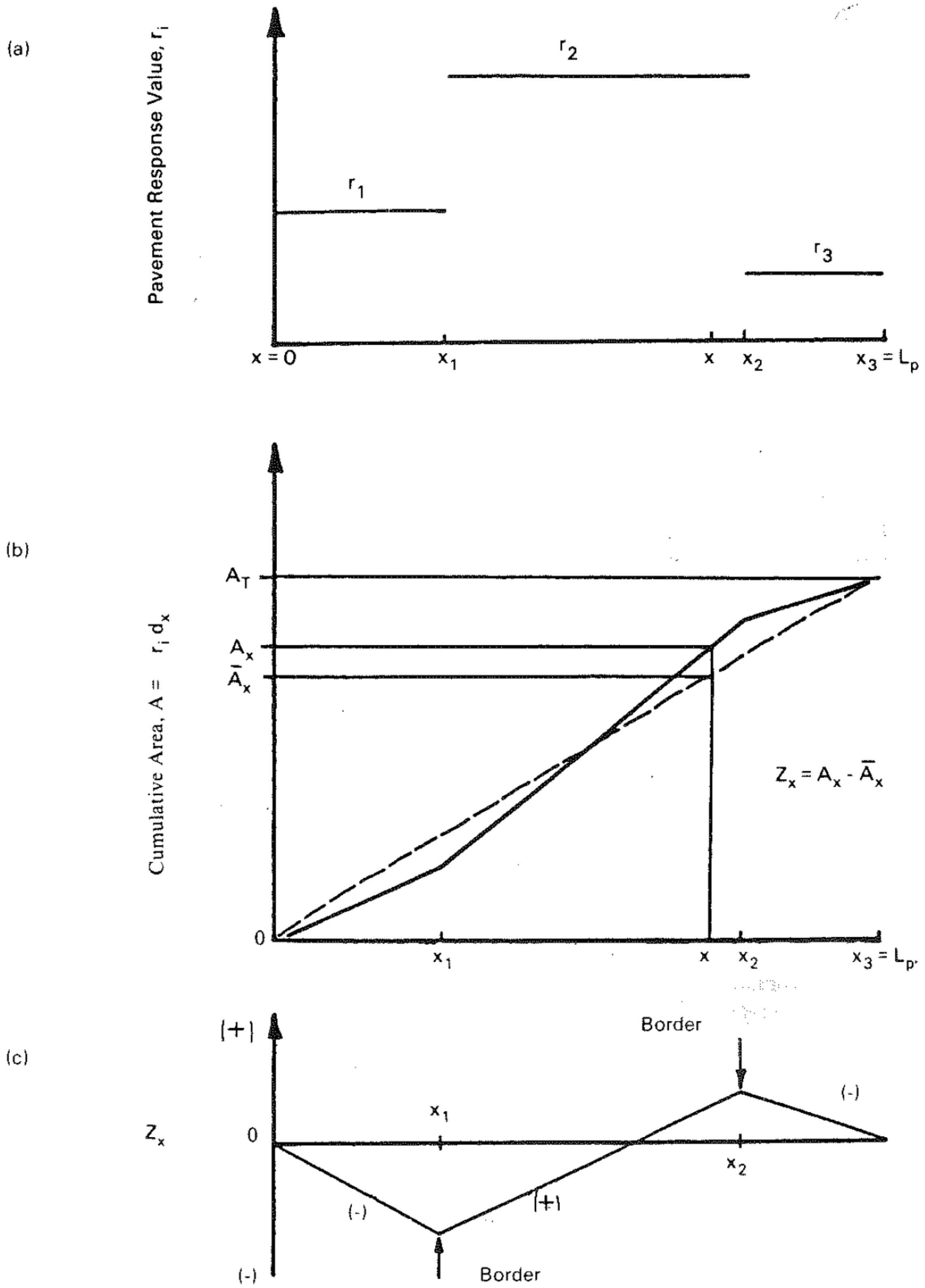
The AASHTO *Guide for Design of Pavement Structures* (AASHTO 1993) describes a method for delineating statistically homogeneous sub-sections from pavement response measurements. The method is called the cumulative difference approach (CDA).

Figure D 1 illustrates the method using the initial assumptions of a continuous and constant response value (r_i) within various intervals (0 to x_1, x_1 to x_2, x_2 to x_3) along a project length. In this example, it is obvious that three unique units having different response magnitudes (r_1, r_2 and r_3) exist along the project. Figure D 1(a) illustrates such a response-distance measurement. Figure D 1(b) shows the cumulative area under the response-distance plot. The solid line indicates the results of the measured responses. The dashed line represents the cumulative area calculated using the average of the three response measurements.

As noted in Figure D 1(b), Z_x is the difference in cumulative area, at a given distance (x_i) between the measured response and the average response. Figure D 1(c) shows the Z_x values plotted against distance. An examination of the plot illustrates that the location of unit boundaries always coincides with the location (along x) where the slope of the Z_x function changes algebraic sign (i.e. from negative to positive or vice versa). Such changes in slope are used to identify the boundaries between homogeneous sub-sections. In the identification of sub-sections with homogeneous pavement deflections, the criteria for sub-section identification also includes a minimum sub-section length reflecting construction practices.

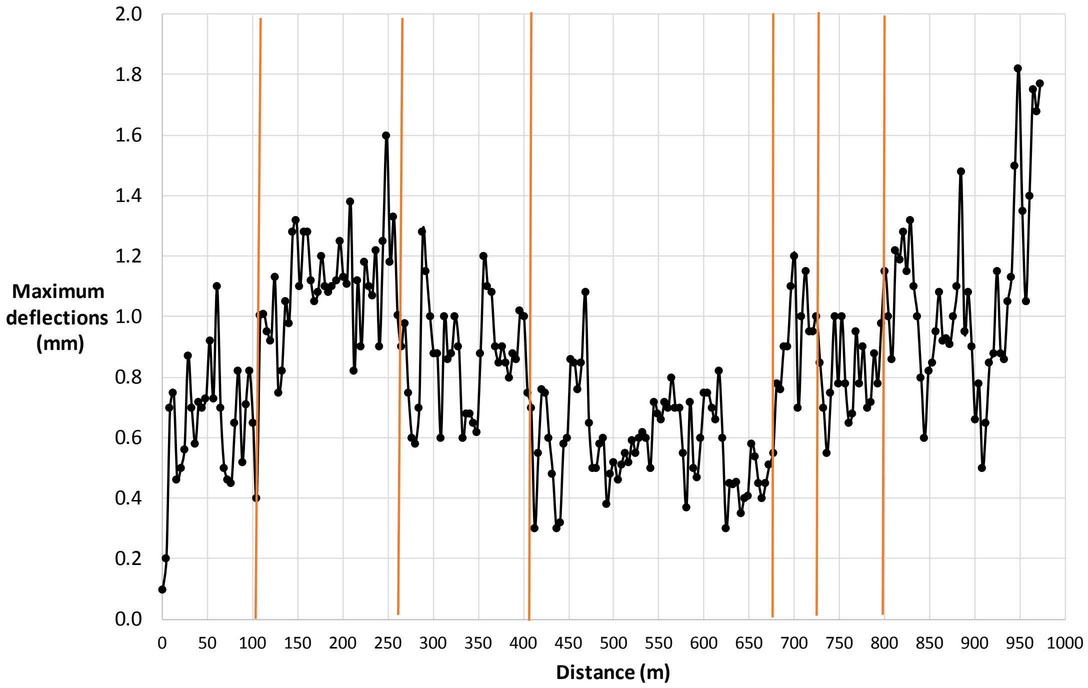
Figure D 2 shows an example of the application of the CDA to analyse deflections including a minimum sub-section length of 50 m.

Figure D 1: Concepts of cumulative difference approach to delineate homogeneous sub-sections



Source: AASHTO (1993).

Figure D 2: Example of CDA applied to maximum deflection measurements



Appendix E Granular Overlay Thickness Design Worksheets

EMPIRICAL DESIGN OF A GRANULAR OVERLAY ON A GRANULAR PAVEMENT	
Project	
Date	
Design traffic loading	ESA
Deflection measurement device (Table 9.2)	
Measured Characteristic Deflection	mm
Seasonal moisture correction factor (Table 9.1)	
Adjusted Characteristic Deflection	…....... × …....... ×…....... = mm
Design deflection (Figure 9.2)	mm
Granular overlay thickness (Figure 9.3)	mm

Appendix F Example of the Empirical Design of a Granular Overlay on a Flexible Pavement

A homogeneous section of spray-sealed granular pavement was tested with a Deflectograph. The measured Characteristic Deflection for a homogeneous section was 0.85 mm.

As the project was located in a temperate climate and the deflections were measured in April, when the pavement would not normally be at its wettest, it was considered appropriate to use a seasonal moisture correction factor of 1.3 in accordance with Table 9.1. From Table 9.2, the deflection standardisation factor for Deflectograph maximum deflections is 1.2. Therefore, the fully adjusted characteristic deflection is $0.85 \times 1.3 \times 1.2 = 1.32$ mm.

The design traffic loading for the project section was 3×10^6 ESA. From Figure 9.2, the design deflection is 0.99 mm.

From Figure 9.3, a 110 mm granular overlay is required to reduce the seasonally corrected characteristic deflection (1.32 mm) to the design deflection (0.99 mm).

Appendix G Composite Modulus Calculation

As described in Section 10.5.1, the calculation of the composite modulus may be useful in guiding the back-calculation of subgrade moduli.

The composite modulus is the weighted mean modulus of the equivalent half space calculated from the surface deflection using Boussinesq's equations (Ullidtz 1987). The composite modulus at any point away from the centre of the FWD load plate is calculated using Equation A5.

$$CM_r = \sigma_0(1-\mu^2)\left(\frac{a^2}{r\ x\ d_r}\right) \qquad \text{A5}$$

where

CM_r	=	composite modulus at a distance r from the centre of the FWD loading plate, (MPa)
σ_0		contact stress of FWD loading plate, (MPa)
μ	=	Poisson's ratio, usually 0.35
a		radius of FWD loading plate, usually 150 mm (mm)
r	=	radial distance from the centre of the FWD loading plate where r > 0, (mm)
d_r	=	measured surface deflection at distance r from the centre of the FWD loading plate (mm)

It is assumed that the FWD loading plate is spread from the pavement layers to the subgrade through load transfer by the layers through a cone of about 45º from the centre of the FWD loading plate. In this case, the FWD surface deflections measured at offsets more than depth from the pavement surface to the top of the subgrade reflect the subgrade moduli variation with depth.

For example, consider a 450 mm thick pavement on which the FWD surface deflections have been measured at offsets of 0 mm, 300 mm, 450 mm, 600 mm, 750 mm, 900 mm, 1200 mm, 1500 mm and 1800 mm. The deflections measured at offsets of 450 mm or more largely reflect the subgrade support (Figure G 1). Figure G 2 shows the calculated composite modulus variation with deflection offset and, hence, with depth in the subgrade. In this case, the composite moduli are increasing with depth, which is indicative of the subgrade modulus increasing with depth. Such information is useful in selecting the seed moduli to commence the back-calculation of the subgrade moduli.

Figure G 1: Example of surface deflections measured at offsets relevant to the subgrade characterisation

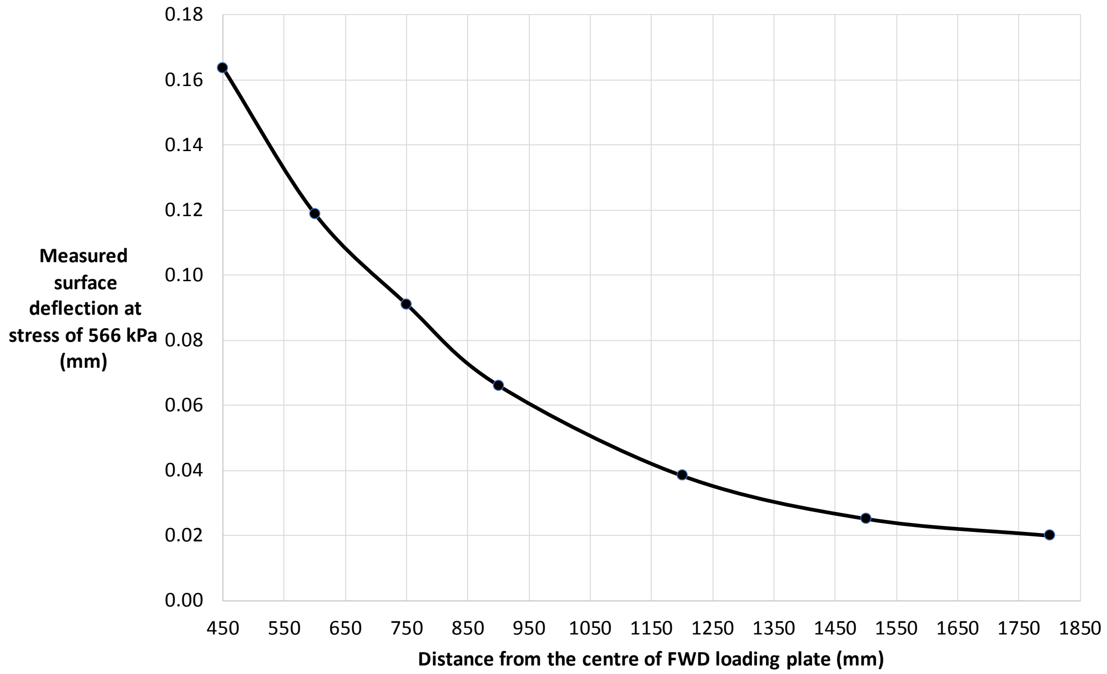
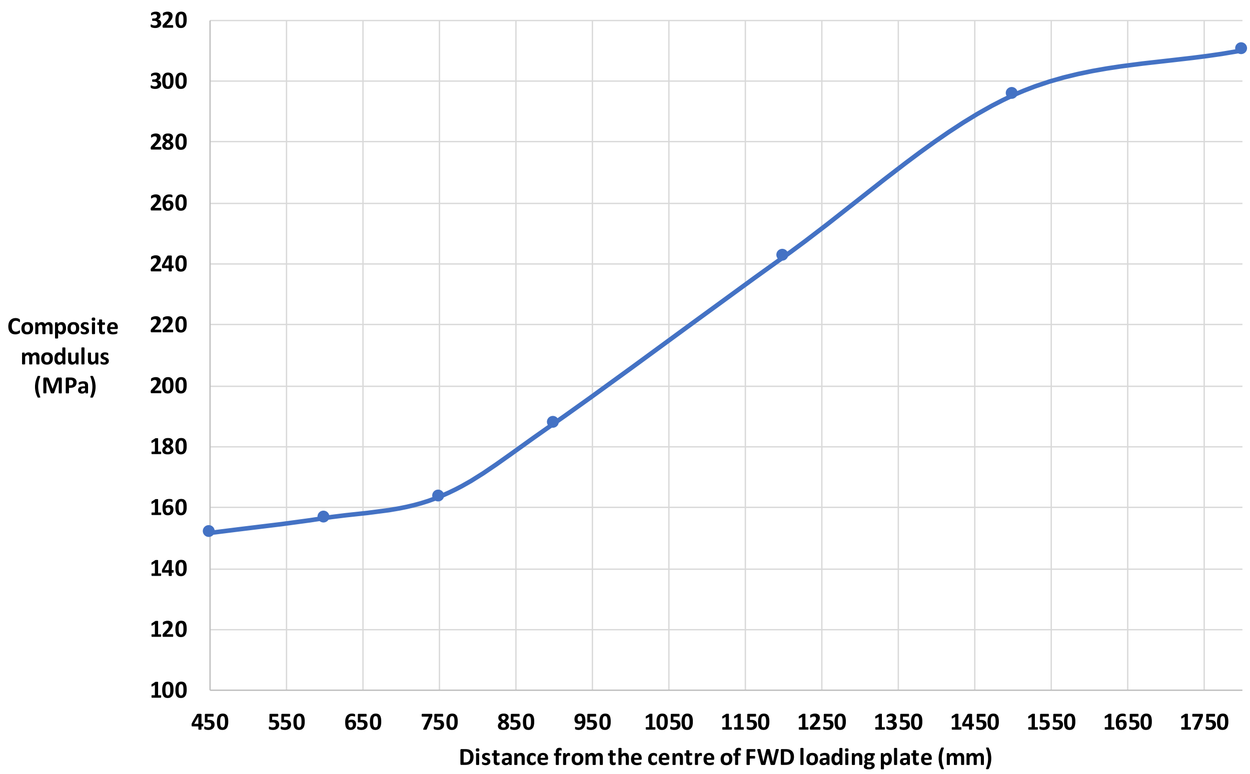


Figure G 2: Example of composite modulus values reflecting subgrade modulus variation with depth



Appendix H Elastic Characterisation of Foamed Bitumen Stabilised Materials

H.1 Introduction

As the design method for new pavements (Austroads 2018a) does not address the thickness design of pavements with foamed bitumen stabilised (FBS) materials, this section details an interim elastic characterisation process for use in the mechanistic-empirical design of strengthening treatments (Section 10).

H.2 Distress Modes

The two load-associated distress modes that have been identified for FBS materials are:

- rutting of the stabilised layer
- fatigue cracking of the stabilised layer.

Rutting of the stabilised layer can generally be avoided by the minimum mix design described in Austroads (2019a). Early-life modulus is enhanced by the use of a secondary binder such as lime or cement.

As with asphalt and cementitiously stabilised materials layers, the distress that commonly dictates the thickness requirements is fatigue cracking, as predicted from horizontal tensile strains at the bottom of the stabilised layer. As discussed in Section 10.9, the allowable load repetitions in terms of fatigue is determined using the Austroads asphalt fatigue relationship.

These thickness design procedures assume the stabilisation treatment:

- comprises a FBS mix which has been designed in accordance with Austroads (2019a)
- includes sufficient quantity of residual bitumen (and secondary stabilising agent) to produce a bound layer with significant tensile strength and, hence, fatigue properties
- does not include so much secondary binder or a type of secondary binder such that an inflexible layer results.

Usually a minimum of 2.5% residual bitumen with lime or cement as secondary binders is required to produce a foamed bitumen layer that meets the above criteria.

H.3 Characterisation for Pavement Design

Since common practice in Australia is to add secondary binder and around 3% residual bitumen, the stress dependency is neglected for design purposes. In addition, the viscoelastic effect of asphalt is not considered in the characterisation of foamed bitumen mixes. Therefore, the FBS mix is approximated as an isotropic, linear elastic solid, the modulus of which depends on pavement temperature and loading rate (which is related to traffic speed).

H.3.1 Factors Affecting Modulus of Foamed Bitumen Mixes

The design modulus of FBS materials is influenced by many factors including:

- quality of construction and construction practices
- properties of the untreated material
- bitumen type and content
- active filler type and content
- foaming characteristics of the bitumen
- in situ curing conditions (temperature, time)

- temperature during mixing
- in-service temperature
- moisture content
- rate of loading
- age.

Due to the complexities involved in estimating the effect of all these factors on the long-term modulus of a FBS layer, the recommended method for determining the design modulus is as follows:

- determine the indirect tensile resilient modulus of soaked specimens[2] in accordance with AS 2891.13.1-2013.
- Adjust the modulus to the in-service pavement temperature.
- Adjust the modulus to the in-service traffic loading rate.

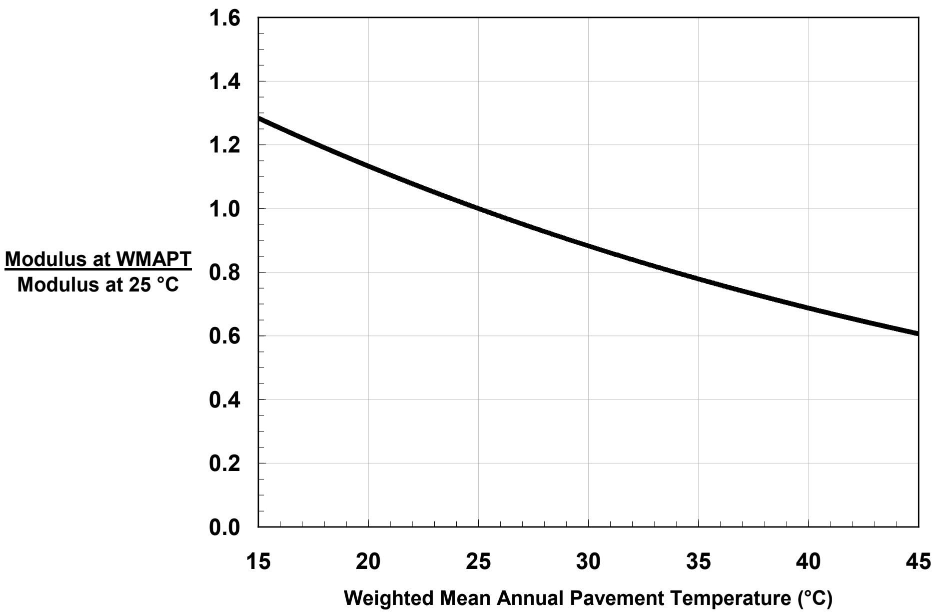
H.3.2 Temperature

Temperature is an important factor in determining the design modulus of FBS mixes.

Commonly, the effect of temperature variations is considered 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.

The variation of mix modulus with temperature is illustrated in Figure H 1.

Figure H 1: Variation with temperature of the ratio of modulus at WMAPT to modulus at 25 °C



H.3.3 Rate of Loading (Traffic Speed)

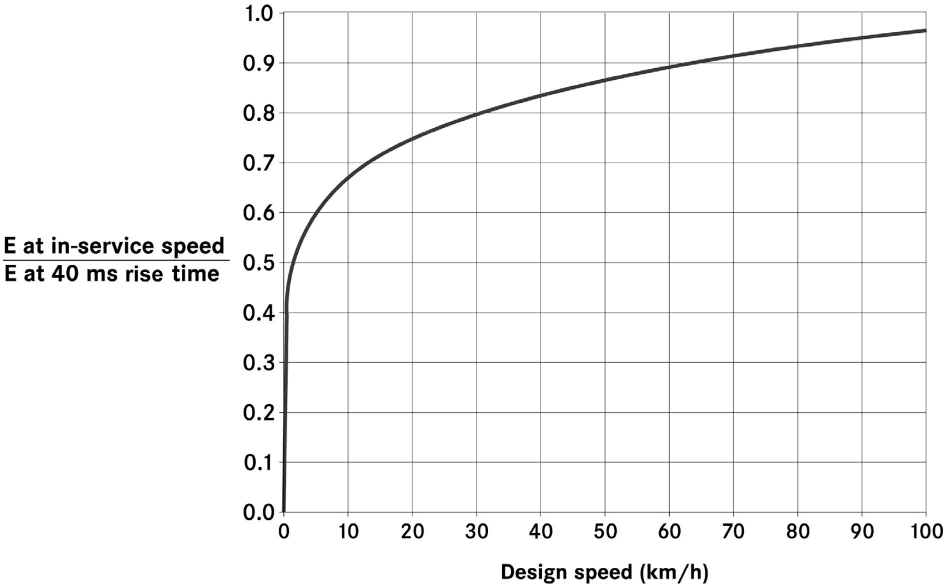
Because of the viscoelastic nature of the bituminous binder, the modulus of foamed bitumen stabilised material is also dependent on the rate at which it is loaded – the slower the rate, the lower the modulus.

2 The testing on soaked specimens should be conducted at 25 °C after 3 days of accelerated air-drying at 40 °C, followed by soaking in water under vacuum for 10 minutes.

When determining the modulus for a given traffic speed, the loading time used will depend on the type of testing device and the shape of the load pulse, as well as the depth below the pavement surface at which the modulus is being sought.

Figure H 2 should be used to adjust the indirect tensile modulus measured at the standard rise time of 40 milliseconds to the modulus at the heavy vehicle design speed.

Figure H 2: Variation with design speed of ratio of modulus at design speed to modulus from standard indirect tensile test (40 ms rise time)



H.4 Determination of Foamed Bitumen Design Modulus and Poisson's Ratio

H.4.1 Definition of Design Modulus

For pavement design purposes, the appropriate design modulus is an estimate of the value obtained from the resilient modulus measured using the standard indirect tensile test (ITT) adjusted to the in-service temperature (WMAPT) and for the rate of loading to which the layer will be subjected in the road bed. If this information is unavailable, then the design modulus 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 FBS mix in its field situation.

H.4.2 Poisson's Ratio

Determination of a value for Poisson's ratio from laboratory testing is difficult. Repeat load triaxial testing (Jenkins 2000) has shown that Poisson's ratio is stress-dependent for mixes without active filler. For design purposes, a Poisson's ratio of 0.40 is assumed for foamed bitumen stabilisation mixes.

H.4.3 Determination of Design Modulus from Measured Modulus

The indirect tensile test (AS 2891.13.1–2013) is the most commonly used laboratory test in Australia for the determination of FBS mix modulus. This is because the testing equipment is relatively inexpensive, and the test is easy to conduct. The following Austroads test methods are used to prepare and test FBS cylinders for modulus:

- AGPT/T301: 2017 *Determining of the Foaming Characteristics of Bitumen*
- AGPT/T302: 2017 *Mixing of Foamed Bitumen Stabilised Materials*
- AGPT/T303: 2017 *Compaction of Test Cylinders of Foamed Bitumen Stabilised Materials Part 1: Dynamic Compaction using Marshall Drop Hammer*
- AGPT/T305: 2017 *Resilient Modulus of Foamed Bitumen Stabilised Materials.*

In the indirect tensile 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 microstrains on the perpendicular diametral plane.

Standard Reference Test Conditions for the ITT are 40 ms rise time (the time for the applied load to increase from 10% to 90% of its peak value) and 25 °C, with a pulse interval 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 is a good simulation of the type of loading produced by a succession of wheel loads.

Specimens should be prepared at the design moisture and binder content. The design moduli are determined from specimens tested after 72 hours of accelerated air-drying at 40 °C, followed by soaking in water under 95 kPa vacuum for 10 minutes.

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 the inherent unstable nature of the bitumen foam, variations in mix constituents and the limits of reproducibility of the test. Due to this variability, designers are advised not to assign a high level of accuracy or precision to a design modulus determined from the mean of a single set of triplicate specimens. Consideration needs to be given to the number of resilient modulus results required to achieve a representative and statistically significant design modulus.

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

1. Determine (from project information) a representative value for heavy vehicle traffic speed (V km/h).
2. Select the WMAPT for the project location (refer to Appendix B).
3. Compact laboratory specimens at the optimum foamed bitumen and secondary binder contents and moisture content. Subject the specimens to 72 hours of accelerated air-drying at 40 °C, followed by soaking in water under 95 kPa vacuum for 10 minutes.
4. Measure the indirect tensile test modulus of each specimen using a rise time of 40 ms and a test temperature of 25 °C.
5. Using the following relationship (Equation A6), calculate the ratio of the modulus at the in-service temperature (WMAPT) to the modulus at the laboratory test temperature (25 °C).

$$\frac{\text{Modulus at WMAPT}}{\text{Modulus at test temperature}} = \exp(-0.025[\text{WMAPT} - \text{T}]) \qquad \text{A6}$$

   This relationship is shown in Figure H 1.

6. Using the following relationship (Equation A7), calculate the ratio of the modulus at the rate of loading in-service to the modulus at the laboratory loading rate in the indirect tensile test (40 ms rise time).

$$\frac{\text{Modulus at speed V}}{\text{Modulus at test loading rate}} = 0.46V^{0.16} \qquad \text{A7}$$

7. Correct the measured mean soaked indirect tensile modulus (Step 4) for temperature and speed by multiplying the measured modulus by the temperature and load rate modulus ratios. The design modulus is this corrected modulus, subject to a maximum value of 2500 MPa.

Appendix I Calculation of Past Traffic Loading

This Appendix describes the process to calculate the past traffic loading (N_p). For mechanistic-empirical design of flexible pavements containing bound materials, the cumulative heavy vehicle axle groups (HVAG), together with the traffic load distribution (TLD) are required to characterise the past traffic loading when considering the fatigue damage to asphalt foamed bitumen stabilised materials, cemented materials and lean-mix concrete.

Austroads (2018a) describes the method of estimating a TLD to calculate the design traffic loading. A similar process applies to estimate a TLD relevant of the calculation of the past traffic loading, except that consideration needs to be given to past axle load changes.

Austroads (2018a) also describes the method used to calculate HVAG to determine the design traffic and these can be utilised to determine the past traffic. Note that in cases where the daily volume of HVAG when the road was first opened to traffic after construction is not known, it can be calculated from the present HVAG volume using Equation A8.

$$\text{initial HVA} = \frac{\text{present HVA}}{(1 + 0.01\text{R})^{\text{P}-1}} \qquad \text{A8}$$

where

R = annual past growth rate (%)

P = number of years from the time the road was initially opened to traffic to the present

Appendix J Asphalt Inlay Design Example

J.1 Introduction

This design example is based on the procedure described in Section 10.10.

A homogeneous section of a pavement with a cracked asphalt and a cracked cemented material subbase (CMS) pavement is situated in a locality where the WMAPT, from Appendix B, is 32 °C. The existing pavement comprises 240 mm asphalt, on 150 mm CMS.

The design traffic loading for a 10-year design period:

- 10^7 heavy vehicle axle groups (HVAG) and 7 x 10^6 ESA
- traffic load distribution (TLD) is in accordance with Appendix G (Austroads 2018a).

Note that the design period was limited to 10 years due to the existing asphalt being cracked and the option of a thin asphalt resurfacing treatment was being considered. For such a treatment, cracking from the existing asphalt is anticipated to propagate through the treatment within 10 years. Hence, designing for fatigue lives in excess of 10 years is inappropriate and not a cost-effective strategy.

A design traffic speed of 60 km/h is appropriate for this project.

FWD testing was conducted in the outer wheelpath at 10 m intervals. Existing pavement and subgrade moduli at the time of deflection testing have been estimated using back-calculation (Section 10.5).

A treatment option identified for evaluation was to:

- mill 80 mm thickness of existing cracked asphalt and
- replace with 80 mm asphalt inlay.

The desired reliability of the rehabilitated project outlasting the design traffic was 95%.

The linear elastic model CIRCLY (Mincad 2009) was used to calculate the critical strains in this design example.

J.2 Procedure

The treatment was designed using the Section 10.10 method.

As described in Section 10.10, the procedure to calculate required thickness of asphalt inlay is similar to that described in Table 8.1, Table 8.2 and Table 8.3 of Austroads (2018a). Steps 1 to 31 below follow this procedure.

Before these design steps are undertaken for some projects, layer moduli are back-calculated from the measured FWD deflection bowls as described in Steps BC1 to BC3 below.

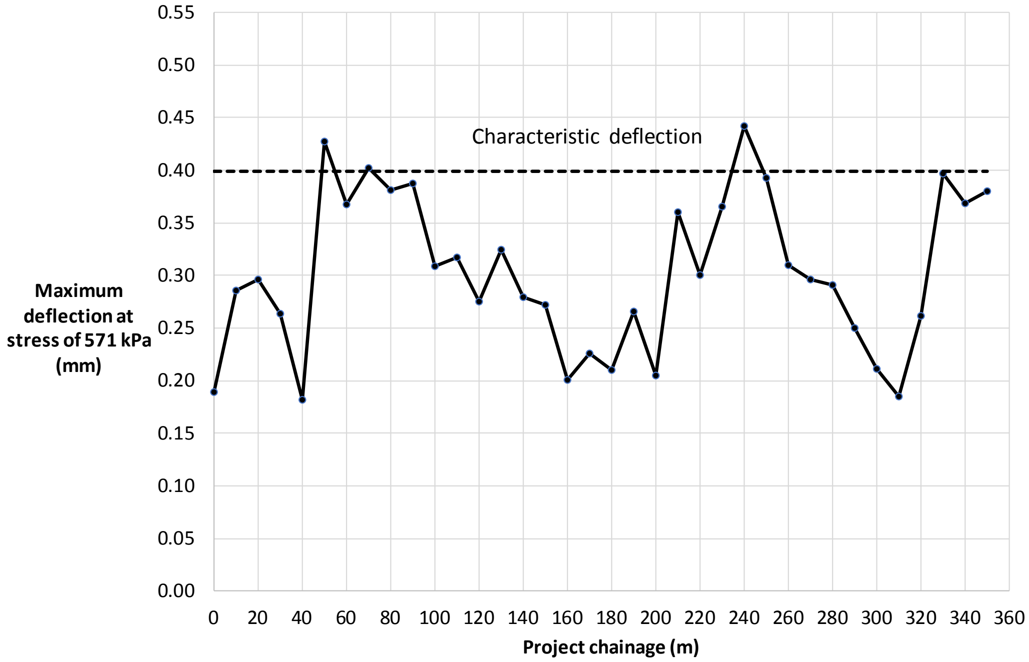
Step BC1

Select a homogeneous section

FWD testing was conducted in the outer wheelpath at 10 m intervals over the 350 m project length. Plotted in Figure J 1 are the maximum deflections linearly adjusted from the applied contact stress to the mean stress of 571 kPa using Equation 3.

The mean of the adjusted maximum deflections is 0.302 mm, the standard deviation 0.074 mm and the coefficient of variation (CV) was 0.25. As the project length was only 350 m and the CV did not exceed 0.25, it was decided that the entire project length was a homogeneous section.

Figure J 1: Variation in maximum deflections along the project



The applied contact stress and measured deflection bowls at each site are given in Figure J 2. The asphalt temperature at the time of the deflection measurements was 42 °C.

Step BC2

Calculate the characteristic maximum deflection (CD)

The CD was calculated after the maximum deflection at each site was linearly adjusted from the plate contact stress during testing to the mean contact stress of 571 kPa. Using Equation 5 with f = 1.31 (N > 30), the CD was calculated:

$$CD = 0.302 + 1.31 \times 0.074 = 0.399 \text{ mm}$$

This value is plotted in Figure J 1. It is apparent that the deflections at chainages 50 m, 70 m, 250 m and 330 m are close to the CD and have low back-calculation error and are therefore most relevant for treatment design.

Step BC3

Back-calculate the layer moduli from the measured deflection bowls

The moduli of existing pavement and subgrade layers were estimated by back-calculation of the measured deflections (Figure J 1). The back-calculation was undertaken with the following pavement and subgrade layers:

- 240 mm thick asphalt layer, constrained with minimum and maximum moduli of 500 MPa and 10 000 MPa respectively
- 150 mm thick cemented material layer, constrained with minimum and maximum moduli of 100 MPa and 5000 MPa respectively
- top 300 mm of subgrade, constrained with minimum and maximum moduli of 20 MPa and 1000 MPa respectively

- a 500 mm thick subgrade layer, the top of which was 300 mm below the top of the subgrade, constrained with minimum and maximum moduli of 20 MPa and 1000 MPa respectively
- a semi-infinite subgrade layer constrained with minimum and maximum moduli of 20 MPa and 10 000 MPa respectively.

The back-calculated moduli at the measurement temperature are given in Figure J 2.

Figure J 2: Layer moduli back-calculated from FWD deflections

Back analysis model constraints

Layer	Material	Thickness (mm)	Emin	Emax
E1	Asphalt	240	500	10000
E2	CTB	150	100	5000
E3	Subgrade	300	20	1000
E4	Subgrade	500	20	1000
E5	Subgrade	0	20	10000

Measured deflections and back-calculated moduli

Chainage	FWD Load	FWD Deflections (mm)							Back-calculated Layer Moduli (MPa)					
(m)	kPa	D0	D200	D300	D450	D600	D900	D1500	E1	E2	E3	E4	E5	Error
0	584	0.195	0.145	0.119	0.095	0.077	0.053	0.028	2041	816	202	250	300	1.0%
10	590	0.297	0.209	0.165	0.121	0.087	0.049	0.021	1320	209	115	249	390	1.0%
20	591	0.309	0.208	0.163	0.120	0.090	0.052	0.025	1017	425	119	224	336	0.8%
30	578	0.269	0.180	0.141	0.106	0.079	0.049	0.024	1197	411	154	243	337	1.1%
40	581	0.186	0.137	0.114	0.087	0.066	0.041	0.021	2586	139	377	380	384	0.1%
50	579	0.437	0.309	0.235	0.162	0.110	0.057	0.025	713	163	79	196	319	2.9%
60	575	0.374	0.274	0.220	0.161	0.120	0.072	0.031	1009	287	72	166	265	2.1%
70	**577**	**0.410**	**0.289**	**0.225**	**0.162**	**0.118**	**0.066**	**0.030**	**867**	**169**	**90**	**174**	**263**	**1.3%**
80	577	0.389	0.279	0.220	0.161	0.118	0.067	0.032	1018	102	120	184	251	0.9%
90	575	0.393	0.294	0.232	0.165	0.118	0.064	0.030	1041	102	113	187	266	3.9%
100	570	0.311	0.245	0.207	0.160	0.123	0.073	0.030	1813	164	69	172	282	0.7%
110	573	0.321	0.238	0.199	0.153	0.116	0.070	0.032	1471	196	92	173	258	0.4%
120	570	0.277	0.203	0.166	0.131	0.104	0.067	0.030	1315	868	78	182	292	1.2%
130	571	0.327	0.246	0.201	0.152	0.114	0.066	0.030	1443	104	117	190	269	0.6%
140	573	0.283	0.204	0.164	0.121	0.091	0.053	0.021	1311	471	83	230	385	1.6%
150	572	0.275	0.210	0.176	0.137	0.099	0.052	0.020	1951	115	94	242	399	2.7%
160	568	0.202	0.152	0.130	0.101	0.078	0.047	0.019	2377	612	89	269	460	0.8%
170	567	0.227	0.173	0.148	0.117	0.090	0.055	0.024	2321	335	98	222	353	0.3%
180	570	0.211	0.146	0.121	0.095	0.073	0.044	0.020	1787	644	134	267	408	1.0%
190	571	0.268	0.193	0.154	0.115	0.085	0.050	0.026	1615	100	177	540	294	0.5%
200	567	0.206	0.134	0.100	0.074	0.056	0.036	0.022	1369	408	248	715	350	1.3%
210	567	0.361	0.283	0.239	0.191	0.152	0.099	0.047	1402	331	67	121	179	0.8%
220	565	0.300	0.229	0.199	0.160	0.132	0.087	0.044	1635	507	89	138	190	0.4%
230	569	0.367	0.269	0.218	0.170	0.132	0.082	0.038	1040	404	74	144	219	0.9%
240	565	0.441	0.334	0.272	0.210	0.165	0.104	0.044	924	403	46	117	192	1.8%
250	**562**	**0.390**	**0.303**	**0.253**	**0.202**	**0.160**	**0.100**	**0.043**	**1260**	**344**	**49**	**121**	**198**	**0.8%**
260	571	0.313	0.217	0.170	0.123	0.091	0.050	0.025	1138	157	163	236	314	1.8%
270	571	0.299	0.208	0.157	0.110	0.078	0.042	0.021	1158	107	238	302	369	1.9%
280	571	0.294	0.203	0.155	0.109	0.077	0.040	0.018	1227	111	182	300	424	0.9%
290	563	0.249	0.179	0.144	0.117	0.095	0.063	0.033	1327	939	132	191	253	1.2%
300	565	0.211	0.142	0.114	0.091	0.075	0.052	0.033	1328	1275	175	546	239	1.2%
310	566	0.185	0.126	0.098	0.072	0.053	0.031	0.015	1727	455	223	368	522	0.8%
320	567	0.262	0.180	0.144	0.111	0.084	0.050	0.023	1275	569	112	231	357	0.5%
330	**558**	**0.391**	**0.298**	**0.252**	**0.200**	**0.158**	**0.103**	**0.048**	**1203**	**300**	**62**	**116**	**173**	**1.0%**
340	558	0.363	0.273	0.227	0.183	0.148	0.098	0.048	1127	545	69	121	175	0.8%
350	557	0.374	0.274	0.220	0.170	0.137	0.089	0.043	944	504	76	132	191	1.8%

As discussed in Section 10.5.2, the moduli most relevant in determination of the design are those associated at weaker areas of pavement as their performance will influence the timing of future rehabilitations. Table J 1 shows the representative moduli at both the measurement temperature and the WMAPT. These moduli are the mean moduli of the values back-calculated at chainages 70 m, 250 m and 330 m which are close to the CD, have low back-calculation error and are therefore most relevant. Note that the asphalt modulus was adjusted for temperature using Equation 15, using k = –0.05 as the existing asphalt was moderately cracked:

$$\text{Modulus at WMAPT} = 1110 \times \exp(-0.05\,(32 - 42)) = 1830 \text{ MPa}$$

Table J 1: Summary of representative back-calculated moduli

State	Asphalt temp. (°C)	Layer modulus (MPa)				
		Asphalt	CMS	Subgrade (top 300 mm)	Subgrade (300–800 mm)	Subgrade (> 800 mm)
During deflection measurements	42	1110	270	67	137	211
At in-service temperature, WMAPT	32	1830	270	67	137	211

Step 1

Select a trial treatment

- Mill 80 mm from existing cracked asphalt surface and
- Replace with 80 mm asphalt inlay in two 40 mm thick layers using size 14 mm asphalt.

Step 2

Elastic parameters for subgrade

In selecting a design modulus for the semi-infinite subgrade, the following was considered:

- The representative back-calculated modulus of the top 300 mm of subgrade of 67 MPa (Table J 1) was obtained at a dry time of the year and, hence, may overestimate an appropriate design modulus.
- Dynamic cone penetrometer testing of the subgrade resulted in estimated in situ CBR at three sites of 7%, 10% and 13%. The associated subgrade vertical moduli were obtained by multiplying these CBR values by 10. However, this testing was undertaken at the dry time of year and may overestimate the design subgrade modulus.
- Laboratory CBR testing was undertaken on a sample of subgrade at the site where the lowest field CBR was measured. The laboratory-soaked CBR value was 5%.
- As the laboratory CBR was measured there was no need to consider the maximum values in Table 10.1.

Based on the above, a subgrade design CBR of 5% was selected, resulting in the following elastic parameters.

E_V = 50 MPa – Section 5.6 of Austroads (2018a)

E_H = 25 MPa

ν_V = $\nu_H = 0.45$ – Section 5.6 of Austroads (2018a)

f = $E_V/(1 + \nu_V) = 34.5$.

Steps 3 and 4 not relevant

Step 5

Elastic parameters for CMS post-cracking

In selecting a design modulus for 150 mm thickness of CMS the following was considered:

1. a representative modulus of 270 MPa was back-calculated from the FWD deflections (Table J 1)
2. the design moduli for presumptive cracked cemented materials of 500 MPa in Austroads (2018a).

As described in Section 10.7.6, as the back-calculated modulus is less than the presumptive value, the back-calculated modulus is used as the design modulus. The elastic parameters adopted were:

E_v = 270 MPa – Section 6.4.3 of Austroads (2018a)

E_H = 135 MPa

ν_v = ν_H = 0.35 – Section 6.4.3 of Austroads (2018a)

f = $E_V/(1 + \nu_V)$ = 200.0.

This layer is not sub-layered in the analysis.

Step 6

Elastic parameters for asphalt

Asphalt inlay

The measured indirect tensile modulus for size 14 mm dense-graded asphalt (Class 320 binder) in close proximity to the project was 4130 MPa at

- in-service air voids of 5%
- a measurement temperature of 25 °C
- a rise time of 40 milliseconds.

Using the procedure described in Section 6.5.5 of Austroads (2018a), the following adjustments were made to the measured modulus:

- the modulus was multiplied by 0.57 to adjust to a temperature of 32 °C
- the modulus was multiplied by 0.85 to adjust to the in-service traffic loading rate (60 km/h).

The resulting design modulus of 2000 MPa was adopted for the size 14 mm dense-graded asphalt:

E_v = E_H = 2000 MPa

ν_v = ν_H = 0.40 – Section 6.5.8 of Austroads (2018a)

Existing cracked asphalt

The temperature-adjusted back-calculated modulus (1830 MPa) is less than the design modulus for a new size 20 mm asphalt (2200 MPa) for the WMAPT and heavy vehicle design speed. (This new asphalt design modulus was calculated using a similar process as for the size 14 mm asphalt inlay described above).

In addition, this temperature-adjusted back-calculated modulus is greater than the presumptive modulus of cracked asphalt of 765 MPa calculated using Equation 17 for a WMAPT of 32 °C.

Following the procedure in Section 10.7.5, the design modulus was calculated as follows:

1. As the existing asphalt is fatigue-cracked, it is necessary to allow for future damage to this asphalt during the design period. Firstly, the traffic ratio (TR) is calculated by dividing the design traffic (7 x 10^6 ESA) by past traffic (3.5 x 10^6 ESA). As the TR = 2, from Figure 10.2 the modulus reduction factor is 0.50.
2. The temperature-adjusted back-calculated modulus (1830 MPa) is then multiplied by this modulus reduction factor to yield a value of 915 MPa.
3. The design modulus of the cracked asphalt is the maximum of the value calculated in Step 2 (915 MPa) and the presumptive cracked moduli of 765 MPa calculated using Equation 17 for a WMAPT of 32 °C.

Based on the above, the following elastic parameters was used for the existing cracked asphalt.

E_v = E_H = 915 MPa

ν_v = ν_H = 0.40 – Section 6.5.8 of Austroads (2018a)

The design moduli used in the analysis are summarised in Table J 2.

Table J 2: Summary of design moduli

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f value
		E_V	E_H	ν_V	ν_H	
Asphalt inlay	80	2000	2000	0.40	0.40	
Existing asphalt base	160(1)	915	915	0.40	0.40	
Existing cemented material	150	270	135	0.35	0.35	200
Subgrade	Semi-infinite	50	25	0.45	0.45	34.5

1. *After milling 80 mm of cracked asphalt.*

Step 7

Permanent deformation allowable loading (Equation 3 of Austroads 2018a):

$$N = \left[\frac{9150}{\mu \in}\right]^7$$

Step 8

The CMS is fatigue cracked, so its fatigue relationship is not relevant.

Step 9

A fatigue relationship is not required for the existing cracked asphalt as its fatigue life was not evaluated.

The fatigue performance of the asphalt inlay is predicted by using the asphalt fatigue relationship (Section 10.9) with an appropriate modulus and volume of binder:

$$N = \frac{SF}{RF}\left[\frac{6918(0.856 \times 10.5 + 1.08)}{2000^{0.36}\mu\varepsilon}\right]^5$$

Assuming volume of bitumen (V_b) of 10.5%.

For project reliability of 95%, shift factor (SF) = 6 and reliability factor (RF) = 6 (Section 6.5 of Austroads (2018a)).

Step 10

- 10^7 heavy vehicle axle groups (HVAG)
- traffic load distribution (TLD) is in accordance with Appendix G of Austroads (2018a).

Step 11

Using the example distribution in Appendix G of Austroads (2018a), ESA/HVAG = 0.7.

Therefore, using Equation 37 of Austroads (2018a):

$$DESA = 0.7 \times 10^7 = 7 \times 10^6$$

Step 12

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 of Austroads 2018a).

Step 13

Critical locations to calculate Standard Axle strains are:

- vertical compressive strain on top of subgrade directly beneath the inner-most tyre of one of the dual tyre sets
- vertical compressive strain on top of subgrade midway between the tyres of one of the dual tyre sets
- horizontal tensile strain bottom of the asphalt overlay directly beneath the inner-most tyre of one of the dual tyre sets
- horizontal tensile strain bottom of the asphalt overlay midway between the tyres of one of the dual tyre sets.

Step 14

Critical strains under Standard Axle load:

- subgrade: maximum vertical compressive strain is 508.8 $\mu\varepsilon$ on top of subgrade midway between the two loaded tyres.
- asphalt inlay: maximum horizontal tensile strain of 143.6 $\mu\varepsilon$ loaded midway between the loaded tyres.

Step 15

Single axle with single tyres at 53 kN axle load:

- tyre-pavement contact stress = 800 kPa
- load radius = 102.4 mm
- two circular areas separated centre-to-centre 2130 mm (Figure 8.2 of Austroads 2018a).

Step 16

Critical locations to calculate single axle single tyre strains are:

- horizontal tensile strain at the bottom of the asphalt inlay

The above strain is calculated directly beneath one of the loaded tyres of the single axle with single tyres.

Step 17

Critical strain resulting from single axle with single tyres with 53 kN axle load:

- asphalt inlay: maximum horizontal tensile strain of 163.5 $\mu\varepsilon$.

Step 18

Permanent deformation allowable load repetitions:

$$N = \left[\frac{9150}{508.8}\right]^7 = 6.1 \text{ x } 10^8 \text{ ESA}$$

Step 19

The allowable loading in terms of permanent deformation is 6.1 x 10^8 ESA compared to design traffic of 7 x 10^6 ESA from Step 11. The trial treatment is acceptable in terms of rutting and shape loss.

Step 20

The fatigue damage to the asphalt inlay needs to be calculated, therefore Steps 21 to 27 must be repeated to calculate the damage under axle each group load of each axle group type.

Step 21

Steps 22 to 26 are repeated for each axle group type present in the traffic load distribution:

- 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.

Step 22

The expected repetitions of each load for each axle group type is calculated in Table J 3 to Table J 7 by entering the data from the TLD (Table G.1 of Austroads (2018a)) 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 are the product of columns 2, 3 and 4.

Table J 3: Calculation of expected repetitions – single axle/single tyre (SAST)

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 J 4: Calculation of expected repetitions – single axle/dual tyres (SADT)

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 J 5: Calculation of expected repetitions – tandem axle/single tyre (TAST)

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 J 6: Calculation of expected repetitions – tandem axle/dual tyres (TADT)

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 J 7: Calculation of expected repetitions – triaxle dual tyres (TRDT)

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

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
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 23

The allowable load repetitions of each axle group type and load level is calculated in Table J 8 to Table J 12. 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 43 of Austroads (2018a). The strains predicted in Steps 14 and 17 are adjusted as follows:

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SAST,53}}{53} = \frac{L_{ij}}{n} \times \frac{163.5}{53}$$ for axles within SAST, TAST groups

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SADT,80}}{80} = \frac{L_{ij}}{n} \times \frac{143.6}{80}$$ for axles within SADT, TADT & TRDT groups

For example, the critical asphalt strain developed under a single axle of a triaxle group (TRDT) with a total group load of 180 kN is calculated as:

$$\mu\varepsilon_{\mathrm{TRDT},60} = \frac{180}{3} \times \frac{143.6}{80} = 107.7\ \mu\varepsilon$$

The allowable load repetitions to fatigue of the inlay of each axle group type and load magnitude are then calculated using Equation 44 (Section 8.2.5 of Austroads 2018a):

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

For example, the allowable repetitions of a triaxle group (TRDT) with a total group load of 180 kN are calculated as:

$$N_{\mathrm{TRDT},180} = \frac{1}{3} \times \frac{6}{6}\left[\frac{6918(0.856 \times 10.5 + 1.08)}{2000^{0.36} \times 107.7}\right]^5 = 4.31 \times 10^7$$

The allowable load repetitions of all axle group/load combinations are shown in Table J 8 to Table J 12.

Steps 24 and 25

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 J 8 to Table J 12.

Table J 8: Calculation of asphalt fatigue damage – single axle/single tyre (SAST)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	11 020	1	30.8	6.76E+10	1.63E-07
20	307 601	1	61.7	2.10E+09	1.47E-04
30	607 578	1	92.5	2.77E+08	2.20E-03
40	617 403	1	123.4	6.55E+07	9.43E-03
50	1 176 642	1	154.2	2.15E+07	5.47E-02
60	915 297	1	185.1	8.62E+06	1.06E-01
70	255 529	1	215.9	3.99E+06	6.40E-02
80	31 216	1	246.8	2.05E+06	1.53E-02
90	4 272	1	277.6	1.14E+06	3.76E-03
100	1 391	1	308.5	6.71E+05	2.07E-03
110	684	1	339.3	4.17E+05	1.64E-03
120	684	1	370.2	2.70E+05	2.54E-03
130	684	1	401.0	1.81E+05	3.78E-03
				Total SAST damage	0.266

Table J 9: Calculation of asphalt damage – single axle/dual tyres (SADT)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	66 334	1	18.0	9.92E+11	6.69E-08
20	166 094	1	35.9	3.14E+10	5.29E-06
30	448 086	1	53.9	4.12E+09	1.09E-04
40	418 863	1	71.8	9.82E+08	4.27E-04
50	320 880	1	89.8	3.21E+08	1.00E-03
60	183 475	1	107.7	1.29E+08	1.42E-03
70	124 150	1	125.7	5.97E+07	2.08E-03
80	88 299	1	143.6	3.07E+07	2.88E-03
90	56 708	1	161.6	1.70E+07	3.33E-03
100	26 606	1	179.5	1.01E+07	2.65E-03
110	7 827	1	197.5	6.24E+06	1.26E-03
120	2 212	1	215.4	4.04E+06	5.47E-04
130	466	1	233.4	2.71E+06	1.72E-04
				Total SADT damage	0.016

Table J 10: Calculation of asphalt fatigue damage – tandem axle group/single tyre (TAST)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	32	2	15.4	1.08E+12	2.95E-11
20	214	2	30.8	3.38E+10	6.33E-09
30	249	2	46.3	4.40E+09	5.65E-08
40	518	2	61.7	1.05E+09	4.94E-07
50	2 600	2	77.1	3.44E+08	7.56E-06
60	9 243	2	92.5	1.38E+08	6.68E-05
70	15 129	2	108.0	6.38E+07	2.37E-04
80	14 949	2	123.4	3.27E+07	4.57E-04
90	14 355	2	138.8	1.82E+07	7.89E-04
100	12 978	2	154.2	1.07E+07	1.21E-03
110	8 797	2	169.7	6.66E+06	1.32E-03
120	5 313	2	185.1	4.31E+06	1.23E-03
130	2 649	2	200.5	2.89E+06	9.16E-04
140	1 385	2	215.9	2.00E+06	6.93E-04
150	760	2	231.4	1.41E+06	5.38E-04
160	385	2	246.8	1.02E+06	3.76E-04
170	208	2	262.2	7.56E+05	2.75E-04
180	123	2	277.6	5.68E+05	2.16E-04
190	65	2	293.1	4.33E+05	1.50E-04
200	50	2	308.5	3.35E+05	1.49E-04
				Total TAST damage	0.009

Table J 11: Calculation of asphalt fatigue damage – tandem axle group/dual tyres (TADT)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	3 740	2	9.0	1.59E+13	2.36E-10
20	14 905	2	18.0	4.96E+11	3.01E-08
30	16 167	2	26.9	6.65E+10	2.43E-07
40	51 204	2	35.9	1.57E+10	3.26E-06
50	168 246	2	44.9	5.13E+09	3.28E-05
60	246 335	2	53.9	2.06E+09	1.20E-04
70	283 346	2	62.8	9.59E+08	2.95E-04
80	253 017	2	71.8	4.91E+08	5.15E-04
90	197 125	2	80.8	2.72E+08	7.25E-04
100	187 568	2	89.8	1.60E+08	1.17E-03
110	162 315	2	98.7	1.00E+08	1.62E-03
120	154 157	2	107.7	6.47E+07	2.38E-03
130	152 240	2	116.7	4.33E+07	3.52E-03
140	169 231	2	125.7	2.99E+07	5.67E-03
150	207 977	2	134.6	2.12E+07	9.81E-03

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
160	148 070	2	143.6	1.53E+07	9.65E-03
170	92 049	2	152.6	1.13E+07	8.13E-03
180	48 252	2	161.6	8.50E+06	5.68E-03
190	22 106	2	170.5	6.50E+06	3.40E-03
200	8 627	2	179.5	5.03E+06	1.72E-03
210	2 075	2	188.5	3.94E+06	5.27E-04
220	834	2	197.5	3.12E+06	2.67E-04
230	414	2	206.4	2.50E+06	1.66E-04
				Total TADT damage	0.055

Table J 12: Calculation of asphalt damage – triaxle group/dual tyres (TRDT)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	74	3	6.0	8.03E+13	9.21E-13
20	2 321	3	12.0	2.51E+12	9.24E-10
30	4 869	3	18.0	3.31E+11	1.47E-08
40	19 492	3	23.9	8.01E+10	2.43E-07
50	61 672	3	29.9	2.61E+10	2.36E-06
60	109 801	3	35.9	1.05E+10	1.05E-05
70	144 700	3	41.9	4.84E+09	2.99E-05
80	123 402	3	47.9	2.48E+09	4.98E-05
90	91 020	3	53.9	1.37E+09	6.63E-05
100	74 429	3	59.8	8.17E+08	9.11E-05
110	54 775	3	65.8	5.06E+08	1.08E-04
120	48 810	3	71.8	3.27E+08	1.49E-04
130	46 576	3	77.8	2.19E+08	2.13E-04
140	49 743	3	83.8	1.51E+08	3.29E-04
150	59 318	3	89.8	1.07E+08	5.55E-04
160	60 902	3	95.7	7.78E+07	7.83E-04
170	71 321	3	101.7	5.74E+07	1.24E-03
180	90 236	3	107.7	4.31E+07	2.09E-03
190	114 448	3	113.7	3.29E+07	3.48E-03
200	124 808	3	119.7	2.54E+07	4.91E-03
210	76 013	3	125.7	1.99E+07	3.82E-03
220	34 617	3	131.6	1.58E+07	2.19E-03
230	11 491	3	137.6	1.27E+07	9.07E-04
240	3 704	3	143.6	1.02E+07	3.62E-04
250	1 339	3	149.6	8.34E+06	1.61E-04
260	118	3	155.6	6.85E+06	1.73E-05
				Total TRDT damage	0.022

Step 26

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 J 13.

Table J 13: Total asphalt inlay fatigue damage

Axle group type	Total group damage
SAST	0.266
SADT	0.016
TAST	0.009
TADT	0.055
TRDT	0.022

Step 27

The total damage to asphalt inlay is the sum of damage resulting from each axle group type:

$$\text{Total fatigue damage to inlay} = 0.266 + 0.016 + 0.009 + 0.055 + 0.022 = 0.37$$

Step 28 is not relevant.

Step 29

Total fatigue damage to the asphalt inlay due to the design traffic is less than or equal to 1, therefore the trial treatment is acceptable.

Step 30 is not relevant.

Step 31

Compare this trial treatment with other options based on whole-of-life costing.

Appendix K Cement-stabilised Base Design Example

K.1 Introduction

An investigation of a homogeneous section of a sprayed bituminous seal-surfaced unbound granular pavement indicated that the rutting was primarily occurring in the non-standard granular base. This section of pavement comprises 450 mm of existing granular material on a subgrade with a design CBR of 7%.

One treatment option identified for evaluation was:

- cementitious stabilisation of the existing granular material to a depth of 350 mm by the addition of 5% cementitious binder to produce a cemented base
- sprayed bituminous seal surfacing.

Based on laboratory unconfined compressive strength (UCS) testing and field density variations with depth achieved on previous stabilisation projects, it was decided to adopt the following cemented base characterisation in the thickness design: design modulus of 5000 MPa, flexural strength of 1.4 MPa and in-service fatigue constant (K) of 235.

The design traffic loading for a 20-year design period:

- 10^7 heavy vehicle axle groups (HVAG)
- traffic load distribution (TLD) is in accordance with Appendix G of Austroads (2018a).

The project reliability adopted was 95%.

Given the cement material base will be surfaced with a sprayed seal surfacing, the post-cracking life of the cemented base is not considered in this analysis. Moreover, given the likelihood of shrinkage cracking of the cemented base reflecting through the thin surfacing within the 20-year design period, allowance will need to be made in the whole-of-life costing for periodic resurfacings.

The linear elastic model CIRCLY (Mincad 2009) was used to calculate the critical strains in this design example.

K.2 Procedure

The treatment was designed using the Section 10.10 method.

As described in Section 10.10, the structural adequacy of the treatment is calculated following the steps in Table 8.1, Table 8.2 and Table 8.3 of Austroads (2018a).

Step 1

The selected trial treatment is cementitious stabilisation of existing pavement to 350 mm depth, desired project reliability is 95%. There remains 100 mm thickness of granular material under the stabilised layer as summarised in Table K 1.

Table K 1: Summary of pavement configuration

Material type	Thickness (mm)
Polymer modified binder sprayed seal	–
Cemented base (E = 5000 MPa)	350
Existing unbound granular material	100
Subgrade, design CBR = 7%	Semi-infinite

Step 2

Elastic parameters for subgrade

Following a process similar to that described in Appendix I, a subgrade design CBR of 7% was adopted. Hence the subgrade elastic characterisation is:

E_v = 70 MPa – Section 5.6 of Austroads (2018a)

E_H = 35 MPa

ν_v = ν_H = 0.45 – Section 5.6 of Austroads (2018a)

f = $E_V/(1 + \nu_V) = 48.3$.

Step 3

After stabilisation to a depth of 350 mm there remains 100 mm thickness of existing granular subbase.

Elastic parameters for top granular sub-layer.

$E_{V\ \text{top granular sub-layer}}$ = $E_{V\ \text{subgrade}} \times 2^{(\text{top granular thickness}/125)} = 70 \times 2^{(100/125)} = 122$ – Section 8.2.3 of Austroads (2018a)

$E_{V\ \text{granular}}$ = 150 MPa – Table 6.4 of Austroads (2018a) for 300 mm thickness of cemented base with a design modulus of 5000 MPa

E_v = minimum (122 MPa, 150 MPa) = 122 MPa

E_H = 61 MPa

ν_v = ν_H = 0.35 – Section 6.3 Part 2

f = 90.4

Step 4

Elastic parameters for other granular sub-layers

Divide the total granular layer thickness into five equi-thick sub-layers (Section 8.2.3 of Austroads 2018a), each of thickness 100/5 = 20 mm.

Calculate the ratio of moduli of adjacent layers:

$$R = \left[\frac{E_{V\ \text{top granular sublayer}}}{E_{V\ \text{stabilised subgrade top sublayer}}}\right]^{\frac{1}{5}}$$, i.e. $R = (122/70)^{1/5} = 1.118$.

Therefore, elastic parameters of the first granular sub-layer on top of the subgrade are:

E_V = $R \times E_{V\ \text{subgrade}} = 1.1175 \times 70 = 78.2$ MPa

E_H = $0.5 \times E_V = 39.1$ MPa

ν_V = $\nu_H = 0.35$

f = $E_V/(1 + \nu_V) = 78.2/(1.35) = 57.9$

Elastic properties of other sub-layers are calculated similarly using the elastic properties of the underlying sub-layer and are listed in Table K 2.

Table K 2: Modelled pavement configuration

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f value
		E_V	E_H	ν_V	ν_H	
Cemented base	350	5000	5000	0.2	0.2	–
Granular	20	122	61	0.35	0.35	90.4
Granular	20	109	54.5	0.35	0.35	80.7
Granular	20	98	49	0.35	0.35	72.6
Granular	20	87	43.5	0.35	0.35	65.2
Granular	20	78	39	0.35	0.35	57.8
Subgrade	Semi-infinite	70	35	0.45	0.45	48.3

Step 5

Elastic parameters for cemented base

$E_v = E_H = 5000$ MPa

$\nu_v = \nu_H = 0.2$

Post-cracking phase of cemented materials life is not considered.

Step 6

Asphalt is not used in the treatment, therefore not relevant.

Step 7

Permanent deformation allowable loading (Equation 3 of Austroads 2018a):

$$N = \left[\frac{9150}{\mu \in}\right]^7$$

Step 8

The fatigue relationship for the cemented base was determined based on a fatigue constant K calculated from the design modulus of 5000 MPa and the design flexural strength of 1.4 MPa (Section 6 of Austroads 2018a):

$$K = 240FS + \frac{919300}{E} - 285$$
$$= 240 \times 1.4 + \frac{919300}{5000} - 285 = 235$$

Using the process in Section 6 of Austroads (2018a) the maximum value of K is 267:

$$K_{max} = \frac{18880}{\sqrt{E}} = 267$$

Using K = 235 and the in-service fatigue relationship (Section 6 of Austroads 2018a), the allowable traffic loading is:

$$N = RF \times \left(\frac{235}{\mu\varepsilon}\right)^{12}$$

For project reliability of 95%, Reliability Factor, RF = 1 (Table 6.8 of Austroads 2018a).

Step 9

Asphalt is not used in the treatment, therefore not relevant.

Step 10

Cumulative HVAG = 10^7.

Traffic load distribution is in accordance with Appendix G of Austroads (2018a).

Step 11

Using the example distribution in Appendix G of Austroads (2018a), ESA/HVAG = 0.7.

Therefore, using Equation 37 of Austroads (2018a):

$$DESA = 0.7 \times 10^7 = 7 \times 10^6$$

Step 12

Standard Axle load is represented as:

- tyre-pavement contact stress = 750 kPa
- tyre load radius = 92.1 mm
- four circular areas separated centre-to-centre 330 mm, 1470 mm and 330 mm (Figure 8.2 of Austroads 2018a).

Step 13

Critical locations to calculate Standard Axle strains are:

- vertical compressive strain on top of subgrade directly beneath the inner-most tyre of one of the dual tyre sets
- vertical compressive strain on top of subgrade midway between the tyres of one of the dual tyre sets
- horizontal tensile strain bottom of cemented material directly beneath the inner-most tyre of one of the dual tyre sets
- horizontal tensile strain bottom of cemented material midway between the tyres of one of the dual tyre sets.

Step 14

Critical strains due to the Standard Axle:

- subgrade: maximum vertical compressive strain is 155 $\mu\varepsilon$ on top of subgrade midway between the two loaded tyres
- cemented material: maximum horizontal tensile strain of 62.6 $\mu\varepsilon$ loaded midway between the loaded tyres.

Step 15

Single axle with single tyres at 53 kN axle load:

- tyre-pavement contact stress = 800 kPa
- tyre load radius = 102.4 mm
- two circular areas separated centre-to-centre 2130 mm (Figure 8.2 of Austroads 2018a).

Step 16

Critical locations to calculate single axle single tyre strains are:

- horizontal tensile strain at the bottom of cemented material
- the above strain is calculated directly beneath one of the loaded tyres of the single axle with single tyres.

Step 17

Critical strain resulting from single axle with single tyres with 53 kN axle load:

- cemented material: maximum horizontal tensile strain of 45.3 $\mu\varepsilon$.

Step 18

Determine the allowable number of load repetitions in terms of permanent deformation.

$$N = \left[\frac{9150}{155}\right]^7 = 4.5 \text{ x } 10^{12}\ ESA$$

Step 19

The allowable load repetitions in terms of permanent deformation is 4.5 x 10^{12} ESA compared to design traffic of 7 x 10^6 ESA from Step 11. As the allowable loading exceeds the design traffic, the candidate treatment is acceptable in terms of rutting and shape loss.

Step 20

Cemented base is the trial treatment; therefore Steps 21 to 27 must be repeated for each axle group load of each axle group type.

Step 21

Steps 22 to 26 are repeated for each axle group type present in the traffic load distribution:

- 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.

Step 22

The expected repetitions of each load for each axle group type are calculated in Table K 3 to Table K 7 by entering the data from the TLD (Table G.1 of Austroads 2018a) 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 are the product of columns 2, 3 and 4.

Table K 3: Calculation of expected repetitions – single axle/single tyre (SAST)

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 K 4: Calculation of expected repetitions – single axle/dual tyres (SADT)

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 K 5: Calculation of expected repetitions – tandem axle/single tyre (TAST)

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

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
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 K 6: Calculation of expected repetitions – tandem axle/dual tyres (TADT)

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 K 7: Calculation of expected repetitions – triaxle dual tyres (TRDT)

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

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
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 23

The allowable repetitions of each axle group type and load level is calculated in Table K 8 to Table K 12. 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 43 of Austroads (2018a). The critical strains calculated in Steps 14 and 17 are adjusted as follows:

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SAST,53}}{53} = \frac{L_{ij}}{n} \times \frac{45.3}{53}$$ for axles within SAST, TAST groups

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SADT,80}}{80} = \frac{L_{ij}}{n} \times \frac{62.6}{80}$$ for axles within SADT, TADT & 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:

$$\mu\varepsilon_{\text{TADT},65} = \frac{130}{2} \times \frac{62.6}{80} = 50.9\ \mu\varepsilon$$

The allowable repetitions of each axle group type and load magnitude is then calculated using Equation 45 of Austroads (2018a):

$$N_{ij} = \frac{1}{n} \times RF \times \left(\frac{K}{\mu\varepsilon_{ij}}\right)^{12} = \frac{1}{n} \times 1 \times \left(\frac{235}{\mu\varepsilon_{ij}}\right)^{12}$$

For example, the allowable repetitions of a tandem group with dual tyres (TADT) with a total group load of 130 kN is calculated as:

$$N_{\text{TADT},130} = \frac{1}{2} \times 1 \times \left(\frac{235}{50.9}\right)^{12} = 4.69 \times 10^{7}$$

The allowable repetitions of all axle group/load combinations are shown to Table K 8 to Table K 12. Note that the post-cracking phase of life is not considered in this analysis.

Table K 8: Calculation of cemented base fatigue damage – SAST

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	11 020	1	8.5	1.99E+17	5.53E-14
20	307 601	1	17.1	4.54E+13	6.78E-09
30	607 578	1	25.6	3.58E+11	1.70E-06
40	617 403	1	34.2	1.11E+10	5.57E-05
50	1 176 642	1	42.7	7.72E+08	1.52E-03
60	915 297	1	51.2	8.74E+07	1.05E-02
70	255 529	1	59.8	1.36E+07	1.88E-02
80	31 216	1	68.3	2.75E+06	1.13E-02
90	4 272	1	76.9	6.63E+05	6.44E-03
100	1 391	1	85.4	1.89E+05	7.38E-03
110	684	1	94.0	5.96E+04	1.15E-02
120	684	1	102.5	2.11E+04	3.24E-02
130	684	1	111.0	1.99E+17	5.53E-14
				Total SAST damage	0.184

Table K 9: Calculation of cemented base fatigue damage – SADT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	66 334	1	7.8	5.59E+17	1.19E-13
20	166 094	1	15.7	1.26E+14	1.31E-09
30	448 086	1	23.5	1.00E+12	4.48E-07
40	418 863	1	31.3	3.21E+10	1.31E-05
50	320 880	1	39.1	2.22E+09	1.44E-04
60	183 475	1	47.0	2.44E+08	7.52E-04
70	124 150	1	54.8	3.87E+07	3.21E-03
80	88 299	1	62.6	7.83E+06	1.13E-02
90	56 708	1	70.4	1.91E+06	2.96E-02
100	26 606	1	78.3	5.34E+05	4.98E-02
110	7 827	1	86.1	1.71E+05	4.58E-02
120	2 212	1	93.9	6.04E+04	3.66E-02
130	466	1	101.7	2.32E+04	2.01E-02
				Total SADT damage	0.197

Table K 10: Calculation of cemented base fatigue damage – TAST

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	32	2	4.3	3.55E+20	8.98E-20
20	214	2	8.5	9.97E+16	2.15E-15
30	249	2	12.8	7.33E+14	3.39E-13
40	518	2	17.1	2.27E+13	2.28E-11
50	2 600	2	21.4	1.54E+12	1.69E-09
60	9 243	2	25.6	1.79E+11	5.16E-08
70	15 129	2	29.9	2.78E+10	5.45E-07
80	14 949	2	34.2	5.54E+09	2.70E-06
90	14 355	2	38.4	1.38E+09	1.04E-05
100	12 978	2	42.7	3.86E+08	3.36E-05
110	8 797	2	47.0	1.22E+08	7.21E-05

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
120	5 313	2	51.2	4.37E+07	1.22E-04
130	2 649	2	55.5	1.66E+07	1.60E-04
140	1 385	2	59.8	6.78E+06	2.04E-04
150	760	2	64.1	2.95E+06	2.58E-04
160	385	2	68.3	1.38E+06	2.80E-04
170	208	2	72.6	6.62E+05	3.14E-04
180	123	2	76.9	3.32E+05	3.71E-04
190	65	2	81.1	1.75E+05	3.71E-04
200	50	2	85.4	9.43E+04	5.30E-04
				Total TAST damage	0.003

Table K 11: Calculation of cemented base fatigue damage – TADT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	3 740	2	3.9	1.15E+21	3.26E-18
20	14 905	2	7.8	2.80E+17	5.33E-14
30	16 167	2	11.7	2.16E+15	7.50E-12
40	51 204	2	15.7	6.32E+13	8.10E-10
50	168 246	2	19.6	4.41E+12	3.81E-08
60	246 335	2	23.5	5.00E+11	4.93E-07
70	283 346	2	27.4	7.92E+10	3.58E-06
80	253 017	2	31.3	1.60E+10	1.58E-05
90	197 125	2	35.2	3.92E+09	5.03E-05
100	187 568	2	39.1	1.11E+09	1.69E-04
110	162 315	2	43.0	3.55E+08	4.57E-04
120	154 157	2	47.0	1.22E+08	1.26E-03
130	152 240	2	50.9	4.69E+07	3.25E-03
140	169 231	2	54.8	1.93E+07	8.75E-03
150	207 977	2	58.7	8.47E+06	2.45E-02
160	148 070	2	62.6	3.92E+06	3.78E-02
170	92 049	2	66.5	1.90E+06	4.85E-02
180	48 252	2	70.4	9.57E+05	5.04E-02
190	22 106	2	74.3	5.01E+05	4.41E-02
200	8 627	2	78.3	2.67E+05	3.23E-02
210	2 075	2	82.2	1.49E+05	1.39E-02
220	834	2	86.1	8.55E+04	9.76E-03
230	414	2	90.0	5.02E+04	8.25E-03
				Total TADT damage	0.284

Table K 12: Calculation of cemented base fatigue damage – TRDT

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	74	3	2.6	9.91E+22	7.47E-22
20	2 321	3	5.2	2.42E+19	9.59E-17
30	4 869	3	7.8	1.86E+17	2.61E-14
40	19 492	3	10.4	5.91E+15	3.30E-12
50	61 672	3	13.0	4.06E+14	1.52E-10
60	109 801	3	15.7	4.22E+13	2.60E-09

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
70	144 700	3	18.3	6.70E+12	2.16E-08
80	123 402	3	20.9	1.36E+12	9.07E-08
90	91 020	3	23.5	3.33E+11	2.73E-07
100	74 429	3	26.1	9.46E+10	7.87E-07
110	54 775	3	28.7	3.03E+10	1.81E-06
120	48 810	3	31.3	1.07E+10	4.56E-06
130	46 576	3	33.9	4.10E+09	1.13E-05
140	49 743	3	36.5	1.69E+09	2.94E-05
150	59 318	3	39.1	7.41E+08	8.01E-05
160	60 902	3	41.7	3.42E+08	1.78E-04
170	71 321	3	44.3	1.66E+08	4.31E-04
180	90 236	3	47.0	8.14E+07	1.11E-03
190	114 448	3	49.6	4.26E+07	2.68E-03
200	124 808	3	52.2	2.31E+07	5.40E-03
210	76 013	3	54.8	1.29E+07	5.90E-03
220	34 617	3	57.4	7.39E+06	4.68E-03
230	11 491	3	60.0	4.34E+06	2.65E-03
240	3 704	3	62.6	2.61E+06	1.42E-03
250	1 339	3	65.2	1.60E+06	8.36E-04
260	118	3	67.8	1.00E+06	1.18E-04
				Total TRDT damage	0.026

Steps 24 and 25

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 K 8 to Table K 12.

Step 26

The cemented base fatigue damage resulting from each axle group type is the sum of the damage caused by each load level of the group. These damages are summarised in Table K 13.

Table K 13: Total cemented base fatigue damage

Axle group type	Cemented base fatigue damage
SAST	0.184
SADT	0.197
TAST	0.003
TADT	0.284
TRDT	0.026

Step 27

Total fatigue damage to the cemented base from the design traffic is the sum of damage resulting from each axle group type:

$$0.184 + 0.197 + 0.003 + 0.284 + 0.026 = 0.694$$

Step 28

Post-cracking phase is not considered in this analysis as when the cemented base fatigue cracks it will readily propagate through the sprayed seal surface.

Step 29

As the fatigue damage to cemented base is less than 1, the trial pavement treatment is acceptable.

Step 30

Given that the fatigue damage (0.69) is considerably less than 1, consideration could be given to assessing the suitability of a treatment with a slightly reduced cemented base thickness.

Step 31

Compare this trial treatment with other options based on whole-of-life costing.

Appendix L Example of Granular Overlay Thickness Design Considering Lime Stabilisation of Subgrade

L.1 Introduction

An investigation of a homogeneous section of a sprayed seal unbound granular pavement indicated that due to excessive moisture the clay subgrade had low strength compared to other sections of the project. This section of pavement comprises 300 mm of existing granular material on a clay subgrade with a design CBR of 3%.

The design traffic loading for a 20-year design period:

- 2 x 10^7 heavy vehicle axle groups (HVAG)
- traffic load distribution (TLD) is in accordance with Appendix G of Austroads (2018a).

A treatment option identified for evaluation was to:

- remove the existing granular materials
- lime stabilise the subgrade to a depth of 250 mm
- replace the existing granular materials
- place a granular overlay over the entire project
- sprayed seal surfacing.

Laboratory testing indicated that the addition of 5% lime to the clay subgrade was the minimum lime content required to achieve long-term changes in material properties. The design CBR of the lime-stabilised layer is 10%.

The linear elastic model CIRCLY (Mincad 2009) was used to calculate the critical strains in this design example.

L.2 Procedure

The treatment was designed using the Section 10.10 method.

As described in Section 10.10, the structural adequacy of the trial treatment is calculated following the steps in Table 8.1, Table 8.2 and Table 8.3 of Austroads (2018a).

Step 1

Trial pavement configuration

Table L 1 summarises the trial treatment to evaluate.

Table L 1: Trial pavement configuration

Material type	Thickness (mm)
Sprayed bituminous seal	–
Granular overlay	100
Removed and replaced existing unbound granular material	300
Lime-stabilised subgrade, design CBR = 10%	250
Subgrade, design CBR = 3%	Semi-infinite

Step 2

Elastic parameters for subgrade and stabilised subgrade

Subgrade

Design CBR	=	3%
E_v	=	30 MPa – Section 5.6 of Austroads (2018a)
E_H	=	15 MPa
ν_v	=	ν_H = 0.45 – Section 5.6 of Austroads (2018a)
f	=	$E_V/(1 + \nu_V)$ = 20.7

Top sub-layer of lime-stabilised subgrade.

Minimum of:

$E_{V\text{ stabilised subgrade top sub-layer}}$	=	$E_{V\text{ subgrade}} \times 2^{(\text{stabilisation thickness}/150)}$
	=	$30 \times 2^{(250/150)}$ = 95 MPa
$E_{V\text{ stabilised subgrade top sub-layer}}$	=	CBR=10 × 10 = 100 MPa – from laboratory CBR testing of lime-stabilised subgrade
E_v	=	minimum (95 MPa, 100 MPa) = 95 MPa
E_H	=	47.5 MPa
ν_v	=	ν_H = 0.45 – Section 5.6 of Austroads (2018a)
f	=	$E_V/(1 + \nu_V)$ = 65.5

Other stabilised subgrade sub-layers

Divide the stabilisation thickness into five equi-thick sub-layers (Section 8.2.3 of Austroads 2018a), each of thickness 250/5 = 50 mm.

Calculate the ratio of moduli of adjacent layers:

$$\mathrm{R} = \left[\frac{E_{\text{V stabilised subgrade top sublayer}}}{E_{\text{Vsubgrade}}}\right]^{\frac{1}{5}}$$, i.e. $R = (95/30)^{1/5} = 1.259$

Therefore, elastic parameters of the first stabilised sub-layer on top of the subgrade are:

E_{V1}	=	R × E_V subgrade = 1.259 × 30 = 37.8 MPa
E_{H1}	=	0.5 × E_{V1} = 18.9 MPa
ν_V	=	ν_H = 0.45
f	=	$E_V/(1 + \nu_V)$ = 37.8/1.45 = 26.1

Elastic properties of other sub-layers are calculated similarly using the elastic properties of the underlying sub-layer and are listed in the following table:

Table L 2: Modelled stabilised subgrade

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f value
		E_V	E_H	ν_V	ν_H	
Stabilised subgrade	50	95	48	0.45	0.45	65.5
Stabilised subgrade	50	75	38	0.45	0.45	52.0
Stabilised subgrade	50	60	30	0.45	0.45	41.3
Stabilised subgrade	50	48	24	0.45	0.45	32.8
Stabilised subgrade	50	38	19	0.45	0.45	26.1
Subgrade	Semi-infinite	30	15	0.45	0.45	20.7

Step 3

Elastic parameters for top granular sub-layer

Minimum of:

$E_{V\ \text{top granular sub-layer}}$ = $E_{V\ \text{stabilised subgrade top sub-layer}} \times 2^{(\text{top granular thickness}/125)} = 95 \times 2^{(400/125)} = 873$ Section 8.2.3 of Austroads (2018a)

$E_{V\ \text{top granular sub-layer}}$ = 350 MPa – Table 6.3 of Austroads (2018a), assuming normal standard crushed rock

E_V = minimum (873 MPa, 350 MPa) = 350 MPa

E_H = 175 MPa

ν_V = ν_H = 0.35 – Table 6.3 of Austroads (2018a)

f = $E_V/(1 + \nu_V)$ = 259.3

Step 4

Elastic parameters for other granular sub-layers

Divide the total granular layer thickness into five equi-thick sub-layers (Section 8.2.3 of Austroads 2018a), each of thickness 400/5 = 80 mm. Calculate the ratio of moduli of adjacent layers:

$$R = \left[\frac{E_{V\ \text{top granular sublayer}}}{E_{V\ \text{stabilised subgrade top sublayer}}}\right]^{\frac{1}{5}}$$, i.e. $R = (350/95)^{1/5} = 1.298$.

Therefore, elastic parameters of the first granular sub-layer on top of the subgrade are:

E_V = $R \times E_{V\ \text{top selected subgrade}} = 1.298 \times 95 = 123.3$ MPa

E_H = $0.5 \times E_V = 61.6$ MPa

ν_V = $\nu_H = 0.35$

f = $E_V/(1 + \nu_V) = 123.3/1.35 = 91.3$

Elastic properties of other sub-layers are calculated similarly using the elastic properties of the underlying sub-layer and are listed Table L 3.

Table L 3: Modelled pavement configuration

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f value
		E_V	E_H	ν_V	ν_H	
Granular	80	350	175	0.35	0.35	259.3
Granular	80	270	135	0.35	0.35	199.7
Granular	80	208	104	0.35	0.35	153.9
Granular	80	160	80	0.35	0.35	118.6
Granular	80	123	62	0.35	0.35	91.3
Stabilised subgrade	50	95	48	0.45	0.45	65.5
Stabilised subgrade	50	75	38	0.45	0.45	52.0
Stabilised subgrade	50	60	30	0.45	0.45	41.3
Stabilised subgrade	50	48	24	0.45	0.45	32.8
Stabilised subgrade	50	38	19	0.45	0.45	26.1
Subgrade	Semi-infinite	30	15	0.45	0.45	20.7

Step 5 not relevant.

Step 6 not relevant.

Step 7

Permanent deformation allowable load repetitions (Equation 3 of Austroads 2018a):

$$N = \left[\frac{9150}{\mu \in}\right]^7$$

Step 8 not relevant.

Step 9 not relevant.

Step 10

- 2 x 10^7 heavy vehicle axle groups (HVAG)
- traffic load distribution (TLD) is in accordance with Appendix G of Austroads (2018a).

Step 11

Using the example distribution in Appendix G of Austroads (2018a), ESA/HVAG = 0.7.

Therefore, using Equation 37 of Austroads (2018a):

$$\text{DESA} = 0.7 \times 2 \times 10^7 = 1.4 \times 10^7$$

Step 12

Standard Axle load is represented as:

- tyre-pavement contact stress = 750 kPa
- tyre load radius = 92.1 mm
- four circular areas separated centre-to-centre 330 mm, 1470 mm and 330 mm (Figure 8.2 of Austroads 2018a).

Step 13

Critical locations to calculate Standard Axle strains are:

- vertical compressive strain on top of lime-stabilised subgrade directly beneath the inner-most tyre of one of the dual tyre sets
- vertical compressive strain on top of lime-stabilised subgrade between the tyres of one of the dual tyre sets
- vertical compressive strain on top of subgrade directly between the tyres of one of the dual tyre sets
- vertical compressive strain on top of subgrade midway between one of the dual tyre sets.

Step 14

Critical strains under a Standard Axle load:

- lime-stabilised subgrade: maximum vertical compressive strain is 743 $\mu\varepsilon$ on top of lime-stabilised subgrade midway between the two loaded tyres
- subgrade: maximum vertical compressive strain is 860 $\mu\varepsilon$ on top of subgrade midway between the two loaded tyres.

Step 15 not relevant

Step 16 not relevant

Step 17 not relevant

Step 18

Determine the allowable load repetitions in terms of permanent deformation (ESA):

Using the strain on top of lime-stabilised subgrade:

$$N = \left[\frac{9150}{743}\right]^7 = 4.3 \text{ x } 10^7 \text{ ESA}$$

Using the strain on top of subgrade:

$$N = \left[\frac{9150}{860}\right]^7 = 1.54 \text{ x } 10^7 \text{ ESA}$$

Step 19

The allowable load repetitions of 4.3 x 10^7 ESA and 1.5 x 10^7 ESA exceeds the design traffic of 1.4 x 10^7 ESA from Step 11.

The trial treatment is acceptable in terms of rutting and shape loss.

Steps 20–30 not relevant.

Step 31

Compare this trial treatment with other options based on whole-of-life costing.

Appendix M Modified Granular Base Design Example

M.1 Introduction

An investigation of a homogeneous section of a sprayed seal unbound granular pavement indicated that the rutting was primarily occurring in the non-standard granular base. This section of pavement had a total 400 mm thickness of existing granular materials on a subgrade with a design CBR of 5%.

The design traffic loading for a 20-year design period:

- 2 x 10^6 heavy vehicle axle groups (HVAG)
- traffic load distribution (TLD) is in accordance with Appendix G of Austroads (2018a).

One treatment option identified for evaluation was to modify the top 150 mm granular base by the addition of 1% cementitious material to increase the rut-resistance of the base. Based on laboratory repeated load triaxial testing, the modification increased the maximum possible modulus from 250 MPa to 700 MPa and there was significant improvement in the permanent deformation characteristics of the base material. The measured unconfined compressive strength (UCS) testing of the treated material was 0.9 MPa and therefore using Austroads (2019a) can be categorised as a modified material. Modified materials are not susceptible to fatigue cracking.

The linear elastic model CIRCLY (Mincad 2009) was used to calculate the critical strains in this design example.

M.2 Procedure

The treatment was designed using the Section 10.10 method.

As described in Section 10.10, the structural adequacy of the treatment is calculated following the steps in Table 8.1, Table 8.2 and Table 8.3 of Austroads (2018a).

Step 1

Try cementitious modification of top 150 mm granular base

Table M 1 lists the pavement configuration with the trial treatment.

Table M 1: Trial pavement treatment

Material type	Thickness (mm)
Sprayed bituminous seal	–
Modified granular base	150
Existing unbound granular material	250
Subgrade, design CBR = 5%	Semi-infinite

Step 2

Elastic parameters for subgrade

CBR = 5%

E_V = 50 MPa – Section 5.6 of Austroads (2018a)

E_H = 25 MPa

ν_V = ν_H = 0.45 – Section 5.6 of Austroads (2018a)

f = $E_V/(1 + \nu_V)$ = 34.5.

Step 3

Elastic parameters for top granular sub-layer

Divide the total thickness (400 mm) of the modified granular base (150 mm) and underlying granular subbase (250 mm) into five equi-thick with the top granular sub-layer being the minimum of:

$E_{V\ \text{top granular sub-layer}}$ = $E_{V\ \text{subgrade}} \times 2^{(\text{top granular thickness}/125)} = 50 \times 2^{(400/125)} = 460$ MPa – Section 8.2.3 of Austroads (2018a)

$E_{V\ \text{modified granular base}}$ = 700 MPa – laboratory repeat load triaxial testing of modified granular base at materials in situ density, moisture content and stress levels under a Standard Axle

E_v = minimum (700 MPa, 460 MPa) = 460 MPa

E_H = 230 MPa

ν_v = ν_H = 0.35 – Section 6.3 of Austroads (2018a)

f = $E_V/(1 + \nu_V)$ = 340.7

Step 3

Elastic parameters for other granular sub-layers

Divide the total granular layer thickness into five equi-thick sub-layers (Section 8.2.3 of Austroads 2018a) each of thickness 400/5 = 80 mm.

Calculate the ratio of moduli of adjacent layers:

$$R = \left[\frac{E_{V\ \text{top granular sublayer}}}{E_{V\ \text{stabilised subgrade top sublayer}}}\right]^{\frac{1}{5}}, \text{ i.e. } R = (460/50)^{1/5} = 1.559$$

Therefore, elastic parameters of the first granular sub-layer on top of the subgrade are:

E_V = $R \times E_{V\ \text{subgrade}} = 1.559 \times 50 = 77.9$ MPa

E_H = $0.5 \times E_V = 39.0$ MPa

ν_V = $\nu_H = 0.35$

f = $E_V/(1 + \nu_V) = 77.9/1.35 = 57.7$

Elastic properties of other sub-layers are calculated similarly using the elastic properties of the underlying sub-layer and are listed in Table M 2.

Table M 2: Modelled pavement configuration

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f value
		E_V	E_H	ν_V	ν_H	
Granular	80	460	230	0.35	0.35	340.7
Granular	80	295	148	0.35	0.35	218.6
Granular	80	189	95	0.35	0.35	140.3
Granular	80	122	61	0.35	0.35	90.0
Granular	80	78	39	0.35	0.35	57.7
Subgrade	Semi-infinite	50	25	0.45	0.45	34.5

Step 5 not relevant.

Step 6 not relevant.

Step 7

Permanent deformation allowable loading (Equation 3 of Austroads 2018a).

$$N = \left[\frac{9150}{\mu \in}\right]^7$$

Step 8 not relevant.

Step 9 not relevant.

Step 10

- 2 x 10^6 heavy vehicle axle groups (HVAG)
- traffic load distribution (TLD) is in accordance with Appendix G of Austroads (2018a).

Step 11

Using the example distribution in Appendix G of Austroads (2018a), ESA/HVAG = 0.7.

Therefore, using Equation 37 of Austroads (2018a):

$$\text{DESA} = 0.7 \times 2 \times 10^6 = 1.4 \times 10^6$$

Step 12

Define Standard Axle load used to calculate strains

Standard Axle load represented as:

- tyre-pavement contact stress = 750 kPa
- tyre load radius = 92.1 mm
- four circular areas separated centre-to-centre 330 mm, 1470 mm and 330 mm (Figure 8.2 of Austroads 2018a).

Step 13

Determine critical locations to calculate strains

Critical locations to calculate strains are:

- vertical compressive strain on top of subgrade directly beneath the inner-most wheel of the dual tyre set
- vertical compressive strain on top of subgrade midway along the vertical axis located symmetrically between a pair of dual tyres.

Step 14

Determine critical strains under a Standard Axle load

Maximum vertical compressive strains from CIRCLY = 1183 $\mu\varepsilon$ on top of subgrade midway between the two loaded tyres.

Steps 15,16 and 17 not relevant

Step 18

Determine the allowable number of load repetitions in terms of permanent deformation (ESA):

$$N = \left[\frac{9150}{1183}\right]^7 = 1.66 \text{ x } 10^6 \text{ ESA}$$

Step 19

Compare allowable traffic loading with design traffic loading

The allowable load repetitions in terms of permanent deformation is 1.66×10^6 ESA which exceeds the design traffic = 1.4×10^6 ESA from Step 10.

The trial pavement composition is acceptable.

Steps 20–30 not relevant.

Step 31

Compare this trial treatment with other options based on whole-of-life costing.

Appendix N Foamed Bitumen Stabilisation Design Example

N.1 Introduction

One treatment option identified for the rehabilitation of an unbound granular pavement (Figure N 1) is strengthening by foamed bitumen stabilisation of the existing granular material and a sprayed seal surface. The existing pavement is founded on a clay subgrade (design CBR 5%) and consists of an unbound granular subbase (250 mm), unbound granular base (150 mm) and a sprayed bituminous seal surfacing.

The design traffic loading for a 20-year design period:

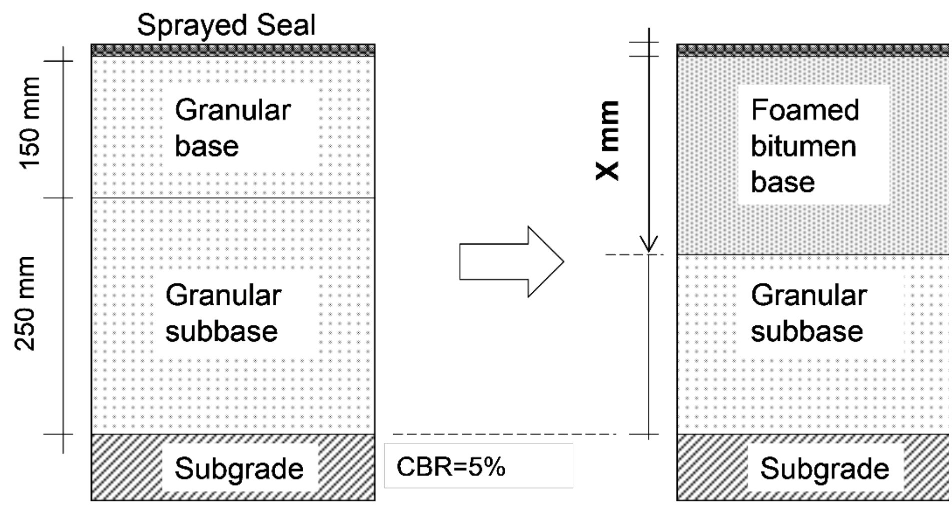
- 10^7 heavy vehicle axle groups (HVAG)
- traffic load distribution (TLD) is in accordance with Appendix G of Austroads (2018a).

The Weighted Mean Annual Pavement Temperature (WMAPT) is 26.5 °C and the heavy vehicle design speed is 90 km/h.

The average indirect tensile resilient modulus is 2438 MPa, measured on 150 mm diameter specimens after 72 hours of accelerated air-drying at 40 °C then soaked in water (95 kPa vacuum) for 10 minutes. The indirect moduli were measured in accordance with AS 2891.13.1-2013, using a rise time of 40 milliseconds and at a test temperature of 25 °C.

The linear elastic model CIRCLY (Mincad 2009) was used to calculate the critical strains in this design example.

Figure N 1: Pavement rehabilitation case study



N.2 Procedure

The treatment was designed using the Section 10.10 method.

As described in Section 10.10, the structural adequacy of the treatment is calculated following the steps in Table 8.1, Table 8.2 and Table 8.3 of Austroads (2018a).

Step 1

Trial the following pavement treatment:

Table N 1 list the trial pavement configuration.

Table N 1: Trial pavement configuration

Material type	Thickness (mm)
Sprayed bituminous seal surface	–
Foamed bitumen stabilised material	300
Unbound granular subbase material	100
Subgrade, CBR = 5%	Semi-infinite

Step 2

Elastic parameters for subgrade

Subgrade design CBR = 5%

E_v = 50 MPa

E_h = 25 MPa

ν_h = ν_v = 0.45

f = $E_v / (1 + \nu_v) = 50/1.45 = 34.5$

Step 3

Elastic parameters for top granular sub-layer

Top granular subbase:

$$E_{v\ \text{top of base}} = E_{v\ \text{subgrade}} \times 2^{(\text{total granular thickness}/125)} = 87\ \text{MPa}$$

$$E_h = 43.5\ \text{MPa}$$

Step 4

Elastic parameters for other granular sub-layers

Divide the total granular layer thickness into five equi-thick sub-layers (Section 8.2.3 of Austroads 2018a), each 100/5 = 20 mm thick.

Therefore, elastic parameters of the granular sub-layer on top of the subgrade are:

E_{v1} = $R \times E_{v\ \text{subgrade}} = 1.117 \times 50.0 = 55.9$ MPa

E_{v2} = $R \times E_{v\ \text{sub-layer 1}} = 1.117 \times 55.9 = 62.4$ MPa

E_{v3} = $R \times E_{v\ \text{sub-layer 2}} = 1.117 \times 62.4 = 69.7$ MPa

E_{v4} = $R \times E_{v\ \text{sub-layer 3}} = 1.117 \times 69.7 = 77.8$ MPa

E_{v5} = $R \times E_{v\ \text{sub-layer 4}} = 1.117 \times 77.8 = 87.0$ MPa

Step 5

Elastic parameters for cemented materials

Not relevant.

Step 6

Elastic parameters for foamed bitumen stabilised (FBS) material

Sub-layering of FBS layer is not required.

Poisson's ratio = 0.40.

Cured soaked indirect tensile modulus measured in the laboratory = 2464 MPa.

Heavy vehicle design speed = 90 km/h.

In-service pavement temperature (WMAPT): 26.5 °C

$$\frac{\text{Modulus at WMAPT}}{\text{Modulus at test temperature (T)}} = \exp(-0.025[26.5\text{-}25])$$

Modulus temperature adjustment = 0.96

$$\frac{\text{Modulus at speed V}}{\text{Modulus at test loading rate}} = 0.46 \times 90^{0.16}$$

Modulus loading rate adjustment = 0.95.

Modulus correction for temperature and loading rate = 2464 MPa x 0.95 x 0.96 = 2200 MPa.

Elastic properties of all materials, including granular sub-layers are listed in Table N 2.

Table N 2: Modelled pavement configuration

Material type	Thickness (mm)	Elastic modulus (MPa)		Poisson's ratio		f value
		E_v	E_h	ν_v	ν_h	
Sprayed seal surface	–	–	–	–	–	–
FBS	300	2200	2200	0.40	0.40	–
Granular	20	87	43.5	0.35	0.35	64.4
Granular	20	78	39	0.35	0.35	57.7
Granular	20	70	35	0.35	0.35	51.6
Granular	20	62	31	0.35	0.35	46.2
Granular	20	56	28	0.35	0.35	41.4
Subgrade	Semi-infinite	50	25	0.45	0.45	34.5

Step 7

Permanent deformation allowable load repetitions (Equation 3 of Austroads 2018a).

$$N = \left[\frac{9150}{\mu \in}\right]^7$$

Step 8

No cemented material therefore not relevant.

Step 9

There is no asphalt in this trial treatment.

Equation 19 is the FBS fatigue relationship:

$$N = \left[\frac{6918(0.856 \times 7 + 1.08)}{2200^{0.36}\mu\varepsilon}\right]^5$$

Assuming a FBS design modulus of 2200 MPa and volume of bitumen (V_b) of 7%.

Step 10

- Cumulative HVAG = 10^7
- traffic load distribution is in accordance with Appendix G of Austroads (2018a).

Step 11

Using the example distribution in Appendix G of Austroads (2018a), ESA/HVAG = 0.7.

Therefore, using Equation 37 of Austroads (2018a):

$$\text{DESA} = 0.7 \times 10^7 = 7 \times 10^6$$

Step 12

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 of Austroads 2018a).

Step 13

Critical locations to calculate Standard Axle strains are:

- vertical compressive strain on top of subgrade directly beneath the inner-most tyre of one of the dual tyre sets
- vertical compressive strain on top of subgrade midway between one of the dual tyre sets
- horizontal tensile strain bottom of FBS material directly beneath the inner-most tyre of one of the dual tyre sets
- horizontal tensile strain bottom of FBS material midway between one of the dual tyre sets.

Step 14

Critical strains under a Standard Axle load:

- subgrade: maximum vertical compressive strain is 333 $\mu\varepsilon$ on top of subgrade midway between the two loaded tyres
- FBS material: maximum horizontal tensile strain of 143.8 $\mu\varepsilon$ between the tyres of one of the dual tyre sets.

Step 15

Single axle with single tyres at 53 kN axle load:

- tyre-pavement contact stress = 800 kPa
  - load radius = 102.4 mm
- two circular areas separated centre-to-centre 2130 mm (Figure 8.2 of Austroads 2018a).

Step 16

Critical locations to calculate single axle single tyre strains are:

- horizontal tensile strain at the bottom of FBS material

The above strain is calculated directly beneath one of the loaded tyres of the single axle with single tyres.

Step 17

Critical strain resulting from single axle with single tyres with 53 kN axle load:

- FBS material: maximum horizontal tensile strain of 108.7 $\mu\varepsilon$.

Step 18

Permanent deformation allowable loading:

$$N = \left[\frac{9150}{333}\right]^7 = 1.2 \text{ x } 10^{10} \text{ ESA}$$

Step 19

The allowable load repetitions in terms of permanent deformation is 1.2 x 10^{10} ESA compared to a design traffic loading of 7 x 10^6 ESA from Step 11. The trial treatment is acceptable in terms of rutting and shape loss.

Step 20

FBS material is present in the candidate treatment, therefore Steps 21 to 27 must be repeated for each axle group load of each axle group type.

Step 21

Steps 22 to 26 are repeated for each axle group type present in the traffic load distribution:

- 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.

Step 22

The expected repetitions of each load for each axle group type is calculated in Table N 3 to Table N 7 by entering the data from the TLD (Table G.1 of Austroads 2018a) 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 are the product of columns 2, 3 and 4.

Table N 3: Calculation of expected repetitions – single axle/single tyre (SAST)

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 N 4: Calculation of expected repetitions – single axle/dual tyres (SADT)

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 N 5: Calculation of expected repetitions – tandem axle/single tyre (TAST)

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

Axle group load (kN)	Proportion of loads	Proportion of axle group	Design traffic (HVAG)	Expected group repetitions
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 N 6: Calculation of expected repetitions – tandem axle/dual tyres (TADT)

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 N 7: Calculation of expected repetitions – triaxle dual tyres (TRDT)

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 23

The allowable repetitions of each axle group type and load level is calculated in Table N 8 to Table N 12.

As a first stage the critical FBS strain for each load level is estimated for a single constituent axle of each axle group type using Equation 43 of Austroads (2018a). The critical strains predicted in Steps 14 and 17 are adjusted as follows:

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SAST,53}}{53} = \frac{L_{ij}}{n} \times \frac{108.7}{53}$$ for axles within SAST, TAST groups

$$\mu\varepsilon_{ij} = \frac{L_{ij}}{n} \times \frac{\mu\varepsilon_{SADT,80}}{80} = \frac{L_{ij}}{n} \times \frac{143.8}{80}$$ for axles within SADT, TADT & TRDT groups

For example, the critical asphalt strain developed under an axle within a triaxle group (TRDT) with a total group load of 180 kN is calculated as:

$$\mu\varepsilon_{\mathrm{TRDT},60} = \frac{180}{3} \times \frac{143.8}{80} = 107.9\ \mu\varepsilon$$

The allowable repetitions of each axle group type and load magnitude are then calculated using Equation 19:

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

For example, the allowable repetitions of a triaxle group (TRDT) with a total group load of 180 kN are calculated as:

$$N_{\mathrm{TRDT},180} = \frac{1}{3} \times \left[\frac{6918(0.856 \times 7 + 1.08)}{2200^{0.36} \times 107.9}\right]^5 = 6.15 \times 10^6$$

The allowable repetitions of all axle group/load combinations are shown in Table N 8 to Table N 12.

Steps 24 and 25

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 N 8 to Table N 12.

Table N 8: Calculation of FBS fatigue damage – single axle/single tyre (SAST)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	11 020	1	20.5	7.46E+10	1.48E-07
20	307 601	1	41.0	2.33E+09	1.32E-04
30	607 578	1	61.5	3.07E+08	1.98E-03
40	617 403	1	82.0	7.28E+07	8.48E-03
50	1 176 642	1	102.5	2.39E+07	4.93E-02
60	915 297	1	123.1	9.55E+06	9.59E-02
70	255 529	1	143.6	4.42E+06	5.78E-02
80	31 216	1	164.1	2.27E+06	1.38E-02
90	4 272	1	184.6	1.26E+06	3.39E-03
100	1 391	1	205.1	7.44E+05	1.87E-03
110	684	1	225.6	4.62E+05	1.48E-03
120	684	1	246.1	2.99E+05	2.29E-03
130	684	1	266.6	2.00E+05	3.41E-03
				Total SAST damage	0.240

Table N 9: Calculation of FBS fatigue damage – single axle/dual tyres (SADT)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	66 334	1	18.0	1.43E+11	4.64E-07
20	166 094	1	36.0	4.46E+09	3.72E-05
30	448 086	1	53.9	5.93E+08	7.55E-04
40	418 863	1	71.9	1.40E+08	2.98E-03
50	320 880	1	89.9	4.60E+07	6.98E-03
60	183 475	1	107.9	1.85E+07	9.94E-03
70	124 150	1	125.8	8.57E+06	1.45E-02
80	88 299	1	143.8	4.39E+06	2.01E-02
90	56 708	1	161.8	2.43E+06	2.33E-02
100	26 606	1	179.8	1.44E+06	1.85E-02
110	7 827	1	197.7	8.94E+05	8.76E-03
120	2 212	1	215.7	5.78E+05	3.83E-03
130	466	1	233.7	3.87E+05	1.20E-03
				Total SADT damage	0.111

Table N 10: Calculation of FBS fatigue damage – tandem axle group/single tyre (TAST)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	32	2	10.3	1.16E+12	2.74E-11
20	214	2	20.5	3.73E+10	5.74E-09
30	249	2	30.8	4.87E+09	5.11E-08

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
40	518	2	41.0	1.16E+09	4.45E-07
50	2 600	2	51.3	3.80E+08	6.84E-06
60	9 243	2	61.5	1.53E+08	6.03E-05
70	15 129	2	71.8	7.07E+07	2.14E-04
80	14 949	2	82.0	3.64E+07	4.11E-04
90	14 355	2	92.3	2.01E+07	7.13E-04
100	12 978	2	102.5	1.19E+07	1.09E-03
110	8 797	2	112.8	7.39E+06	1.19E-03
120	5 313	2	123.1	4.77E+06	1.11E-03
130	2 649	2	133.3	3.21E+06	8.26E-04
140	1 385	2	143.6	2.21E+06	6.27E-04
150	760	2	153.8	1.57E+06	4.84E-04
160	385	2	164.1	1.13E+06	3.40E-04
170	208	2	174.3	8.39E+05	2.48E-04
180	123	2	184.6	6.30E+05	1.95E-04
190	65	2	194.8	4.81E+05	1.35E-04
200	50	2	205.1	3.72E+05	1.34E-04
				Total TAST damage	0.008

Table N 11: Calculation of FBS fatigue damage – tandem axle group/dual tyres (TADT)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	3 740	2	9.0	2.29E+12	1.64E-09
20	14 905	2	18.0	7.14E+10	2.09E-07
30	16 167	2	27.0	9.41E+09	1.72E-06
40	51 204	2	36.0	2.23E+09	2.29E-05
50	168 246	2	44.9	7.40E+08	2.27E-04
60	246 335	2	53.9	2.97E+08	8.30E-04
70	283 346	2	62.9	1.37E+08	2.07E-03
80	253 017	2	71.9	7.02E+07	3.60E-03
90	197 125	2	80.9	3.89E+07	5.06E-03
100	187 568	2	89.9	2.30E+07	8.16E-03
110	162 315	2	98.9	1.43E+07	1.14E-02
120	154 157	2	107.9	9.23E+06	1.67E-02
130	152 240	2	116.8	6.21E+06	2.45E-02
140	169 231	2	125.8	4.28E+06	3.95E-02
150	207 977	2	134.8	3.03E+06	6.86E-02
160	148 070	2	143.8	2.19E+06	6.75E-02
170	92 049	2	152.8	1.62E+06	5.68E-02
180	48 252	2	161.8	1.22E+06	3.96E-02
190	22 106	2	170.8	9.29E+05	2.38E-02
200	8 627	2	179.8	7.18E+05	1.20E-02
210	2 075	2	188.7	5.64E+05	3.68E-03
220	834	2	197.7	4.47E+05	1.87E-03
230	414	2	206.7	3.58E+05	1.16E-03
				Total TADT damage	0.387

Table N 12: Calculation of FBS fatigue damage – triaxle group/dual tyres (TRDT)

Axle group load (kN)	Expected group repetitions	Axles in group	Critical strain (microstrain)	Allowable group repetitions	Damage
10	74	3	6.0	1.16E+13	6.40E-12
20	2 321	3	12.0	3.62E+11	6.42E-09
30	4 869	3	18.0	4.76E+10	1.02E-07
40	19 492	3	24.0	1.13E+10	1.72E-06
50	61 672	3	30.0	3.70E+09	1.67E-05
60	109 801	3	36.0	1.49E+09	7.38E-05
70	144 700	3	41.9	6.97E+08	2.08E-04
80	123 402	3	47.9	3.57E+08	3.46E-04
90	91 020	3	53.9	1.98E+08	4.60E-04
100	74 429	3	59.9	1.17E+08	6.38E-04
110	54 775	3	65.9	7.24E+07	7.57E-04
120	48 810	3	71.9	4.68E+07	1.04E-03
130	46 576	3	77.9	3.14E+07	1.48E-03
140	49 743	3	83.9	2.16E+07	2.30E-03
150	59 318	3	89.9	1.53E+07	3.87E-03
160	60 902	3	95.9	1.11E+07	5.49E-03
170	71 321	3	101.9	8.19E+06	8.71E-03
180	90 236	3	107.9	6.15E+06	1.47E-02
190	114 448	3	113.8	4.71E+06	2.43E-02
200	124 808	3	119.8	3.65E+06	3.42E-02
210	76 013	3	125.8	2.86E+06	2.66E-02
220	34 617	3	131.8	2.26E+06	1.53E-02
230	11 491	3	137.8	1.81E+06	6.35E-03
240	3 704	3	143.8	1.46E+06	2.53E-03
250	1 339	3	149.8	1.19E+06	1.12E-03
260	118	3	155.8	9.80E+05	1.21E-04
				Total TRDT damage	0.151

Step 26

The FBS fatigue 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 N 13.

Table N 13: Total FBS fatigue damage

Axle group type	Total group damage
SAST	0.240
SADT	0.111
TAST	0.008
TADT	0.387
TRDT	0.151

Step 27

Total fatigue damage is the sum of damage resulting from each axle group type:

Total FBS fatigue damage = 0.240 + 0.111 + 0.008 + 0.387 + 0.151 = 0.90.

Step 28 is not relevant.

Step 29

Total fatigue damage to the FBS layer is less than 1, therefore the trial pavement is acceptable.

Step 30

As the total FBS fatigue damage (0.90) was close to 1, there is limited potential to evaluate a lower FBS thickness.

Step 31

Compare this trial treatment with other options based on whole-of-life costing.

Appendix O Example of the Design of Concrete Overlays on Flexible Pavements

A concrete overlay is to be placed on an existing pavement consisting of 120 mm of sound asphalt with 150 mm thick granular base and 200 mm thick subbase material. The design traffic loading is 10^7 HVAG.

The equivalent design subgrade strength of the existing pavement was calculated using the following inputs:

- Based on particle size distribution, plasticity and visual inspection, the granular base appears sound quality, consistent with a material with an in situ CBR of 60%.
- Based on DCP measurements a design CBR of 30% was adopted for the 200 mm thick granular subbase.
- Based on laboratory-soaked CBR testing, the design CBR of 5% was selected for the subgrade.

An equivalent subgrade design strength of 13% was calculated as follows using Equation 20.

$$CBR_m = \left[\frac{0.15\ x\ 60^{0.333} +\ 0.20 \text{ x } 30^{0.333} + 0.65 \text{ x } 5^{0.333}}{0.15 +\ 0.20 + 0.65}\right] = 12\%$$

Reference to Table 9.1 of Austroads (2018a) indicates that, for a design traffic of 10^7 HVAG, the minimum subbase requirement for the concrete overlay is 170 mm of bound material or 125 mm of lean-mix concrete subbase. After costing these alternatives it was decided to place a 50 mm thick asphalt over the existing 120 mm of asphalt. As the existing asphalt is sound, the resulting total thickness of bound material is 170 mm.

Figure 9.1 of Austroads (2018a) indicates that with this 170 mm bound subbase on a subgrade design CBR of 12%, the effective subgrade CBR is 75%.

Having reached this stage, the design process is identical to that for the design of new rigid pavements as described in Section 9 of Austroads (2018a) and illustrated by the example in Appendix L of Austroads (2018a).

Appendix P Example of the Design of Asphalt Overlays on Rigid Pavements

A plain jointed concrete pavement with 4.5 m slab length located in the western region of Sydney is to receive an asphalt overlay. FWD testing on the existing pavement was undertaken in accordance with Section 4.10 resulting in a characteristic value for maximum deflection at the outer edge of transverse joints of 0.70 mm and a characteristic value of differential deflection (across a joint) of 0.10 mm. As described in Section 12.2, these values exceed the tolerable values of:

- 0.57 mm for mean deflection and
- 0.08 mm for differential deflections.

To protect against reflective cracking caused by load-induced differential movements across joints and cracks, the maximum deflection (0.70 mm) needs to be reduced to 0.57 mm and the differential deflection (0.10 mm) needs to be reduced to 0.08 mm. Each 5 mm of overlay achieves a 1% reduction.

The required percentage reduction in maximum deflection is:

$$\frac{0.70 - 0.57}{0.70} \times 100 = 19\%$$

The overlay thickness to achieve this percentage reduction is 19 × 5 = 95 mm.

The required percentage reduction in differential deflection is:

$$\frac{0.10 - 0.08}{0.10} \times 100 = 20\%$$

The overlay thickness to achieve this percentage reduction is 20 × 5 = 100 mm.

Hence, a minimum overlay thickness of 100 mm is required to inhibit load-induced cracking.

As noted in Section 12.2, measures to inhibit reflective cracking may also need to be considered.

Appendix Q Example of Economic Evaluation of Alternatives

Q.1 Example of the use of the Procedures for the Evaluation of Alternative Treatments for a Flexible Pavement

An existing pavement comprises 120 mm of cracked asphalt on granular material. Two alternative treatments have been selected for evaluation:

Option A:

- mill 100 mm of existing cracked asphalt and replace a 100 mm thick asphalt overlay. This treatment has an estimated life of 20 years
- zero subsequent rehabilitation costs during the 20-year analysis period
- routine maintenance.

Option B:

- initially mill 40 mm existing cracked asphalt and replace. It is estimated that this treatment will have a life of 5 years
- at years 5, 10 and 15, mill 40 mm cracked asphalt and replace
- routine maintenance.

Year 2019 was selected as the evaluation year and an analysis period of 20 years was adopted. Costs for both options were calculated using a discount rate of 7%, with sensitivity analysis at discount rates of 4% and 10%.

The 2019 unit rates used in the analysis are shown in Table Q 1.

Table Q 1: 2019 unit rates

Item	2019 unit rate ($/m^2)
Asphalt 40 mm depth	15
Asphalt 100 mm depth	34
Milling 40 mm depth	5
Milling 100 mm depth	13

Table Q 2 and Table Q 3 summarise the whole-of-life road agency costs for the two options. As the condition of the pavement at the end of the analysis period would be similar in the two options, salvage value of the two options is the same.

It is apparent that Option A has the lowest whole-of-life costs at discount rates of 4% and 7%. At a discount rate of 10% the costs of the two options are similar. It would be expected that road user costs would be greater for Option B due to delay costs associated with the five yearly overlays.

Note that as the salvage value of the pavements at 20 years following Options A and B were considered the same, the salvage value has not been included in the costings.

Table Q 2: Option A: costing analysis ($/m²)

Year	Description of work	Unit rate	Routine Mtce	Total	PV at 4%	Cum PV at 4%	PV at 7%	Cum PV at 7%	PV at 10%	Cum PV at 10%
0	Initial construction	47.00		47.00	47.00	47.00	47.00	47.00	47.00	47.00
1			0.50	47.50	0.48	47.48	0.47	47.47	0.45	47.45
2			0.50	48.00	0.46	47.94	0.44	47.90	0.41	47.87
3			0.50	48.50	0.44	48.39	0.41	48.31	0.38	48.24
4			0.50	49.00	0.43	48.81	0.38	48.69	0.34	48.58
5			0.50	49.50	0.41	49.23	0.36	49.05	0.31	48.90
6			0.50	50.00	0.40	49.62	0.33	49.38	0.28	49.18
7			0.50	50.50	0.38	50.00	0.31	49.69	0.26	49.43
8			0.50	51.00	0.37	50.37	0.29	49.99	0.23	49.67
9			0.50	51.50	0.35	50.72	0.27	50.26	0.21	49.88
10			0.50	52.00	0.34	51.06	0.25	50.51	0.19	50.07
11			0.50	52.50	0.32	51.38	0.24	50.75	0.18	50.25
12			0.50	53.00	0.31	51.69	0.22	50.97	0.16	50.41
13			0.50	53.50	0.30	51.99	0.21	51.18	0.14	50.55
14			0.50	54.00	0.29	52.28	0.19	51.37	0.13	50.68
15			0.50	54.50	0.28	52.56	0.18	51.55	0.12	50.80
16			0.50	55.00	0.27	52.83	0.17	51.72	0.11	50.91
17			0.50	55.50	0.26	53.08	0.16	51.88	0.10	51.01
18			0.50	56.00	0.25	53.33	0.15	52.03	0.09	51.10
19			0.50	56.50	0.24	53.57	0.14	52.17	0.08	51.18
20	Salvage	0.00	0.50	57.00	0.23	53.80	0.13	52.30	0.07	51.26

Summary of discounted costs

Discount rate	Construction	Maintenance	Salvage	Total
4%	47.00	6.80	0	53.80
7%	47.00	5.30	0	52.30
10%	47.00	4.26	0	51.26

Table Q 3: Option B: costing analysis ($/m²)

Year	Description of work	Unit rate	Routine mtce	Total	PV at 4%	Cum PV at 4%	PV at 7%	Cum PV at 7%	PV at 10%	Cum PV at 10%
0	Initial construction	20.00		20.00	20.00	20.00	20.00	20.00	20.00	20.00
1			0.50	0.50	20.00	20.00	0.47	20.47	0.45	20.45
2			0.50	0.50	0.48	20.48	0.44	20.90	0.41	20.87
3			0.50	0.50	0.46	20.94	0.41	21.31	0.38	21.24
4			0.50	0.50	0.44	21.39	0.38	21.69	0.34	21.58
5	Mill and replace 40 mm asphalt	20.00	0.50	20.50	0.43	21.81	14.62	36.31	12.73	34.31
6			0.50	0.50	16.85	38.66	0.33	36.64	0.28	34.60
7			0.50	0.50	0.40	39.06	0.31	36.95	0.26	34.85
8			0.50	0.50	0.38	39.44	0.29	37.25	0.23	35.09
9			0.50	0.50	0.37	39.80	0.27	37.52	0.21	35.30
10	Mill and replace 40 mm asphalt	20.00	0.50	20.50	0.35	40.16	10.42	47.94	7.90	43.20
11			0.50	0.50	13.85	54.01	0.24	48.18	0.18	43.38
12			0.50	0.50	0.32	54.33	0.22	48.40	0.16	43.54
13			0.50	0.50	0.31	54.64	0.21	48.61	0.14	43.68
14			0.50	0.50	0.30	54.94	0.19	48.80	0.13	43.81
15	Mill and replace 40 mm asphalt	20.00	0.50	20.50	0.29	55.23	7.43	56.23	4.91	48.72
16			0.50	0.50	11.38	66.61	0.17	56.40	0.11	48.83
17			0.50	0.50	0.27	66.88	0.16	56.56	0.10	48.93
18			0.50	0.50	0.26	67.14	0.15	56.71	0.09	49.02
19			0.50	0.50	0.25	67.38	0.14	56.84	0.08	49.10
20	Salvage	0	0.50	0.50	0.24	67.62	0.13	56.97	0.07	49.17

Summary of discounted costs

Discount rate	Construction	Maintenance	Salvage	Total
4%	20.00	47.85	0	67.85
7%	20.00	36.97	0	56.97
10%	20.00	29.17	0	49.17

Austroads' **Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design** provides advice for the investigation of existing sealed road pavements and the selection and design of pavement strategies/treatments. It covers pavement investigation, testing and evaluation, identification of causes and modes of distress, and description of treatment options.

Guide to Pavement Technology Part 5

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