Manual

Pavement rehabilitation

February 2020

Queensland Government

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1 Overview

1.1 Introduction

The Department of Transport and Main Roads *Pavement Rehabilitation Manual* is written as a supplement to *Part 5: Pavement Evaluation and Treatment Design* of the Austroads *Guide to Pavement Technology* (AGPT05) hereafter referred to as AGPT05. The *Pavement Rehabilitation Manual*, used in conjunction with AGPT05 and the other components of the Pavement Design System, as outlined in the Transport and Main Roads' *Pavement Design Supplement* (PDS), provides guidance and gives requirements for the evaluation of existing pavements and design of rehabilitation treatments for departmental road infrastructure projects.

Designers are referred to all other relevant documents (for example, the Austroads *Guide to Pavement Technology*) including the following departmental documents:

- *Pavement Design Supplement* (PDS)
- *Road Planning and Design Manual* (RPDM), and
- *Road Drainage Manual* (RDM).

1.2 Purpose

The purpose of this *Manual* is to provide, in conjunction with AGPT05, an evaluation procedure and range of appropriate design methods for the effective rehabilitation of the different (existing) road pavements for which the department is responsible. This *Manual* reflects departmental experience.

1.3 Scope

For the purposes of this *Manual*, the term ‘pavement rehabilitation’ is defined as ‘any activity that improves the functional or structural condition of a pavement while using some or all of its existing structure’.

This *Manual* is intended as a guide for professional, trained, experienced and knowledgeable pavement rehabilitation designers who are required to:

- work within the confines of departmental organisational policies, guidelines and road network requirements
- be aware of, assess and apply risk management and budgetary constraints to the entire road system and its various components
- consider local area or project specific issues, and
- optimise initial designs and in-service treatments to suit budget and whole-of-life costs (WOLC) issues.

It provides guidelines for undertaking project-level rehabilitation investigations. This involves:

- collecting and interpreting pavement condition data
- evaluating the existing pavement, and
- developing rehabilitation treatments, if required.

If rehabilitation treatment is required, the following are also involved:

- developing and designing treatment options to rehabilitate the existing pavement
- calculating the WOLC for each option
- evaluating the options, and
- providing recommendations to identify the most appropriate option(s) and how they are to be implemented.

This *Manual* is organised to reflect this process:

- Chapter 1 introduces the *Manual* and provides a brief overview of pavement rehabilitation in the Transport and Main Roads context.
- Chapter 2 discusses the collection of pavement condition data and the evaluation of pavements.
- Chapter 3 describes the typical alternative treatment options to treat various forms of pavement distress encountered across the state-controlled road network (managed by Transport and Main Roads).
- Chapter 4 details how the treatment options selected are to be designed, with additional information given in Chapter 5.
- Chapter 5 gives the methodology to be used for designing pavement rehabilitation treatments using either the general mechanistic procedure (GMP) or the overlay design procedure using design charts (deflection reduction method).
- Chapter 6 provides details about how options and strategies are to be compared.

Terms, definitions, abbreviations and acronyms used, and their meaning, are outlined in Table 1.14.

1.3.1 In scope

The types of pavements to which this *Manual* can be applied include:

- granular pavements surfaced with bituminous chip seals or asphalt
- full-depth or deep-strength asphalt pavements
- pavements with modified or stabilised layers, and
- concrete (rigid) pavements.

This *Manual* can be used to design a variety of pavement rehabilitation treatments including:

- granular overlays
- asphalt overlays
- inlays
- treatments that include a cementitiously modified or stabilised layer
- treatments that include a foamed bitumen (and lime) stabilised layer
- treatments that include a lime stabilised layer, and
- combinations of these.

1.3.2 Out of scope

This *Manual* cannot be applied to:

- bridge decks
- unsealed roads, or
- pavements other than those subject to general road traffic (for example, the *Manual* cannot be used to rehabilitate pavements at container handling yards, at ports, or for car parks).

In addition, because this *Manual* focuses on state-controlled roads which Transport and Main Roads manages and requires the use of departmental pavement rehabilitation and pavement design systems, it may not describe all the distress mechanisms, treatments or other considerations that may be applicable to other roads or contexts (for example, roads in other states and territories, roads in Defence bases). The user / designer must assess whether the *Manual* is applicable for any specific case.

1.4 Basis of Manual

The relationships between distress mechanisms and their causes are based either on the outcomes of departmental research programs or on the research findings of other organisations (for example, Austroads).

The rehabilitation techniques presented in this *Manual* are supported by field trials carried out by Transport and Main Roads. Trials of various treatment options are part of an ongoing program of research. This *Manual* presents some techniques that, while research continues, have shown positive indications of success. Techniques proven unsuccessful in trials are not included or discussed except where a caution against their use is considered appropriate.

1.5 Document precedence

For evaluating pavements and selecting and designing pavement rehabilitation treatments (on state-controlled roads), this *Manual* shall take precedence over all other manuals, guidelines, and so on. Where a more recently published document conflicts with this *Manual*, the department's Pavement Rehabilitation Unit must be contacted (email ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au) for clarification about what is to apply.

Refer to sections 1.1, 1.8, 1.9 and 1.10 for additional guidance.

1.6 Currency of documents referenced

This *Manual* contains references to documents current at the time of publication. Readers, designers and other users of the *Manual* must obtain and reference the latest version. Specifically, copies of the latest departmental technical publications, specifications and guidelines must be obtained by the reader, designer or other user before reading, interpreting or applying this *Manual*.

Where a document published more recently conflicts with this *Manual*, the department's Pavement Rehabilitation Unit must be contacted (email ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au) for clarification about what is to apply.

1.7 Feedback

Feedback about the *Manual* is encouraged. The feedback of users will help improve the *Manual* (for example, add to content) and provide direction for future versions. By providing feedback, readers will help maintain the relevance of the *Manual*. Feedback should be emailed to tmr.techdocs@tmr.qld.gov.au.

1.8 Pavement design

The PDS is an important reference and designers will need to refer to it to complete pavement rehabilitation investigations and designs. This *Manual* relies upon, but does not detail, the ‘design system’ presented in the PDS; however, this *Manual* takes precedence over the PDS for the assessment of existing pavements and for other pavement rehabilitation-related matters.

In some circumstances, there may be a question about which manual to apply. Following are some guidelines about when the PDS is to be used:

- for new pavements, including those used to widen an existing pavement; for widenings, the new pavement designs must be adjusted so the structural integrity and service life of the new and rehabilitated pavements are not compromised (for example, adjust layer thicknesses of widening pavement to allow for pavement drainage and/or provide subsurface pavement drains), and
- where a pavement evaluation and/or rehabilitation investigation recommends the existing pavement be reconstructed rather than rehabilitated.

For a single project, the ‘total’ design solutions may be based, in part, on both this *Manual* and the PDS; for example, in the case of pavement widening and overlay, the design for the new widening pavement must be designed using the PDS while the overlay must be designed in accordance with this *Manual*.

1.8.1 Pavement design system

The PDS describes the department’s ‘pavement design system’ and its ‘scope and applicability’ (for example, see sections 1 and 2 of the PDS). As noted previously, the system also applies to this *Manual* except this *Manual* takes precedence with respect to the assessment of existing pavements and for other pavement rehabilitation-related matters.

Reference should be made to the PDS for further details about the pavement design system.

1.9 Pavement surfacings

The designer must refer to the following guides for general information and to address the selection and design of the most appropriate surfacing treatment;

- Austroads *Guide to Pavement Technology Part 3: Pavement Surfacing* (AGPT03)
- Austroads Technical Report: *Update of Double / Double Design for Austroads Sprayed Seal Design Methods* (AP-T236), and/or
- Austroads Technical Report: *Update of the Austroads Sprayed Seal Design Method* (AP-T68).

Where there is a conflict between this *Manual* and these guides, the designer must check with the department’s Pavements, Research and Innovation Unit.

1.10 Austroads Guide to Pavement Technology

Austroads has published the *Guide to Pavement Technology*, an important reference for designers. This *Manual* must be read in conjunction with the Austroads *Guide to Pavement Technology*;

however, the department's specifications, policies, manuals, and so on take precedence over the Austroads *Guide to Pavement Technology*.

The Parts of the Austroads *Guide to Pavement Technology* at the date of publication of this chapter are given in Table 1.10. Also included in Table 1.10 are comments about each Part.

Table 1.10 – The current Austroads Guide to Pavement Technology and selected departmental manuals

Austroads *Guide to Pavement Technology*		**Transport and Main Roads manual**	**Comments**
	Title		
1	Introduction to Pavement Technology	No manual	The PDS and this *Manual* each contains a specific introduction. They also provide an overview with respect to pavement design and pavement rehabilitation in the departmental context. Austroads *Guide to Pavement Technology Part 1 Introduction to Pavement Technology* (AGPT01) provides a more general overview of pavement technology.
2	Pavement Structural Design	PDS	The PDS is a supplement to Austroads *Guide to Pavement Technology Part 2 Pavement Structural Design* (AGPT02). Notwithstanding this, the departmental PDS takes precedence over AGPT02.
3	Pavement Surfacings	No manual	The following guides provide guidance about pavement surfacings: • Austroads *Guide to Pavement Technology Part 3 Pavement Surfacings* (AGPT03) • AP-T236 *Update of Double / Double Design for Austroads Sprayed Seal Design Methods* (AP-T236), and • AP-T68 *Update of the Austroads Sprayed Seal Design Method* (AP-T68).

Austroads *Guide to Pavement Technology*		Transport and Main Roads manual	Comments
	Title		
4 and its sub-parts	Pavement Materials with sub-parts about granular base and sub-base materials, asphalt, materials for concrete road pavements, stabilised materials, recycled materials, bituminous binders, geotextiles and geogrids, test methods (discontinued), earthworks materials, aggregate and source rock, seals, and stabilising binders	No dedicated manual	Pavement Materials with sub-parts about granular base and sub-base materials, asphalt, materials for concrete road pavements, stabilised materials, recycled materials, bituminous binders, geotextiles and geogrids, test methods (discontinued), earthworks materials, aggregate and source rock, seals, and stabilising binders.
5	Pavement Evaluation and Treatment Design	This *Manual*	This *Manual* is a supplement to Austroads *Guide to Pavement Technology* Part 5 *Pavement Evaluation and Treatment Design* (AGPT05). Notwithstanding, this *Manual* takes precedence over AGPT05. AGPT05 provides additional guidance.
6	Unsealed Pavements	No manual	Austroads *Guide to Pavement Technology* Part 6 *Unsealed Pavements* (AGPT06) provides guidance with respect to unsealed road pavement technology.
7	Pavement Maintenance	No manual	In the first instance, refer to departmental manuals, and so on referenced in this *Manual* (Section 1.15). Austroads *Guide to Pavement Technology* Part 7 *Pavement Maintenance* (AGPT07) provides guidance with techniques and methods for carrying out a number of routine maintenance tasks, with emphasis on ad hoc activities (not pre-programmed) such as pothole filling and 'holding actions'. It complements AGPT05.
8	Pavement Construction	No dedicated manual Refer to the department's technical notes, specifications (MRSs) and technical specifications (MRTSs).	In the first instance, refer to the PDS and this *Manual*, which provide some guidance about pavement construction, and other relevant departmental documents (for example, technical notes, specifications and technical specifications). Similarly, Austroads *Guide to Pavement Technology* Part 8 *Pavement Construction* (AGPT08) provides additional guidance on how to ensure the 'As Constructed' pavement layers meet the design requirements.

Austroads *Guide to Pavement Technology*		Transport and Main Roads manual	Comments
	Title		
9	Pavement Work Practices (discontinued)	No dedicated manual Refer to the department's technical notes, specifications (MRSs) and technical specifications (MRTSs).	In the first instance, refer to the department's technical notes, engineering notes, specifications, technical specifications and the *Western Queensland Best Practice Guidelines* (WQBPG). Austroads' *Pavement Work Tips* (AP-PWT00) is an additional source of technical notes and similar publications related to pavement work practices produced by Austroads member organisations and industry associations. Austroads and the Australian Asphalt Pavement Association (AAPA) have, jointly, published a number of 'work tips' which are also useful references.
10	Subsurface Drainage	No dedicated manual.	In the first instance, refer to the PDS and this *Manual* for guidance about subsurface drainage. Also refer to the department's *Road Drainage Manual* (RDM), technical notes, engineering notes, specifications, technical specifications and the WQBPG. Austroads *Guide to Pavement Technology* Part 10 *Subsurface drainage* (AGPT10) provides guidance on the types of pavement subsurface drainage systems and procedures to design these systems, materials used for pavement subsurface drainage and construction and maintenance considerations for pavement subsurface drainage systems.

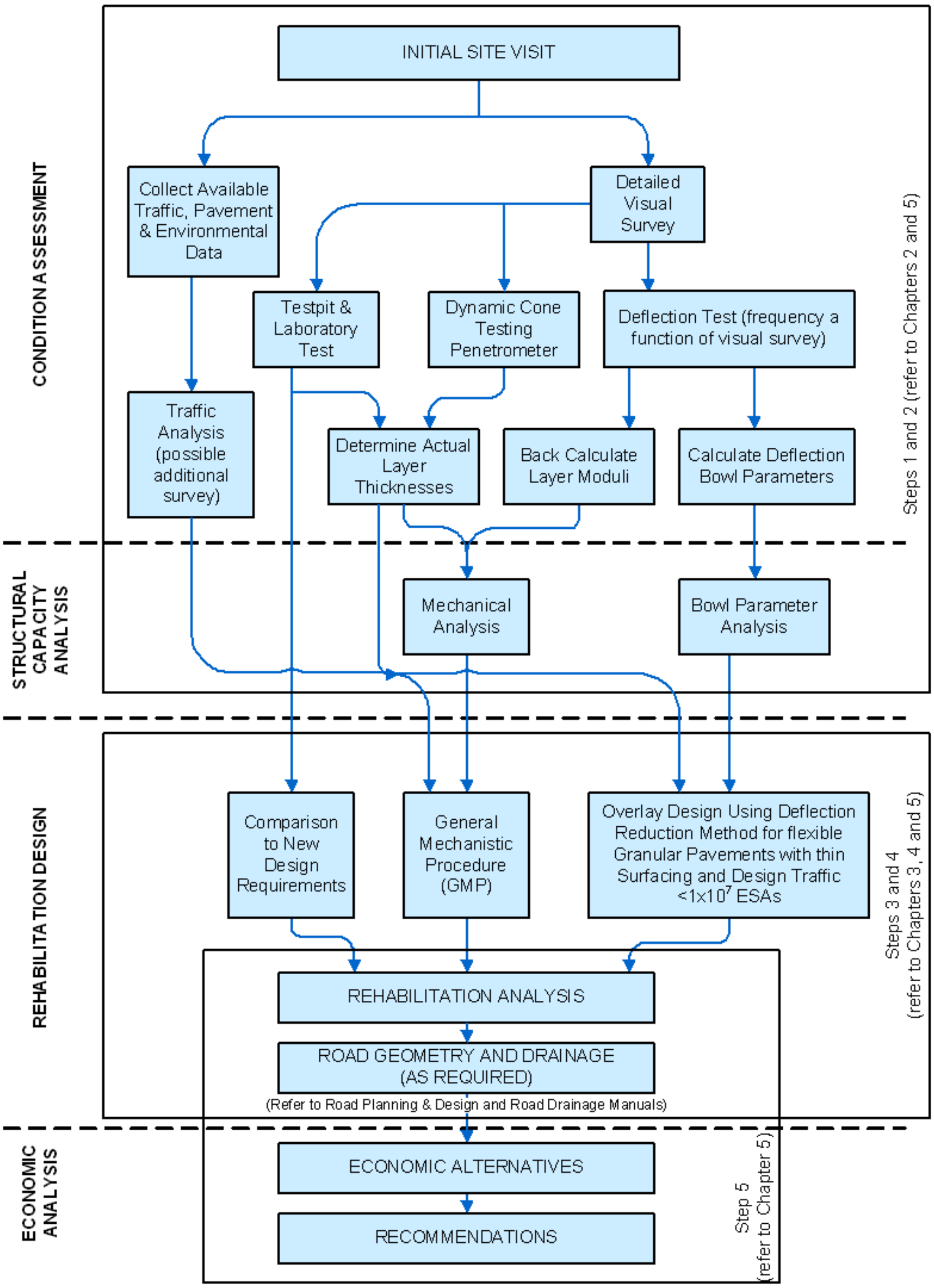
1.11 Asset management and asset maintenance

Though indicators of the need for maintenance and rehabilitation may be common, and maintenance may be required pending rehabilitation of the existing pavement being completed, this *Manual* does not specifically address maintenance issues. A rehabilitation technique may, however, still be applicable on a limited scale to a maintenance task.

1.12 Pavement rehabilitation system

The pavement rehabilitation system on which this *Manual* is based consists of five steps, which are depicted in Figure 1.12.

Figure 1.12 – Pavement rehabilitation system



1.12.1 Step 1: Determine purpose and scope

The purpose of the rehabilitation investigation is determined (refer to Section 1.13). To do this, questions must be asked and answered to identify the scope and purpose, and therefore the subsequent direction, of the investigation and design process. Such questions may be:

- Is work on the pavement considered necessary because of its forecast condition?
- Is work on the pavement considered necessary because of its current condition?
- Have increasing traffic volumes necessitated an upgrade of the road's structural capacity?

1.12.2 Step 2: Evaluate existing pavement and subgrade

The pavement's present condition is evaluated, as is the subgrade's condition. This is described in Chapter 2. Chapter 5 describes the overarching methodology to be used when completing Step 2 and Step 4.

It involves firstly collecting data from several sources, including historical records, routine or specific condition assessments, and testing programs. These data are then interpreted to characterise the existing pavement (and its constituent materials) so representative sections can be defined, pavement distress mechanisms can be identified, the condition and serviceability of pavements assessed, and the structural capacity of pavements evaluated. The evaluation process needs to be carried out irrespective of whether the purpose of rehabilitation is to restore the pavement to an acceptable condition or to upgrade the structural capacity of the pavement. In all cases, the present pavement condition is relevant.

1.12.3 Step 3: Develop alternative strategies

A number of alternative rehabilitation strategies and options are developed. Chapter 3 describes this process. In developing strategies, the identified purpose, the pavement type and configuration, the distress mechanism(s), history of maintenance expenditure and relevant design and construction considerations are considered to determine which of the wide variety of available rehabilitation treatments are appropriate. Departmental Asset Managers should be consulted to obtain some, or all, of this information and determine their views about possible strategies.

1.12.4 Step 4: Design treatments

The technical details of the alternative strategies are determined, the design method(s) chosen. and the treatments designed. Chapters 4 and 5 must be used to do this. The general range of treatments is covered in Chapter 4 while a more specific overlay design procedure is presented in Chapter 5. Chapter 5 also describes the GMP and the overarching methodology to be used when completing Step 2 and Step 4.

1.12.5 Step 5: Select option(s) and make recommendations

The options and strategies developed are compared and the optimal solution(s) selected. The basis of the comparison presented in Chapter 6 is primarily economic. Design and construction considerations are considered in Step 3.

1.13 Purpose of rehabilitation

The first step in carrying out a pavement rehabilitation investigation is to determine the purpose of the exercise. This may involve rectifying a pavement that has reached a state where, for the safety of road users, its rehabilitation cannot be delayed. Alternatively, it may deal with a situation where the natural growth in traffic has necessitated an upgrade of the pavement's structural capacity, width and/or alignment.

1.13.1 Synergies and opportunities

Designers must be cognisant of how the road corridor, as a whole, is managed. Further, they must be aware of the synergies or opportunities presented by works being undertaken, now or in the future, whatever the reason for them.

Where a pavement is to be rehabilitated primarily because of its current or forecast condition, consideration should also be paid to its cross-section, crossfall, superelevation (if applicable), drainage, shape and alignment. It may be possible to upgrade / correct some, or all, of these characteristics concurrently for only a marginal increase in cost; for example, where an existing substandard base gravel is to be stabilised, the marginal cost of widening the pavement at the same time would be significantly less than carrying out the widening at some later date.

Conversely, where action is required for reasons other than those related to the pavement, the pavement must still be evaluated. This will provide information about whether the road pavement can function in its present condition for the next design period and, if not, provide information enabling interventions to be designed. The evaluation will also identify any non-critical distress mechanisms that, if addressed, can mitigate or prevent premature failure of the pavement. In a manner similar to described previously, if works are being carried out for reasons other than those related to the pavement (for example, widening to improve safety, shape correction to correct the application of superelevation, minor realignment), then it may be an opportune time to investigate the pavement, and rehabilitate it if required. Doing this at the same time as other works may mean the pavement can be rehabilitated at a relatively marginal increase in cost.

1.14 Terms

Table 1.14 provides additional terms, definitions, abbreviations and acronyms used in this *Manual*.

Table 1.14 – Terms, definitions, abbreviations and acronyms and their meaning.

Term	Meaning
AAPA	Australian Asphalt Pavement Association https://www.aapa.asn.au/
AASHTO	American Association of State Highway and Transportation Officials https://www.transportation.org/
AC	Asphaltic concrete https://en.wikipedia.org/wiki/Asphaltic_concrete
AC7	Dense graded asphalt with a maximum nominal aggregate size of 7mm compliant with MRS30 and MRTS30, as relevant
AC10	Dense graded asphalt with a maximum nominal aggregate size of 10mm compliant with MRS30 and MRTS30, as relevant
AC14	Dense graded asphalt with a maximum nominal aggregate size of 14mm compliant with MRS30 and MRTS30, as relevant.
AC20	Dense graded asphalt with a maximum nominal aggregate size of 20mm compliant with MRS30 and MRTS30, as relevant
AEP	Annual exceedance probability http://www.bom.gov.au/water/awid/id-703.shtml
Agricultural Lime	This term refers to ground or pulverised limestone ($CaCO_3$). This has no engineering application but can be used for agricultural purposes to reduce soil acidity. https://en.wikipedia.org/wiki/Agricultural_lime
AMC	Asphalt master curve
ARMIS	A Road Management Information System. A departmental database system of both condition and inventory information relating to the road infrastructure managed by the department.
ARRB	Australian Road Research Board https://www.arrb.com.au/

Term	Meaning
AS	Australian Standard www.standards.org.au
ASTM	American Standard Test Method http://www.astm.org/Standard/index.html
Austroads	The association of Australasian road transport and traffic agencies https://www.austroads.com.au/
Back analysis	Estimation of pavement layer properties, commonly the vertical elastic moduli, based on the shape and magnitude of the deflection bowl. Back analysis (also referred to as back calculation) software attempts to match the observed or measured deflection bowl with a theoretical bowl drawn from many pavement configurations. Typically, a forward calculation process based on a linear elastic pavement model is iterated with different layer properties until the difference between the theoretical bowl and the observed bowl are within a specified tolerance. In theory, it should be possible to vary both the layer modulus and thickness to arrive at a solution; however, because of the limited number of deflection values within a bowl, and to reduce the run time, it is often assumed the layer thicknesses and other properties are known. Where the forward calculation is time-consuming, as in the case of a finite element method model, a catalogue of deflections bowls can be generated over a long period of time to provide an alternative to the iterative process.
BCR	Benefit-cost ratio
BSP	British Standard Pendulum, a commonly used device for measuring skid resistance
C170	Class 170 bitumen compliant with MRTS17
C320	Class 320 bitumen compliant with MRTS17
C600	Class 600 bitumen compliant with MRTS17
CBR	California Bearing Ratio https://en.wikipedia.org/wiki/California_bearing_ratio
Cement	Cement compliant with the relevant MRS or MRTS (for example, MRTS07B)
Cementitious blend	A cementitious blend compliant with the relevant MRS or MRTS (for example, MRTS07B)
CF	Curvature function, derived from surface deflections
CIP	*Cycling Infrastructure Policy* https://www.tmr.qld.gov.au/Travel-and-transport/Cycling/Cyclists
CIRCLY	Mechanistic-empirical pavement design software from MINCAD Systems https://pavement-science.com.au/softover/circly/
CTB	Cementitiously treated base, an upper layer of the pavement treated with either lime, cement or both, and often in combination with other additives including fly ash
CTMeter	Circular track meter, a portable device used to measure pavement surface texture (Test Method ASTM E2157) http://www.nippou.com/en/products/ct.html
CTSB	Cementitiously treated sub-base
CV	Coefficient of variation https://en.wikipedia.org/wiki/Coefficient_of_variation

Term	Meaning
D	Surface deflection normally assumed to be D_0. Surface deflections are measured by various devices. Typical devices in Queensland are a falling weight deflectometer, a heavy falling weight deflectometer, a Benkelman Beam, a traffic speed deflectometer and deflectograph
D_0	Surface deflection measured at the point of maximum deflection (0mm offset)
D_{900}	Surface deflection measured at an offset of 900mm
D_r	Representative D
DCP	Dynamic cone penetrometer; testing must comply with Transport and Main Roads Test Method Q114B
Deflection bowl	A representation of the shape of the elastic deformation of the pavement surface when a load is applied
Deflectograph	A deflectograph is an automated deflection measuring system based on the deflection beam principle. A deflectograph measures the deflection of the road surface under the action of a rolling wheel travelling at between 3–4 km/h at standard loading conditions to assess the structural condition of pavements.
Degree of Anisotropy	The ratio of vertical over horizontal modulus ($\frac{E_v}{E_h}$) of a material, where the modulus is defined as the ratio of the stress over the strain ($\frac{\sigma}{\varepsilon}$). Austroads adopts a Degree of Anisotropy of 1 for bound materials and 2 for unbound granular materials; however, foreign road agencies and companies often adopt a value of 1 for all materials, bound or otherwise, and adjustments (AGPT05) need to be made to results obtained from design software obtained from these sources.
DESAs	Design equivalent standard axles
DGA	Dense graded asphalt compliant with MRS30 and MRTS30 as relevant
Dielectric constant	The measure of resistance encountered when forming an electric field in a specific medium compared to a vacuum; also referred to as the relative permittivity https://en.wikipedia.org/wiki/Relative_permittivity#Terminology
Discount rate	As for real discount rate
DMAF	Deflection moisture adjustment factor
DOS	Degree of saturation, the proportion of voids in a pavement material filled with water, typically expressed as a percentage.
DR	Deflection ratio, a parameter, D_{250}/D_0, derived from surface deflections
DR_r	Representative DR
DVR	Digital Video Road, a departmental system that records and allows viewing of images versus the chainage and global navigation satellite system coordinates of road carriageways captured on roads managed by the department.
EfromD3	A software package used to back analyse the pavement using deflection results, it was developed and distributed by the Australian Road Research Board (ARRB). It uses CIRCLY and requires the user to have a licensed copy of CIRCLY installed.

Term	Meaning
ESA	Equivalent standard axle described at https://globalroadtechnology.com/equivalent-standard-axle-first-principles/
EVA	Ethylene vinyl acetate, a polymer used to modify bitumen, which approaches elastomeric materials in softness and flexibility yet can be processed like other thermoplastics https://en.wikipedia.org/wiki/Ethylene-vinyl_acetate
Expansion ratio (for foamed bitumen)	The ratio of the maximum volume of foamed bitumen to the volume of unfoamed bitumen
FEM	Finite element method, a numerical method for solving partial differential equations https://en.wikipedia.org/wiki/Finite_element_method
Fly ash	One of the residues generated by coal combustion, composed of the fine particles driven out of the boiler with the flue gases, it is commonly added during the construction of cement treated pavement layers to slow the rate of curing, extend the working time and reduce the likelihood of block cracking https://en.wikipedia.org/wiki/Fly_ash
Foamed bitumen	Bitumen, typically with a foaming agent added, foamed by the addition of water
Foamed bitumen stabilisation	Stabilisation using at least foamed bitumen as a binder; for departmental works, this normally requires the use of a primary binder, the foamed bitumen, and a secondary binder, typically lime
Foaming agent (for foamed bitumen)	A chemical agent mixed into the bitumen to enhance its foaming characteristics, such an additive would typically enhance the expansion ratio and/or the half-life of the foamed bitumen
Foaming water content (for foamed bitumen)	The percentage of water added to hot bitumen to induce foaming
Forward calculation	Calculation of the response of a pavement model to a load model. A linear elastic pavement model typically includes layer properties such as thickness, vertical elastic modulus, Degree of Anisotropy, Poisson's Ratio, and shear modulus. Where a FEM model is used, several additional parameters may be required including bulk density, residual compaction stress, cohesion, internal friction angle and several associated with the stress dependency model for unbound granular material. CIRCLY is an example of software that performs the forward calculation.
FWD	Falling weight deflectometer https://en.wikipedia.org/wiki/Falling_weight_deflectometer
GAR	Geotextile absorption rate, used for geotextile SAMs or SAMIs
GB cement	Type GB (general blend) cement compliant with AS 3972
General mechanistic procedure	A pavement design procedure for non-rigid pavements, it is based on mechanistic design principles which consider (some) pavement material properties and behaviours. The procedure is as described in AGPT Parts 2 and 5 but as modified by this *Manual* (Chapter 5) and the PDS.
Geotextile	Unless otherwise described, in this *Manual,* 'geotextile' means a paving geotextile compliant with MRS57 and MRTS57 https://en.wikipedia.org/wiki/Geotextile

Term	Meaning
GIMS	Geospatial information management system, a departmental system of electronic copies of many of the department's registered plans of roads managed or formerly managed by Transport and Main Roads (or by Queensland Transport)
GIS	Geographic information system, a computer application designed to capture, store, manipulate, analyse, manage, and present all types of spatial or geographical data https://en.wikipedia.org/wiki/GIS
GMP	General mechanistic procedure
GNSS	Global navigation satellite system, which records the location using geographic coordinates of latitude, longitude and elevation. Proprietary systems include the US NAVSTAR global positioning system and Russian global navigation satellite system. https://en.wikipedia.org/wiki/Satellite_navigation
GP cement	Type GP (general purpose) cement compliant with AS 3972
GPR	Ground penetrating radar, a technology that transmits electromagnetic signals into the ground and receives reflections from subsurface features. The results can be interpreted by experts to provide information about the pavement. https://en.wikipedia.org/wiki/Ground-penetrating_radar
Half-life (for foamed bitumen)	The time taken for the maximum foamed volume of bitumen to settle to half this volume
HIPAR	Hot in-place asphalt recycling
HWD	Heavy falling weight deflectometer, a heavy-duty version of the FWD, intended for measurement of deflections on pavements containing significant depths of bound material. Capable of delivering greater than a 100 kN load to the pavement. Typically used for industrial hard stand areas and airport runways.
Hydrated lime	Calcium hydroxide ($Ca(OH)_2$), also known as hydrated or slaked lime, is produced by slaking or hydrating quicklime by adding water. This process can take place immediately following quicklime manufacture at the lime kiln, onsite using conventional water trucks, or in purpose-built mixing tanks. Hydrated lime supplied from the plant is typically a very fine, dry powder. By popular connotation, hydrated lime is a dry powdered hydrate, whereas slaking involves more water, producing wet hydrates. https://en.wikipedia.org/wiki/Calcium_hydroxide
IRI	International roughness index, a measure of road roughness https://en.wikipedia.org/wiki/International_Roughness_Index
IWP	Inner wheel path in a lane not adjacent to a pavement edge and therefore less likely to be affected by roadside moisture infiltration — commonly the wheel path to which deflection moisture adjustment is applied
kml	Keyhole Markup Language https://en.wikipedia.org/wiki/Keyhole_Markup_Language
.kmz	Commonly used GIS file format that is described adequately at https://en.wikipedia.org/wiki/Keyhole_Markup_Language
Lime stabilisation optimum	Optimum lime content determined through pH and UCS testing – an additional 1% lime should be added in the field to allow for variations caused by losses and uneven mixing

Term	Meaning
Limestone	A general term for carbonate-rich natural deposits composed primarily of calcium carbonate ($CaCO_3$), with varying amounts of impurities https://en.wikipedia.org/wiki/Limestone
LSO	Lime stabilisation optimum
LTS	Laser texture scanner, a portable device used to measure pavement surface texture https://amesengineering.com/all-products/
May	Indicates the existence of an option; any option applied must be based on sound engineering judgement and documented
MATTA	MATerial Testing Apparatus, laboratory apparatus that can test for, amongst other things, resilient modulus and creep
MCA	Multi-criteria analysis https://en.wikipedia.org/wiki/Heuristic_analysis
MRS	Transport and Main Roads Specification (Measurement); for example, MRS05 refers to departmental Specification (Measurement) MRS05
MRTS	Transport and Main Roads Technical Specification; for example, MRTS05 refers to departmental Technical Specification MRTS05
NAASRA	National Association of Australian Road Authorities, it has been replaced by Austroads
NPV	Net present value, an absolute measure equal to discounted benefits (user + non-users) project less discounted costs, all calculated over the life of the project https://en.wikipedia.org/wiki/Net_present_value
OG14	OGA with a maximum nominal aggregate size of 14mm compliant with MRS30 and MRTS30 as relevant
OGA	Open graded asphalt compliant with MRS30 and MRTS30 as relevant
OMC	Optimum moisture content, the moisture content of a soil at which a specified amount of compaction will produce the maximum dry density under specified test conditions
OnQ	The departmental project management framework, it includes policies, principles, project governance, methodology, templates, tools and project support
OWP	Outer wheel path, the wheel path in a lane closest to the edge of the pavement and more susceptible to moisture infiltration from the roadside than IWPs and deflection moisture adjustment is not applied. Appropriately constructed shoulders and side drains can be used to reduce such infiltration.
PAFV	Polished aggregate friction value, a measure, on a scale of 0–100, of the resistance of an aggregate to polishing under the action of traffic as determined in standard laboratory tests
Pavement maintenance	Routine maintenance tasks, with emphasis on ad hoc activities (not pre-programmed) such as pothole filling and 'holding actions'
Pavement rehabilitation	Any activity that improves the functional or structural condition of a pavement while using some or all of its existing structure
PBD	Polybutadiene, a polymer used to modify bitumen which is a synthetic rubber also used in the manufacture of tyres https://en.wikipedia.org/wiki/Polybutadiene

Term	Meaning
PI	Plasticity index, the size of the range of water contents where the soil exhibits plastic (non-resilient or inelastic) properties https://en.wikipedia.org/wiki/Atterberg_limits
PMB	Polymer modified binder; in this *Manual*, when PMB is used, it refers to a binder compliant with MRTS18
Pozzolan	A pozzolan is a finely-divided siliceous and/or aluminous material that, in the presence of water and calcium hydroxide, will form a cemented product. The cemented products are calcium-based hydrates, which are essentially the same hydrates that form during the hydration of Portland cement. https://en.wikipedia.org/wiki/Pozzolan
PWOC	Present worth of costs, future costs discounted to present costs using the real discount rate
PWT	Work Tips, Austroads and Australian Asphalt Pavement Association (AAPA) https://www.aapa.asn.au/technology-publications/work-tips/
Quicklime	Quicklime is principally comprised of calcium oxide (CaO) and is typically supplied as a granular powder. Quicklime hydrates readily when in contact with water to form calcium hydroxide ($Ca(OH)_2$) or hydrated lime. The hydration process results in a rapid release of heat and water vapour / steam. Heating $CaCO_3$ at elevated temperatures produces calcium oxide (CaO), also known as burnt lime. The reaction here is: $CaCO_3$ + heat (>1000°C) gives CaO + CO_2. https://en.wikipedia.org/wiki/Calcium_Oxide
R	R statistical computing language, a language and environment for statistical computing and graphics which provides a wide variety of statistical (linear and non-linear modelling, classical statistical tests, time-series analysis, classification, clustering, and so on) and graphical techniques, and is highly extensible. One of R's strengths is the ease with which well-designed, publication-quality plots can be produced, including mathematical symbols and formulae where needed. R is available as free software under the terms of the Free Software Foundation's GNU General Public License in source code form. It complies with and runs on a wide variety of platforms including Windows. https://www.r-project.org/
RAP	Reclaimed asphalt pavement
Real discount rate	The rate used to discount future costs to costs in present-day terms, this rate is expressed in real terms (it excludes inflation). For departmental projects, the discount rate should be obtained from Queensland Treasury before the analysis begins.
Reclaimer / Stabiliser	A single-rotor mix-in-place plant of a type (plant that mixes insitu) specifically designed for the dual task of reclamation and stabilisation work
Rehabilitation	As for pavement rehabilitation
RMPC	Road Maintenance Performance Contract https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Road-maintenance-performance-contract-manual

Term	Meaning
RMS	Roads and Maritime Services was an agency of the New South Wales Government responsible for building and maintaining road infrastructure and managing the day-to-day compliance and safety for roads and waterways. In April 2019, it was revealed that the agency is to cease to exist with all of its functions transferred to Transport for NSW. Legislation was passed in the NSW Parliament in November 2019, and it was dissolved on 1 December 2019. https://en.wikipedia.org/wiki/Roads_and_Maritime_Services
RPDM	The department's Supplement to Austroads' *Guide to Road Design*, the *Road Planning and Design Manual*
RTA	Roads and Traffic Authority of the State of New South Wales, which became the New South Wales Department of Roads and Maritime Services (RMS) and subsequently Transport for New South Wales
SAM	Strain alleviating membrane
SAMI	Strain alleviating absorbing membrane interlayer
SARA	Standard axle repetitions for asphalt
SASW	Spectral analysis of surface waves, a possible alternative to deflection testing on rigid pavements. Inversion (similar to back-calculation) of measurements allows layer moduli to be estimated and the thickness of at least the first layer to be identified. https://en.wikipedia.org/wiki/Exploration_geophysics#Spectral-Analysis-of-Surface-Waves https://en.wikipedia.org/wiki/Surface_wave_inversion
SBR	Styrene-butadiene rubber, a polymer used to modify bitumen
SBS	Styrene-butadiene-styrene, a polymer used to produce elastomeric bitumen
SCA	Statistical cluster analysis or clustering is the task of grouping a set of objects in such a way objects in the same group (called a cluster) are more similar (in some sense or another) to each other than to those in other groups (clusters). It is a main task of exploratory data mining, and a common technique for statistical data analysis, used in many fields, including machine learning, pattern recognition, image analysis, information retrieval, and bioinformatics. https://en.wikipedia.org/wiki/Cluster_analysis It is currently used as a tool within pavement rehabilitation to separate deflection bowls into groups representing portions of the pavement with similar structural properties. The metric includes components comparing both the magnitude and shape of the bowl.
SEM	Standard error of the mean https://en.wikipedia.org/wiki/Standard_error#Standard_error_of_the_mean
Shall	Indicates a statement is mandatory
Should	Indicates a recommendation. Where the word ‘should’ is used, it is considered to be recommended usage, but not mandatory. Any recommendation not applied must be based on sound engineering judgement and documented.
Slaking (in context of stabilisation)	The addition of water to fully hydrate a material
SMA	Stone mastic asphaltic concrete
SPTD	Sand patch texture depth

Term	Meaning
SSD	Sample standard deviation, commonly referred to as *s*, where $$s = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}}$$ https://en.wikipedia.org/wiki/Standard_deviation#Sample_standard_deviation
SSDCF	Sample standard deviation correction factor, an unbiased estimate of the standard deviation **σ** of the population obtained by multiplying the SSD, *s*, by SSDCF, **σ**. The SSDCF is the reciprocal of the c4 factor described in https://en.wikipedia.org/wiki/Unbiased_estimation_of_standard_deviation
Stabiliser	A single-rotor mix-in-place plant (plant that mixes insitu) of a type specifically designed for stabilisation work
TBAR	Total bitumen application rate used for geotextile SAMs or SAMIs
TSD	Traffic speed deflectometer, a rolling wheel deflectometer for network-level bearing capacity measurement which uses patented Doppler technology to measure pavement deflection while travelling at normal traffic speed – up to 80 km/h
UCS	Unconfined compressive strength
VMP	Vehicle mounted profilometer, equipment approved for use on state-controlled roads which typically employs accelerometers in combination with fixed and scanning lasers to measure roughness, texture and rut depth. Inclinometers and gyroscopes are used to report road geometry including gradient, crossfall, horizontal and vertical curvature. DVR-compliant video is also recorded typically using at least four cameras (forward, rear, left and right views). The linear reference or chainage at which measurements are recorded are commonly derived from a rotary encoder attached to the driveshaft of the vehicle. An additional locational reference is obtained from GNSS equipment which records the latitude, longitude and elevation of the results.
VR	Visual rating
WH&S	Work, health and safety
WMAPT	Weighted mean annual average pavement temperature
WOLC	Whole-of-life costs, all ownership and user costs necessary to provide a serviceable pavement over the assessment period

1.15 Referenced documents

Table 1.15 – Referenced documents

Reference	Title
AGAM15	*Guide to Asset Management Technical Information Part 15: Technical Supplements*, Austroads https://austroads.com.au/publications/asset-management/agam15
AGAMT006	*Test Method AGAM-T006 Pavement Deflection Measurement with a Falling Weight Deflectometer (FWD)*, Austroads https://austroads.com.au/publications/asset-management/agam-t006-11
AGPT01	*Guide to Pavement Technology Part 1: Introduction to Pavement Technology*, Austroads https://austroads.com.au/publications/pavement/agpt01

Reference	Title
AGPT02	*Guide to Pavement Technology Part 2: Pavement Structural Design*, Austroads https://austroads.com.au/publications/pavement/agpt02
AGPT03	*Guide to Pavement Technology Part 3: Pavement Surfacings*, Austroads https://austroads.com.au/publications/pavement/agpt03
AGPT04A	*Guide to Pavement Technology Part 4A: Granular Base and Subbase Materials*, Austroads https://austroads.com.au/publications/pavement/agpt04a
AGPT04B	*Guide to Pavement Technology Part 4B: Asphalt*, Austroads https://austroads.com.au/publications/pavement/agpt04b
AGPT04C	*Guide to Pavement Technology Part 4C: Materials for Concrete Road Pavements*, Austroads https://austroads.com.au/publications/pavement/agpt04c
AGPT04D	*Guide to Pavement Technology Part 4D: Stabilised Materials*, Austroads https://austroads.com.au/publications/pavement/agpt04d
AGPT04F	*Guide to Pavement Technology Part 4F: Bituminous Binders*, Austroads https://austroads.com.au/publications/pavement/agpt04f
AGPT04G	*Guide to Pavement Technology Part 4G: Geotextiles and Geogrids*, Austroads https://austroads.com.au/publications/pavement/agpt04g
AGPT04K	*Guide to Pavement Technology Part 4K: Selection and Design of Sprayed Seals*, Austroads https://austroads.com.au/publications/pavement/agpt04k
AGPT04L	*Guide to Pavement Technology Part 4L: Stabilising Binder*, Austroads https://austroads.com.au/publications/pavement/agpt04l
AGPT05	*Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design*, Austroads https://austroads.com.au/publications/pavement/agpt05
AGPT06	*Guide to Pavement Technology Part 6: Unsealed Pavements*, Austroads https://austroads.com.au/publications/pavement/agpt06
AGPT07	*Guide to Pavement Technology Part 7: Pavement Maintenance*, Austroads https://austroads.com.au/publications/pavement/agpt07
AGPT08	*Guide to Pavement Technology Part 8: Pavement Construction*, Austroads https://austroads.com.au/publications/pavement/agpt08
AGPT10	*Guide to Pavement Technology Part 10: Subsurface Drainage*, Austroads https://austroads.com.au/publications/pavement/agpt10
AGPT-T250	*Modified Surface Texture Depth (Pestle Method)*, Austroads https://austroads.com.au/publications/pavement/agpt-t250-08
AGPT-T301	*Determining the Foaming Characteristics of Bitumen,* Austroads https://austroads.com.au/publications/pavement/agpt-t301-17
AGRD03	*Guide to Road Design Part 3: Geometric Design*, Austroads https://austroads.com.au/publications/road-design/agrd03

Reference	Title
AGRD05A	*Guide to Road Design Part 5A: Drainage – Road Surface, Networks, Basins and Subsurface*, Austroads https://austroads.com.au/publications/road-design/agrd05a
AP-C87	*Glossary of Terms*, Austroads https://austroads.com.au/publications/corporate-reports-and-plans/ap-c87-15
AP-PWT00	*Linked Work Tips Index*, Austroads https://austroads.com.au/publications/pavement/ap-pwt00-18
AP-R203	*Economic Evaluation of Road Investment Proposals: Risk Analysis*, AP-R203-02, Austroads https://austroads.com.au/publications/economics-and-financing/ap-r203-02
AP-R374	*Guidance for the Development of Policy to Manage Skid Resistance*, AP-R374-11, Austroads https://austroads.com.au/publications/asset-management/ap-r374-11
AP-R511	*Improved Design Procedures for Asphalt Pavements*, AP-R511-16, Austroads https://austroads.com.au/publications/pavement/ap-r511-16
AP-R569	*Guidelines and Specifications for Microsurfacing*, AP R569-18, Austroads https://austroads.com.au/publications/pavement/ap-r569-18
AP-T68	*Update of the Austroads Sprayed Seal Design Method*, Austroads https://austroads.com.au/publications/pavement/ap-t68-06
AP-T177	*Review of Skid Resistance and Measurement Methods*, AP-T177-11, Austroads https://austroads.com.au/publications/asset-management/ap-t177-11
AP-T235	*Guide to the Selection and use of Polymer Modified Binders and Multigrade Bitumens*, Austroads https://austroads.com.au/publications/pavement/ap-t235-13
AP-T236	*Update of Double / Double Design for Austroads Sprayed Seal Design Methods*, AP-TR236-13, Austroads https://austroads.com.au/publications/pavement/ap-t236-13
AP-T248	*Improved Design Procedures for Asphalt Pavements: Pavement Temperature and Load Frequency Estimation*, AP-T248-13, Austroads https://austroads.com.au/publications/pavement/ap-t248-13
AS 3705	*Geotextiles – Identification, marking and general data*, AS 3705:2012, Standards Australia www.standards.org.au
AS 3706.0	*Geotextiles – Methods of test – General introduction and list of methods*, AS 3706.0:2013, Standards Australia www.standards.org.au
AS 3706.1	*Geotextiles – Methods of test – General requirements, sampling, conditioning, basic physical properties and statistical analysis*, AS 3706.1:2012, Standards Australia www.standards.org.au
AS 3706.2	*Geotextiles – Methods of test – Determination of tensile properties – Wide strip method*, AS 3706.2:2012, Standards Australia www.standards.org.au

Reference	Title
AS 3706.3	*Geotextiles – Methods of test – Determination of tearing strength – Trapezoidal method*, AS 3706.3:2012, Standards Australia www.standards.org.au
AS 3706.4	*Geotextiles – Methods of test – Determination of burst strength – California bearing ratio (CBR) – Plunger method*, AS 3706.4:2012, Standards Australia www.standards.org.au
AS 3706.5	*Geotextiles – Methods of test – Method 5: Determination of puncture resistance – Drop cone method*, AS 3706.5:2014, Standards Australia www.standards.org.au
AS 3706.6	*Geotextiles – Methods of test – Determination of seam strength*, AS 3706.6:2012, Standards Australia www.standards.org.au
AS 3706.7	*Geotextiles – Methods of test – Method 7: Determination of pore-size distribution – Dry-sieving method*, AS 3706.7:2014, Standards Australia www.standards.org.au
AS 3706.9	*Geotextiles – Methods of test – Determination of permittivity, permeability and flow rate*, AS 3706.9:2012, Standards Australia www.standards.org.au
AS 3706.10.1	*Geotextiles – Methods of test – Determination of transmissivity – Radial method*, AS 3706.10.1:2012, Standards Australia www.standards.org.au
AS 3706.11	*Geotextiles – Methods of test – Determination of durability – Resistance to degradation by light, heat and moisture*, AS 3706.11:2012, Standards Australia
AS 3706.12	*Geotextiles – Methods of test, Method 12: Determination of durability – Resistance to degradation by hydrocarbons or chemical reagents*, AS 3706.12:2008, Standards Australia www.standards.org.au
AS 3706.13	*Geotextiles – Methods of test – Determination of durability – Resistance to certain microbiological agents*, AS 3706.13:2012, Standards Australia www.standards.org.au
AS 3972	*General purpose and blended cements*, AS 3272:2010, Standards Australia www.standards.org.au
ASTM D276	*Standard Test Methods for Identification of Fibers in Textiles*, ASTM D276-12, ASTM International https://www.astm.org/Standards/D276.htm
ASTM D6140	*Standard Test Method to Determine Asphalt Retention of Paving Fabrics Used in Asphalt Paving for Full-Width Applications*, ASTM D6140-00(2014) ASTM International https://www.astm.org/Standards/D6140.htm
ASTM E2157	*Standard Test Method for Measuring Pavement Macrotexture Properties Using the Circular Track Meter*, ASTM E2157-15, ASTM International https://www.astm.org/Standards/E2157.htm
ATAPG	*Australian Transport Assessment and Planning Guidelines* https://atap.gov.au/

Reference	Title
CTLD	*Class-Specific Traffic Load Distributions*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Pavement-design-supplement
DFBSP	*Design, construction and performance of insitu foamed bitumen stabilised pavements*, J Ramanujam, 2009, Queensland Department of Main Roads http://librarycat/docs/QldRoads/2009mar/QR_Mar09_dcapoifbsp.pdf (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
MRS05	Specification (Measurement) MRS05 *Unbound Pavements*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS07A	Specification (Measurement) MRS07A *Insitu Stabilised Subgrades Using Quicklime or Hydrated Lime*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS07B	Specification (Measurement) MRTS07B *Insitu Stabilised Pavements Using Cement or Cementitious Blends*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS07C	Specification (Measurement) MRS07C *Insitu Stabilised Pavements Using Foam Bitumen*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS11	Specification (Measurement) MRS11 *Sprayed Bituminous Surfacing (Excluding Emulsion)*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS12	Specification (Measurement) MRS12 *Sprayed Bituminous Emulsion Surfacing*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS13	Specification (Measurement) MRS13 *Bituminous Slurry Surfacing,* Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS22	Specification (Measurement) MRS22 *Supply of Cover Aggregate*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS30	Specification (Measurement) MRS30 *Asphalt Pavements*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing

Reference	Title
MRS39	Specification (Measurement) MRS39 *Lean Mix Concrete Sub-base for Pavements*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS40	Specification (Measurement) MRS40 *Concrete Pavement Base*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS57	Specification (Measurement) MRS57 *Geotextiles for Paving Applications*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS58	Specification (Measurement) MRS58 *Subgrade Reinforcement using Pavement Geosynthetics*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRS70	Specification (Measurement) MRS70 *Concrete*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/2-Bridges-Marine-and-Structures
MRS71	Specification (Measurement) MRS71 *Reinforcing Steel*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/2-Bridges-Marine-and-Structures
MRS104	Specification (Measurement) MRTS104 *Retarding Pavement Reflective Cracking using Asphalt Geosynthetics*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS04	Technical Specification MRTS04 *General Earthworks*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/3-Roadworks-Drainage-Culverts-and-Geotechnical
MRTS05	Technical Specification MRTS05 *Unbound Pavements*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS07A	Technical Specification MRTS07A *Insitu Stabilised Subgrades Using Quicklime or Hydrated Lime*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS07B	Technical Specification MRTS07B *Insitu Stabilised Pavements Using Cement or Cementitious Blends*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing

Reference	Title
MRTS07C	Technical Specification MRTS07C *Insitu Stabilised Pavements using Foamed Bitumen*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS08	Technical Specification MRTS08 *Plant-Mixed Heavily Bound (Cemented) Pavements*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS09	Technical Specification MRTS09 *Plant-Mixed Pavement Layers Stabilised Using Foamed Bitumen*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS11	Technical Specification MRTS11 *Sprayed Bituminous Surfacing (Excluding Emulsion)*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS12	Technical Specification MRTS12 *Sprayed Bituminous Emulsion Surfacing*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS13	Technical Specification MRTS13 *Bituminous Slurry Surfacing*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS17	Technical Specification MRTS17 *Bitumen and Multigrade Bitumen*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS18	Technical Specification MRTS18 *Polymer Modified Binder (including Crumb Rubber)*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS19	Technical Specification MRTS19 *Cutter and Flux Oils*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS20	Technical Specification MRTS20 *Cutback Bitumen*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS21	Technical Specification MRTS21 *Bituminous Emulsion*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS22	Technical Specification MRTS22 *Supply of Cover Aggregate*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing

Reference	Title
MRTS23	Technical Specification MRTS23 *Supply and Delivery of Quicklime and Hydrated Lime for Road Stabilisation*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS30	Technical Specification MRTS30 *Asphalt Pavements*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS39	Technical Specification MRTS39 *Lean Mix Concrete Sub-base for Pavements*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS40	Technical Specification MRTS40 *Concrete Pavement Base*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS42	Technical Specification MRTS42 *Supply of Wax Emulsion Curing Compound for Concrete*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS57	Technical Specification MRTS57 *Geotextiles for Paving Applications*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS58	Technical Specification MRTS58 *Subgrade Reinforcement using Pavement Geosynthetics*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS70	Technical Specification MRTS70 *Concrete*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/2-Bridges-Marine-and-Structures
MRTS71	Technical Specification MRTS71 *Reinforcing Steel*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/2-Bridges-Marine-and-Structures
MRTS101	Technical Specification MRTS101 *Aggregates for Asphalt*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing
MRTS104	Technical Specification MRTS104 *Retarding Pavement Reflective Cracking using Asphalt Geosynthetics*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Specifications/5-Pavements-Subgrade-and-Surfacing

Reference	Title
MTM	*Materials Testing Manual*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
OnQ	*OnQ Project Management Framework*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/OnQ-Project-Management-Framework
PCDCP	*Pavement Condition Data Collection Policy*, Queensland Department of Transport and Main Roads https://intranet.tmr.qld.gov.au/corp/pip/tsam/Pages/Data_Collection_Policy.aspx (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
PDS	*Pavement Design Supplement*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Pavement-design-supplement
PMPG	*Project Management Practices Guideline*, Queensland Department of Transport and Main Roads https://inside.tmr.qld.gov.au/corp/pmi/Proj-mgt/PM/Pages/PMPG.aspx (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
PPM	*Preconstruction Processes Manual*, Department of Transport and Main Roads https://www.tmr.qld.gov.au/Business-industry/Technical-standards-publications/Preconstruction-processes-manual.aspx
Q115	*Test Method Q115: Unconfined compressive strength of stabilised materials*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
Q138A	*Q138A - Preparation and compaction of laboratory mixed foamed bitumen stabilised material,*Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
Q139	*Resilient modulus of stabilised material - indirect tensile method,* Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
Q146	*Test Method Q146: Degree of saturation of soils and crushed rock*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
Q707A	*Test Method Q707A: Permeability of road surfacing and granular materials – even flow field permeameter*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual

Reference	Title
Q707B	*Test Method Q707B: Permeability of road surfacing and granular materials – rapid flow field permeameter*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
Q708B	*Test Method Q708B: Road roughness – surface evenness – two laser profilometer*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
Q708C	*Test Method Q708C: Road roughness – surface evenness – static level and staff*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
Q708D	*Test Method Q708D: Road roughness – surface evenness – ARRB walking profiler*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Materials-testing-manual
RDM	*Road Drainage Manual*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Road-drainage-manual.aspx
RMF	*Risk Management Framework*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Risk-management-framework
RMG	*Routine Maintenance Guidelines*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Routine-maintenance-guidelines
RMPCM	*Road Maintenance Performance Contract Manual*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Road-maintenance-performance-contract-manual
RMS MC	*Continuously Reinforced Concrete Pavement* – CRCP, Standard Drawings – Pavement, Volume MC, Roads and Maritime Services https://www.rms.nsw.gov.au/business-industry/partners-suppliers/document-types/standard-drawings/pavement.html
RMS MJ	*Jointed Reinforced Concrete Pavement* – JRCP, Standard Drawings – Pavement, Volume MJ, Roads and Maritime Services https://www.rms.nsw.gov.au/business-industry/partners-suppliers/document-types/standard-drawings/pavement.html
RMS MP	*Plain Concrete Pavement* – PCP, Standard Drawings – Pavement, Volume MP, Roads and Maritime Services https://www.rms.nsw.gov.au/business-industry/partners-suppliers/document-types/standard-drawings/pavement.html
RMS R83	*Concrete Pavement Base*, QA Specification R83, Roads and Maritime Services https://www.rms.nsw.gov.au/business-industry/partners-suppliers/document-types/specifications/qa/roadworks.html

Reference	Title
RP 1143	*Cold Overlay (Slurry Seal) Trials – Nerang Broadbeach Road – Gold Coast Springbrook Road*, Internal Report RP1143, Queensland Department of Transport and Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results/?qu=RP+1143&x=64&y=22 (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
RP 1200	*Crack control trial on cement treated base: Bruce Highway – Beerburrum Deviation*, Internal Report RP1200, Queensland Department of Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Crack+control+trial+on+cement+treated+base&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
RP 1256	*Beerburrum deviation crack control trial condition assessment report No. 1*, Report No. RP1256, Queensland Department of Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Beerburrum+deviation+crack+control+trial+condition+assessment+report+No&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
RP 1369	*Beerburrum deviation crack control trial condition assessment report No. 2*, Report No. RP1369, Queensland Department of Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Beerburrum+deviation+crack+control+trial&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
RP 1568	*Hot In-Place Asphalt Recycling: Bruce Highway and Redcliffe Sub-Arterial*, Internal Report RP 1568, Queensland Department of Transport and Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=%22RP+1568%22# (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
RP 2103	*Beerburrum deviation crack control trial condition assessment report No. 3*, Report No. RP2103, Queensland Department of Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Beerburrum+deviation+crack+control+trial+condition&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
RP 2374	*Beerburrum deviation crack control trial condition assessment report No. 4*, Report No. RP2374, Queensland Department of Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Beerburrum+deviation+crack+control+trial+condition+assessment+report&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)

Reference	Title
RP 2383	*Pavement performance report: Beerburrum Deviation cement treated pavements*, Report No. RP2383, Queensland Department of Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Pavement+performance+report%3A+Beerburrum+Deviation&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
RPDM	*Road Planning and Design Manual*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Road-planning-and-design-manual-2nd-edition
RPDM3	*Road Planning and Design Manual (2nd edition) Volume 3: Guide to Road Design*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Road-planning-and-design-manual-2nd-edition
RPDM3-03	*Road Planning and Design Manual (2nd edition) Volume 3 Part 3: Geometric Design*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Road-planning-and-design-manual-2nd-edition
SRMP	*Skid Resistance Management Plan*, Queensland Department of Transport and Main Roads https://inside.tmr.qld.gov.au/corp/pip/tsam/ElementManagement_Documents/SRMP.pdf (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
T192	*Test Method T192 – Determination of the texture depth of road surfacing by the TRL mini texture meter*, Roads and Maritime Services https://www.rms.nsw.gov.au/business-industry/partners-suppliers/document-types/test-methods/volume-1.html
TIAMP	*Transport Infrastructure Asset Management Policy*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Transport-Infrastructure-Asset-Management-Policy
TN143	*Technical Note TN143: Rehabilitation Technique and Treatment Prioritisation Method of Plain Concrete Pavements*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Technical-Notes/Technical-Notes-Index
TN163	*Technical Note TN163 Third party utility infrastructure installation in state-controlled roads technical guideline*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Technical-Notes/Technical-Notes-Index
TN175	*Technical Note TN175 Selection and Design of Sprayed Bituminous Treatments* https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Technical-Notes/Technical-Notes-Index

Reference	Title
Guideline	*Structural design procedure of pavements on lime stabilised subgrades guideline*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Structural-design-procedure-of-pavements-on-lime-stabilised-subgrades-guideline
Guideline	*Statistical cluster analysis of deflection data for pavement structural identification and design*, Queensland Department of Transport and Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Statistical-cluster-analysis-of-deflection-data-for-pavement-structural-identification-and-design
TNMCP	*Transport Noise Management Code of Practice, Volume 1: Road Traffic Noise. Volume 2: Construction Noise and Vibration*, Queensland Department of Transport and Main Roads (excludes Volume 3 in this *Manual)* https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Transport-noise-management-code-of-practice.aspx
WQBPG	*Western Queensland Best Practice Guidelines*, Queensland Department of Main Roads https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Western-Queensland-best-practice-guidelines

1.16 Additional references

Table 1.16 – Additional references

Reference	Title
AASHTO, 1993	*Guide for the Design of Pavement Structures 4th Edition*,1993, American Association of State Highway Transport Organisations https://store.transportation.org/Item/CollectionDetail?ID=86
AASHTO, 2015	*Mechanistic-Empirical Pavement Design Guide: A Manual of Practice, 2nd Edition*, American Association of State Highway Transport Organisations https://store.transportation.org/Item/CollectionDetail?ID=145
AASHTO M288	*Standard Specification for Geosynthetic Specification for Highway Applications*, American Association of State Highway Transport Officials https://store.transportation.org/search?q=Standard%20Specification%20for%20Geosynthetic%20Specification%20for%20Highway%20Applications&categoryCode=&index=storeitem&type=All&pageNum=1&pageSize=10&sortBy=Relevance&itemType=All
Angell, 1988	*Technical Basis for the Pavement Design Manual*, D.J Angell, 1988, Queensland Department of Transport and Main Roads http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/detailnonmodal/ent:$002f$002fSD_ILS$002f0$002fSD_ILS:12024/one?qu=Technical+Basis+for+the+Pavement+Design+Manual&te=ILS (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
ARRB, 1984	*21st ARRB Regional Symposium*, 1984, Darwin, NT http://114.111.144.247/Presto/search/SearchResults.aspx?q=KENvbmZlcmVuY2VfYW5kX0pvdXJuYWwuQ086KFJlZ2lvbmFsIFN5bXBvc2l1bSAyMXN0IDE5ODQgRGFyd2luKSk=&qcf=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5

Reference	Title
ASTM STP479	*Suggested Method of Mixture Design Procedure for Lime Treated Soil*, Special Technical Publication 479, M R Thompson, 1970, American Society for Testing Materials https://www.astm.org/DIGITAL_LIBRARY/STP/PAGES/STP38539S.htm
Austroads and ARRB, 1996	*Performance of unbound and stabilised pavement materials under accelerated loading: summary report of Beerburrum II ALF trial* (APRG Research Report. No. 16 and ARR 286), Austroads Pavement Research Group and ARRB Transport Research, Vermont South, Vic http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Performance+of+unbound+and+stabilised+pavement+materials+under+accelerated+loading+%3A+summary+report+of+Beerburrum+II+ALF+trial&te=ILS (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
Bullen, 1992	*Effective maintenance using geotextile reinforced spray seals*, F Bullen and A Nardi, 16th Australian Road Research Board Ltd (ARRB) Conference, 1992, Perth http://114.111.144.247/Presto/content/Detail.aspx?ctID=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5&rID=MzE5&qrs=RmFsc2U=&q=QnVsbGVuIE5hcmRp&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==
Button, 1984	*Fabrics in asphalt overlays – design, construction and specifications*, J.W Button and J.A Epps, 1984, Report (final) for the Texas State Department of Highways and Public Transportation Planning Division, research report number 261-3F https://tti.tamu.edu/publications/catalog/record/?id=1705
Caltabiano, 1991	*Reflection Cracking in Asphalt Overlays (with discussion)*, M.A Caltabiano and J.M Brunton, Association of Asphalt Paving Technologists (AAPT) Asphalt Paving Technology Conference 1991, Seattle, Washington https://trid.trb.org/view.aspx?id=486832
CCAA, 1988	*Recycling Pavements by Cement Stabilisation*, 1988, Cement and Concrete Association of Australia (CCAA) http://www.worldcat.org/title/recycling-pavements-by-cement-stabilisation/oclc/27611696
Chen, 1999	*Determination of Bedrock Depth from Falling Weight Deflectometer Data*, Dar-Hao Chen, 1999, Transportation Research Board Paper No. 99-0589, TRR1655 http://www.trb.org/Publications/Publications.aspx
CSIR, 2002	*Interim Technical Guidelines: The Design and Use of Foamed Bitumen Treated Materials*, CSIR, Pretoria https://scholar.sun.ac.za/handle/10019.1/44319
Dickinson, 1984	*Bituminous Roads in Australia*, E.J Dickinson, 1984, ARRB http://114.111.144.247/Presto/content/Detail.aspx?ctID=NmQ5ZmY0YWQtNTc4NS00YzZiLTk3MTItNjEzYWQxZTgyMTRl&rID=MTIyMA==&qrs=RmFsc2U=&q=Qml0dW1pbm91cyBSb2FkcyBpbiBBdXN0cmFsaWE=&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==
FHWA, 1985	*Pavement Rehabilitation Manual*, 1985, Department of Transportation, Federal Highway Administration (FHWA), (Supplemented: April 1986, July 1987, March 1988, February 1989 and October 1990) http://www.worldcat.org/title/pavement-rehabilitation-manual/oclc/932234352&referer=brief_results

Reference	Title
Fredlund, 1994	*Equations for the Soil Water Characteristic Curve*, March 1994, Canadian Geotechnical Journal, 31(3): 521–532, D.G Fredlund and A Xing https://www.semanticscholar.org/paper/Equations-for-the-soil-water-characteristic-curve-Fredlund-Xing/fe7ccc21350387b329b993c5902a0828e0da7d1b
Genuchten, 1980	*A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils*, September 1980, Soil Science Society of America Journal 44(5): 892–898, Martinus Th. Van Genuchten https://www.researchgate.net/publication/250125437_A_Closed-form_Equation_for_Predicting_the_Hydraulic_Conductivity_of_Unsaturated_Soils1
Gerke, 1982	*Subsurface Drainage - Some Do's and Don'ts*, R.J Gerke, 19th ARRB Regional Symposium, Wagga Wagga, 1982 http://114.111.144.247/Presto/content/Detail.aspx?ctID=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5&rID=NjU0NQ==&qrs=RmFsc2U=&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==
Hicks, 1977	*Pavement Management Guide*, B.E Hicks, 1977, Roads and Transportation Association of Canada Pavement Management Committee https://books.google.com.au/books?id=vMA1AQAAIAAJ&redir_esc=y
Jameson, 1990	*Trials with Proprietary Product Stabilising Agents*, G Jameson, August 1990, 15th ARRB Conference, Darwin, Australia http://114.111.144.247/Presto/content/Detail.aspx?ctID=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5&rID=MTE2MA==&qrs=RmFsc2U=&q=KENvbmZlcmVuY2VfYW5kX0pvdXJuYWwuVEk6KFRyaWFscyB3aXRoIFByb3ByaWV0YXJ5IFByb2R1Y3QgU3RhYmlsaXNpbmcgQWdlbnRzKSkgQU5EIChDb25mZXJlbmNlX2FuZF9Kb3VybmFsLkFVOihHIEphbWVzb24pKSBBTkQgKENvbmZlcmVuY2VfYW5kX0pvdXJuYWwuQ086KChBUlJCKSBDb25mZXJlbmNlIDE1dGggMTk5MCBEYXJ3aW4pKQ==&qcf=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==
Koerner, 1988	*Durability and aging of geosynthetics*, R.M Koerner (editor), *Seminar on Durability and Aging of Geosynthetics*, 1988, Drexel University, Philadelphia, USA https://trid.trb.org/view.aspx?id=1175508
Little, 1987	*Soil Stabilisation for Roadways and Airfields*, Report ESL-TR-86-19, D.N Little, M.R Thompson, R.L Terrel, J.A Epps, and E.J Barenberg, 1987, Air Force Services and Engineering Centre, Tyndall Air Force Base, Florida https://ia802800.us.archive.org/25/items/DTIC_ADA183382/DTIC_ADA183382.pdf
Lorenz, 1987	*New Mexico Study of interlayers used in reflective crack control*, V.M Lorenz, 1987, Transportation Research Record (TRR) 1117, Transportation Research Board (TRB) https://pubsindex.trb.org/view/1987/C/282307
Maccarrone, 1994	*Pavement Recycling Using Foamed Bitumen*, S Maccarrone, G Holleran, D.J Leonard and S Hey, 1994, 17th ARRB Conference http://114.111.144.247/Presto/content/Detail.aspx?ctID=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5&rID=Mjk4Nw==&qrs=RmFsc2U=&q=UGF2ZW1lbnQgUmVjeWNsaW5nIFVzaW5nIEZvYW1lZCBCaXR1bWVu&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==

Reference	Title
McCallister, 1990	*Property Changes in Lime Treated Expansive Clays Under Continuous Leaching*, Technical Report, GL-90-17 http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Property+Changes+in+Lime+Treated+Expansive+Clays+Under+Continuous+Leaching&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
Markow, 1985	*Optimal Rehabilitation Frequencies for Highway Pavements*, Michael J Markow and Wayne S Balta, 1985, Transportation Research Board (TRB) https://pubsindex.trb.org/view/1985/C/272078
Metcalf, 1981	*Principles and Application of Soil Stabilisation*, ARR 47A, J.B Metcalf, Australian Road Research Board (ARRB) http://114.111.144.247/Presto/content/Detail.aspx?ctID=NmQ5ZmY0YWQtNTc4NS00YzZiLTk3MTItNjEzYWQxZTgyMTRl&rID=Mzkx&qrs=RmFsc2U=&q=UHJpbmNpcGxlcyBhbmQgQXBwbGljYXRpb24gb2YgU29pbCBTdGFiaWxpc2F0aW9uIE1ldGNhbGY=&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==
Mildenhall, 1986	*A manual for the maintenance and repair of concrete roads*, H.S Mildenhall and G.D.S Northcott, 1986, Department of Transport and Cement and Concrete Association, Great Britain http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/detailnonmodal/ent:$002f$002fSD_ILS$002f6$002fSD_ILS:6516/ada (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
Muller, 2001	*Trial ground penetrating radar (GPR) pavement investigations in Border District*, W Muller, J Jones, M Kendall, *Road System and Engineering Technology Forum*, 2001, Brisbane, Queensland, Australia http://librarycat/closed/catdept/Forum/200111005/19_tf2000_M_Kendall.pdf
NAPA, 1982	*Asphalt Overlays on Cracked and Seated Concrete Pavements*, 1982, Offprint from Paving Forum, National Asphalt Pavement Association (NAPA) http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/detailnonmodal/ent:$002f$002fSD_ILS$002f4$002fSD_ILS:4979/ada (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
NAPA, 1986	*Does PCC cracking and seating work* (NAPA, 1986, *Better Roads*, National Asphalt Pavement Association (NAPA)) https://www.researchgate.net/publication/293717126_DOES_PCC_CRACKING_AND_SEATING_WORK
NAPA, 1987	*Cracking and Seating of PCC Pavements Prior to Overlaying with Hot Mix Asphalt*, Campbell Crawford, 1987, National Asphalt Pavement Association (NAPA) http://www.worldcat.org/title/cracking-seating-of-pcc-pavements-prior-to-overlaying-with-hot-mix-asphalt-state-of-the-art/oclc/221147585&referer=brief_results
NHI, 1993	*Instructional Text C: Techniques for pavement rehabilitation: a training course*, 1993, National Highway Institute http://www.worldcat.org/title/techniques-for-pavement-rehabilitation-a-training-course/oclc/32006451

Reference	Title
Pickering, 2000	*The effective use of geotextiles in bituminous surfacings*, Report RP15473, 2000, Queensland Department of Main Roads, Brisbane http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/detailnonmodal/ent:$002f$002fSD_ILS$002f39$002fSD_ILS:39902/ada (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
Ramanujam, 2000	*Characterisation of Foamed Bitumen Stabilisation*, J.M Ramanujam and J.D Jones, August 2000, *Road System and Engineering Technology Forum*, Brisbane, Australia http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Characterisation+of+Foamed+Bitumen+Stabilisation&qf=AUTHOR%09Author%09Ramanujam%2C+Jothi+M.%09Ramanujam%2C+Jothi+M.&qf=AUTHOR%09Author%09Jones%2CJD%09Jones%2CJD (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
Ramanujam, 2001	*Foamed Bitumen Stabilisation: the Queensland Experience*, M Kendall, B Baker, P Evans and J Ramanujam, 2001, 20th ARRB Conference, Melbourne, Victoria, Australia http://114.111.144.247/Presto/content/Detail.aspx?ctID=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5&rID=MzEwNQ==&qrs=RmFsc2U=&q=Rm9hbWVkIEJpdHVtZW4gU3RhYmlsaXNhdGlvbiBLZW5kYWxs&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==
Ramanujam, 2009	*Recycling of Asphalt Pavements*, J.M Ramanujam, 2009, Queensland Department of Transport and Main Roads https://trid.trb.org/view/1158562
Rigo, 1990	*Thermal resistance of polypropylene geotextiles used in interlayers retarding the reflective cracking in pavements*, J.M Rigo, 4th International Conference on Geotextiles, Geomembranes and Related Products 1990, The Hague, Netherlands, page 879 https://trid.trb.org/view.aspx?id=1181759
Saarenketo, 2000	*Road Evaluation with Ground Penetrating Radar*, T Saarenketo and T Scullion, Journal of Applied Geophysics, 43:119–138 http://directory.umm.ac.id/Data%20Elmu/jurnal/J-a/Journal%20Of%20Applied%20Geophysics/Vol43.Issue2-4.2000/1219.pdf
Saarenketo, 2006	*Electrical properties of road materials and subgrade soils and the use of ground penetrating radar in traffic infrastructure surveys*; T Saarenketo, PhD Thesis, University of Oulu, Oulu, Finland http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Electrical+properties+of+road+materials+and+subgrade+soils+and+the+use+of+ground+penetrating+radar+in+traffic+infrastructure+surveys&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)

Reference	Title
Sherwood, 1993	*Soil Stabilisation with Cement and Lime*, P.T Sherwood, 1993, *State of the Art Review*, Transport Research Laboratory (TRL) http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Soil+Stabilisation+with+Cement+and+Lime&qf=AUTHOR%09Author%09Sherwood%2CPT%09Sherwood%2CPT (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
Snow, 1981	*Overlays – construction (A.C., Granular, C.T.B.)*, B.K Snow, 1981, Queensland Department of Transport and Main Roads Northern Division Seminar, Townsville http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Overlays&qu=-&qu=construction+%28A.C.%2C+Granular%2C+C.T.B.%29&te= (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
TNZ RR127	*Mechanistic Design of Pavements Incorporating A Stabilised Subgrade*, Research Report 127, 1988, Transfund New Zealand http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/detailnonmodal/ent:$002f$002fSD_ILS$002f36$002fSD_ILS:36363/ada (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)
TRB, 1987	*Lime Stabilization: Reactions, Properties, Design and Construction*, *State of the Art* Report 5, 1987, Transportation Research Board (TRB) https://pubsindex.trb.org/view/1987/C/283897
TRRL RR225	*Recycled asphalt wearing courses*, A.C Edwards and H.C Mayhew, 1989, Research Report 225, Transport and Road Research Laboratory (TRRL) https://trl.co.uk/reports/RR225
Vuong, 1988	*'Analyses of a heavy duty granular pavement using finite element method and linear elastic back-calculation models,'* B Vuong, D.W Potter and P Kadar, Proceedings of the ARRB Conference, 14th, Part 8, pp284–298. Australian Road Research Board, Vermont South, Vic http://114.111.144.247/Presto/content/Detail.aspx?ctID=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5&rID=MzUyNA==&qrs=RmFsc2U=&q=QW5hbHlzZXMgb2YgYSBoZWF2eSBkdXR5IGdyYW51bGFyIHBhdmVtZW50IHVzaW5nIGZpbml0ZSBlbGVtZW50IG1ldGhvZCBhbmQgbGluZWFyIGVsYXN0aWMgYmFjay1jYWxjdWxhdGlvbiBtb2RlbHM=&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==
Walter, 1985	*Surface enrichment of sprayed seals using bitumen emulsion in New South Wales*, 1985, Bitumen Emulsion Workshops, Melbourne and Sydney, Australia http://114.111.144.247/Presto/content/Detail.aspx?ctID=MjE1ZTI4YzctZjc1YS00MzQ4LTkyY2UtMDJmNTgxYjg2ZDA5&rID=NjcxNQ==&qrs=RmFsc2U=&q=UCBEIFdhbHRlcg==&ph=VHJ1ZQ==&bckToL=VHJ1ZQ==&rrtc=VHJ1ZQ==
Welke, 1984	*Cracking and Seating of Jointed Portland Cement Concrete Pavements in Michigan*, R.A Welke, A.B Webb Jr and C Van Deusen, 1984, Proceedings of the Association of Asphalt Paving Technologists Technical Sessions, Association of Asphalt Paving Technologists (AAPT), Arizona.
Wirtgen AE	*Ancillary Equipment*, Wirtgen GmbH, Windhagen, Germany https://www.wirtgen.de/en/line-products/cold-recyclers-soil-stabilizers/ancillary-equipment/

Reference	Title
Wirtgen CRM	*Cold Recycling Manual*, Wirtgen GmbH, Windhagen, Germany http://media.wirtgen-group.com/media/02_wirtgen/infomaterial_1/kaltrecycler/kaltrecycling_technologie/kaltrecycling_handbuch/Cold_recycling_Manual_EN.pdf
Wirtgen CRSS	*Cold Recyclers and Soil Stabilizers*, Wirtgen GmbH, Windhagen, Germany https://www.wirtgen.de/en/news-media/information-material/cold-recyclers-soil-stabilizers/cold-recylcers-soil-stabilizers.php
Wirtgen HR	*Hot Recyclers*, Wirtgen GmbH, Windhagen, Germany https://www.wirtgen.de/en/news-media/information-material/hot-recyclers/
Zanten, 1986	*Geotextiles and geomembranes in civil engineering* (text), R Veldhuijzen Van Zanten (editor), 1986 http://qdot.ent.sirsidynix.net.au/client/en_AU/tmrlibrary/search/results?qu=Geotextiles+and+geomembranes+in+civil+engineering&qf=AUTHOR%09Author%09Veldhuijzen+van+Zanten%2CR%09Veldhuijzen+van+Zanten%2CR (for users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au)

2 Pavement evaluation

This chapter discusses the process of pavement evaluation. Chapter 3 discusses how the outcomes of a pavement evaluation are used to identify a range of appropriate rehabilitation treatments for different pavement types. Chapters 4 and 5 provide the specific technical detail and references relating to these rehabilitation techniques.

This *Manual* must be read in conjunction with AGPT05. It should not be assumed this *Manual* contains all relevant information, even where the subject or headings are the same; for instance, the designer must refer to the visual assessment sections in both this *Manual* and AGPT05 to access the complete discussion.

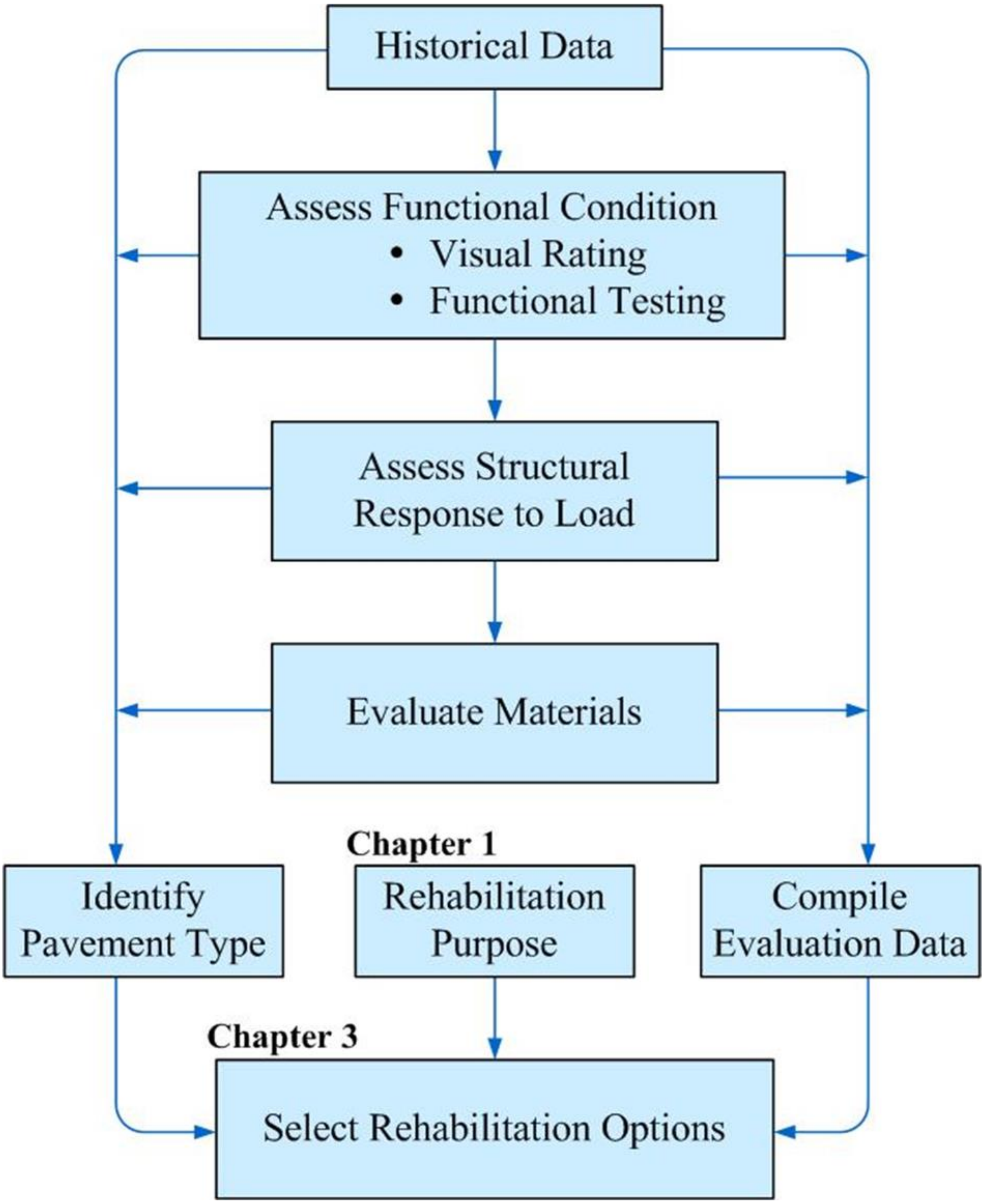
Figure 2 shows the pavement evaluation process. Pavement evaluation involves:

- relating the symptoms of pavement distress to their causes
- explaining how the distress(es) developed (the mechanism[s] causing distress), and
- detailing appropriate methods to complete an investigation and the sequencing of them.

Further, it involves assessing the functional and structural condition of the existing pavement and assessing it against the predicted future conditions (for the required design period).

The driver of a rehabilitation investigation may not necessarily be pavement distress (for example, works may be required to provide extra traffic capacity) (refer to Chapter 1 for a discussion about 'synergies and opportunities'). A pavement may not be showing distress (not at the end of its design life) but will be upgraded to provide service for another design period (such as upgrade from four lanes to six before the existing pavement has reached the end of its design life). In such cases, while pavement distress may not be evident, an evaluation is still required.

Figure 2 – Pavement evaluation process



2.1 Use of data and sources of data

Data should not be used in isolation; rather, the data collected from various sources should be used together to evaluate the pavement(s) and it is best to gather data from multiple sources rather than rely on only one or two sources.

2.2 Geometric assessment

This chapter focuses on the evaluation of the road pavement(s). Austroads *Guide to Road Design Part 3: Geometric Design* (AGRD03) and the Transport and Main Roads supplement (RPDM3-03) provide guidance on when a geometric assessment should be undertaken. Situations where a geometric assessment is recommended include:

- a full shoulder seal
- shoulder widening
- overlay and widening
- rehabilitation (for example, stabilisation) and widening, and
- duplication of an existing carriageway.

Therefore, a geometric assessment may be warranted in parallel with a pavement rehabilitation investigation. Refer to Section 2.12.2 of this *Manual*, RPDM3-03 and AGRD03 for further guidance.

2.3 The pavement evaluation concept

2.3.1 Purpose

The purpose of a pavement evaluation is primarily to determine the condition of the existing pavement so appropriate rehabilitation treatments may be identified (if required). A sound pavement evaluation will:

- enable the designer to assess the existing pavement and determine its current condition
- identify the causes / mechanisms of any observed pavement distress, if any
- ascertain whether the existing pavement must be rehabilitated to withstand the predicted conditions for the required design period, and
- provide the foundation for identifying what treatments / interventions are required, if any.

Firstly, factors affecting the performance of the existing pavement are investigated to determine its functional condition (for example, how the road satisfies the needs of road users in terms of cost, comfort, convenience and safety) and structural condition (how it responds to load[s]). Secondly, its ability to withstand the traffic and other environmental conditions expected over the required design period are determined in terms of its functional and structural capacity. If rehabilitation measures are necessary, the evaluation will also allow these to be designed so the required life is attained.

Premature conclusions about the causes of pavement distress should be avoided as they may cause the designer to focus on justifying them, rather than being open to all possibilities. As a result, the designer may fail to identify the real cause(s) of the pavement distress.

2.4 Sources of information

Information used in a pavement evaluation is available from several different sources. These include:

- historical records
- pavement condition assessments
- an assessment of the drainage systems, including any sub-surface pavement drainage systems
- materials testing, and
- the pavement's structural response to load.

The following sections provide more details about some of these sources. While not all of these sources may be relevant in any specific case, the rehabilitation designer should be aware of their potential contribution towards determining why the present conditions exist.

Pavement condition data may be collected and recorded by a variety of methods, including:

- as part of a formalised visual rating (VR) or visual condition survey
- a vehicle mounted profilometer (VMP) which collects information such as rut depth, roughness and photographic images against (gazettal) chainage and global navigation satellite system (GNSS) coordinates
- pavement roughness surveys, and
- deflection surveys.

Alternatively, pavement condition data may be gathered via a simple visual inspection done by the evaluating officer.

Figure 2.4 – Vehicle mounted profilometer

a – front mounted discrete lasers	b – rear mounted scanning lasers
	

2.5 Presentation of data

To facilitate the effective interpretation of data, attention should be paid to how data are prepared and presented. The designer will find the use of both data mapping and graphical techniques of considerable benefit. One example of the output of a visual survey is shown in AGPT05 Section 3.3.1, Figure 3.1 *Visual Condition Data*.

A data map should consist of a schematic plan of the pavement under evaluation, on which some or all the following detail may be symbolically, diagrammatically or graphically represented:

- location, nature, extent and severity of defects including patches; Appendix A of this *Manual* and AGPT05 Section 3.3.1 *Visual Condition Data* contain information about the identification of visual distress
- load response (for example, deflections as recorded by falling weight deflectometer (FWD), heavy falling weight deflectometer (HWD), traffic speed deflectometer (TSD), deflectograph or Benkelman Beam testing) against chainage
- identification of representative sections as determined by condition data (for example, by using deflection results)
- values characterising representative sections (for example, characteristic moduli derived from back analysis)
- test results (for example, of subgrade California Bearing Ratio (CBR))
- changes in pavement or subgrade configuration or type
- details and extents of typical cross-sections
- geometric features (for example, longitudinal sections, superelevations / crossfalls, alignment)
- drainage features
- topographical features (for example, cuts, fills, cross-grades)
- degrees of saturation
- moisture contents, and/or
- photographs (for example, from Digital Video Road (DVR) or field inspection).

The benefits of such a map include:

- assistance with identifying relationships between various observations or measured parameters
- provision of an interim record of field observations to save repeated site inspections
- enabling the information to be visually appraised
- enabling the road to be divided more readily into representative sections for analysis, and
- provision of a permanent record of pavement information of value in subsequent evaluations.

Note: Once rehabilitation is completed, the data may no longer be observable. A map provides a record that may be referenced in the future).

2.6 Historical data

A considerable quantity of historical data relevant to determining an appropriate rehabilitation strategy may be available from a variety of sources.

Historical data may be available from the sources discussed in the following sections. The designer should contact the relevant departmental regional or district office in the first instance to obtain the available, relevant historical information.

2.6.1 Advantages

Some or the advantages of gathering historical data include:

- gathering data can be a relatively quick process
- such data can provide the pavement's history
- historical data can help explain current distress(es), and
- historical data provide a good base for developing further investigations (for example, testing).

2.6.2 Disadvantages and limitations

The gathering of historical data does have some disadvantages and limitations associated with it.

Gathering historical information is a useful starting point; however, the designer must use this information with care. It is not unusual for the actual (insitu) pavement to differ from expected, given the historical information, therefore, historical information should only be used in conjunction with project specific testing.

2.6.3 Original design

Obtaining the original design data will assist the designer to determine whether the distress is caused by a change in the design parameters, the constructed pavement or the performance expectations (for example, there may be a difference between the (assumed) design values and the actual conditions experienced, such as design traffic versus actual traffic).

This information may be available from:

- the department's A Road Management Information System (ARMIS) database (see Section 2.6.6)
- original scheme documents (for example, drawings / plans, specifications, the geospatial information management system (GIMS))
- investigation results, and
- previous design or investigation reports.

Information garnered from these sources includes:

- preconstruction testing
- design assumptions (for example, design CBR strength, design traffic)
- extents of various pavement types
- type cross-sections
- superelevation and crossfall details
- road drainage (for example, stormwater, cross-drainage)
- topographic information
- alignment details
- locations of utilities / services (needed for field investigations, see also Section 2.8.6)
- pavement structure and materials, and
- moisture control system design (for example, subsurface pavement drainage, drainage blankets).

There is no certainty the pavement was constructed in accordance with the design. Information must be checked by obtaining 'As Constructed' information and/or undertaking field testing / investigations.

2.6.4 Construction detail

Refer to AGPT05, Section 3.2.2 *Construction Details*.

2.6.5 Digital Video Road

The department also provides a DVR software product that allows images collected along a (surveyed) road to be viewed against both chainage and GNSS coordinates. These images are normally captured using compliant vehicle mounted equipment.

DVR allows the user to:

- view images from the 'straight ahead', left, right and rear directions (captured against gazettal chainage)
- export still images, and
- take rudimentary, approximate measurements (for example, lane width).

The designer must be cognisant of the fact the images are historical and they may not represent the current state of the pavement or road corridor.

It is important to note the images are not of high resolution, therefore, it may not be possible to use them to complete VR surveys (for example, to map location and extent of cracking). DVR often cannot replace a visual survey undertaken during a site visit. A visual survey should be completed where required.

DVR is a useful tool to be used in an investigation, especially in the initial stages (for example, to identify the location and extents of cuts and fills and possibly to confirm some field observations).

2.6.5.1 Access to Digital Video Road

Historical and current DVR-compliant video files are stored on departmental computer network resources. Both the software and images are accessible by departmental officers; however, access to these resources by individuals or organisations external to the department is likely to be restricted. External parties may obtain a copy of DVR information from the relevant departmental regional or district office.

Video imagery available from such products as Google Street View may be used to supplement, but not replace, DVR information, and shall consider the date of such imagery.

2.6.6 A Road Management Information System

Transport and Main Roads has a database of selected information about the roads it manages. This database, ARMIS, contains a range of historical pavement information. This may include information about:

- scheme numbers / names and the date of their construction
- pavement configuration (for example, layer thicknesses, material types)
- roughness
- rutting
- surface texture
- deflections (for example, from network deflectograph testing)
- traffic information
- age of the surfacing
- age of the pavement, and
- skid resistance.

The extent of information, and its accuracy, age, content and level of aggregation, can vary; for example, ARMIS may contain aggregated roughness values for the whole length of a road but skid resistance test results for only some sections of the road. Consequently, ARMIS information should not be used on its own and should be checked / verified through other (for example, field) investigations.

The designer should be aware data in ARMIS are often in an aggregated form (for example, average rut depth for a 100m segment) and/or it is the result of 'coarse' network surveys. Consequently, ARMIS data alone may not be adequate for a detailed investigation. Where possible, the original disaggregated information should be obtained, and/or supplementary investigations should be undertaken.

Notwithstanding these limitations, the ARMIS database is a useful starting point in an investigation.

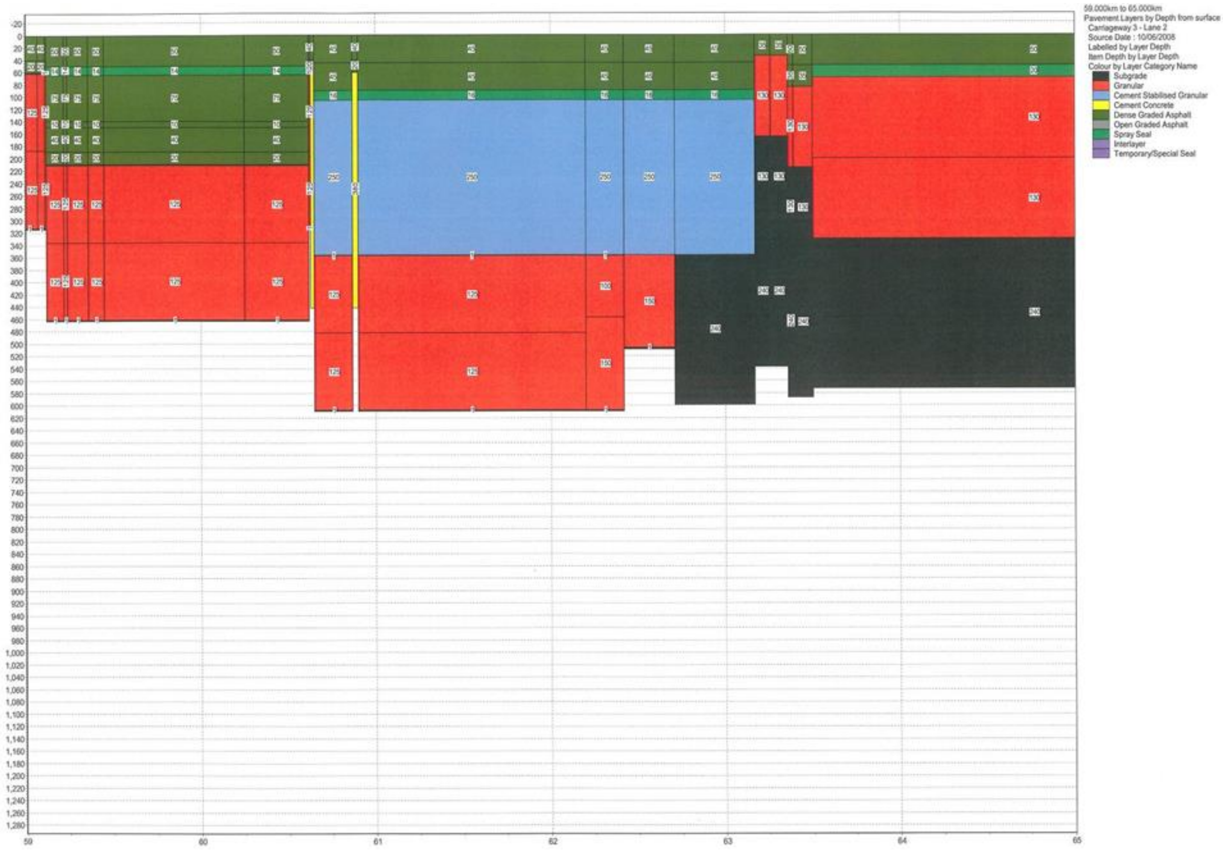
2.6.6.1 Access to A Road Management Information System

ARMIS information can be accessed by departmental officers. Individuals or organisations outside the department may not be able to access ARMIS information directly. External parties may obtain ARMIS information from the relevant departmental regional or district office.

2.6.6.2 Interrogating A Road Management Information System and viewing information

Provided the user has access to ARMIS data, the *MapInfo* and *Chartview* software programs, and ARMIS Query Library are all useful tools for interrogating the database and viewing information.

Figure 2.6.6.2. – Example of Chartview output based on A Road Management Information System information (thicknesses of layers shown)



2.6.7 Geospatial information management system: Digital plan room

The department has a GIMS with electronic copies of some of the department’s registered plans. Users with access can search for, view and print drawings (contained in the GIMS database). These can be design or 'As Constructed' plans ('As Constructed' plans have not been produced for all schemes, so many plans are design plans).

2.6.7.1 Access to geospatial information management system

GIMS can be accessed by departmental officers; however, individuals or organisations outside the department may not be able to access it. External parties may obtain electronic copies of the department’s registered plans from the relevant departmental regional or district office.

2.6.8 Maintenance records

Refer to AGPT05 Section 3.2.3 *Maintenance and Rehabilitation Records*.

Maintenance information may also include data about the rate of pavement deterioration.

Transport and Main Roads' Asset Managers should be consulted to obtain some or all this information and determine their views about past pavement performance.

2.6.9 Climatic conditions

Records of climatic conditions experienced during construction, if available, are an important reference.

2.6.9.1 Rainfall and dry periods

Rainfall records, in terms of intensity, duration and distribution, may be of value when related to the original design assumptions and the development of pavement distress. High intensity falls may exceed the capacity of the drainage systems (for example, stormwater, cross-drainage culverts) leading to a 'back-up' of water into and through a subsoil drainage system. On the other hand, a prolonged wet period, even if intensities are low, may lead to a fall in subgrade strength through a general rise in moisture levels.

Conversely, an extended dry period may result in unexpected subgrade shrinkage with consequent damage to the pavement.

2.6.9.2 Temperature

Temperature affects the performance of bituminous seals, asphalt layers and concrete. When undertaking a pavement investigation and/or designing treatments, the designer will need to take account of the effects of temperature. This *Manual*, the PDS, AGPT02 and AGPT05 provide details of how temperature can affect pavement performance and how it needs to be considered in design.

2.6.10 Traffic

Refer to AGPT05 Section 3.2.5 *Effect of Traffic on Past Performance*.

The Traffic Surveys and Data Management website provides access to Transport and Main Roads' traffic survey applications. A spreadsheet (*Class-Specific Traffic Load Distributions* (CTLD)) is also available which generates a traffic load distribution for use in pavement design. It combines class-specific traffic load distributions with classified vehicle counts to produce a single combined traffic load distribution.

These applications shall be used to extract relevant traffic data and reports for the rehabilitation project.

2.7 Visual assessment of pavement condition

It is often useful to assess the pavement visually, especially in the initial stages of a pavement investigation. Mapping defects at this time is also recommended. The type, severity and extent of visual distress provides the most immediate and direct indicator of either the mode, or modes, of distress and associated causes. Such an assessment also helps guide further investigations and helps the designer evaluate how and why the observed distress has occurred.

AGPT05 Section 3.3 *On-site Data* discusses visual assessments further. AGPT05 Appendix A *Identification, Causes and Treatment of Visual Distress* also contains pertinent information for the visual survey of rigid pavements.

During a visual assessment, note:

- the effectiveness of surface and, where possible, subsurface drainage and any links between drainage and distress
- links between distress and other recorded features (for example, cut / fill boundaries, drainage and vegetation)
- links between distress and other data recorded (for example, deflections, construction history and geology), and
- links between distress and traffic loading (for example, where distress is related to road grade, points of entry and exit of heavy vehicles).

For departmental project-level pavement rehabilitation investigations, the visual assessment shall be conducted in accordance with AGPT05 Appendix A *Identification, Causes and Treatment of Visual Distress*.

A visual assessment is an opportunity to check network survey results (for example, validate rut depths measured using a VMP) or take measurements for the first time (for example, note the location and extent of ruts and their depth at regular intervals).

2.8 Ground penetrating radar

The discussion in this section pertains to the use of ground penetrating radar (GPR) for investigating road pavements. It does not cover all possible uses of GPR nor does it cover all available GPR units or systems.

GPR systems vary and so does their capability. The type of, or actual, system chosen will depend on what data are being sought. The designer should choose the GPR system or provider carefully. In so doing, the designer must consider what data are required and the capabilities of the various systems available.

Description

GPR is a non-destructive testing technique providing information relatively quickly. GPR systems transmit electromagnetic signals (microwave energy) into the ground and receive reflections from subsurface features. GPR systems 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 simple analogy to GPR is sonar; however, unlike sonar, GPR uses electromagnetic waves instead of mechanical ones to form images beneath the surface. Within the pavement, some of the transmitted microwave energy rebounds off material boundaries (for example, subgrade / sub-base interface, services, boulders, and so on) and some continues downwards. The greater the difference in electrical conductivity (the greater the dielectric contrast) between materials at a boundary, the stronger is the reflection. The time to detect reflections is, in part, a product of the depth. The longer it takes to receive a reflection, the deeper the feature is (assuming material properties are consistent with depth).

Once a GPR survey has been completed, the raw data must be analysed and interpreted to yield useful information. This may take some time and requires specific expertise. Like all non-destructive testing methods, calibration against physical measurements, such as those obtained from trenches or cores, is strongly recommended to make the results as representative as possible. Additional fieldwork (for example, coring, trenching) and/or destructive testing is normally required to validate its results.

Calibration

GPR measures the time taken for electromagnetic waves to travel from the transmitting antenna to the receiving antenna and the magnitude of returning electromagnetic reflections; however, these measurements are not particularly useful unless they can be associated with a distance (depth).

A reflection occurs where a difference in the dielectric properties of the pavement material is encountered. The larger the difference, the larger the magnitude of the reflection.

While it is possible to detect structural interfaces, the GPR measurements cannot be used to reveal the properties of the material between these boundaries. The person interpreting the GPR results, therefore, should have a good knowledge of typical pavement construction practices at the location being investigated so he or she can interpret the results in an informed manner.

To convert the measured time to a distance, it is necessary to know the velocity of the electromagnetic waves through the materials encountered between the transmitter and receiver. The velocity of the electromagnetic wave is related to dielectric permittivity (also known as dielectric constant) of the material through which it is travelling.

The electromagnetic wave velocity is strongly influenced by the presence of moisture and GPR investigations should, therefore, avoid periods where water tables may be high.

Information on the type of material present from cores / trenches is used to calibrate the assumed relative dielectric permittivity (dielectric constant) of the pavement layers, which determines the velocity of the electromagnetic waves.

The electromagnetic wave velocity varies with the pavement dielectric properties. The accuracy attained in layer depth estimates, therefore, depends on how variable the pavement materials are along the road and how many cores or trenches are taken to calibrate for changes in the material properties.

A typical calibration uses specialised software to reconcile core reports, supplied by the client, with GPR measurements, and, in so doing, provides estimates of the dielectric permittivity for each layer:

1. Enter the location, layers, depths and materials indicated by core reports.
2. The software assigns an initial (presumptive seed value) dielectric permittivity to each layer based on the indicated (core) material type.
3. Allow software to adjust the initial dielectric permittivity of layers until the difference between measured (core) and estimated (GPR) layer depths is reduced to a minimum.

Alternatively, research into techniques such as the surface reflection coefficient or multi-offset GPR could provide a method to calculate the electromagnetic wave velocity and calibrate layer depth predictions without cores or trenches.

The dielectric permittivity of material extracted from the pavement (Section 2.9) can be measured using a variety of methods. Techniques include resonator cavities, large-coaxial cells and free-space methods. If measured, this information should be used to initialise the dielectric permittivity of each layer during calibration.

2.8.1 Advantages

Advantages of using GPR include:

- 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 so proceed with more confidence
- longitudinal and, sometimes, transverse changes can be identified, allowing the pavement to be sectioned with more confidence, and
- investigations and, hence, back analysis, designs and costs, are better optimised.

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

2.8.2 Disadvantages and limitations

Disadvantages and limitations of using GPR include:

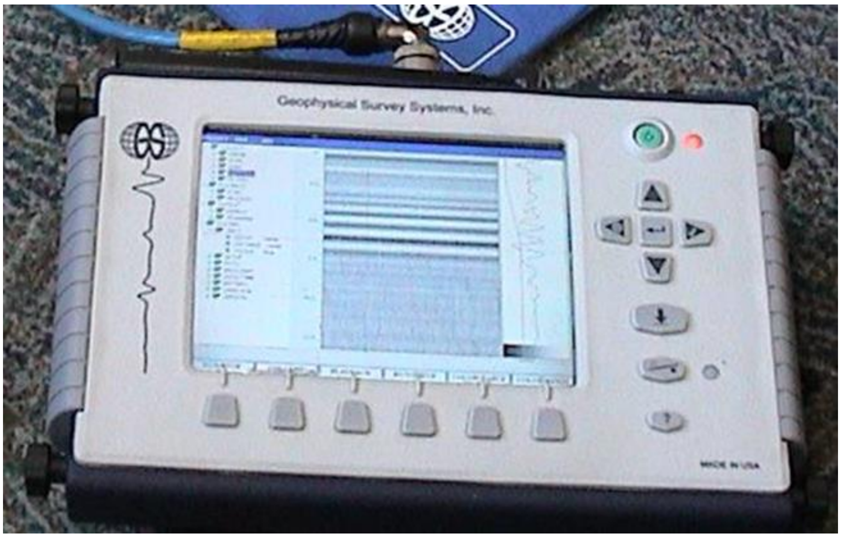
- its cost
- field investigations are normally required to obtain accurate results; the use of GPR does not eliminate the need for the other fieldwork, investigations and testing (for example, trenching, sampling and laboratory testing)
- analysis is time-consuming, which affects the cost and the timeframe of the investigation
- analysis and interpretation require input from a GPR expert and the person or team undertaking the pavement investigation, and
- there are currently a limited number of providers (for example, affects availability, cost).

The technology is still developing, productivity and techniques may improve, and costs may reduce, with time.

2.8.3 Equipment

A GPR setup for road investigations typically consists of a GPR control unit (refer Figure 2.8.3(a)) or units connected to one, two or more antennae. Some units can record Global Positioning System coordinates with the GPR data, enabling the data to be related to other Global Positioning System-based systems. In addition, some systems can capture other related information as well (for example, photographs of the road environment and road surface).

Figure 2.8.3(a) – Example of a single-channel portable ground penetrating radar control unit



Accurate positioning is critical to pavement investigations. This is especially so with GPR. There is little point identifying subsurface features with GPR if their location cannot be determined accurately and reported reliably. Most GPR setups for pavement investigation use a rotary encoder on the survey vehicle or trailer axle. The encoder, connected to the GPR control unit, measures distance and triggers the GPR pulse rate. The user can then specify a pulse at a set spacing (for example, every 50mm, 70mm or 500mm). The spacing used depends on the speed of the survey, the objectives of the investigation, traffic management requirements and the limitations of the equipment.

As GPR deals with waves approaching the speed of light, one of the main limitations in survey speed is the computing power of the control unit(s). Substantial improvements in survey speed and capacity of GPR units are expected in the future as computing power improves.

Antennae used for pavement investigations may be either ground-coupled (placed on the ground and towed, see Figure 2.8.3(b)) or air-coupled (suspended on a vehicle or on a towed frame, see Figure 2.8.3(c)). It is usually possible to undertake much quicker surveys with air-coupled antennae; however, their penetration is typically less than a ground-coupled antenna (they may provide less information about subsurface features).

Figure 2.8.3(b) – Example of a ground-coupled antenna with distance-measuring wheel set-up behind a vehicle

Figure 2.8.3(c) – Example of an air-coupled antenna mounted on the front of a vehicle

Most commercial GPR units are of the ’impulse’ type (the GPR emits discrete bursts of energy into the pavement and measures the response after each).

In recent years, step-frequency GPR units have become available, particularly for three-dimensional applications. Though much more computer-intensive, this approach has several advantages including better sensitivity and the ability to operate over a much wider range of frequencies (and, therefore, depths). They provide more information about subsurface features than other systems at pavement depths.

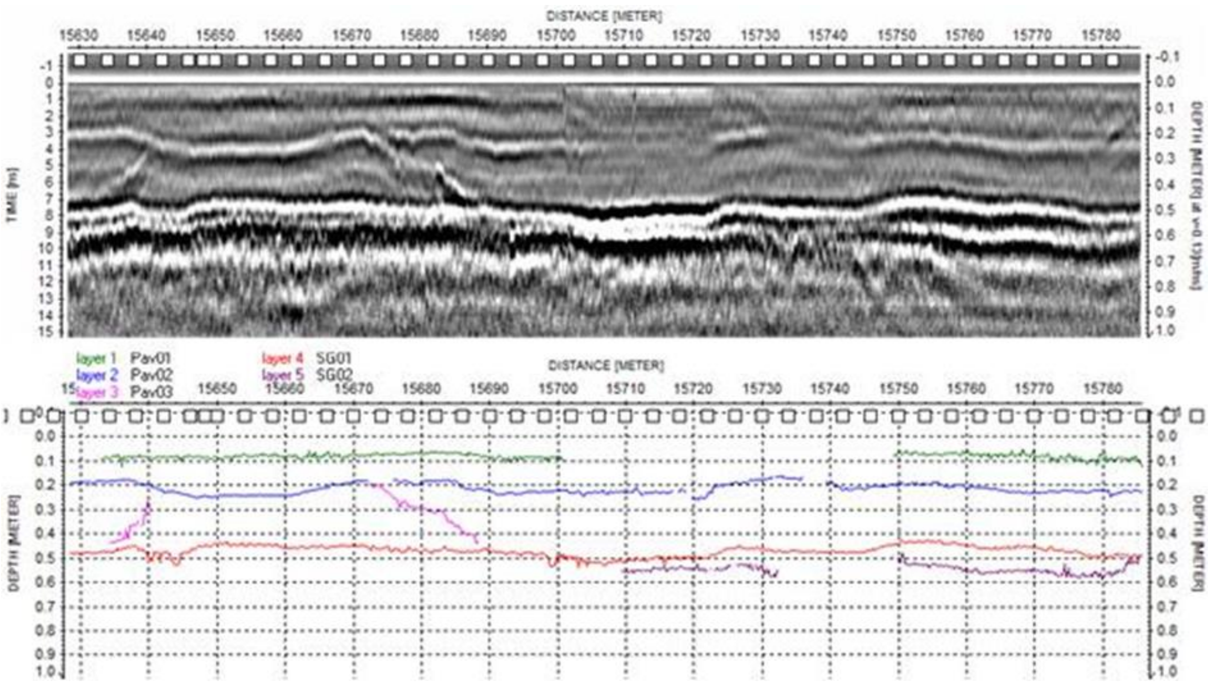
Another recent development is the use of three-dimensional GPR systems. These consist of multiple synchronised antennae spaced at intervals across a part of a lane (for example, the general maximum width of a vehicle, which is 2.5m). The results are combined to produce a three-dimensional representation. These data can then be sliced incrementally in any orientation to identify features of interest.

2.8.4 Ground penetrating radar output

Output from a single GPR antenna is usually displayed as a two-dimensional plot of time (used to determine depth) versus position. An indication of the strength of reflection is also included (represented by colour or shades of grey). The result is called a 'radargram'. The upper portion in Figure 2.8.4(a) shows a typical two-dimensional radargram of a road pavement shown in ‘greyscale’. Underneath is the interpretation of each layer within the pavement shown in a long section. The depth is estimated from the known typical travel velocities through pavement materials which can then be refined (calibrated) by adjusting predicted depths to match actual physical measurements (for example, to depths obtained from cores or trenches). Accurate positioning is again critical here; there is little point trying to match layers found in the core, trench or pit with the GPR data if the information is not for the same location.

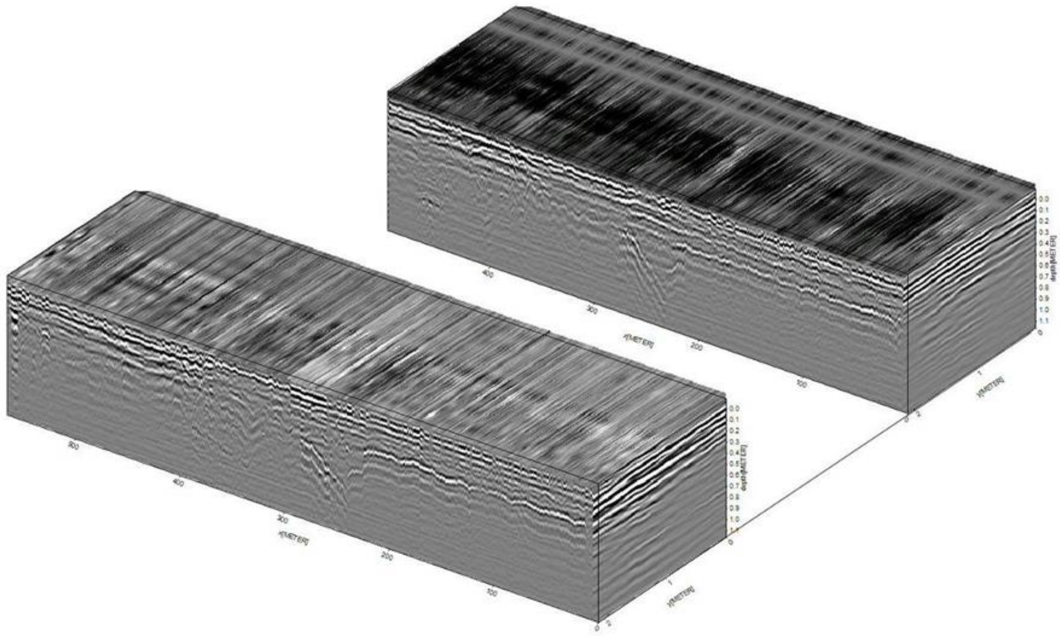
Figure 2.8.4(a) and Figure 2.8.4(b) show an example of three-dimensional GPR output of information collected from a pavement.

Figure 2.8.4(a) – Example of a two-dimensional radargram of a road pavement (top) and corresponding layer interpretation (bottom)



Note: Several changes of construction can be seen.

Figure 2.8.4(b) – Example of three-dimensional radargrams of a road pavement



2.8.5 Ground penetrating radar and pavement investigations

For pavement investigations, the transverse location of tests may correspond to deflection testing; for instance, the outer wheel path (OWP) in the outer lane may be targeted with a single run in one direction or multiple runs can be used to investigate other wheel paths.

Often the pavement will need to be cored, trenched or have test pits excavated in it so the GPR outputs can be verified. A combination of these techniques is, therefore, normally required. Consequently, GPR is a complementary tool (for example, to deflection testing and fieldwork) rather than a technique which can be relied upon on its own.

The cost of a GPR survey and the associated analysis and interpretation can be significant when compared to the cost of the total investigation. When compared to the cost of the constructed project, however, the cost to use GPR can be put into perspective.

Whether a GPR is used for a project investigation will depend on an assessment of cost against likely benefits. When making this assessment, consider:

- the expected variability of road pavements – the higher the likely variability, the greater the justification for the use of GPR
- the function and importance of the road – the higher the function or importance of the road, the greater the justification for the use of GPR, and
- whether economies of scale can be achieved by undertaking surveys for several projects in one commission.

2.8.6 Appropriate uses

GPR is particularly well suited to determining the 'As Constructed' pavement structure and the thickness of pavement layers in situations such as:

- measuring pavement-layer depths and/or monitoring the variation in pavement layer thicknesses over the length of a project site
- locating changes of construction within a pavement
- locating patches, trenches and the like, and their extent – these may be hidden by the relatively recent resurfacing but may need to be removed before rehabilitation treatments are applied
- locating subgrade anomalies (for example, number and extent of large boulders, air voids, and tree roots)
- determining the thickness of pavement above sensitive infrastructure (for example, thickness of an asphalt deck wearing surface over bridge decks and concrete cover to reinforcing), and
- locating man-made features such as public utilities (for example, communications, gas, electricity, water, sewer and stormwater items), abandoned objects (fuel tanks, concrete waste, and so on) and past construction (for example, tram tracks, concrete foundations). Details of some of these may not be contained in historical records or such records may not even exist. GPR is one of the only methods able to detect non-metallic items such as fibre optic or polyvinyl chloride / poly-pipe.

The following applications involve a further degree of complexity and understanding of GPR theory and may require specific GPR systems and expertise, but have been shown to be possible using GPR:

- location of 'wet areas' within a pavement
- detection of stripping defects within asphalt
- detection of voids under concrete slabs in concrete pavements, and/or
- estimation of air-void content in new asphalt works. This requires the use of an air-coupled antenna (refer to the work by Saarenketo and Scullion (Saarenketo, 2000)).

2.8.6.1 Preconstruction

If used to aid preconstruction activities, such as pavement rehabilitation investigations, it is recommended the GPR investigation be undertaken as early as possible. The GPR data can then be used to:

- verify or challenge the accuracy of existing records
- determine the location and extent of 'consistent' sections of existing pavement; from this information, the number and location of trenches / cores can be determined on a more informed basis, maximising the effectiveness of these investigations – anomalous areas can be located and either avoided (if trying to locate representative areas) or investigated further (if warranted).
- use estimated pavement layer thicknesses calibrated against physical measurements in conjunction with deflection data to facilitate mechanistic back analysis (this may provide greater overall confidence in the results of the back analysis)
- compare the GPR results with visual surveys, deflection results or roughness results to see if anomalies detected by GPR correlate with the pavement's appearance, deflection properties or roughness respectively, and/or
- verify or challenge assumptions as to the causes of pavement failure mechanisms (for example, detecting strong reflections indicative of excess moisture within a pavement may lead to conclusions regarding the rehabilitation strategy, detecting soft spots in subgrade may explain surface failures, and so on).

2.8.6.2 Construction and/or post-construction

GPR surveys should be undertaken early, thereby allowing construction personnel to act on the results. GPR surveys may be used to:

- locate changes of construction and subgrade anomalies (this can assist personnel in making informed selections about the location and extent of pavement lots, and in scheduling the project works)
- help determine whether the pavement in the shoulder is adequate for trafficking as part of temporary traffic diversions
- locate the extent of anomalies, such as cementitiously modified / stabilised patches or asphalt plugs, to be removed / replaced, targeted or avoided

- verify the location and depth of public utility plant (GPR is one of the only methods able to detect non-metallic services such as fibre optic or polyvinyl chloride / poly-pipe)
- determine the extent / location of boulders or other subgrade obstacles (this may allow more appropriate machinery to be selected early on, with less guesswork on extents of 'unsuitable' materials and associated delays / costs)
- detect, track and determine the size of air voids such as those often found under bridge-relieving slabs or adjacent to culverts, and/or
- detect the location of previous trenching (GPR can often differentiate materials disturbed previously from surrounding undisturbed materials).

In summary, there is a wide range of applications that can be undertaken using GPR with respect to road pavements. This list presents some of the more common applications; however, there are many more potential applications that may be considered.

2.8.7 Inappropriate uses

GPR is inappropriate to use for:

- determining pavement strength – GPR operation is based on the travel time of electromagnetic waves through pavements (mechanical wave methods, such as ultrasound, which are related to material stiffness can be used to infer strength)
- determining density
- identifying boundaries between layers or material types precisely; for example, it may be difficult to decide whether a boundary is the subgrade-sub-base interface with certainty (while the strength and depth may indicate it is most probably the subgrade-sub-base interface, a core or the excavation of a trench or test pit is required to confirm the prediction; however, once a layer is positively identified, it can usually be tracked quite reliably with the GPR)
- penetrating metals – its ability to penetrate other materials decreases rapidly with increasing conductivity; for example, the penetration depth is usually very shallow in wet clays, or where there is a lot of moisture and ions in the pore-water solution
- measuring compaction of fill, and/or
- determining moisture content.

The velocity of electromagnetic waves is dependent on the relative electrical permittivity, also referred to as dielectric constant (ε_r), of the material. Electromagnetic waves travel faster in air than in pavement materials.

A sound understanding of their behaviour is required when interpreting the results of a GPR survey; for example, compaction can influence the GPR response (the lower the compaction, the more air, the faster the travel time and quicker the reflection arrives back at the antenna). By contrast, electromagnetic waves travel slower in locations where conductivity is increased, such as where there is an increased moisture content.

While it is true compaction will affect the GPR response, other factors such as high moisture content may produce the opposite effect (compared to when compaction is low). These interactions need to be understood by the GPR interpreter when undertaking his or her analysis.

2.8.8 Investigation considerations

The following should be considered prior to undertaking the GPR investigation.

2.8.8.1 Identify what is being looked for and where is it likely to be?

It is important to define clearly the size, likely depth range and material type of the defect or object being targeted. This may determine what GPR system is to be used. Further, the GPR system will be initialised with settings (pulse spacing, depth of penetration, and so on), based on the expected properties of the target. It is also important to consider the properties of the surrounding material and, in some cases, the target's geometry. GPR relies on bouncing waves off a target different, in some way, from its surrounds. If the target is too small or electrically very similar to the surrounding material, very little signal may reflect, making detection difficult.

2.8.8.2 How slow can the ground penetrating radar go / what is the sampling frequency required?

The faster GPR units travel, the greater the interval between readings. This may or may not be acceptable, depending on the type of targets to be detected. In most systems, there is an inherent trade-off between speed and detail. This needs to be considered, based on the objectives of the investigation.

2.8.8.3 What is needed as an output?

GPR is usually displayed as a real-time output on a screen and is also recorded on a hard disk for further processing. Once processed, it can be displayed as a printout of the radargrams, or a GPR interpreter can use software to identify layers and output this into a drawing package, producing output as shown in the lower portion of Figure 2.8.4(a). The simplest for those unfamiliar with GPR is the most time consuming and, therefore, most expensive to produce. As a minimum, it is recommended a hard copy of the radargrams be requested, along with the raw and processed data (for future reference or in case further analysis is needed).

2.8.8.4 Who can explain ground penetrating radar?

On first appearance, GPR data may be quite confusing; it is recommended a person experienced in GPR investigations reviews the results and explains the findings. Once familiar with this sort of output, the expert advice may not be necessary for simple projects but may still be needed for complex projects or where the target is challenging to detect.

2.8.8.5 Expected performance and other comments

GPR inspections are usually successful in investigating most types of pavements. The depth of penetration can range up to around 0.5m with a ground-coupled 1.5 GHz antenna, 3–4m with a larger 400 MHz antenna, and even deeper with lower-frequency antennae.

Note: While using lower frequencies results in greater penetration, the price of their use will be reduced resolution (it will only be possible to detect / distinguish between larger objects).

2.8.8.6 Further reading

Relevant publications include:

- *Electrical properties of road materials and subgrade soils and the use of ground penetrating radar in traffic infrastructure surveys* (Saarenketo, 2006).
- *Trial Ground Penetrating Radar (GPR) Pavement Investigations in Border District* (Muller, 2001).
- *Road evaluation with ground penetrating radar* (Saarenketo, 2000).

2.9 Test pits

This type of evaluation is invariably used to calibrate deflection testing and GPR results.

Excavation is disruptive to traffic and is relatively expensive when compared with other types of investigation.

The type of excavation and equipment required is dependent on the type of material likely to be encountered and information being sought.

2.9.1 Core

This method is used to obtain information on the layer type, thickness and condition of bound and/or rigid pavements containing asphalt, stabilised / modified base and/or concrete. Figure 2.9.1(a) shows a thick asphalt core. Cores taken are normally 100mm or 150mm in diameter; however, larger cores up to 500mm may be taken depending on the purpose of the investigation. Figure 2.9.1(b) shows the typical coring equipment with a 480mm diameter core barrel attached. Cores greater than 150mm should only be undertaken on thin asphalt layers less than 100mm thick as thick layers are too heavy for manual retrieval of the core. In addition to information about layer thickness, coring also allows for the inspection and verification of surface crack propagation and condition of interlayer joints. Figure 2.9.1(c) shows the presence of moisture at the different layer interface. Coring cannot be undertaken on unbound gravel base.

Figure 2.9.1 – Coring

(a) – Thick asphalt core	(b) – Equipment	(c) – Moisture
		

2.9.2 Auger

This method is used to obtain information on the pavement layer profile, condition and to collect samples of the existing pavement material down to and including the natural subgrade. Augers used for investigations are normally between 350mm–450mm attached to an appropriately-sized excavator capable of cutting through the expected material type. Figure 2.9.2(a) shows the augering investigation.

The augering process is undertaken slowly, in layers, with attention to the material being excavated. Experience and extra care are required to prevent cross-contamination between materials from each layer. A limiting factor of this type of investigation is the volume of material that can be sampled is limited by the size of the auger and the thickness of the layer. Figure 2.9.2(b) shows the test pit marked out to identify the various layers. When the subgrade is reached, dynamic cone penetrometer (DCP) tests may be undertaken to estimate the insitu CBR of the natural subgrade.

Augering provides information on the pavement materials at the point of excavation but may not represent the material across the whole road cross-section. Accordingly, when undertaking this type of investigation, it would be desirable to undertake a series of auger excavation at staggered locations.

Figure 2.9.2 – Augering

(a) – Auger	(b) – Test pit with marked layers
	

2.9.3 Trench

This method is used to obtain information on the existing pavement layer. The advantage of a trench is the ability to evaluate the pavement profile across the cross-section of a lane. The investigation may require the use of several plant items. A skid steer road profiler would be required to cut and remove the asphalt surface and any heavily-bound base layer. Thin asphalts and sprayed seals can be removed by saw cutting and the use of an excavator. Lightly-bound base layer may also be removed using an excavator.

Care must be taken to excavate the pavement material in thin layers down to the natural subgrade, to prevent cross-contamination between pavement layers.

Trench investigation allows an in-depth visual evaluation of the uniformity of the pavement profile across the road section, its thickness, condition, appearance and field moisture content. It allows for the sampling of the pavement material at the desired location based on the visual examination of the pavement profile. This type of investigation allows for the collection of a significant volume of material from each pavement layer.

When the subgrade is reached, DCP tests can be undertaken to estimate the insitu CBR of the natural subgrade.

For safety reasons and to prevent risk of collapse due to localised vehicle movement, the investigation trench must not exceed 1.5m below the existing finished surface level unless appropriate trench collapse prevention support is in place.

Figure 2.9.3 – Trenching

(a) – Excavator	(b) – Trench with marked layers
	

2.9.4 Reinstatement of test pits

The full depth of the base, sub-base and the subgrade material shall be reinstated to match the existing layer materials and thicknesses as follows:

- The subgrade shall be backfilled using only good-quality excavated base, sub-base and imported granular material. The reinstatement shall be machine compacted to a maximum finished layer thickness of 150mm, and 97% of Maximum Dry Density (refer to Technical Specification MRTS04 *General Earthworks* (MRTS04), Clause 15 *Compaction*).

  Note:

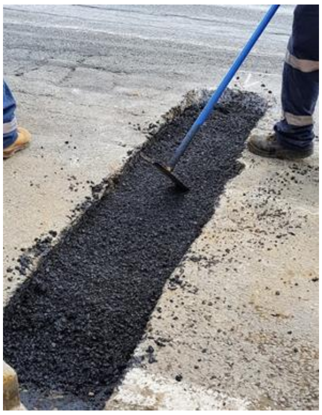
  1. Good quality excavated subgrade material may be used in the reinstatement of subgrade layer if approved by the Transport and Main Roads supervisor.
  2. If any of the excavated material is not suitable to be compacted, it shall be removed from site.

- Reinstatement of granular material layer shall be backfilled with plant-mixed cement modified base (unconfined compressive strength (UCS) 1.5 MPa in 28 days) and placed in accordance with Technical Specification MRTS05 *Unbound Pavements* (MRTS05).
- Reinstatement of cemented material layer shall be backfilled with Lean Mix Concrete (no slump, target 5 MPa), placed and compacted in layers of maximum thickness of 150mm (refer Technical Specification MRTS39 *Lean Mix Concrete Sub-base for Pavements* (MRTS39)).

- Reinstatement of surfacing shall be with hot-mix asphalt conforming to AC14 (A15E) asphalt and placed in accordance with Technical Specification MRTS30 *Asphalt Pavements* (MRTS30).
- Prior to placement of asphalt, the surface shall be brushed clean of any loose material and finished in a manner which shall promote adhesion of bituminous emulsion to the surface. A bitumen emulsion tack coat shall be applied to the bottom and sides of the trench / auger (refer to Technical Specification MRTS12 *Sprayed Bituminous Emulsion Surfacing* (MRTS12) Clause 10 and Figure 2.9.4(e)).
- Asphalt shall be placed in accordance with MRTS30 in two compacted layers, each of 50mm maximum thickness.
- All the backfill shall be machine compacted.

For reinstatement of utility trenches, where protection of underlying services is required, refer Technical Note TN163 *Third party utility infrastructure installation in state-controlled roads technical guideline* (TN163).

Figure 2.9.4 – Trench reinstatement

(a) – Subgrade reinstatement, compaction using a pad foot roller attachment	(b) – Final surface preparation of subgrade using an upright mechanised compactor	(c) – Placement of lean mix concrete to prepared subgrade
		
(d) – Final layer surface preparation of lean mix concrete with a plate compactor	(e) – Bitumen emulsion applied to top of finished lean mix concrete & sides of existing asphalt	(f) – Hot mix asphalt being hand placed
		
(g) – Bottom layer of asphalt compacted with a upright mechanised compactor	(h) – Final compaction and finishing with a plate compactor	
		

2.9.5 Selection of location and extent of test pits

The preferred location of test pits has traditionally been based on a variety of information including field observations, historical records, GPR and deflection results. The methods employed largely relied on identification of broad homogenous sections with the midpoint of such sections adopted as the test pit location.

The statistical cluster analysis (SCA) method, described in Appendix A of this *Manual,* allows a more informed decision on the location and extent of excavations. Specifically, the SCA method provides a tool with which to minimise the number of test pits, while retaining adequate samples of predominant pavement structures.

Although the SCA method was developed primarily to classify deflection data for back analysis (*Guideline: Statistical cluster analysis of deflection data for pavement structural identification and design*), use of broader section definitions with relaxed discriminants can lead to the identification of similar pavement structures across the entire project and reduce the number of test pits.

The process begins by classifying the deflection data as a single road section and site. GNSS coordinates provide the primary reference to individual bowls; however, to reuse the process developed for back analysis, lanes must remain unique and should be assigned a sequence number. Wheel paths used for back analysis (L, R, I or O) remain unaltered.

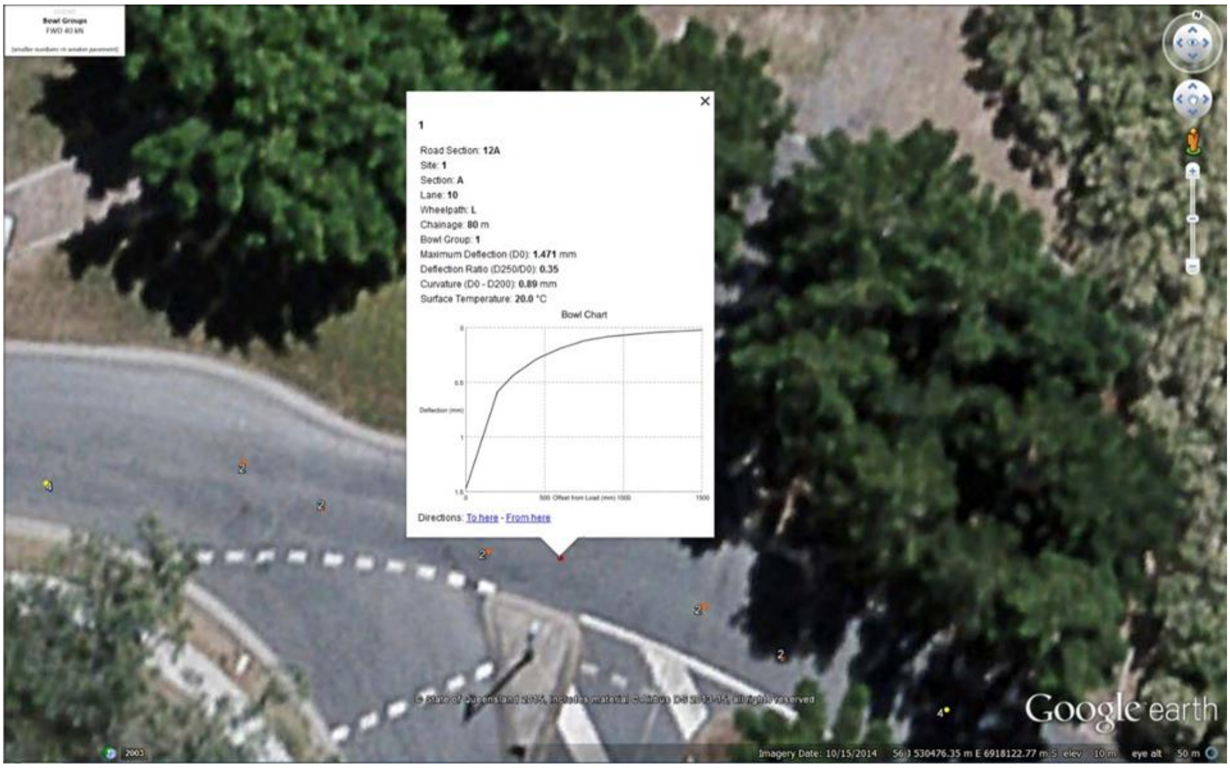
Because of the revised use of road section, site and lane classifiers, the geospatial charts and bowl chart thumbnails used for back analysis are not especially useful. Instead, the GNSS coordinates are used to generate a geographic information system (GIS)-compatible product called the 'Bowl Group kmz file' (intended for use within Google Earth). This file contains all deflection bowls in the project separated into bowl groups which indicate structural similarity. Figure 2.9.5(a) and Figure 2.9.5(b) provide an example of the use of the kmz file within Google Earth. In this example, the FWD data were collected from two lanes; however, the SCA method has been applied to the entire project. As a result, the similarity or otherwise of pavement structures on the site is easily identified.

In Figure 2.9.5(a), the bowl group labels are numbered in sequence based on the maximum deflection, with lower numbers corresponding to weaker pavements. This sequence is also reflected in the colour of the bowl icon, with the red end of the spectrum representing weaker bowl groups, and the blue end, stronger bowl groups.

Figure 2.9.5(a) – Bowl groups in Google Earth

In addition to broad structural details, the 'SCA kmz file' contains individual bowl details which can be accessed by clicking on the bowl icon. Figure 2.9.5(b) provides an example of the details for bowl group 1, the weakest bowl on the site. The maximum deflection of nearly 1.5mm at 40 kN and bowl shape suggest pavement in this vicinity is on the verge of failure; indeed, examination of the imagery suggests the presence of pothole repairs (this defect and others were later confirmed during a site visit.) The site, section and lane relate to the entire project; however, wheel paths are as reported in the deflection data.

Figure 2.9.5(b) – Bowl details



Details on how to generate the 'Bowl Group kmz file' are contained in Appendix A in this *Manual*.

2.10 Project brief

The brief for the pavement investigation, or the project, should be referenced when evaluating a pavement: Pertinent information may include:

- information about the project, including the Transport and Main Roads Road ID, as the pavement investigation may be one part of a much larger project
- project schedule / timeline
- the year of opening
- the design period to be used for pavement rehabilitation designs
- forecast traffic
- extents of the investigation
- details about the scope of the investigation, and
- project constraints (see also Section 2.12.3).

If this information is not contained in the brief, then the designer should seek it from the customer, or obtain it from Transport and Main Roads.

Where the designer has not received a brief, it is recommended the designer prepare a detailed written project proposal and use this to negotiate and agree a written brief (for the pavement rehabilitation investigation) with the customer.

It is important the scope of the investigation does not place undue limits on the investigation as this may preclude treatments that may become viable as the investigations progress (for example, do not rule out stabilisation early in the investigation).

2.10.1 Level of detail required

Projects go through several phases as they develop. In Transport and Main Roads nomenclature, these are (*Project Management Practices Guideline (*PMPG)):

- concept phase that includes options analysis and the development of a business case
- development phase that includes preliminary design and detailed design
- implementation phase that includes construction, and
- finalisation phase that includes handover.

The purpose and level of detail required, including the accuracy of cost estimates, varies in each phase. Given this, the designer should confirm the phase, purpose and level of detail required with the customer at the start of the rehabilitation investigation.

Further information about project phases can be found in the department's *Preconstruction Processes Manual* (PPM).

2.11 Future planning

The road may be subject to change in the future; for example, a complete realignment of a road may be planned in ten years' time. In this example, the future works may affect the design period to be used and the treatments options to be explored. The designer should, therefore, investigate what future planning exists, assess how it will affect the investigation and consider this effect when developing options. Some of this information may be contained in the project brief (see Section 2.10).

2.12 Other information

Additional information can be gathered from field (site) inspections or other organisations. In the case of the former, this can be done in conjunction with a visual assessment of pavement condition (refer to Section 2.7) and/or via one or more visits to the site. Ideally, such information will be collected towards the start of an investigation and before testing of the pavement occurs.

The following information should be collected:

- topography, geology and pedology
- climate
- road geometry and cross-section information
- project constraints
- types, condition and location of drainage
- types and locations of utilities / services, and
- the nature of land use adjacent to the road.

2.12.1 Topography, geology and pedology

Refer to AGPT05 Section 3.3.2 *Site and Environment Information*. Following are examples relating pavement distress to other features:

- the water associated with swampy areas may cause weakening of the subgrade, resulting in shape loss, rutting and so on, and/or
- cut / fill interfaces are often associated with poor subsurface drainage reflected in shape loss, rutting and cracking.

2.12.2 Road geometry and cross-section

Where possible, information about the road’s horizontal and vertical alignment, as well as cross-sectional information, including type cross-sections, should be collected. Primary references for this information are drawings / plans from past road projects (refer to Section 2.6.3). For pavement rehabilitation investigations, the following details should be noted for the current road and its proposed future states:

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

In some cases, a geometric assessment will be required as part of the overall project (refer to Section 2.2). Normally, the designer undertaking the pavement rehabilitation investigation is concerned with information affecting pavement performance. Assessment of the adequacy of geometry or drainage, except pavement drainage, is not usually within scope of a pavement / pavement rehabilitation designer’s investigation. If assessment of the geometry or drainage is required, it is recommended a specialist designer, or designers, be engaged to undertake this part of the investigation.

The driver of a rehabilitation investigation may not necessarily be pavement distress. Changes required because of deficient geometry are also a common driver for a road upgrade (refer to Chapter One for a discussion about 'synergies and opportunities'). In such circumstances, possible alterations to the road geometry need to be considered when designing the rehabilitation treatment. In addition, alterations to the geometry of a road may impose constraints on rehabilitation options (for example, tolerable increase in surface heights).

2.12.3 Project constraints

Project constraints should be identified in the early stages of the investigation. This includes the type and extent of the constraints. They may be identified in the Project Brief, through meetings with the region or district and/or during a visual inspection of the road. Potential constraints may include:

- limitations / restrictions on increases in pavement surface levels to:
  - maintain current vertical clearances
  - avoid raising kerb, kerb and channel, drainage gullies and/or or road safety barrier
  - ensure the loadings on structures do not increase, or
  - avoid formation widening required because of an increase of surface heights (for example, due to an overlay)
- the availability of materials, plant / equipment, and so on that may affect the viability of certain options
- regional or district policies, practices or experiences
- requirements for re-establishing access to adjoining properties, and/or
- existing or planned utilities / services plant (for example, underground services / utilities, transmission lines).

The designer should not assume a current constraint will remain or a perceived constraint is a constraint to the region or district (for example, the region or district may be planning or willing to raise a road safety barrier). Constraints should be confirmed with the region or district before the development of rehabilitation options commences.

2.12.4 Drainage

The influence, and potential influence, of surface and the subsurface water must be considered when assessing an existing pavement and designing rehabilitation treatments. Often a large portion of expenditure on pavement rehabilitation is related, at least in part, to moisture-related pavement distress.

It is often possible to identify moisture-related distress in the existing pavement during a visual survey. Additional field or laboratory testing can also reveal where moisture is relatively high and how this affects pavement performance. The cause of any identified moisture-related distress needs to be addressed if any rehabilitation treatment is to achieve its intended purpose. Examples of distress caused by the ingress of water are:

- in rigid (concrete) pavements, pumping, the formation of voids, cracking, joint deterioration and corner breaks, and
- in pavements with asphalt layers, stripping, rutting, loss of surface shape (depressions), fatigue cracking and potholes.

In some cases, a drainage assessment will be required as part of the overall project (refer to Section 2). 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 a specialist drainage designer, or designers, be engaged to undertake this part of the investigation.

Refer to AGPT05 Section 3.3.2 *Site and Environment Information* for further discussion.

2.12.5 Utilities / services

Information on utilities / services can be important for several reasons, namely:

- so field investigations can be undertaken safely, without damaging utilities / services and without disrupting utilities / services
- to understand the reasons for existing pavement conditions, and
- to plan any rehabilitation treatment.

It is recommended:

- utilities / services information be obtained early and before any excavations are undertaken
- the 'Dial Before You Dig' service be accessed to this end, and
- a professional services locator be employed.

Note: 'Dial Before You Dig' service is not the only information source.

Refer also to AGPT05 Section 3.3.2 *Site and Environment Information*.

2.12.6 Land use

The nature of current and future land use adjacent to the road can help in assessing the current state of the road and designing future rehabilitation treatments. Specifically, land use can affect traffic volumes. Ideally, traffic forecasts will allow for changes in land use along the road corridor and within the road system / network.

2.13 Pavement condition / distress and testing

AGPT05 Section 4 and Appendices A and B should be referenced for much of the information about this subject. The following sections supplement and take precedence over AGPT05.

2.13.1 Intervention levels

General intervention levels for maintenance are contained in the department's *Routine Maintenance Guidelines* (RMG). These levels have been largely developed by considering safety, community expectations, research, and standards adopted by other road authorities. The department's *Road Maintenance Performance Contract* (RMPC) manual permits these ‘general intervention levels’ to be varied for individual roads by way of a documented agreement between the parties to the contract, with the clear indication the ‘safety of road users is not compromised’ by such agreement.

2.13.2 Investigatory levels

Where appropriate, information about investigatory levels is given in the relevant section following. AGPT05 also provides some general guidance.

2.13.3 Structural condition / capacity

The objective of a structural evaluation is to determine the present structural condition of a pavement and its capacity to withstand traffic and environmental forces over the design period. It is also the foundation for the design of any (structural) rehabilitation treatments that may be required.

As structural capacity cannot be determined directly, several indirect indicators are used. These include:

- defects observable during a visual survey (for example, rutting, cracking and shoving, see Section 2.7)
- structural response to load (deflection data, see Section 2.13.10)
- properties of pavement and subgrade materials (for example, as determined from field sampling and laboratory testing), and
- moisture-related defects as noted in AGPT05 Section 3.3.2 *Site and Environment Information*.

2.13.4 Roughness

Roughness is used to represent the riding quality of a pavement and can be an indicator of the serviceability and/or structural condition of a pavement. It is influenced by surface irregularities, distortions and deformations. The roughness of a pavement usually increases with time from initial construction to ultimate retirement. As roughness increases, the structural condition of the pavement decreases. In addition, as roughness increases, so too does the dynamic pavement loading.

Note: Roughness can develop from loading of the pavement and from other factors (for example, from material volume changes associated with moisture changes). Identifying the cause/s of roughness can be critical with respect to selecting an appropriate rehabilitation treatment.

The department uses both international roughness index (IRI) and National Association of Australian Road Authorities (NAASRA) roughness counts to report roughness. A two-laser vehicle mounted profilometer (*Test Method Q708B Road roughness – surface evenness – two laser profilometer* (Q708B)) should be used to measure roughness, although, depending on the circumstances, less productive devices such as a static level and staff (*Test Method A708C: Road roughness – surface evenness – static level and staff* (Q708C)) or a walking profilometer (*Test Method Q708D: Road roughness – surface evenness – ARRB walking profiler* (Q708D)) may be permitted (MRTS30). The results of network-level surveys collected using a VMP are typically reported in IRI for both wheel paths and the lane (Quarter Car), with NAASRA counts estimated from the Lane IRI.

In accordance with the recommendations of the Austroads *Guide to Asset Management Part 15: Technical Supplements* (AGAM15), Lane IRI shall be calculated using the Quarter Car (IRI Averaging), method. If required, the NAASRA count may then be estimated from the Lane IRI using the formula ($NRM = 26.5 \times \text{Lane } IRI_{qc} - 1.27$) indicated in AGPT05.

Statewide roughness surveys are conducted annually (*Pavement Condition Data Collection Policy* (PCDCP)). Network survey results can be accessed via ARMIS; they are typically presented graphically (for example, in *Chartview* software using ARMIS data).

2.13.4.1 Test procedures and measures

Transport and Main Roads undertakes network and project level road roughness data collection through external service providers using a VMP.

Calibration and validation of roughness using a VMP shall be carried out in accordance with Q708B.

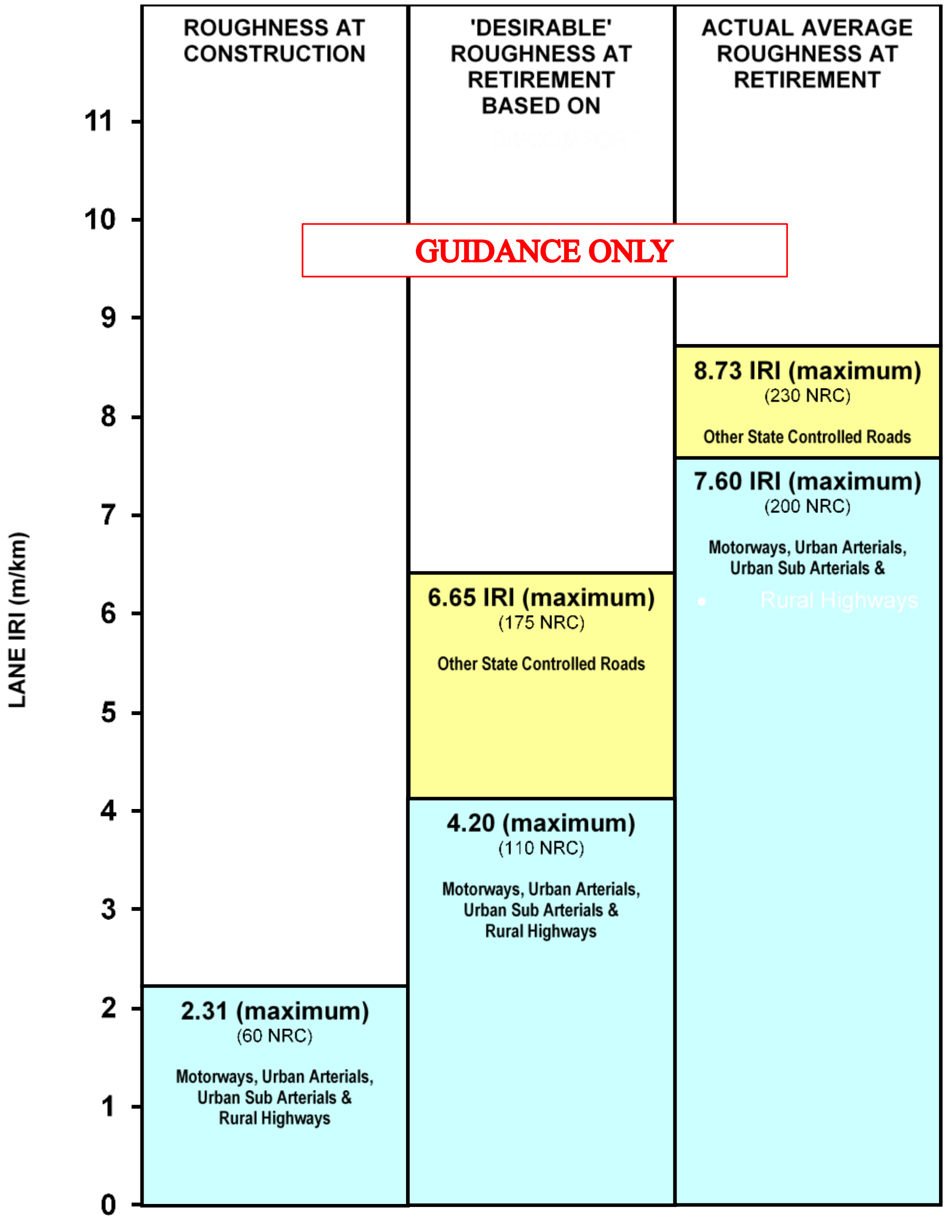
Network-level results are reported in ARMIS at 100m intervals; however, through individual requests, data can be processed and reported for road segments of a specific length. A section with a length of

100m is typically used (for example, reported in ARMIS), although a higher or lower level of aggregation may be used in some circumstances.

Figure 2.13.4.1 shows various roughness values throughout the life of the pavement for various road categories, from initial construction to the desirable maximum average roughness prior to reconstruction. These limits are provided as a guide to the condition of the pavement with respect to riding quality and are not necessarily an indication of structural integrity (AGPT05). The values in the figure are not mandatory limits to in-service roughness (reaching the value does not mean reconstruction must be undertaken), they are to be used as a guide only.

For pavements with measured roughness below those given in Figure 2.13.4.1, 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.

Figure 2.13.4.1 – Roughness: a guide indicating the functional adequacy of the pavement



2.13.5 Geometric form

The geometry of a road includes its horizontal and vertical alignment, cross-section widths and shape. Cross-sectional detail includes the application of superelevation, crossfalls, the width of cross-section elements (for example, lanes, shoulders, verges) and subsurface drainage. Surface drainage is influenced by geometry (refer to Section 2.13.8).

The implementation of a rehabilitation treatment may be an opportune time to review the geometry and vice versa. Chapter 1 outlines how designers should be aware of 'synergies and opportunities' and how the department's regions or districts may be able to take advantage of them.

Reasons why the road geometry may need to be reviewed include:

- changes in land use (for example, increase in traffic, change function of road, change in road user or vehicle types)
- an increase in traffic so the volume of traffic exceeds the road's capacity, or alters road function or road user expectations
- edge break, reducing the effective seal width (for example, Figure 2.13.5)
- interim widenings creating changes in crossfall exceeding those permitted in design standards (for example, crossover crown issues)
- changes in design standards
- surface drainage is inadequate (refer to Section 2.13.8), and
- the upgrade of adjoining sections making the standard of the section of road under consideration inconsistent with the design standard of adjoining sections.

Figure 2.13.5 – Edge break (which can reduce seal width)

2.13.5.1 Test procedures and measures

There are no test procedures for road geometry but there are certain applicable design standards. For road geometry, the primary references include the RPDM3-03 and the AGRD03. To determine whether the road complies with these standards, details of the geometry will need to be collected and assessed (for example, refer to Section 2.12.2). It may require a model of the road and ground surfaces and properties to be developed through a survey or other similar means. A specialist geometric designer may need to be engaged to assess the geometry of the road.

2.13.6 Noise and vibration

Noise is defined by the *Transport Noise Management Code of Practice* (TNMCP) as unwanted sound transmitted through air or another medium. Sound is defined as any pressure variation (in air, water or some other medium) human ears can detect.

The TNMCP details how noise and vibration are managed by the department, including test procedures, measures, and causes.

2.13.7 Skid resistance and surface texture

> *Skid resistance is a condition parameter characterising the contribution that a road surface makes to the level of friction ('grip') between a road surface and a vehicle tyre.*
>
> *Skid Resistance Management Plan (SRMP)*

The level of skid resistance available depends on, among other things, the micro-texture of the aggregate, the macro-texture of the road surface (surface texture) and the thickness of any water film on the road surface. Other factors that can influence the level of skid resistance available are discussed in Section 2.13.7.2.

Studies of traffic crash histories around the world have consistently found a disproportionate number of crashes occur where the road surface has a low level of surface friction and/or surface texture, particularly when the road surface is wet (SRMP).

Skid resistance is one of many factors that may influence the risk of crashes (for example, other factors include driver behaviour, geometry and vehicle characteristics).

The following two sections are general in nature. For a detailed discussion about skid resistance and surface texture, reference should be made to the following documents:

- for details about the management of skid resistance by the department, refer to the SRMP
- Technical Specification MRTS101 *Aggregates for Asphalt* (including Annexure) (MRTS101) contains polished aggregate friction value (PAFV) requirements for asphalt
- project specific requirements for major projects which usually include texture and skid resistance requirements
- a technical note is being prepared to provide further details on PAFV and texture requirements
- a future edition of the PDS will refer to AGPT03 and Technical Note TN175 *Selection and Design of Sprayed Bituminous Treatments* for guidance on surfacing selection
- until future publications become available, the department's Pavements, Materials and Geotechnical Directorate shall be contacted for advice about appropriate PAFV, surface texture values and test methods.

These documents also contain further references to which the designer may refer.

2.13.7.1 Test procedures and measures

Skid resistance testing is always invariably undertaken on a wet road surface to give a 'worse-case scenario' at a location. Testing in dry conditions is very rare and would only typically be considered where it would be prudent as part of an incident investigation (SRMP).

For its routine network-level testing, Transport and Main Roads uses a sideways-force coefficient routine investigation machine, the testing services of which shall be procured by Transport and Main Roads from a reliable and experienced operator and service provider.

For both reactive, and research and development testing, a range of devices may be used.

Vehicle mounted devices include:

- sideways-force coefficient routine investigation machine
- ViaFriction
- Griptester, and
- Dynamometer (Vericom)*.

Light weight portable devices include:

- British Standard Pendulum (BSP).

*Note: While the dynamometer is widely used by police and crash investigators, because of work health and safety (WH&S) considerations, it commonly operates on a dry road surface (AGAM15), and a test method which includes for control of water film thickness is not available. This device is not suitable for recording continuous skid resistance measurements.

Detailed technical information regarding these test devices and their operation is not provided in this *Manual*, but can be obtained from the SRMP and/or from the following Austroads documents (all devices):

- AGAM15
- *Guidance for the Development of Policy to Manage Skid Resistance* (AP-R374), and
- *Review of Skid Resistance and Measurement Methods* (AP-T177).

Note: Each of these devices measures slightly different things and produces its own unique output; hence, the results obtained for a site from one device cannot be compared directly to results obtained using another device.

Transport and Main Roads will procure network-level testing using a sideways-force coefficient routine investigation machine with two test pods / wheels, the device collects test data from the left and right wheel paths concurrently. This enables monitoring of a parameter known as the differential friction level which is simply the numerical difference between the sideways-force coefficient routine investigation machine test value obtained in the left and right wheel paths. The differential friction level can be of interest during an incident investigation, as it is often held to be an important parameter in terms of overall vehicle stability.

The sideways coefficient routine investigation machine's basic constituents are a heavy vehicle (truck) cab and chassis with an integrated water tanker body, test pod/s (which can be as required) and instrumentation. Each test pod contains a test wheel held at an angle of 20º to the longitudinal axis of its normal travelling wheels (this is known as the 'yaw angle'). The test wheel is free to rotate during testing but is restrained by the yaw angle. A consistent vertical load of 200 kg, which is independent of the movement of the truck chassis, is also applied to the test wheel. A machine has at least one test pod to test one wheel path.

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. Surface (water) spray and the noise generated from the tyre-road interaction are also affected by surface texture as described in the AGPT03.

For roads controlled by the department, the surface texture is determined using at least one of the following methods:

- by converting the results of a vehicle mounted profilometer survey to an equivalent sand patch texture depth (SPTD)
- by converting the results of a Transport Research Laboratory texture meter survey to an equivalent SPTD (Technical Specification MRTS40 *Concrete Pavement Base* (MRTS40), *Test Method T192 – Determination of the texture depth of road surfacing by the TRL mini texture meter* (T192)) and,
- the 'sand patch' test with testing undertaken in accordance with Austroads *Modified Surface Texture Depth (Pestle Method)* (AGPT-T250).

Portable texture measurement devices such as the laser texture scanner (LTS) or circular track meter (CTMeter) (*Standard Test Method for Measuring Pavement Macrotexture Properties Using the Circular Track Meter* (ASTM E2157)) may be used where sand patch tests are likely to be impractical (for example, porous or open graded surface material, or where the texture depth is too small to be measured reliably by the sand patch method).

The LTS offers several advantages over the CTMeter: it is more compact, lighter (four versus 13 kg), does not require to be tethered to a power source and records the location of the test from an embedded GNSS receiver. In addition to being able to measure macro-texture, the LTS is often able to measure into the micro-texture range, which may eventually allow the results to be used to estimate skid resistance. A test method for the LTS has yet to be developed.

Texture depth measured by the sand patch test is expressed in millimetres. Some laser profilometers can measure texture depths expressing results as mean profile depths or, by use of correlation factors, as equivalent SPTD. The use of equivalent SPTDs derived from laser profilometers should be used with care.

For a project level investigation including skid resistance, texture measurements shall also be collected to allow the adjustment of skid resistance for slip speed to be carried out.

Refer to AGAM15 for further information.

2.13.7.2 Causes

The available surface friction depends on a multitude of factors including vehicle speed, surface texture, water depth, tyre characteristics, tyre condition, vehicle suspension characteristics, distribution of mass, seasonal influences and road geometry (SRMP). In the case of the latter, how superelevation is applied, the degree of superelevation provided, and horizontal curve radius are important influences.

Table 4.4 of AGPT05 outlines some factors contributing to reduced skid resistance. Additional factors include:

- the accumulation of surface water (for example, from poor surface drainage or depressions)
- surface contamination (for example, debris filling voids of open graded asphalt (OGA))
- pavement markings with low friction, and
- crack seals / sealants with low friction.

Other factors can exacerbate or otherwise contribute to reduced skid resistance such as:

- depressions (for example, ruts or shoves creating depressions)
- inappropriate geometry (for example, incorrect application of superelevation, the degree of superelevation provided is too low, a horizontal curve radius is too small)
- corrugations
- poor maintenance
- poor surface drainage (for example, from inappropriate drainage design, geometry or depressions)
- inadequate sight distances
- changing the shape of the pavement through a treatment or intervention without checking surface drainage (for example, checking for aquaplaning)
- inappropriate selection of a surfacing (for example, inadequate surface texture) and/or inappropriate materials used in a surfacing (for example, use aggregate with a PAFV less than recommended for a site), and
- changes to standards.

2.13.8 Surface drainage and water spray

How surface water is drained (surface drainage) will help determine water film thicknesses, water spread, and so on. It can be particularly difficult to solve surface drainage problems where superelevation is developed for horizontal curves and/or where grade and other geometric combinations result in long flow paths for surface water. Road drainage must be designed in accordance with the RDM and geometry must be designed in accordance with the RPDM3-03 (refer Section 2.2 of this *Manual*). The Austroads *Guide to Road Design Part 5A: Drainage – Road Surface, Networks, Basins and Subsurface* (AGRD05A) includes guidance on maximum allowable water film thicknesses for new roads. A suitably qualified designer, or designers, should be consulted to assess and solve surface drainage issues. This may require the surface of the road and road corridor to be captured (for example, via a detailed survey). A review of the crash history of a road may also reveal locations where surface drainage may be of concern.

Some surfacings like OGA can reduce water spray, particularly early in life (for example, when there are few, fine, or debris-filling voids in the OGA). The department has no objective test to measure water spray produced by vehicles and how it affects drivers; OGA is the best asphalt at reducing spray, followed by stone mastic asphaltic (SMA) concrete with dense graded asphalt (DGA) least able to reduce water spray (AGPT03).

For more information about the performance of surfacings, refer AGPT03. The department's Pavements Material and Geotechnical Directorate can also provide advice.

2.13.9 Defects observable during a visual assessment

A visual assessment of the existing pavement (refer to Section 2.7) can reveal distresses indicating the condition of the pavement.

A defect observed during a visual survey is evidence of an undesirable condition in a pavement. It may simply affect its serviceability, and/or it may indicate a lack of structural capacity. The most common such indicators are potholes and patches, rutting, cracking and shoving.

Appendix A of AGPT05 provides more details about distress types, including photographs.

2.13.9.1 Potholes and patches

Potholes provide a dramatic indication of pavement failure. They may be structural in nature, wholly related to the surfacing, or a combination of the two.

Patches are usually repairs to a pavement and can indicate where issues exist or are likely to occur in the future. Their size can vary from small patches (for example, a few square metres) to large / extensive patches (for example, full lane width for several hundred metres). In addition, their thickness can vary (for example, a surface repair 50mm deep versus a structural repair 150mm deep).

Patches, whether temporary expedients or more permanent restorations, may only be considered effective if they, and the pavement immediately adjoining them, have survived several seasonal cycles without failure recurring. Materials used in the construction of patches include asphalt, 'cold-mix' asphalt, crack sealants and (insitu or imported) modified / stabilised material – a combination of materials may be used in any one patch. Given the thickness and type of materials can vary, especially when compared to the balance of the pavement, the properties of patches can influence how rehabilitation options are implemented; for instance, a treatment incorporating foamed bitumen stabilisation of the existing pavement may require the removal of unsuitable patches such as those 150mm deep and constructed only of asphalt.

Analysing the location, frequency and distribution of patched failures will often assist in determining their cause. Undertaking a visual assessment (refer to Section 2.7), including mapping of patches, may be advantageous.

Provided the patch is tested, load-deflection testing will normally also indicate whether the condition is caused by a pavement strength (structural) deficiency or by a problem with the surfacing only (surfacing deficiency) (large patches are more likely to be included in deflection testing than small patches; accurate patch details (for example, location) are required to relate deflection results to patches).

2.13.9.2 Rutting

Rutting is a longitudinal deformation (depression) located in wheel paths and is most commonly found in flexible pavements. The layer(s) suffering the deformation will be evident from associated indicators or may be determined by inspection of test pits or trenches that reveal the pavement (cross-)section through (across) the rut(s).

In addition to its effect on serviceability, deformation in base layers may lead to a reduction in the effective pavement thickness and, if left untreated, to the premature development of deformation in the subgrade. This deformation may progress to shoving if the rutting becomes so severe, surface cracking occurs, and moisture enters and weakens the underlying layers and/or the subgrade.

To assist in identifying the cause of rutting, the existence or absence of associated shoving is an important attribute. For a flexible granular pavement, where ruts are wide and there is little or no evidence of shoving, it is more likely related to deformation at depth in the pavement (for example, at the subgrade level) because of either insufficient pavement strength and/or compaction of the pavement under traffic. In this case, 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 depths at the time of deflection testing. The higher the correlation of rut depth and deflection, the more likely the rutting is at least partly due to inadequate pavement strength. If rut depths do 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.

Where rutting is associated with shoving, it is usually indicative of the shear strength in the upper pavement layers being inadequate to withstand the applied traffic loads. In this case, there will be poor correlation between the severity of rut depth and measured deflections. Trenching across the full width of the lane(s) and/or asphalt coring within and between ruts can also be used to identify the critical pavement layer or layers. Laboratory testing of the affected materials sampled from the pavement will further assist in evaluating whether the shear deformation is related to deficiencies in the specification or standard for that material; deficiencies in construction; or the use of non-conforming material.

Finally, rutting (and shoving) can also disrupt / change surface water flows, and/or pond water leading to undesirable water (film) thicknesses that may subsequently lead to aquaplaning or loss of control of vehicles. The RDM provides guidance with respect to road drainage, including surface drainage. Where surface drainage is an issue, refer to the RDM and Section 2.13.8 in this *Manual*.

2.13.9.2.1 Test procedures and measures

Rutting is measured:

- as the maximum vertical displacement in the transverse profile (perpendicular to flow of traffic)
- across a single wheel path or both wheel paths within a traffic lane, and
- relative to a reference datum.

It can be measured in each direction.

Various measures are available including:

- deviations from a 1.2 m long straight edge (for example, measured during a visual assessment), and/or
- rutting calculated using the results of a VMP survey (typically, the results calculate an equivalent rut depth such as from a point on a 2m straight edge).

The 1.2m straight edge or equivalent results are used by Transport and Main Roads during visual assessments.

Automated methods are being used increasingly; however, manual measurements can still have a place in project level investigations – for example, manual measurements can be used to validate automated measurements, to check specific locations (spot checks) or to compare current rut depths against those recorded in the last automated survey (which may have occurred some time ago).

The results from automated methods can be processed and reported for road segments of a specific length (for example, average rut depth over section length, percentage of section with a rut depth >10mm). Usually, statewide rutting surveys are conducted annually (PCDCP). Network survey results can be accessed via ARMIS and they can be presented graphically (for example, in *Chartview* software using ARMIS data).

Refer to AGAM15 for further details.

2.13.9.2.2 Causes

Rutting may occur because of permanent deformation in granular bases, asphalt surfacings or bases, in the subgrade, or in a combination of these. Where excess bitumen has been placed in a seal, or many seals have been placed over time, resulting in a significant total thickness of seals, rutting may occur in the seal/s. Back analysis of deflection data may reveal pavement layers with an unusually low modulus, and therefore, a potential cause of rutting.

Deformation of a granular base could be a consequence of very high (volume and/or mass) traffic loading, poor material quality, excessive moisture or inadequate compaction / construction. Deformation of an asphalt base may be a result of very high traffic loading (for example, >107 equivalent standard axles (ESAs)), inappropriate mix design, inappropriate asphalt selection, interaction between layers (for example, excess cutter in a priming seal penetrating an overlying asphalt layer) or inadequate compaction. The evaluation should consider, and focus materials testing on, these aspects.

One criterion used as part of the pavement design procedure is to limit rutting of the subgrade, or the cumulative permanent deformation caused by vertical compressive strain at the top of the subgrade layer, to a certain level. Subgrade rutting may indicate:

- the pavement is performing in accordance with the (original) design assumptions
- the (original) design traffic has been exceeded
- the effective subgrade strength is / was less than the design strength adopted in the (original) design, or
- the insitu condition of the subgrade is / was different from the design condition adopted in the (original) design (for example, moisture content is higher), or
- the pavement has suffered from one or more overloads (for example, an over-mass vehicle traversing the pavement).

Permanent deformation of the subgrade accumulates with the passage of each (heavy vehicle) axle (group).

2.13.9.3 Cracking

Cracking can indicate many things including:

- oxidation (aging) of the binder
- permanent severe deformation of the subgrade caused by repetitive loading
- instability in the upper pavement layers
- cracking of underlying cementitiously bound layers (reflection cracking)
- settlement, or
- repeated deflection causing fatigue in the asphalt layer/s.

Some commonly encountered cracks are:

- transverse cracks
- fatigue (crocodile) cracks
- age-related cracks
- longitudinal cracks, and
- block cracks.

The type, location, distribution and severity of cracking provides a guide to the cause of pavement distress(es). AGPT05 Appendix A contains further details about these, and other, crack types and their causes.

In this *Manual*, terminal cracking of asphalt is defined as fatigue (crocodile) cracking that is, on average, 2mm wide and cracked in a clearly defined pattern. Table 2.13.9.3 contains suggested terminal values for pavement including asphalt. To date, only fatigue (crocodile) cracking has been included in the terminal criteria.

Table 2.13.9.3 – Suggested terminal criteria for asphalt fatigue (crocodile) cracking

Type of road	Proportion of road section with cracks ≥2mm wide (% of area affected)
Motorways, Urban Arterials and Urban Sub-Arterials	5
Rural Highways, Transport and Main Roads state-controlled roads and Developmental Roads	20
Secondary Roads	30

2.13.9.3.1 Test procedures and measures

Automated crack (and other defect) detection systems are now incorporated into network-level profilometer survey vehicles. Alternatively, cracks can also be rated manually by visual inspection of pavement video footage taken during network-level surveys. These results typically include the percentage of pavement exhibiting specific crack types and can be obtained from ARMIS.

2.13.9.3.2 Causes

AGPT Appendix A outlines causes of cracking of flexible pavements. Other causes of cracking include:

- cracking of rigid pavements due to inadequate concrete thickness, edge effects (for example, edge of slab too close to wheel path), support or all of these
- cracking in an asphalt surfacing due to the oxidation of the binder
- volume change of the subgrade
- reflection of joints in concrete pavements
- lack of bond to underlying layers
- shrinkage cracking of stabilised materials (for example, reflective cracking in a cementitiously treated base (CTB) pavement), and/or
- void formation under concrete slabs.

2.13.9.4 Shoving

Shoving represents a gross deformation of the pavement, rapidly leading to disintegration: it cannot be tolerated.

It occurs because of one or more of the following:

- inadequate strength in surfacing or base material
- poor bond between pavement layers
- insufficient containment of the pavement edge, and/or
- inadequate pavement thickness, overstressing the subgrade.

If the cause of the shoving is not immediately evident, excavation of a trench perpendicular to the direction of shoving should provide an indication of the failure mechanism and what further materials testing may be necessary.

Finally, shoving (and rutting) can also disrupt / change surface water flows, and/or pond water leading to undesirable water (film) thicknesses that may subsequently lead to aquaplaning of vehicles. The RDM provides guidance with respect to road drainage, including surface drainage. Where surface drainage is an issue, refer to the RDM and Section 2.13.8 of this *Manual*.

2.13.10 Surface deflections

AGPT05 Section 4.8 contains a discussion relating to the use of surface deflections to assess a pavement. Surface deflections are a measure of the structural response of a pavement to a load.

Note: The department's methodology for the use of surface deflections follows the philosophy outlined in AGPT05; however, the details are different (for example, the deflection charts, design of overlays using deflection). This *Manual* takes precedence over AGPT05 for departmental projects and roads controlled by Transport and Main Roads.

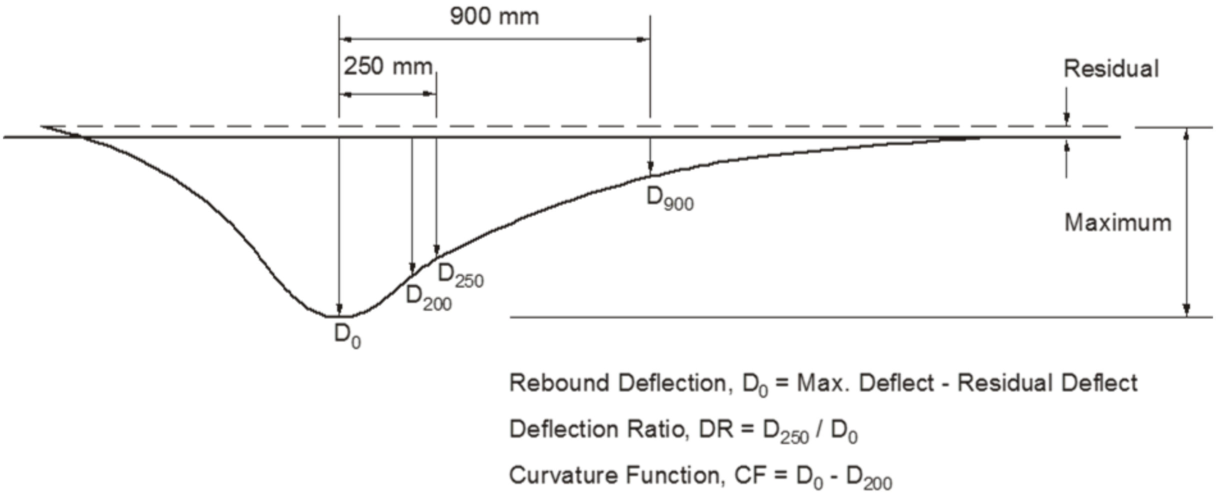
Deflection data must be representative of the current pavement condition when the data is to be used to characterise or evaluate the current condition of a pavement. An important aspect to consider, when deciding about whether the data can be so used, is the age of data. A general rule of thumb is, if deflection data are more than two years old, they are unlikely to be representative of the current condition of a pavement. This is not always the case, however, and the decision must be reviewed on a case-by-case basis by the Designer and the Client.

If the existing pavement contains bound layers, the deflection is usually very low. As a result, the deflection bowl measured is usually very flat, which makes the interpretation of bowl data less reliable. If the existing pavement is rigid, then the use of, and results from, deflection testing is somewhat different than for non-rigid pavements.

Depending on the pavement type (rigid versus non-rigid) and apparatus used to test the pavement, it is possible to use deflections to characterise a substantial length of pavement at one time.

The shape of a typical load-deflection response ('deflection bowl') of a point on the pavement surface in relation to a passing load is shown in Figure 2.13.10.

Figure 2.13.10 – Typical deflection bowl with load-deflection parameters for a Benkelman Beam test on a non-rigid pavement



2.13.10.1 Test equipment

The more common test apparatuses include the Benkelman Beam (Figure 2.13.10.1(a)), with its automated variants such as the deflectograph (Figure 2.13.10.1(b)), the FWD (Figure 2.13.10.1(c)) and the HWD. The TSD (Figure 2.13.10.1(d)) is a recent addition to this range of equipment and predominantly used for network-level program testing.

Figure 2.13.10.1(a) – An example of a Benkelman Beam

Figure 2.13.10.1(b) – An example of a deflectograph

Figure 2.13.10.1(c) – An example of a falling weight deflectometer

Figure 2.13.10.1(d) – An example of a traffic speed deflectometer

The Benkelman Beam method was routinely used by the department in the past and, while seldom used by the department today, may still be used. It is most suited to project level investigations with the quicker, automated deflectograph (and deflectolab) and TSD better suited to network surveys. Since 2014, Transport and Main Roads has engaged the use of TSD equipment for network deflection surveys.

The Benkelman Beam is used for point testing at user-specified locations; both wheel paths may be tested. It is a manually-operated, low-cost, mechanical testing system. It measures a 'departure bowl' and uses a beam longer than used in a deflectograph.

The deflectograph has beams like a pair of small automated Benkelman Beams with some exceptions. The beams are mounted under a truck; point testing is done in both wheel paths. Measurements are taken relative to a reference frame pulled along the pavement by the truck. Measurements are usually taken at regular, closely spaced intervals (4–5 metres) along a test section. Consequently, it is well suited to testing roads varying significantly in strength, for identifying weak sections for local strengthening and/or for testing long lengths of road. Differences to the Benkelman Beam include shorter beams (resulting in shorter deflection bowl measurements), and it measures the approach side of the bowl. Consequently, the results of the deflectograph and the Benkelman Beam cannot be compared directly.

The following must be noted:

- the Benkelman Beam uses an 80 kN loading on a standard dual tyre single axle with a tyre pressure of 550 kPa
- in general, the Benkelman Beam is recommended only for flexible granular pavements up to about 450mm thick (and not other pavement types)
- the load on the Benkelman Beam and deflectograph cannot be increased to obtain useable deflection values from stiff (for example, full depth asphalt, deep strength asphalt, rigid) pavements
- for stiff pavements, the bowl measurements taken with the Benkelman Beam and deflectograph are affected by the movement of the reference beam in the deflection bowl
- how the pieces of equipment are used differs, depending on pavement type – the deflectograph is not normally used on roads with rigid pavements as user-specified locations cannot be designated (for example, at joints, refer to Section 2.13.10.7)
- the bowl measurements taken with the deflectograph are affected by deflection of the reference frame under the influence of the front and rear truck axle loads
- a tyre pressure of 750 kPa is considered more representative of today’s traffic and this value is now used in pavement design (AGPT02).

Therefore, the limitations of the Benkelman Beam and the deflectograph need to be considered before deciding on whether to use them. The same limitations need to be considered when designing using their results. Alternatives include the FWD, HWD and/or TSD.

Both falling weight and traffic speed devices challenge the notion the deflection of the pavement is modelled adequately by assuming the load applied to the pavement is static and the deflection of the pavement has reached equilibrium (for example, *CIRCLY* software). The main reasons for this are explained in terms of the dynamic response to the inertia (mass) of the pavement and the viscous damping forces of bituminous pavement material (for example, asphalt). Notwithstanding, the response of most pavements, are in fact, modelled adequately by the assumption of a layered system at equilibrium.

FWDs / HWDs use a falling weight, as opposed to a loaded rolling wheel, to load the pavement through a 300mm diameter plate. The impact of the falling weight causes surface (Rayleigh) waves to radiate from the area of impact at a group velocity varying between 100–700 m/s. Particles influenced by Rayleigh waves exhibit movement normal to the pavement surface and radial from the centre of load; however, the geophones of FWDs typically only measure the normal component.

The load can be varied and has an applied load pulse duration of between 25–35 ms (*Test Method AGAM-T006 Pavement Deflection Measurement with a Falling Weight Deflectometer (FW)*) (AGAMT006). The load pulse duration is, however, a function of the load, impact velocity and the pavement structure. The fundamental loading frequency corresponds to 1/2T or an equivalent (wheel) loading rate of π/T km/h (refer Section 2.13.10.4.9.2), where T is the load pulse duration in seconds. A 25–35 ms load pulse duration corresponds to a rate of loading of between 125 km/h and 90 km/h respectively, therefore, back-calculated asphalt moduli should be adjusted to the expected operating speed of the pavement, using the appropriate asphalt master curve (AMC) (AGPT02).

The geophones are used to measure peak deflections of the Rayleigh waves at specific distances (offsets, typically 0, 200, 300, 450, 600, 750, 900 1200 and 1500mm) from the centre of the load; however, the spacing of the geophones limits the ability of FWD / HWD equipment to test the top 200mm of pavement, with only one deflection point representing this region able to be collected – therefore, it is most suitable for thicker or stiffer pavements.

- The structural analysis of flexible pavements and chart-based overlay design procedures described in this *Manual* are for deflections measured with a FWD using a contact stress of 566 kPa (that is, equivalent to 40 kN) and a load pulse duration of about 25–35 ms.
- The rate of loading will influence the modulus of bituminous layers such as asphalt and foamed bitumen treated material, and therefore the measured deflection – refer Section 2.13.10.4.9.2 for further details.
- Deflections measured with a HWD may differ from those that would be obtained from a FWD with the same contact stress.
- HWD deflections may need to be standardised to FWD values before using the design procedures described in this *Manual*.
- Where the actual contact stress is different to the target contact stress, the measured deflections may need to be normalised to the target contact stress (for example, normalise to a stress of 566 kPa before using the design procedures described in this *Manual*).
  FWD deflections obtained using a load of other than 40 kN may need to be standardised to the equivalent 40 kN FWD values before using the design procedures described in this *Manual*.

To make these corrections (referred to as load normalisation), deflections are usually assumed to be related linearly to stress level. Since the assumption of linearity is an approximation made only over small load variations, the contact stress during testing should be as close as practicable to the target stress and be within 15% of it.

Note: This does not permit deflections at 40 and 60 kN to be estimated from 50 kN results).

Deflections measured at target loads of 40, 60 and 80 kN can be used to apply a potentially more accurate, non-linear load normalisation to results.

FWD / HWD equipment should be calibrated and validated in accordance with AGAMT006.

Notwithstanding the use of a standard 566 kPa contact stress, where FWD / HWD deflections are measured for use with the GMP for design (on non-rigid pavements), higher contact stresses may be required to reduce measurement errors for stiff pavements with very low deflections. As a guide, the maximum deflection (D_0) of the pavement should not be less than 0.2 mm.

To undertake back analysis of non-rigid pavements in accordance with the GMP, the maximum deflection, along with deflections at seven or more offsets from the point of application of the load, must be measured. Commonly used offsets are 0, 200, 300, 450, 600, 750, 900, 1200 and 1500mm (nine geophones, although as many as 17 geophones are deployed on some makes).

While a FWD / HWD allows more precise estimates of the deflection bowl parameters at a site, the deflectograph is highly productive when compared to the FWD / HWD and can be used to undertake extensive testing to uncover any variations in pavement strength between wheel paths as well as longitudinally (for non-rigid pavements). A FWD/HWD is not as cost-effective as the deflectograph in measuring deflections at closely spaced intervals (4–5m) in both wheel paths if this is required to characterise accurately non-rigid-pavements with variable strength. Nevertheless, the significant limitations of the deflectograph, see previous guidance) must also be considered.

Currently, Transport and Main Roads uses a FWD / HWD for project level investigations; more recently, the TSD has been introduced to service and is used for statewide network-level surveys. FWD / HWD is recommended for project level investigations. TSD results can be used to identify specific locations within a road network where more detailed testing (additional lanes) may be warranted or for asset management at the network level.

Transport and Main Roads' Strategic Asset Management unit manages the statewide sealed road network TSD survey program under the *Transport Infrastructure Asset Management Policy* (TIAMP). Although the bulk of its program consists of network-level testing, the TSD may also be available for limited project level work. The intent of the TSD testing program is to collect as much of the sealed network as can be collected in the allocated time; however, due to operational constraints of the vehicle, some sections of the sealed network are not able to be tested. These include:

- sections where manoeuvrability issues prevent turn arounds
- isolated seals (TSD might be damaged by travelling across unsealed road to access these)
- roads with significant curves, resulting in a travel speed less than 50 km/hr, and/or
- sections of extreme roughness.

In addition to these constraints, the Doppler lasers are only equipped in the left wheel path of the TSD, which means moisture adjustment which considers deflection in the IWD is seldom possible.

The deflection reduction-based, overlay design methods (see Chapter 5) were based on measurements with the Benkelman Beam and deflections obtained by using any other device must therefore be converted to equivalent Benkelman Beam deflections for use with these methods.

The conversion factors for deflection data from deflection testing devices used by the department are:

- 1 unit of Benkelman Beam D_0 deflection is equivalent to 1 unit of 40 kN FWD D_0 deflection
- 1 unit of Benkelman Beam D_0 deflection is equivalent to 1.17 units of deflectograph D_0 deflection.

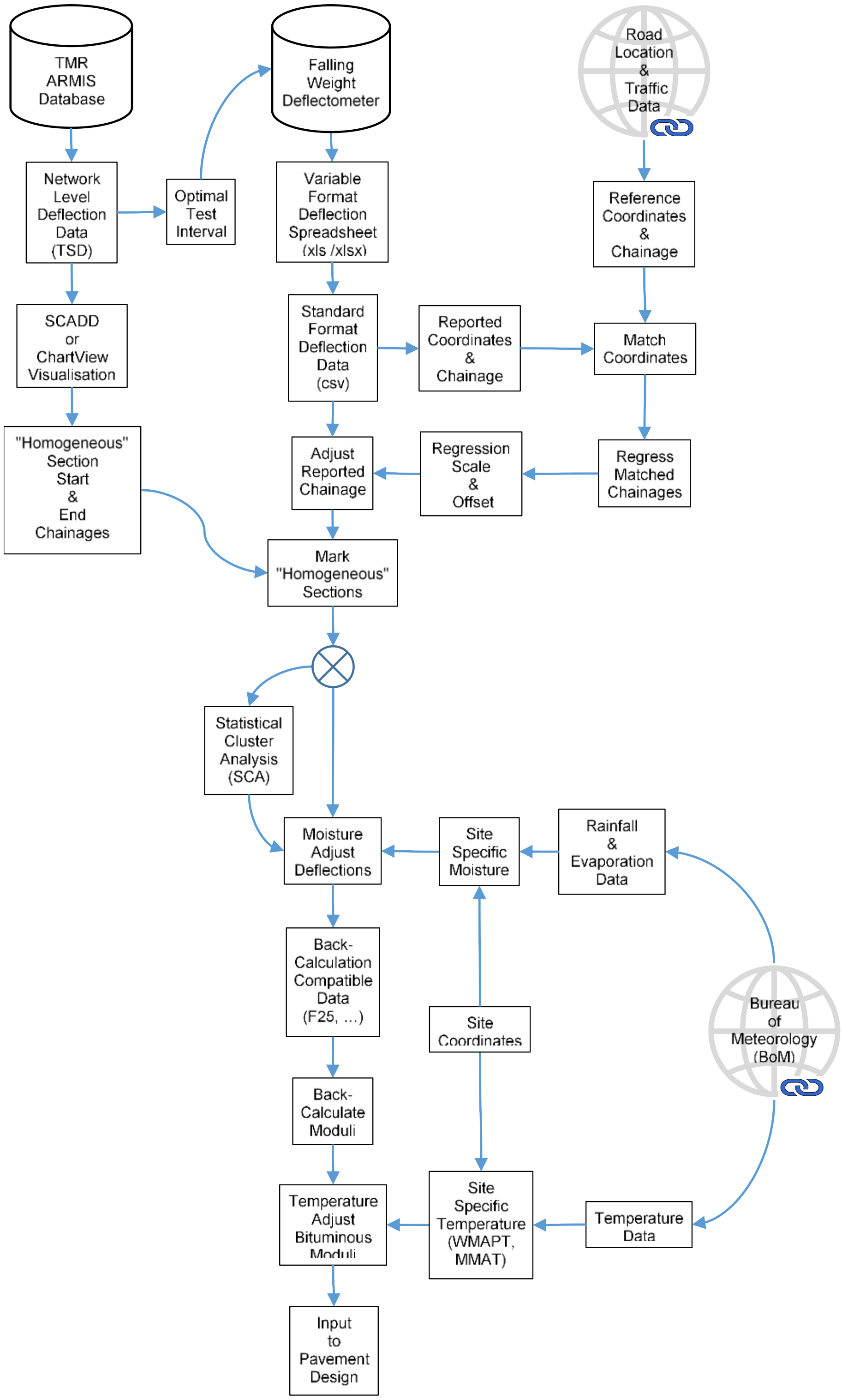
The curvature measured by the Benkelman Beam, 40 kN FWD and deflectograph is taken to be equal (1 unit of Benkelman Beam curvature = 1 unit of 40 kN FWD curvature = 1 unit of deflectograph curvature).

These factors were considered when the charts included in Appendix 5A of Chapter 5 were developed; no correction needs to be applied when using 40 kN FWD and deflectograph deflections and the charts in Appendix 5A.

2.13.10.2 Some testing considerations

Figure 2.13.10.2 indicates the various procedures associated with the collection and back-calculation of FWD deflection data.

Figure 2.13.10.2 – Falling weight deflectometer data collection and back calculation procedures



To determine the frequency of the test using a Benkelman Beam or FWD / HWD on a non-rigid pavement, a compromise must be made between:

- obtaining results that allow sections with different structural conditions or configurations to be identified (use a testing pattern that allows the pavement to be subsectioned)
- obtaining the minimum number of test points (results) necessary for a meaningful statistical analysis of each subsection
- traffic management requirements
- time
- cost, and
- maintaining a reasonable production rate.

Spacing for rigid pavements depends on the deflection testing proposed (AGPT05), see also Section 2.13.10.7 of this *Manual*.

The spacing used will depend on the importance of the route and the frequency of changes in the topography, pavement type (rigid versus non-rigid), spacing in cuttings and along embankments, subgrade, pavement configuration and soil or construction characteristics.

Note: The test interval in each wheel path should be between 5–100m, refer Table 2.13.10.2. The test interval is normally staggered between adjacent wheel paths (alternate wheel paths longitudinally offset by half the test interval) to achieve lane sampling at half the wheel path interval.

Table 2.13.10.2 – Recommended falling weight deflectometer / heavy falling weight deflectometer test intervals for non-rigid pavement

Wheel path test interval* (m)	Environment
5–10	may be used for sections of high distress or very short sections
20–30	should be used for urban areas or short sections
50	may be used for rural areas or long sections
100	may be used where uniform pavement conditions exist or long sections of multilane road

*Note: Staggered to achieve lane sampling at half the interval.

Small sample sizes tend not to sample the tails of the deflection distribution. This results in an underestimate of standard deviation of the population (where a large number of tests are carried out). Since 90th percentile deflection D_r is based on the standard deviation, this will also be underestimated.

Therefore, the number of test points within each section shall not be less than 6, resulting in an underestimate of D_r by no more than 5% (refer Section 2.13.10.4.2).

FWDs are unlikely to be charged out for less than half a day and are commonly charged out to a full day; therefore, the number of test points should be chosen to ensure the device is fully occupied for the period being charged.

For a deflectograph, the longitudinal spacing of test sites is a function of equipment geometry and test speed (measurements are usually taken at regular intervals, commonly 4–5m, along a test section). Consequently, only the transverse locations of wheel path positions need to be determined when a deflectograph is used.

In all cases and where it is practical to do so, the transverse position of test sites should be selected while keeping in mind any proposed changes to the road alignment.

Where deflection results are inconsistent with observed visual condition of a pavement, there may be seasonal effects to consider; for example, ineffective moisture control may lead to high deflection in wet seasons. Other factors, such as deterioration of the surfacing or instability of base layers, should also be considered. When asphalt temperatures exceed 60°C, the results of Benkelman Beam deflection tests become unreliable.

2.13.10.3 Reference load

Deflection measurements used by the department are referenced (normalised) to a FWD load of 40 kN.

Where deflections are measured at some other load, a linear elastic model of the pavement response results in the deflection being inversely proportional to the applied load. This approach is often used to adjust measured deflections to the reference load.

By comparing the theoretical response of the pavement to a FWD plate and a Standard Axle, it can be demonstrated a 40 kN FWD plate load provides a close approximation to the pavement response of a 50 kN dual tyre load and is, therefore, the preferred reference load for the FWD device. Other loads can be used to measure deflection but should be reported at 40 kN by multiplying the deflections by 40/L where L is the load in kN used to measure deflection, and where the adjustment does not amount to more than ±15% of L (Section 2.13.10.1).

2.13.10.4 Granular pavements with a thin asphalt surfacing or seal

The structural adequacy of existing granular pavements with a thin asphalt (total thickness no greater than 50mm) surfacing or seal, in terms of resistance to permanent deformation, may be assessed by comparing the characteristic deflection to the design deflection (see also Chapter 5). Alternatively, the GMP for overlay thickness design may be used (see Chapter 5).

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

2.13.10.4.1 Maximum deflection

For the deflectograph and FWD / HWD, this is the maximum reading recorded for each test site. For the Benkelman Beam, the 'maximum deflection' is taken as the total deflection minus the residual deflection (the rebound deflection). For all devices discussed in this *Manual,* the maximum deflection is denominated as D_0.

2.13.10.4.2 Representative sections and deflections (D_r)

Analysis of deflection survey results can help determine rehabilitation requirements. To this end, (relatively) homogeneous subsections (representative sections) must be identified for each test run (for example, OWP). Initial identification of representative sections can be performed by visual inspection of plotted deflection results (for example, Figure 2.13.10.4.2). Data about the (past and forecast) traffic, environment and pavement type / history / configuration can be used in conjunction with deflection results to divide further the initial subsections into representative sections. The results of a GPR survey (for example, to identify layer details and changes in pavement type) may also assist.

Deflections on a section may vary throughout the year because of moisture changes (due to seasonal variations) and, where the pavement has an asphalt layer greater than 50mm thick, because of temperature variations. Deflection measurements need to be corrected for these effects, as set out in Section 2.13.10.4.9.

To identify representative sections, the following statistics are calculated for each proposed representative section: mean, standard deviation and coefficient of variation (CV) (see Equation 2.13.10.4.2(a)). The CV provides an indication of the variability within each section.

Equation 2.13.10.4.2(a) – Coefficient of variation

$$CV = \frac{\sigma \times 100}{x}$$

where:

CV = coefficient of variation

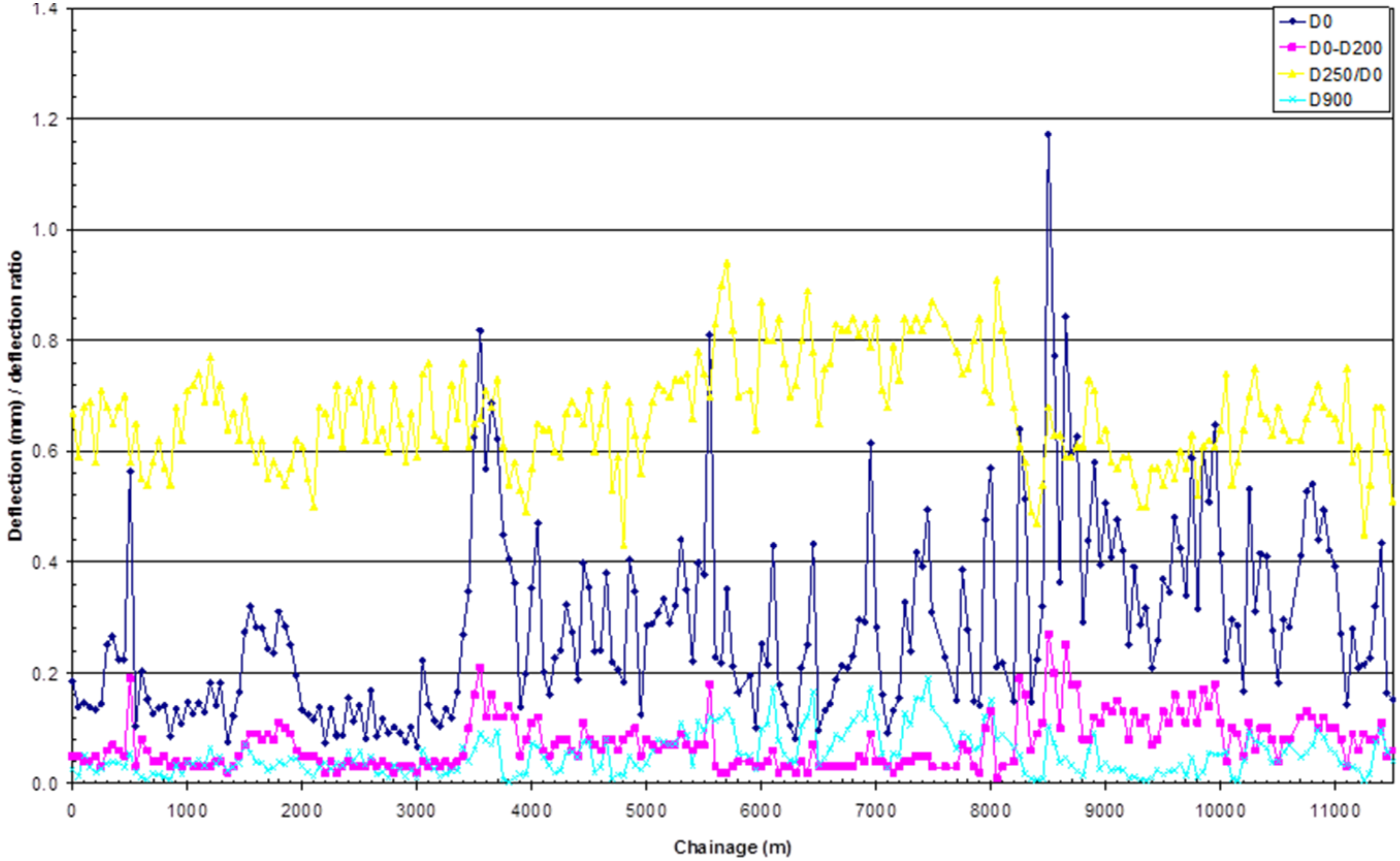
σ = standard deviation of selected deflection results, typically D_0 values, for section under consideration

x = mean of selected deflection results, typically D_0 values, for the section under consideration.

If the CV is less than 30%, the proposed representative section may be regarded as relatively homogenous and can be classified as a representative section; however, it is desirable to obtain a CV of 25% or less. It is important, because of the variable nature of pavements, to exercise caution during statistical analysis to ensure all high or low values not representative of the general pavement condition (for example, results for tests on patches and/or culverts) or those remedied during rehabilitation, are excluded. Once the sections have been determined, their deflection response can be characterised.

In this *Manual*, the representative deflection (D_r) of a section corresponds to the 90th percentile (90% highest) rebound deflection, based on the assumption the deflection results are described adequately by normal statistical distribution.

Figure 2.13.10.4.2 – Example plot of deflection results from a non-rigid pavement



A representative deflection is determined for each test run in each section (for example, for the inner and/or outer wheel path of each lane, D_r (Inner Wheel Path (IWP) or OWP) according to Equation 2.13.10.4.2(b):

Equation 2.13.10.4.2(b) – Representative deflection

$$D_r = x + (1.28 \times \sigma)$$

where:

D_r = representative deflection

σ = standard deviation of selected deflection results, typically D_0 values, for section under consideration.

x = mean of selected deflection results, typically D_0 values, for section under consideration.

Note: σ is the standard deviation of the population (very large sample) which is typically underestimated by the sample standard deviation (SSD) *s*.

Table 2.13.10.4.2 indicates the degree to which the SSD underestimates the standard deviation of a normally distributed population.

Table 2.13.10.4.2 – Underestimate of the standard deviation of a normally distributed population

Sample size (n)	Underestimate
2	20.2%
3	11.4%
4	7.9%
5	6.0%
6	4.8%
7	4.1%
8	3.5%
9	3.1%
10	2.7%
15	1.8%
20	1.3%
25	1.0%
30	0.9%
50	0.5%
100	0.3%
200	0.1%

From Table 2.13.10.4.2, it can be seen a sample size of 6 underestimates the standard deviation of the population by less than 5%; therefore, subject to the constraints of Table 2.13.10.4.2, a sample size of at least 6 should be used and *s* multiplied by the sample standard deviation correction factor (SSDCF) of Equation 2.13.10.4.2(c) to provide an unbiased estimate of σ.

Equation 2.13.10.4.2(c) – Sample standard deviation correction factor

$$SSDCF = \sqrt{\frac{n-1}{2}}\frac{\Gamma\left(\frac{n-1}{2}\right)}{\Gamma\left(\frac{n}{2}\right)}$$

where:

n = the sample size

Γ= the gamma function

Relevant additional correction factors should then be applied as required (see Section 2.13.10.4.9).

D_r can be used to verify pavement performance (see Section 2.13.10.5) to predict future performance and to design an overlay using the deflection reduction method (refer to Chapter 5).

When designing using the deflection reduction method, the D_r for design is selected. This is normally the larger D_r of the corrected IWP / OWP test runs.

2.13.10.4.3Residual deflection

Residual deflections are relevant when a Benkelman Beam or deflectograph is used. A residual deflection is the deformation of the test site remaining when the test vehicle moves beyond the range of influence. For a thin flexible pavement, values of ≥0.15mm or >25% of the maximum deflection indicate either a potential weakness in the upper layer of the pavement or deformation of the surfacing course. Further investigations should be carried out when such values are measured to determine why this is the case, particularly if an overlay is being considered.

Caution should be exercised when testing stiff pavements (for example, pavements with a cementitiously stabilised base, deep strength asphalt pavements, rigid (concrete) pavements), as high residuals may result. This is caused by the feet of the Benkelman Beam being within the bowl at the start of the test. In this case, high residual deflection values may not indicate structural inadequacy.

A substantial (>0.15mm) 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 cementitiously modified / stabilised 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.

2.13.10.4.4Deflection ratio

The deflection ratio (DR) is the ratio of the deflection at a point 250mm from the maximum rebound deflection (D_{250}) to the maximum rebound deflection (D_0) (see Equation 2.13.10.4.4(a)).

The representative DR is the 10th percentile lowest DR assuming a ‘normal’ statistical distribution (see Equation 2.13.10.4.4(b)).

Equation 2.13.10.4.4(a) – Deflection ratio

$$DR = \frac{D_{250}}{D_0}$$

where:

DR = deflection ratio

D_{250} = deflection at a point 250mm from the maximum rebound deflection

D_0 = maximum rebound deflection.

Equation 2.13.10.4.4(b) – Representative deflection ratio

$$DR_r = x - (1.28 \times \sigma)$$

where:

DR_r = representative deflection ratio

σ = standard deviation of deflection ratios for section under consideration (refer use of SSDCF in Section 2.13.10.4.2)

x = mean of deflection ratios for section under consideration.

The DR is used to delineate sections of road pavement that are bound, unbound or excessively weak, but not rigid. For results derived using a FWD with a 40 kN loading, the Benkelman Beam or the deflectograph:

- DR of greater than 0.8 would indicate a bound pavement
- DR of between 0.6–0.7 would be expected for a good quality unbound pavement with a thin asphalt surfacing or seal, and
- DR of less than 0.6 would indicate a possible weakness in unbound pavement with a thin asphalt surfacing or seal.

These values are not applicable for results using different apparatus or loads.

2.13.10.4.5Curvature function

The shape (curvature) of the deflection bowl is used to estimate the likelihood of fatigue cracking in an asphalt layer. The curvature is defined by the curvature function (CF) as given in Equation 2.13.10.4.5.

Equation 2.13.10.4.5 – Curvature function

$$CF = D_0 - D_{200}$$

where:

CF = curvature function

D_{200} = deflection at a point 200mm from the maximum rebound deflection

D_0 = maximum rebound deflection.

The representative curvature function, CF_r, for a section of pavement is taken to be the mean CF.

For granular pavements with thin bituminous surfacings, the curvature function is likely to be 25–35% of the maximum deflection. Values higher than this may indicate the granular base course has low strength.

High values of the CF (for example, 0.4mm for results derived using a FWD with a 40 kN loading, the Benkelman Beam or deflectograph) may indicate a pavement is lacking stiffness, a very thin pavement, or a pavement with a cracked asphalt surface. Low values of the CF (for example, <0.2mm for results derived using a FWD with a 40 kN loading, the Benkelman Beam or deflectograph) indicate a stiff pavement.

2.13.10.4.6 Subgrade response (D_{900})

The deflection at a point 900mm from the point of maximum deflection is referred to as the D_{900} value.

The representative D_{900} value is taken as the 90th percentile D_{900} (refer Equation 2.13.10.4.6).

Equation 2.13.10.4.6 – Representative D_{900}

$$D_{900\,r} = x + (1.28 \times \sigma)$$

where:

$D_{900\,r}$ = representative D_{900}

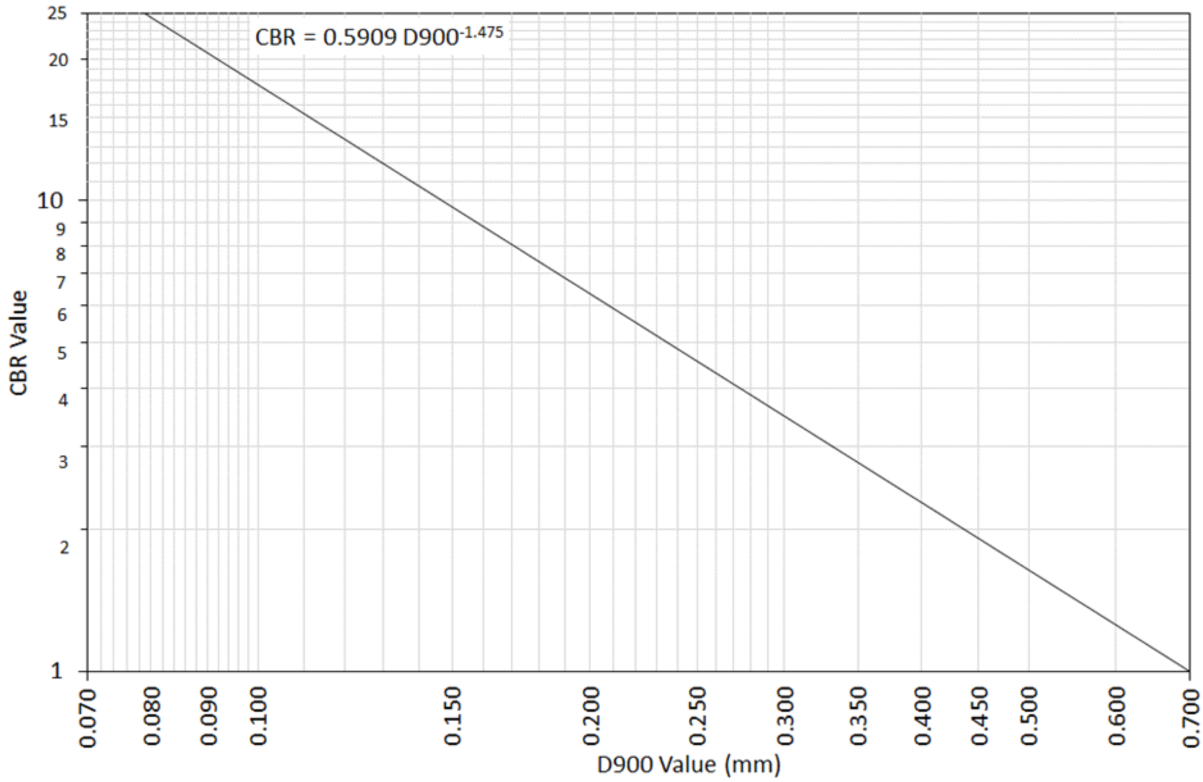
σ = standard deviation of D_{900} values for section under consideration (refer use of SSDCF in Section 2.13.10.4.2)

x = mean of D_{900} values for section under consideration.

Though a subgrade's CBR is sometimes less important in overlay design, the subgrade response versus CBR relationship may assist in supporting other design options (for example, assessing subgrade strength to design a stabilisation option).

As indicated, the subgrade CBR value at the time of testing is largely independent of all but the D_{900} deflection and essentially unaffected by the structure of the overlying pavement. The relationship between D_{900} and CBR is shown in Figure 2.13.10.4.6 for results derived using a FWD with a 40 kN loading (see also charts of Section 2.13.10.4.8). The relationship shown in Figure 2.13.10.4.6 is not applicable for results using different apparatus or loads.

Figure 2.13.10.4.6 – D_{900} versus California Bearing Ratio for Benkelman Beam, deflectograph and normalised 40 kN falling weight deflectometer results for a granular pavement with a thin asphalt surfacing or seal



2.13.10.4.7 Statistical classification of deflection bowls

The maximum deflection, deflection ratio, curvature and subgrade response can be seen as an attempt to classify a deflection bowl in terms of its magnitude and shape.

If any two bowls are compared by means of simple linear regression, excluding the intercept, the regression slope is effectively a measure of the relative magnitude of the two bowls while the coefficient of determination ($\boldsymbol{R^2}$) is a measure of the relative shape or pavement structure of the two bowls. In the limiting case where the two bowls are identical, the slope and $\boldsymbol{R^2}$ will be equal to 1. Conversely, if the two bowls are very different in magnitude and shape, the slope will be either much higher or lower than 1 and $\boldsymbol{R^2}$ will approach 0. Additionally, if the intercept is excluded and one is required, $\boldsymbol{R^2}$ will degrade, implicitly representing the intercept. Together, these facts imply regression slope and $\boldsymbol{R^2}$, suitably transformed, can be used as the components of a metric which can be used by a SCA process to separate deflection bowls into groups with similar magnitude and shape.

Cluster analysis uses a 'dissimilarity' matrix comprised of the metric for all bowl combinations. The pavement is typically divided into representative sections as described in Section 2.13.10.4.2 which prevents the matrix from becoming too large to process and may also reduce the need to include a significant regression slope as a component of the metric, in which case, bowls will be grouped within each representative section based entirely on their shape.

Details of the cluster analysis process for deflection bowls are included in Appendix A. This appendix also identifies several advantages associated with using cluster analysis:

- Core, Test Pit and Trench locations can be easily selected by plotting bowl groups within a geospatial context. The geospatial report can also be used to assess and potentially reduce the requirement for more expensive investigations involving trenches.
- The mean bowl shape of each group can be used to reduce amplification of deflection measurement errors through the back-analysis process and significantly reduce the number of bowls to be back analysed, an important feature where slower but more accurate structural design using the finite element method (FEM) is anticipated.

2.13.10.4.8Back analysis

Appendix A indicates deflection measurement errors can be amplified by the back-analysis process. For this reason, only the representative mean bowl shape of each group identified in the cluster analysis described in Appendix A should be used for back analysis.

Within each bowl group, the deflections of wheel paths other than the OWP should be adjusted with the deflection moisture adjustment factor (DMAF) described in Section 2.13.10.4.9.3 and detailed in Appendix B.

The mean bowl shape should then be assigned a maximum deflection corresponding to the representative (90th percentile) maximum deflection of all bowls (including both moisture-adjusted and otherwise) in the group. The resulting bowl is to be back analysed.

A normalised load of 40 kN is to be used for deflections submitted for back analysis. Where a linear elastic model is used to predict the pavement response (for example, *CIRCLY*), deflections should be multiplied by 40/L where L is the actual load in kN.

For an unbound granular pavement with thin bituminous surfacing where the vertical modulus of the granular material increases by a factor of 2 every 125mm of thickness to maximum achievable value of 350 MPa (CBR 80) (AGPT02), Figure 2.13.10.4.8(a) to Figure 2.13.10.4.8(c) can be used to estimate the granular thickness, vertical elastic modulus, subgrade CBR and design traffic (for a traffic multiplier of 1.6) from FWD D_{900} and D_0 values.

Figure 2.13.10.4.8(a) indicates the CBR contours are nearly vertical (independent of D_0) where the pavement does not contain a significant amount of granular material at the maximum achievable modulus (350 MPa), allowing the CBR to be approximated as a function of D_{900} as indicated in Figure 2.13.10.4.6; however, as the amount of granular material at the maximum achievable modulus increases in thickness, the CBR needs to be expressed as a function of both D_0 and D_{900}.

Figure 2.13.10.4.8(c) has been compiled by matching the deflections produced by a 40 kN FWD plate load with the vertical strain at the top of the subgrade produced by a full standard axle with a contact pressure of 750 kPa.

Note: Grey traffic contours are indicative only and should not be used for design purposes.

Figure 2.13.10.4.8(a) – Falling weight deflectometer thickness back analysis chart for unbound granular pavement with thin bituminous surfacing, 40 kN plate load

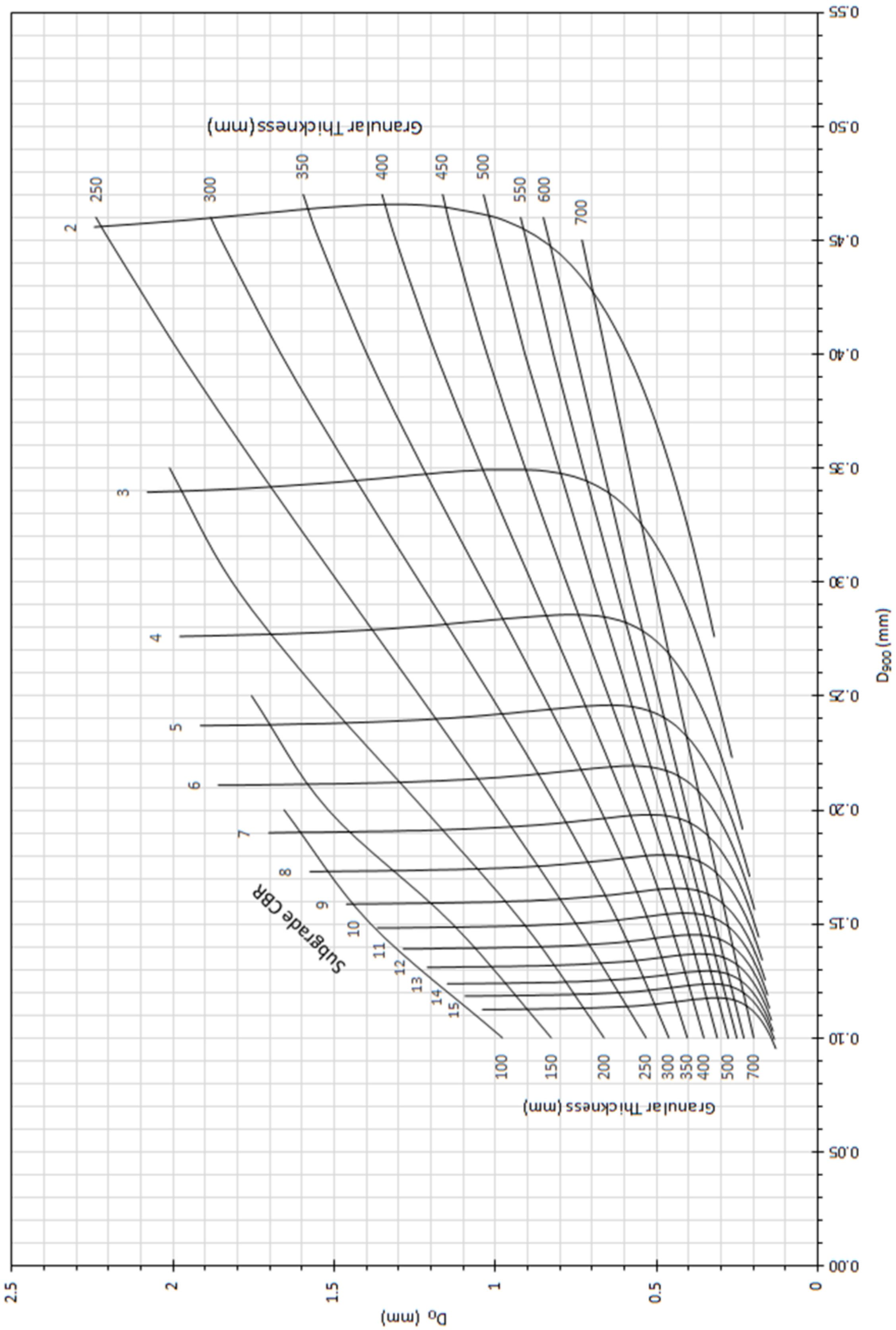


Figure 2.13.10.4.8(b) – Falling weight deflectometer modulus back analysis chart for unbound granular pavement with thin bituminous surfacing, 40 kN plate load

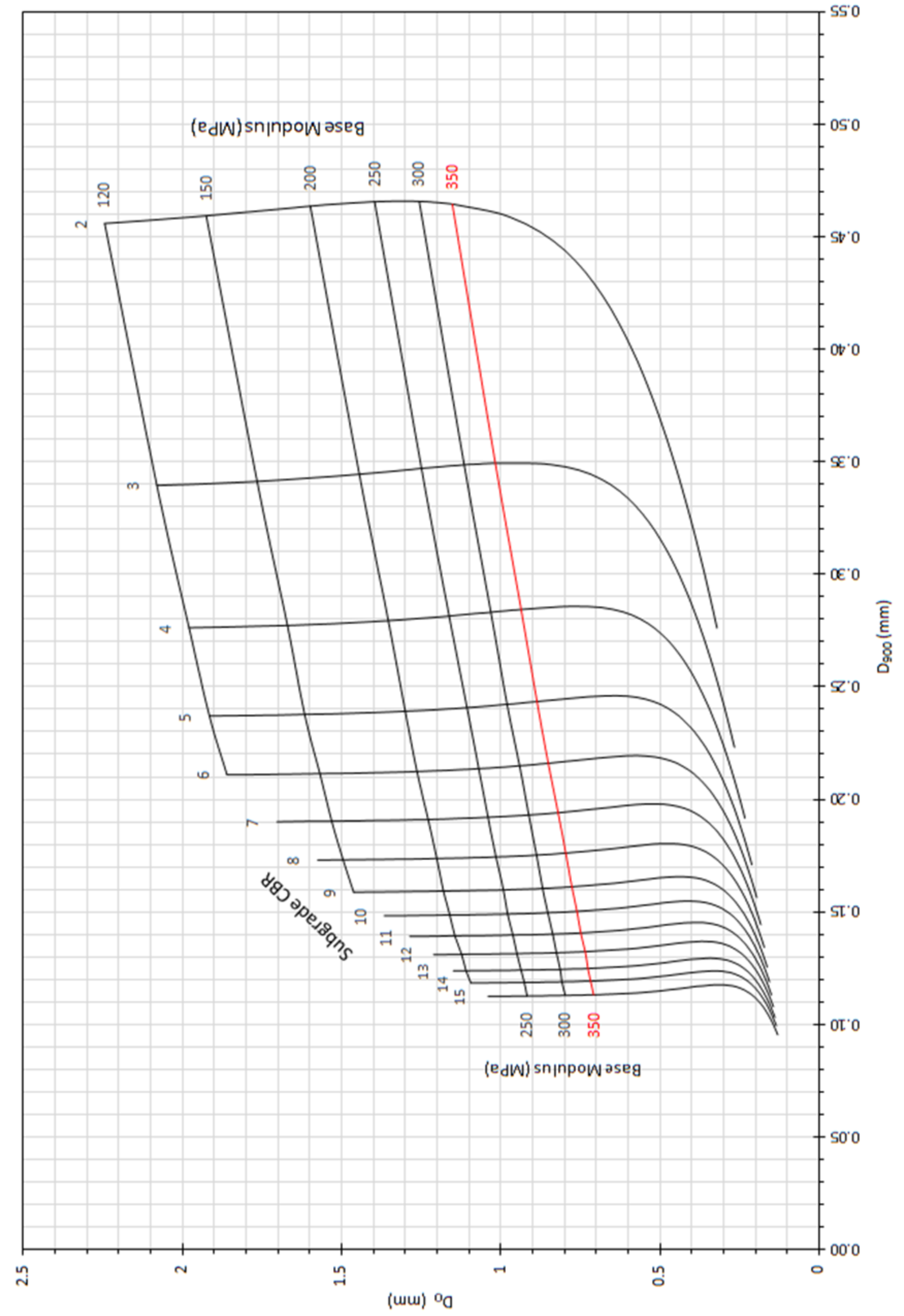
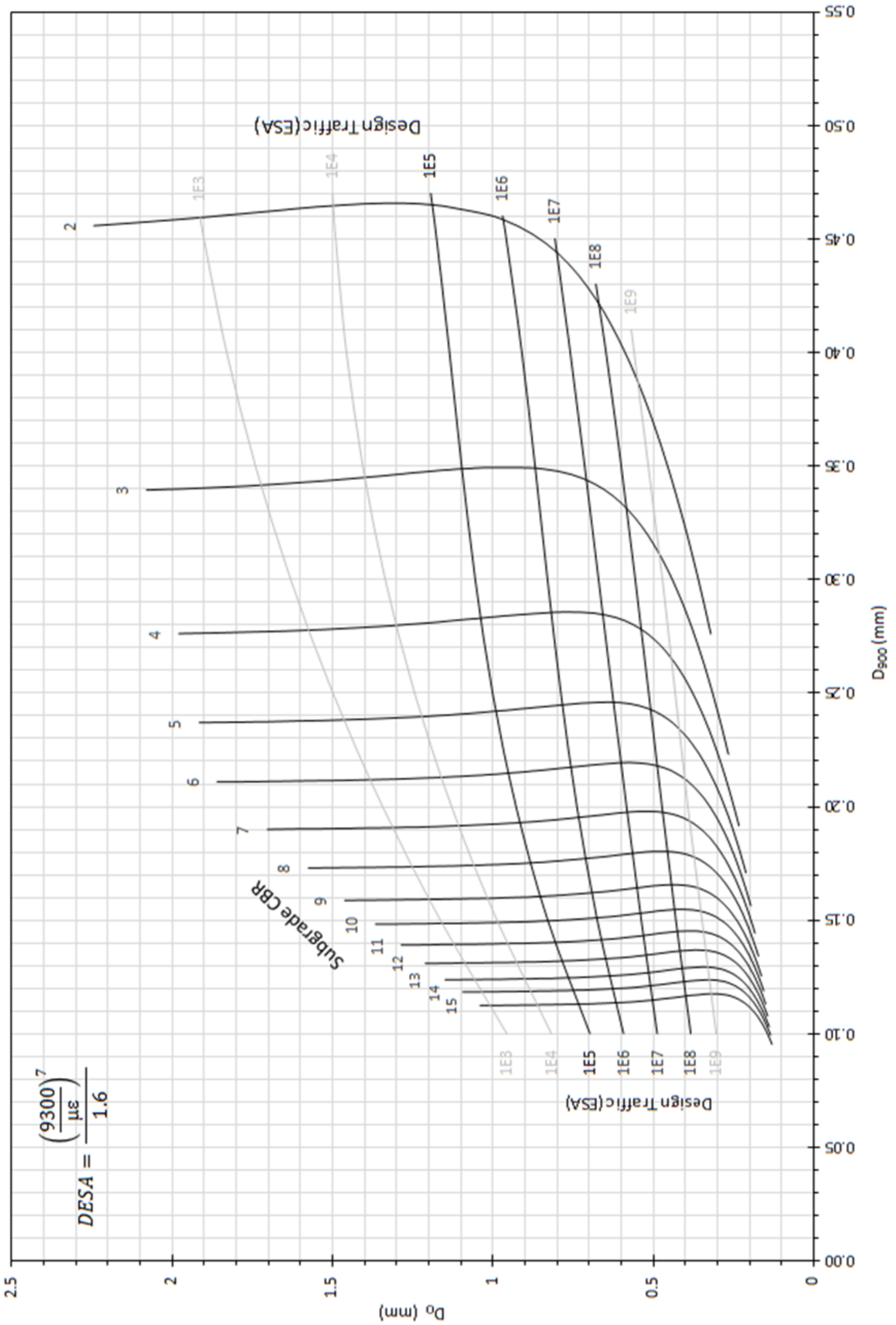


Figure 2.13.10.4.8(c) – Falling weight deflectometer traffic back analysis chart for unbound granular pavement with thin bituminous surfacing, 40 kN plate load



2.13.10.4.9 Deflection adjustments

2.13.10.4.9.1 Temperature adjustment

Deflection and back analysis results must be adjusted to the average working temperature of the pavement for the specific location. This average working temperature is referred to as the weighted mean annual average pavement temperature (WMAPT). For design expediency, WMAPT zones have been derived for the state as shown in Figure 2.13.10.4.9.1(a). Alternatively, WMAPTs for some populated centres are available (AGPT02).

The temperature of the asphalt at a depth of 25mm should be measured at regular intervals, and when weather conditions change. The temperature at this depth is considered equivalent to the average temperature throughout the layer. The frequency at which the temperature is measured needs to be carefully considered. More frequent measurements are required when:

- the temperature is changing quickly, or
- thick asphalt layers exist as such layers are more highly influenced by the asphalt temperature.

Where asphalt temperatures exceed 60°C, testing should cease as the results obtained become unreliable. Appropriate adjustments can be made during the analysis and design phases, including design by deflection (only).

Asphalt moduli obtained from back analysis should be moderated to take account of the pavement temperature at the time of testing. Strictly, a temperature correction should be applied, for each test, to each point on the deflection bowl; however, the correction factor is not constant – for example, for one unit of correction for D_0, the correction factors will be 0.44 units for D_{300}, 0.16 units for D_{600} and 0.20 units for D_{900} (interpolation for points between these offsets is acceptable). It is often impractical to apply these varying corrections to all points in all bowls. In such cases, it is acceptable to apply a correction factor obtained from Figure 2.13.10.4.9.1(b) to the moduli obtained from back analysis.

When designing using the deflection reduction method, individual D_0 and CF results are adjusted to correct for the difference in performance between the measured field temperatures and the WMAPT. The design steps for asphalt surfaced granular pavements with a total thickness of asphalt of 50 mm or more are:

- determine the ratio of WMAPT for the site to the measured temperature at the time of testing
- determine the correction factors from Figure 2.13.10.4.9.1(b) and Figure 2.13.10.4.9.1(c) as appropriate, and
- multiply the deflections (D_0s) and curvatures (CFs) by the corresponding correction factors.

For pavements with asphalt with a total thickness less than 50mm, temperature correction is not required.

Where the infrared sensor of FWD / HWD or TSD equipment is used to measure the pavement surface temperature, the Modified Bell's Equation (*Improved Design Procedures for Asphalt Pavements: Pavement Temperature and Load Frequency Estimation* (AP-T248)) should be used to estimate the mid-depth temperature of the asphalt and the relevant AMC (AGPT02) used to adjust the modulus to the WMAPT.

Note: To adjust measured deflections for temperature, the existing asphalt thickness is required. Asphalt thicknesses may be obtained from historical data (Section 2.6), measuring pavement cores, measurement taken during trenching (or excavation of test pits) or GPR results (accurately calibrated against cores, test pits or trenches).

Figure 2.13.10.4.9.1(a) – Weighted mean annual average pavement temperature zones

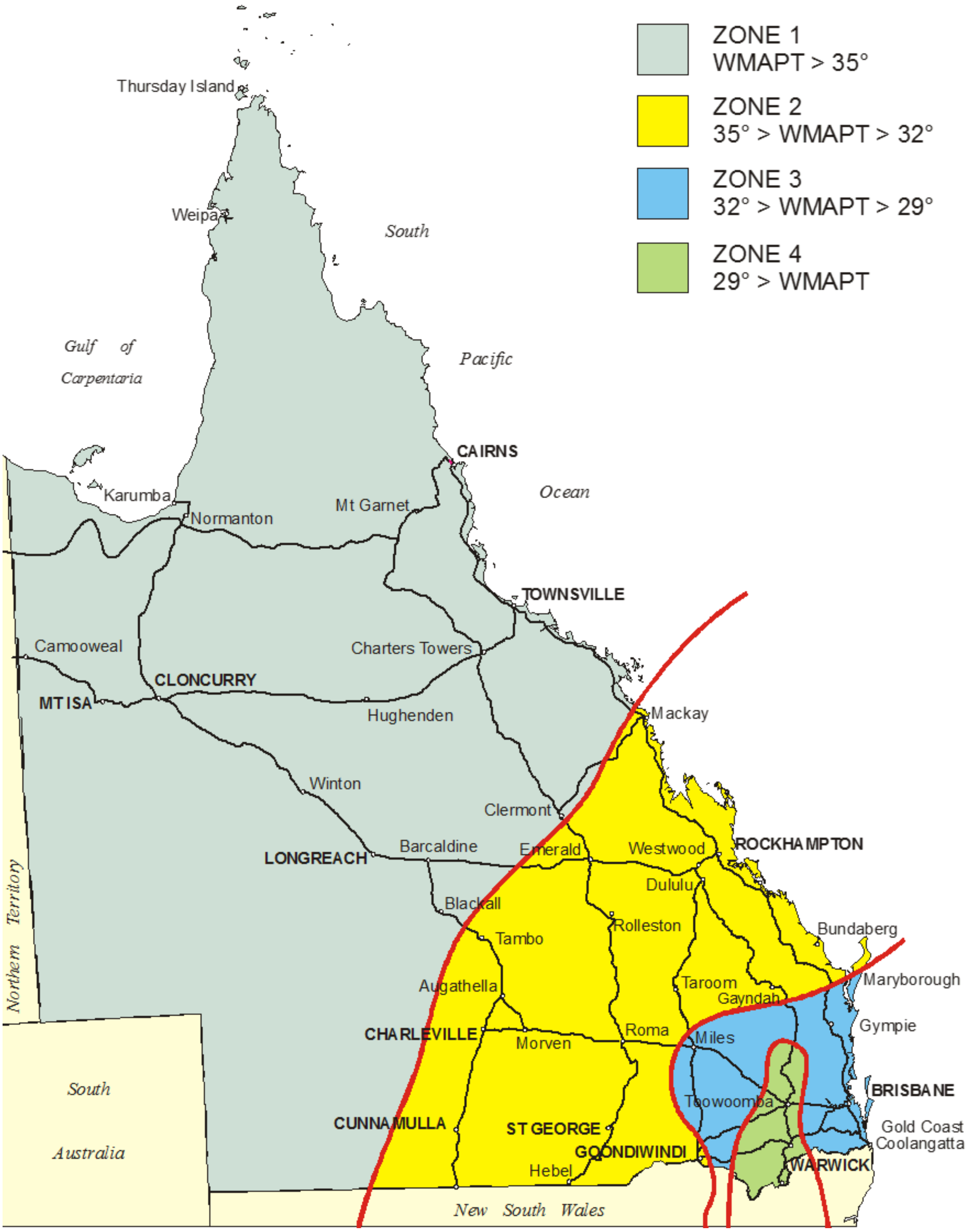


Figure 2.13.10.4.9.1(b) – Temperature correction for Benkelman Beam, deflectograph and normalised 40 kN falling weight deflectometer deflections results for a pavement with a thin asphalt surfacing or seal

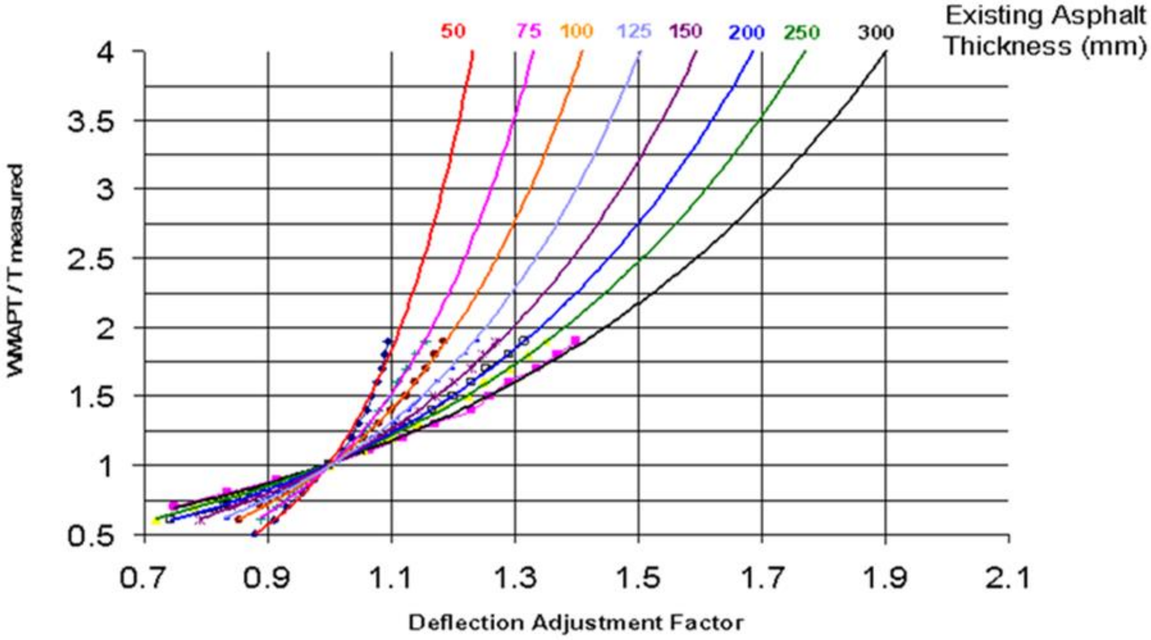
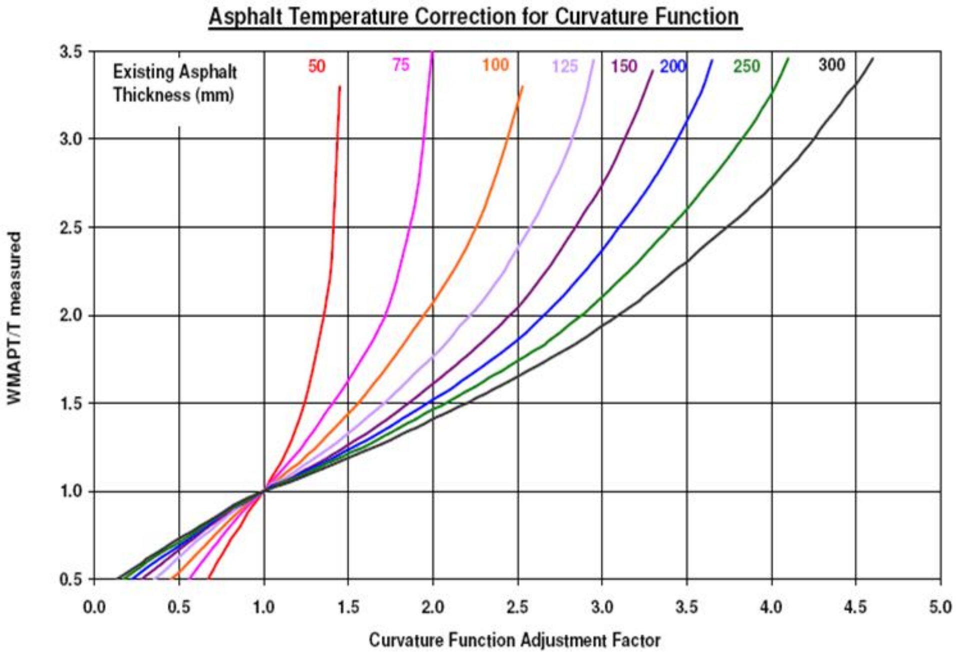


Figure 2.13.10.4.9.1(c) – Temperature correction for Benkelman Beam, deflectograph and normalised 40 kN falling weight deflectometer curvatures for a pavement with a thin asphalt surfacing or seal



2.13.10.4.9.2 Adjustment of deflection results for speed of loading

The modulus of asphalt increases as the rate of loading (load frequency) increases. Modulus values have been adopted for asphalt based on the normal in-service rate of loading and these values have been used to produce the overlay design charts (refer to Chapter 5). Adjustments to take account of the speed of loading during testing are only applied if the rate of testing the deflection response of a pavement is much slower or faster than the speed of loading experienced under 'real' traffic. The

Benkelman Beam is one type of testing equipment requiring such an adjustment. The deflectograph is another.

No correction for speed of loading during deflection testing is applied if FWD or HWD data are used to design the overlay; however, the rate of loading will influence the modulus of bituminous layers such as asphalt and foamed bitumen treated material, and therefore the measured deflection. The velocity V of a vehicle (km/h) is related to the loading frequency f (Hertz) by Equation 2.13.10.4.9.2(a) (AGPT02).

Equation 2.13.10.4.9.2(a) – Loading velocity of rolling wheel

$$\mathrm{V} = 2\pi\mathrm{f}$$

where the load pulse duration of falling weight equipment is T_p (seconds), the fundamental load frequency f_0 (Hertz) is given by Equation 2.13.10.4.9.2(b) (*Improved Design Procedures for Asphalt Pavements* (AP-R511)).

Equation 2.13.10.4.9.2(b) – Fundamental loading frequency of falling weight device

$$\mathrm{f_0} = \frac{1}{2\mathrm{T_p}}$$

By combining Equation 2.13.10.4.9.2(a) and Equation 2.13.10.4.9.2(b), Equation 2.13.10.4.9.2(c) is obtained.

Equation 2.13.10.4.9.2(c) – Equivalent loading velocity of falling weight device

$$V = \frac{\pi}{T_p}$$

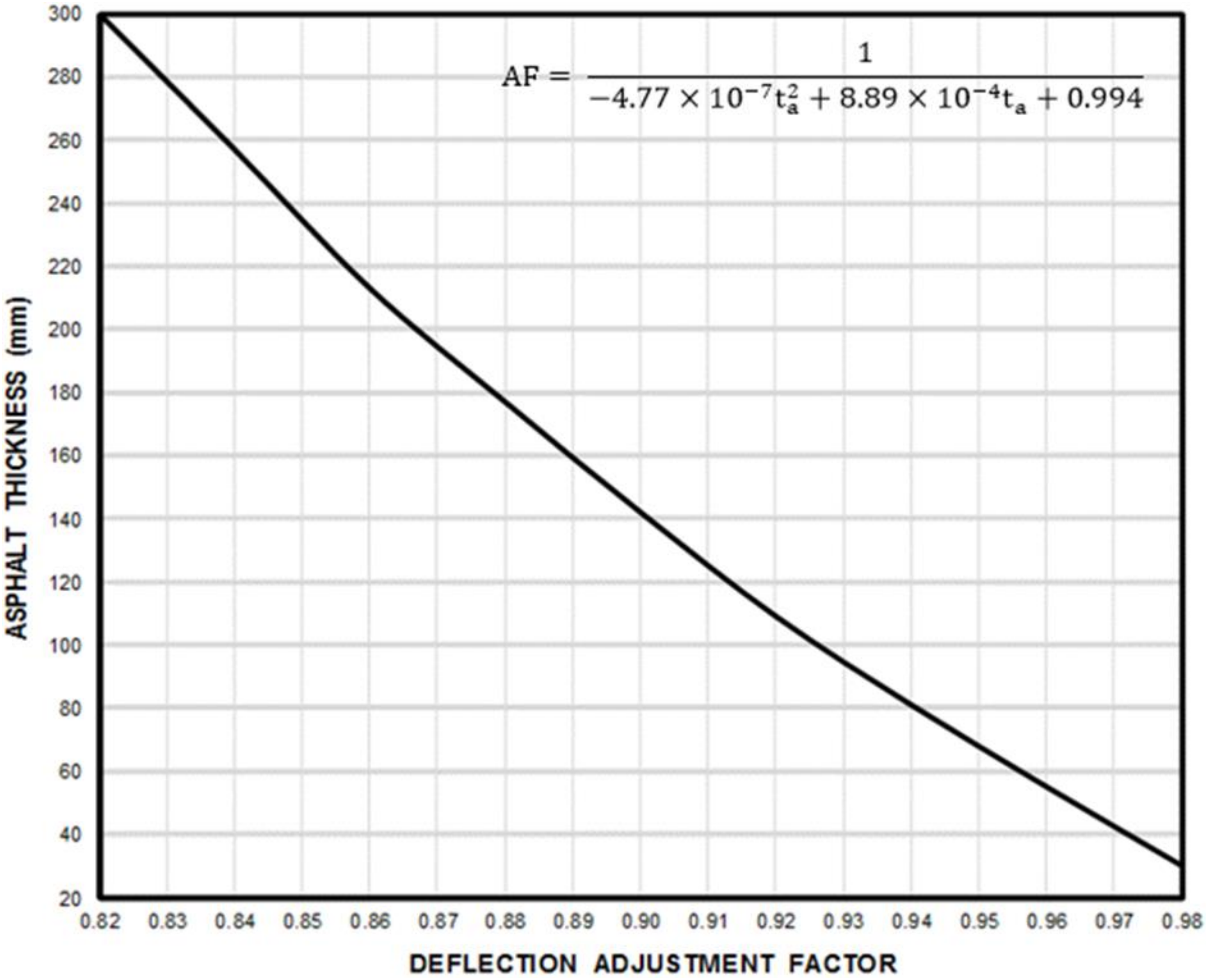
Section 2.13.10.1 indicates typical load pulse durations of the FWD correspond to load velocities between 90–125 km/h which is likely to exceed the operating speed of the pavement.

In such cases, it is recommended the back-calculated modulus be adjusted to the heavy truck operating speed of the pavement from the FWD load pulse duration using the appropriate AMC (AGPT02). Where mix-specific model parameters cannot be ascertained, a presumptive model may be adopted using equations 7 and 8 and Table 2.2 of AP-R511.

The rate of loading for Benkelman Beam and deflectograph is much slower than the operating speed of traffic; therefore, values derived from deflection testing using a Benkelman Beam and deflectograph should be corrected to figures that represent expected rate of loading.

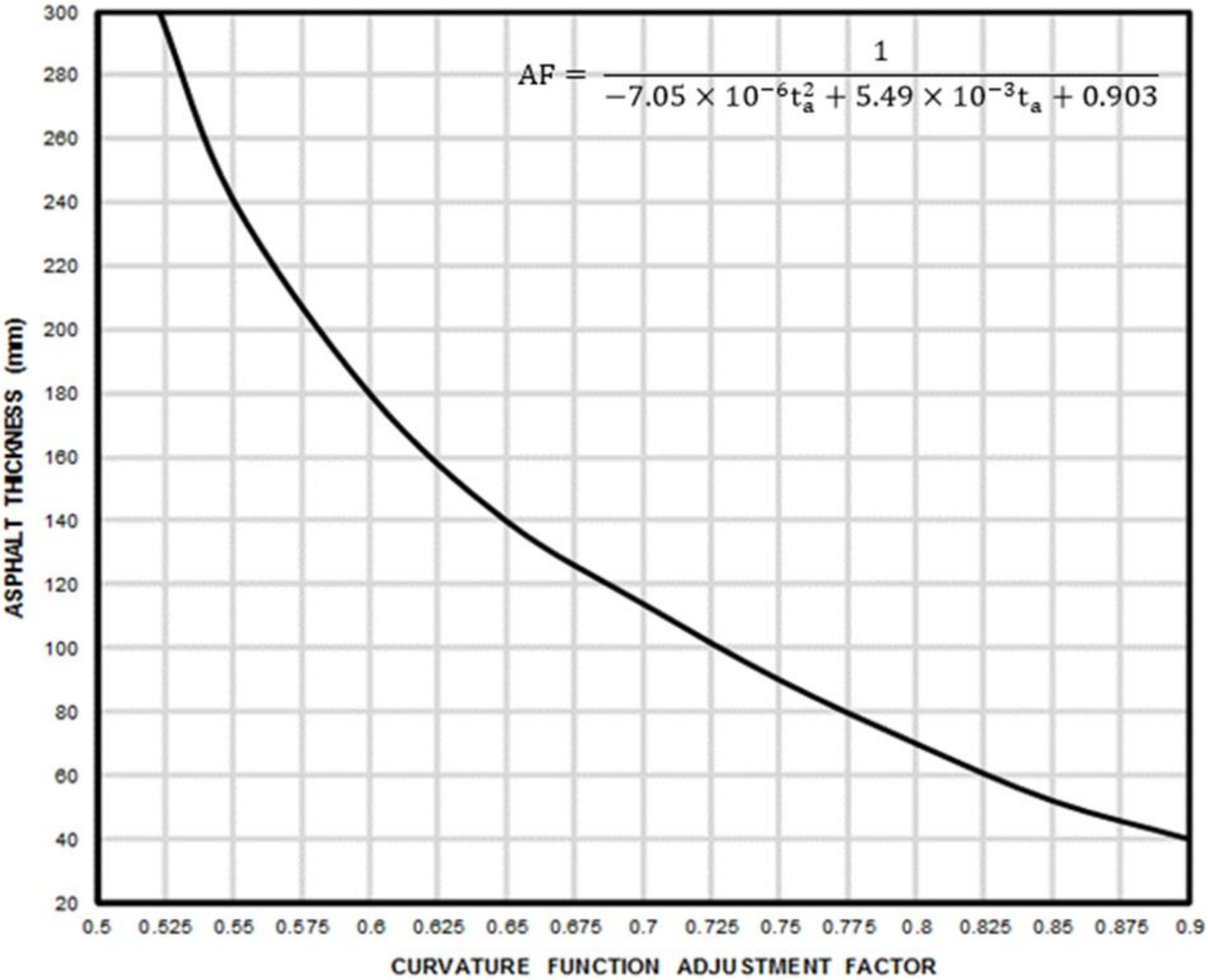
TSD equipment, as the name suggests, travels at traffic speed and adjustments for the speed of loading of bituminous material are likely to be minimal; notwithstanding, adjustments should be applied to back-calculated moduli rather than deflections (AGPT02).

Figure 2.13.10.4.9.2(a) – Speed correction of deflections for Benkelman Beam and deflectograph results derived for a pavement with a thin asphalt surfacing or seal



When designing using the deflection reduction method for pavements containing asphalt, correction of the recorded D_r and the representative CF to reflect the normal loading induced by vehicles at the 'normal' (vehicle) operating speed is required. Figure 2.13.10.4.9.2(a) and Figure 2.13.10.4.9.2(b) have been developed for this purpose. They provide adjustment factors for various total thicknesses of asphalt t_a. D_r is corrected by multiplying it by the appropriate factor determined from Figure 2.13.10.4.9.2(a). CF is corrected by multiplying it by the appropriate factor determined from Figure 2.13.10.4.9.2(b).

Figure 2.13.10.4.9.2(b) – Speed correction of curvature for Benkelman Beam and deflectograph results derived for a pavement with a thin asphalt surfacing or seal



2.13.10.4.9.3 Deflection moisture adjustment

The modulus of pavement materials, including the subgrade, are dependent on the moisture content.

The traditional method of compensating for moisture levels was to multiply pavement deflections by a factor determined by the pavement strength, subgrade type, depth to water table and climate zone.

If possible, however, deflection testing should be undertaken when the subgrade is in the weakest condition, normally at the end of the wet season. Since adjustment factors are influenced by things such as subgrade type, rainfall, location of water table and pavement types, they should be developed from studies conducted by each departmental Region or District.

Where studies of the local moisture environment are not possible, guidance is provided by Equation 2.13.10.4.9.3(a). The DMAF values in this equation are for use when producing designs based on the deflection reduction method. To adjust for moisture, the D_r and representative D_{900} obtained from testing in a specific season are multiplied by the DMAF given by the equation. Refer to Appendix B for the origins of Equation 2.13.10.4.9.3(a).

Equation 2.13.10.4.9.3(a) – Deflection moisture adjustment factor

$$DMAF = Au^3 + Bu^2 + Cu + D$$

where

Equation 2.13.10.4.9.3(b) – Deflection moisture adjustment factor subvariate

$$u = a\, Season^b\, Subgrade^c\, Climate^d\, Deflection^e + f$$

A =	8.2	a =	0.5389
B =	-30.7	b =	1
C =	38.9	c =	0.8327
D =	-15.4	d =	0.3518
		e =	0.7652
		f =	1

The variables of Equation 2.13.10.4.9.3(b) should be assigned in accordance with Table 2.13.10.4.9.3.

Table 2.13.10.4.9.3 – Deflection moisture adjustment factor variables

Variable	Instructions	Value	Interpretation
Season	Value determined from daily rainfall and evaporation, refer Appendix B	0	Wet, residual moisture level above equilibrium
		1	Dry, residual moisture level below equilibrium
Subgrade	Subgrade material to be determined from site investigation	0.41	Subgrade material other than silty or clayey silt
		1	Silty and clayey silt subgrade material
Climate	Value determined from AEP 20%, 15-minute extreme rainfall, refer Appendix B and Bureau of Meteorology intensity, frequency, duration tables	0.08	Semi-arid
		0.3	Sub-tropical
		1	Tropical
Deflection	Pavement strength determined from 40 kN representative maximum deflection D_r results obtained from Benkelman Beam, deflectograph, FWD or TSD equipment	0.76	D_r ≤0.9mm
		0.88	0.9mm <D_r <1.5mm
		1	D_r ≥1.5mm

In the wet tropics, stronger pavements are considered to exhibit a greater sensitivity to moisture infiltration than weaker pavements and are therefore assigned a higher DMAF.

Subgrades classified as either silty or clayey silt are considered to exhibit a greater sensitivity to moisture infiltration and are therefore assigned a higher DMAF than other types of subgrade.

In situations where the water table is within one metre of the subgrade level throughout most of the year, a DMAF of 1 (no adjustment) is to be applied.

The factors produced by Equation 2.13.10.4.9.3(a) are for guidance in situations where more reliable information is not available. They relate to pavements of full width construction with fair drainage conditions. Caution should be exercised when applying them to other situations (for example, to existing ‘boxed’ pavements).

The OWPs are susceptible to environmental forces while the moisture of the other wheel paths are constant. For this reason, DMAFs are only applied to the deflections measured in wheel paths other than the OWP, to attempt to simulate the anticipated weakest condition of the pavement (end of wet season).

When comparing OWP deflections with moisture adjusted deflections in other wheel paths, the following should be noted:

- The greater of the two deflection/s should be used to determine D_r.
- Where the adjusted deflections are lower than the corresponding OWP deflection/s, a check should be made to ensure other factors are not controlling the OWP deflection (for example, 'box type' construction trapping moisture or providing inadequate lateral restraint). If there are no obvious defects in the OWP that would cause high deflections, then the OWP deflections should be used to determine D_r.

2.13.10.5 Verification of pavement performance in overlay design using deflections (only)

A valuable evaluation procedure, and one essential element prior to design, is assessing whether the condition of pavement is consistent with the measured level of deflections. This may be done by:

- determining the past traffic (refer to Section 2.6.10)
- determining the D_{900} value (refer to Section 2.13.10.4.6)
- reading the tolerable deflection from the appropriate chart in Appendix C of Chapter 5, and
- comparing this with the D_r of the existing pavement (refer to Section 2.13.10.4.2).

The tolerable deflection and D_r may be compared to determine whether pavement distress is load associated or due to other causes. Where there is a reasonable correlation, the structural analysis is validated, and designs may be completed using the deflection reduction method.

Where D_r is higher than would be expected, considering the road’s condition (its tolerable deflection), the four most probable causes are:

- Deflection testing was carried out in extreme climatic conditions which do not represent a normal range of climatic conditions.
- The pavement’s moisture control system is inadequate and causing unusually high deflection results (for example, the surface is deeply rutted and cracked, or a boxed pavement is not draining properly, as illustrated in Figure 2.13.10.5). Solutions could include correcting the surface profile and/or providing an effective seal or providing adequate subsurface drainage (either subsoil drains or widening to provide a full width pavement). Overlay design based on moisture corrected deflection levels would then be appropriate.
- A recent reseal is giving a false impression of surface condition.
- The deflection response may be atypical for the type of pavement tested, in which case, further testing of pavement and subgrade material properties is required to determine the cause.

Figure 2.13.10.5 – Failures as a result of 'boxed' construction with inadequate drainage

Where D_r is lower than would be expected, considering the road's condition, there are two possible causes:

- the distress results from the inadequacy of pavement material to respond elastically under load – additional testing of the pavement and subgrade materials to determine their properties is needed to check this and, hence, determine the most appropriate remedial treatment, and/or
- the distress has been significantly increased by active factors not associated with load (for example, the degradation of pavement materials under environmental influences).

In either case, it would be inappropriate to proceed with an overlay design based on the deflection reduction method. Rather, alternative remedial treatments or design methods should be investigated.

2.13.10.6 Full depth asphalt, deep strength asphalt and flexible composite pavements

As with granular pavements with a thin asphalt surfacing or seal, full depth asphalt, deep strength asphalt and flexible composite pavements can be sectioned (refer to Section 2.13.10.4.2); however, the evaluation and design of such pavements needs to be carried out using the GMP (refer to Chapter 5).

2.13.10.7 Rigid pavements

Refer to Section 4.8 of this *Manual* and AGPT05 Section 4.9. Where the surface response of the pavement is likely to be insufficient to provide reliable back-calculation results, spectral analysis of surface waves (SASW) or similar seismic technology may be deployed to provide supplementary information. Inversion (back-calculation) of SASW measurements will be required to determine layer moduli. Where possible, the thickness of, at least, the rigid layer should also be reported. Receiver spacing should be selected to target a measurement range slightly greater than the anticipated depth to subgrade.

2.13.11 Materials properties

In addition to the techniques following, back analysis using deflection results can also provide information about the insitu strength of existing pavement materials.

2.13.11.1 Non-rigid pavements

2.13.11.1.1 Approach to the materials evaluation

Materials evaluation should follow and be guided by:

- the gathering and interpretation of historical data (Section 2.6)
- a visual assessment (Section 2.7)
- GPR surveys (if used, Section 2.8)
- the project brief (Section 2.10)
- future planning (Section 2.11)
- the gathering and interpretation of other relevant information (Section 2.12), and
- deflection testing (Section 2.13.10).

The evaluation of materials properties has two aims:

- it serves a diagnostic purpose by resolving anomalies (for example, between the pavement's condition and its load / deflection response) or by determining why the material has performed in a certain manner, and
- it is used to obtain parameters for back analysis and the design of rehabilitation treatments (if these are required).

The efficiency of materials sampling may be enhanced by collecting a greater quantity of samples than may have initially appeared necessary, and/or by conducting extra insitu testing. In doing this, the need to return to the site may be avoided (for example, this avoids the need to re-establish sampling and testing resources or to undertake fresh excavations; the latter of these may lead to further pavement failures or cause additional disruption to road users). By retaining excess samples following evaluation testing, further specialised tests can subsequently be conducted if an uncommon rehabilitation treatment is appropriate. It also allows test results to be checked (for example, by retesting a material when a 'suspect' result is obtained in the first instance).

As listed previously, information from several sources should be used to determine the most appropriate locations for materials sampling and testing. Further, they will often give a good indication of the extent and nature of the testing required. It may often prove as important to test an area of sound pavement as one suffering significant distress. This will assist in determining whether variations in a material's characteristics or in material types contributed towards or caused the problem. This may include variations arising from construction or environmental exposure.

Distress at the interface between different materials may indicate either compaction difficulties during previous stages of construction or differences in the permeability of materials.

At times, it may prove more informative to carry out testing of fundamental material qualities such as strength (for example, CBR) and permeability than the usual indicator tests such as moisture content, grading and Atterberg Limits (*21st ARRB Regional Symposium*) (ARRB, 1984)).

Where records of the pavement configuration are unavailable or considered unreliable, sampling from test pits or trenches will enable the transverse (pavement) profile to be observed and representative samples to be taken.

2.13.11.1.2 Granular materials

The first stage in evaluating a granular paving material involves identifying the various material types within the pavement structure and across the extent of the area under investigation (for example, within each subsection). This information may be available from construction records or, to some extent, be indicated by the surface condition, but may still need to be confirmed by field investigations.

Disturbed samples may be recovered from test pits or trenches or be obtained by drill probing. Though more efficient, the latter method less accurately identifies layer thicknesses and can alter the gradings of samples. It may also result in the contamination of samples (for example, materials mix at layer boundaries). If this occurs, then the test results may not be representative. Importantly, staff carrying out field sampling and testing should be fully briefed on the purpose of the investigation so they may appropriately respond to the discovery of the unexpected.

As part of the field-testing process, the insitu characteristics of materials such as density, strength and possibly permeability should be measured. The exact tests required will depend on a preliminary assessment of the distress mechanism or the rehabilitation options within scope.

Laboratory testing of recovered samples to remain unbound should be based on Transport and Main Roads' Specification (Measurement) MRS05 *Unbound Pavements* (MRS05) and Technical Specification MRTS05 *Unbound Pavements* (MRTS05). Where the materials fail to meet these standards, consideration should be given to whether mechanical or chemical modification would enable compliance with MRS05 and MRTS05 to be achieved. Alternatively, the unbound materials can be modified or stabilised; however, the pavement may no longer be a flexible one (see Chapters 4 and 5). Acceptance of a marginal or non-standard material as part of a rehabilitated pavement must only be with the approval of the department's District or Regional Director. Alternatively, if materials do not comply with MRS05 and MRTS05, modification or stabilisation may be considered. Further, testing additional to that prescribed in MRS05 and MRTS05 may be appropriate.

Laboratory testing of unbound granular samples may extend to tests that allow their suitability for stabilisation to be investigated; for example, additional material / samples may be required so stabilising testing can be completed. This may require separate combined samples to be taken that represent the proposed stabilised layer (for example, take top 250mm of granular material in one sample) and subjected to modification / stabilisation testing. This may include laboratory testing to determine:

- supporting conditions (for example, potential reactivity of subgrade, strength of support)
- the type of binder(s) suitable, if any
- whether sulphates that inhibit stabilisation reactions are present
- the mix design (if stabilisation is viable), and
- what strengths may be used in the rehabilitation treatment design process (if treatment is required).

When combined with more traditional testing of unbound granular materials, this allows multiple options to be explored (an example would be an overlay-only solution where insitu unbound granular materials remain in their current state versus a stabilisation and overlay option where existing materials are modified insitu).

Where a pavement has failed prematurely, close attention should be paid to insitu moisture contents and possibly the insitu degree of saturation. Examination of the transverse (or vertical) moisture gradient within pavement layers can be particularly worthwhile. There is considerable evidence significant pore pressures will develop in paving materials under traffic loadings where the degree of saturation exceeds 70%. Notwithstanding, problems may occur in MRS05 and MRTS05 Type 1 material (crushed rock) at levels of saturation as low as 60%. The department's *Test Method Q146: Degree of saturation of soils and crushed rock* (Q146) describes the method to be used for determining the degree of saturation of compacted paving materials.

2.13.11.1.3 Stabilised and modified materials

The performance of stabilised and modified materials can be assessed against the original design expectations. The assessment of their insitu condition also allows rehabilitation design to account for their expected future performance. If the purpose of the stabilisation or modification is simply to modify its granular characteristics or improve the material (for example, lime stabilisation of a gravel with high plasticity), then it may be tested and evaluated in a similar manner to an unbound granular material. If the purpose had been to bind the material to improve strength, then other properties such as its UCS should be tested. This will require cores to be taken from the insitu material.

Layer thicknesses, compaction and the effectiveness of the modification / stabilisation through the full depth of the layer should also be checked (for example, test the compaction of the top half of the modified / stabilised layer and then test the bottom half of the modified / stabilised layer).

The extent of delamination and erosion between (bound) layers and at cracks should also be assessed in bound materials.

2.13.11.1.4 Asphalt

If inappropriate asphalt mixes / designs are selected / used, or inappropriate or inadequate construction techniques are used, failures of asphalt layers can be a problem. This is of concern for areas of high traffic loading intensity, in terms of vertical loading or shear stress (for example, at intersections), or in warmer climates. Where both conditions exist, the potential for asphalt failures are highest.

The evaluation of an existing asphalt layer may address any of the following characteristics:

- stability, flow, stiffness and air voids
- permeability
- viscosity
- flushing and stripping
- aggregate degradation or poor aggregate used in construction
- delamination, and/or
- surface texture and skid resistance.

The use of non-standard asphalt or proprietary mixes is not widespread, and their use in the department's works is uncommon. Where such a mix is present in the existing pavement and is found to need some form of remedial treatment or strengthening, the department's Pavements, Research and Innovation Unit (of Pavements, Materials and Geotechnical Directorate, Engineering and Technology) should be consulted.

2.13.11.1.4.1 Stability, flow, stiffness and air voids

Design parameters are determined by Marshall, wheel tracker and four point bending tests, which relate to the rutting and fatigue behaviour of the asphalt mix. Tests may be carried out on core samples and the results compared with the original design and quality control test values. Poor construction practice may result in inadequate compaction, segregation or reduced layer thickness. Over-compaction may lead to instability and permanent deformation of the asphalt mix when the asphalt consolidates under traffic loading. It may also lead to a flushed surface when the asphalt is subject to traffic.

Core samples may be used to help identify whether rutting exists solely within the asphalt layer or if it is caused by the instability of lower layers. Trenching provides an even clearer indication of the cause and location of rutting.

2.13.11.1.4.2 Permeability

Water may enter the pavement because of defects in the asphalt such as a high voids content, a ravelling surface or an oxidised binder, or because of cracking. Amongst other things, it can lead to stripping of the binder leading to failure of the asphalt.

A styrene-butadiene-styrene (SBS) polymer modified waterproofing seal must be provided under asphalt surfacing layers until such time the asphalt contractor has demonstrated a history of compliance with the insitu air voids requirements (PDS). In addition, during construction, it is desirable asphalt layers below the final surface layer not be left exposed to rain unless sealed (for example, an AC20 base layer trafficked during construction may need to be sealed to ensure it is not exposed to rain), or unless they have a characteristic value of compaction of at least 93%.

In pavements with multiple asphalt layers, and where the surfacing is OGA, a waterproofing seal between the asphalt surfacing and the intermediate layer is typically provided. Further, OGA layers should always be placed over a surface that has free transverse drainage provided on the low side of the OGA layer. Placing OGA over a surface formed by cold milling can lead to accelerated stripping of the underlying asphalt. Grooves formed in the underlying DGA from the milling process may allow small water reservoirs to develop at the interface. In addition, poor level control during milling can result in depressions at the interface within the pavement which tend to allow the ponding of water on a larger scale. If OGA surfacing is to be used over a milled surface, a correction course of a small stone DGA such as AC10 must be placed on the milled surface to provide an interface to facilitate the shedding of water which permeates through the OGA. An alternative treatment is to use a fine-toothed mill, followed by a 7mm or 10mm seal. This may avoid the need for the DGA correction layer.

OGA must be placed on a surface that drains (for example, is free of depressions that can pond water). The placement of an asphalt corrector course may be required to meet this requirement.

2.13.11.1.4.3 Viscosity

Problems with bitumen viscosity may lead to either flushing or ravelling of the asphalt. Standard procedures exist for both sampling the asphalt and testing the viscosity of its bitumen binder. These results may be compared with the data obtained from the previous construction quality control testing to determine the remaining life of the oxidised material. Where recycling is being considered, the additional bitumen or other additives required to achieve the required viscosity must be assessed via sampling and testing.

2.13.11.1.4.4 Flushing

Flushing can be the result of sealing over an already flushed surface, having too low a value for air voids, sealing over patches which have not had sufficient time to strengthen (routine maintenance should precede a reseal by at least two months), using an inappropriate spray rate for an underlying seal (for example, too high), using excessive cutter in an underlying seal or by placing an asphalt layer before the amount of cutter remaining in the underlying primer seal or prime has reduced to an acceptable level (for example, via evaporation).

2.13.11.1.4.5 Aggregate degradation or poor-quality aggregate used in construction

Where degradation of the aggregate is suspected of causing ravelling or skid resistance problems, the condition of the stone should be compared in areas with and without traffic loading. If an observable difference is noted, the aggregate should be subjected to weathering, crushing or polishing tests as appropriate. If it is suspected poor (for example, non-complying) aggregate was used in the construction of other layers, that aggregate should be similarly tested.

2.13.11.1.4.6 Delamination

Where areas of an asphalt layer have been totally removed from the underlying layer or where crescent cracking is evident, there are several possible causes. If the exposed surface (of the underlying layer) is free of adhered bitumen and the adjoining edges of the layer can be lifted, the delamination is probably the result of a poor bond caused by either contamination of the original surface, poor construction (for example, tack coat omitted) or water entering through the surface and breaking the bond between layers. If, however, there are traces of asphalt still adhering to the exposed surface (of the underlying layer) and the surrounding surface (of the overlying layer) is flushed, then the delamination is probably caused by the asphalt adhering to vehicle tyres.

2.13.11.1.4.7 Surface texture and skid resistance

These should be tested as described in Section 2.13.7 and the SRMP.

2.13.11.1.5Sprayed bituminous seals

The evaluation of a bitumen seal may address any of the following characteristics:

- permeability
- viscosity
- flushing and stripping
- stone degradation or poor-quality aggregate used in construction
- de-lamination and/or
- surface texture and skid resistance.

2.13.11.1.5.1 Permeability

Water may enter the pavement because of defects or deficiencies in the seal such as: stripped stones, insufficient residual bitumen (for example, application rate too low), binder oxidation, or because of surface cracking. The department's *Test Method Q707A: Permeability of road surfacing and granular materials – even flow field permeameter* (Q707A) and *Test Method Q707B: Permeability of road* *surfacing and granular materials – rapid flow field permeameter* (Q707B) describe field permeability test procedures.

2.13.11.1.5.2 Viscosity

Problems with bitumen viscosity may lead to flushing or stripping of the seal. Transport and Main Roads' *Technical Specifications MRTS17 Bitumen and Multigrade Bitumen* (MRTS17) and *MRTS18 Polymer Modified Binder (including Crumb Rubber)* (MRTS18) specify the sampling and testing of the viscosity of the bitumen in a spray seal, the results of which may then be compared with the design value. The data may also be used to determine the remaining life in an aged seal.

2.13.11.1.5.3 Flushing

Flushing and stripping are often the result of an 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, using an inappropriate spray rate for an underlying seal (for example, too high), sealing over patches which have not had sufficient time to strengthen (routine maintenance should precede a reseal by at least two months), using excessive cutter in the binder or by placing a seal before the amount of cutter remaining in the underlying primer seal or prime has reduced to an acceptable level (for example, via evaporation). Stone penetration into the granular base is also a common cause of flushing; therefore, flushing may be indicative of poor pavement surface preparation or a base with inadequate strength, poor compaction and/or a high moisture content.

2.13.11.1.5.4 Stripping

Stripping can result from the use of dusty or poorly pre-coated aggregate, insufficient residual binder, a poor aggregate spread rate, insufficient cutter in the binder in cool or cold conditions, construction when the temperature is too low 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 areas subject to high shear stress (for example, at intersections where turning and braking occur). Stripping can also occur where there has been excessive penetration of the binder into an underlying granular base course, if it is particularly porous.

When polymer modified binders (PMB) are used during adverse weather conditions, the chances of stripping are greatly increased, especially if no specific mitigating measures are employed.

2.13.11.1.5.5 Stone degradation or poor-quality aggregate used in construction

Where degradation of the stones is suspected as the cause of stripping or skid resistance problems, their condition should be compared in areas with and without traffic loading. If an observable difference is noted, the stone should be subjected to weathering, crushing or polishing tests as appropriate. If it is suspected poor (for example, non-complying) aggregate was used in the construction of underlying layers, that aggregate should be similarly tested.

2.13.11.1.5.6 Delamination

Where areas of the seal have been totally removed from the underlying material, a visual inspection will normally identify the cause. If the exposed surface (of the underlying layer) is free from adhered bitumen and the adjoining edges of the seal can be lifted (for example, Figure 2.13.11.1.5.6), the delamination is probably caused by poor adhesion (as for an asphalt). If, however, there are traces of bitumen still adhering to the exposed surface (of the underlying layer) and the surrounding surface (of the seal) is flushed, then the delamination is probably caused by the seal adhering to vehicle tyres.

Delamination can also be a result of placing a seal on a poorly prepared substrate. A dusty substrate, for example, will mean the bond between the seal and substrate is suboptimal and this may lead to delamination.

Figure 2.13.11.1.5.6 – Delamination of a seal

2.13.11.1.5.7 Surface texture and skid resistance

These should be tested as described in Section 2.13.7 and the SRMP.

2.13.11.1.6 Subgrade

The support provided by the subgrade is one of the most important determinants of the thickness of any new pavement, including widenings. This is also the case when stabilisation / modification is to be considered. For overlays, however, the insitu stiffness of the whole pavement becomes the controlling factor.

The PDS provides guidance for determining subgrade CBR values for use in new pavement design. An additional approach to determine the effective in-service CBR for design of new pavements involves (insitu or laboratory) testing of a similar subgrade beneath a sealed pavement with similar moisture conditions. This method is suited to pavement widenings where an ideal simulation of the proposed subgrade is provided. Testing the subgrade of an existing pavement is also required to choose and design rehabilitation treatments for an existing pavement. If records have been kept of the existing pavement’s design subgrade CBR, they can be compared with test results. This is of great benefit as it helps to determine the extent to which in-service strength has varied, due to changes in moisture content or density.

Where the design thickness of a proposed pavement widening is less than the existing pavement, there is a possibility the step-up in the subgrade could act as a barrier to the movement of water. The permeability of the subgrade material at its anticipated in-service density should be compared with that of the base or sub-base material of the widening. Where it is lower, subsurface pavement drainage should be provided (for example, strip drains at the interface between the existing pavement and the new widening). Similarly, differences between pavement layers (for example, difference in total asphalt thickness) can also prohibit movement of water, and drainage may need to be provided.

With partial replacement or insitu stabilisation / modification of an existing pavement, it may be impractical to rework the subgrade. If this is the case, insitu CBR values, corrected for possible moisture content changes, should be adopted. Where replacement of the subgrade is required, usually in selected locations, care should be taken to ensure the depression in the surface of the subgrade does not become a trap / sink for water.

The measurement of subgrade moisture contents and their variation along and across the pavement will also serve as a guide to the performance and effectiveness of the pavement's moisture control system.

Finally, testing to determine the support conditions is important (for example, potential reactivity of subgrade, strength of support).

2.13.11.2 Rigid pavement materials

Material-related distress in pavements containing concrete will usually result from inadequacy of its strength or durability. A strength deficiency will appear as uncontrolled cracking and spalling at the edges of slabs (the latter being for jointed concrete pavements). The construction history of the pavement should be checked to determine whether the cracking occurred because of poor timing of joint saw cutting (for jointed concrete pavements), construction not complying with the design / specifications / standards (for example, constructed too thinly) or has occurred over the service life of the pavement.

Core samples of the concrete may be taken to determine its density and compressive strength, and by inference, its flexural strength for comparison with the original design values. These cores will also serve to confirm layer thicknesses and can be used as an input for the design of rehabilitation treatments (if required).

Spalling at expansion joints in jointed concrete pavement may occur because of debris wedging in the unsealed or ineffectively-sealed joints. Joints should always be sealed at initial construction. Subsequently, the maintenance regime should include regular inspections to ensure joints are clean and sealed with an effective joint filler. Joints may need to be resealed during the life of a jointed concrete pavement.

Durability problems will lead to erosion of the surface. Durability tests may also need to be carried out on core samples.

The Austroads *Guide to Pavement Technology Part 4C: Materials for Concrete Road Pavements* (AGPT04C) contains further discussion about materials used in the construction of rigid pavements.

Apart from concrete materials, the materials properties described in Section 2.13.11.1 apply to non-rigid pavements (for example, for subgrades, refer to Section 2.13.11.1.6).

2.14 Moisture control system

Drainage of pavement and subgrade layers has a great influence on pavement performance. Many pavement designs and materials will perform quite adequately, provided moisture is limited (to favourable levels); however, if the pavements are permitted to 'wet up' (moisture contents allowed to rise), rapid deterioration can occur. In undertaking a pavement evaluation, it is important to determine the adequacy of the current surface and subsurface drainage systems in preventing water infiltration into the pavement or, if it does penetrate, removing water from the pavement.

It is also important to determine the extent to which the performance of the pavement relies on the integrity of the moisture control system; for example, the failure of a drainage system designed to relieve conditions associated with water springs would have a greater and more immediate effect on the performance of a pavement than the slow increase in permeability of an aging seal.

A moisture control system may fully meet its design parameters but be ineffective for the actual in-service conditions. Records of the purpose of specific drainage measures are essential as they provide an initial basis for assessing performance. Even where there are no specific drainage structures, and moisture is controlled by the selection or combination of materials with certain characteristics, design records will provide information vital in ensuring the rehabilitation process does not detract from the functionality of the overall moisture control system.

A range of factors may affect the moisture conditions in a pavement and, though they should have been considered during the initial design phase, they may require reassessment as part of the evaluation process. Table 2.14 details the more significant of these factors.

In addition to the water intentionally or inadvertently built-in during construction, there are a variety of mechanisms that may allow wetting of the pavement or subgrade. These should also be considered as part of the moisture control system evaluation. They include:

- seepage of groundwater
- movement of a water table under a road
- rainfall infiltration through the road surfacing
- capillary moisture from the verges
- capillary water from a water table
- vapour movements from below the road
- lateral movement of moisture from pavement materials comprising the road shoulder.

Table 2.14 – Some factors affecting the moisture condition of road pavements

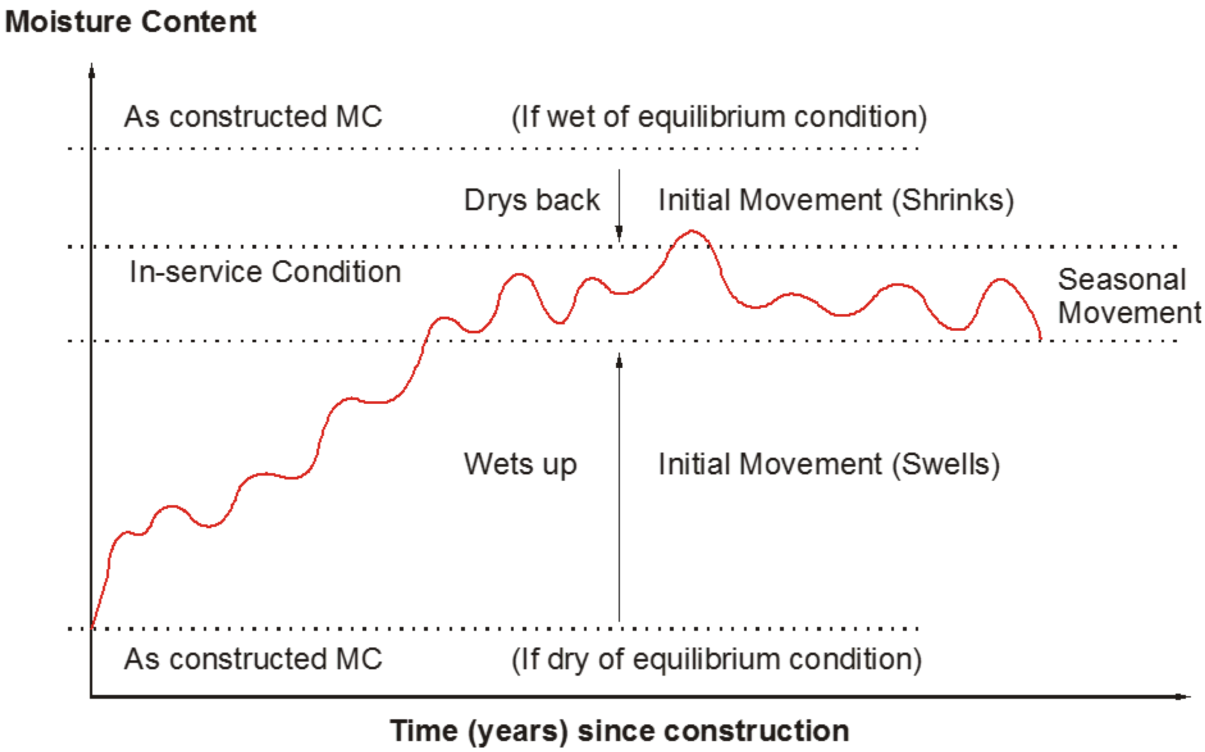
Aspect	Factors
General drainage condition	• position / configuration / depth of catch drains, table drains and subsoil drains • crossfall including shoulder crossfall • superelevation and how it is applied • longitudinal grade • vegetated shoulders versus sealed shoulders • cut versus fill versus cut / fill transitions • presence of subsurface drainage systems • presence of stormwater and cross-drainage systems and structures • type cross-section (which shows how some of these elements are combined)
Position of the water table	• static versus variable • deep versus shallow
Climate	• amount of rainfall • evaporation • temperatures • thermal gradients.

Aspect	Factors
Construction	• boxed versus full width pavement • presence of trenches (for example, public utility plant trenches) • moisture content at time of compaction and sealing
Maintenance	• whether subsoil drains or the components or drainage systems are maintained / flushed • whether surfacings are maintained (for example, surfacing replaced before it reaches the end of its service life, cracks sealed promptly)
Surrounding landform / topography	• presence of drainage depressions, swamps, and so on • presence of adjacent rivers, dams, irrigation areas and so on • presence of vegetation • runoff and permeability of soil strata.
Subgrade, pavement and shoulder materials	• particle size • pore size • density • mineralogy • shrink / swell properties • grading • permeability • salinity.

2.14.1 Moisture environment

If an area of ground is sealed by an impermeable layer such as a pavement, the moisture conditions beneath the central section of that area change until a long-term equilibrium moisture condition is attained. This long-term equilibrium moisture condition is independent of initial conditions, does not significantly vary from season to season, and is in equilibrium with a groundwater table and/or average climatic influences at the edges of the sealed area (refer Figure 2.14.1). Even in very wet areas, the equilibrium moisture content of well-drained pavement layers is equal to, or drier than, the optimum moisture content (OMC). Equilibrium moisture conditions are, therefore, quite favourable.

Figure 2.14.1 – Moisture content versus time



The time taken to change from the initial moisture conditions to long-term equilibrium conditions varies according to the texture or permeability of the subgrade soil. With heavy clay soils, it may take three to four years. This is of practical consequence: the effects of soaking the subgrade during construction can persist for a long time and detrimentally affect pavement performance.

At the edges (for example, shoulders, OWP), the moisture conditions are much more variable. Consequently, investigations should always include examination of the insitu moisture content towards the edges of a pavement.

In addition to these, the following factors are to be noted:

- Reducing the permeability of pavement layers by selecting appropriate dust ratios for gravels and effective compaction of all layers is important in controlling moisture, particularly over sensitive subgrades.
- Moisture must be tightly controlled during construction and overlying layers should not be constructed until the underlying layer has reached (for example, dried out to) an appropriate moisture content. During insitu stabilisation / modification, care should be taken to ensure excess water is not added.
- High fills on permeable strata can cut off moisture flows by compressing the layer(s) underneath the fill material.
- Low fills can include perched water tables.
- Rock bars or other low-permeability strata can trap moisture in soil strata on the upslope side.
- Rock cuts may require special treatment (for example, install a drainage layer).

2.14.2 Effects of subsurface drainage on moisture content

Where evaporation and soil suction are drawing moisture to the surface of a cracked pavement, the moisture content of the pavement may increase, and its effective strength decrease, when the surface is sealed.

For subgrades with low permeability ($k<10^{-7}$ m/s) trench drains may have little effect on lowering the water table midway between the trenches unless there are significant cracks, fissures or more permeable seams within the material. The reason for this is capillary rise. Typical values for the height of capillary rise in various soils are given in Table 2.14.2 and Figure 2.14.2 (*Subsurface Drainage – Some Do's and Don'ts* (Gerke, 1982)).

Drainage blankets and systems can easily irrigate the subgrade. It is important to ensure:

- outlets are above the inverts of table drains and standing water levels, and
- blanket thicknesses exceed the expected capillary rise.

The drainage systems must have an adequate (resultant) grade and be free from depressions so as not to pond water.

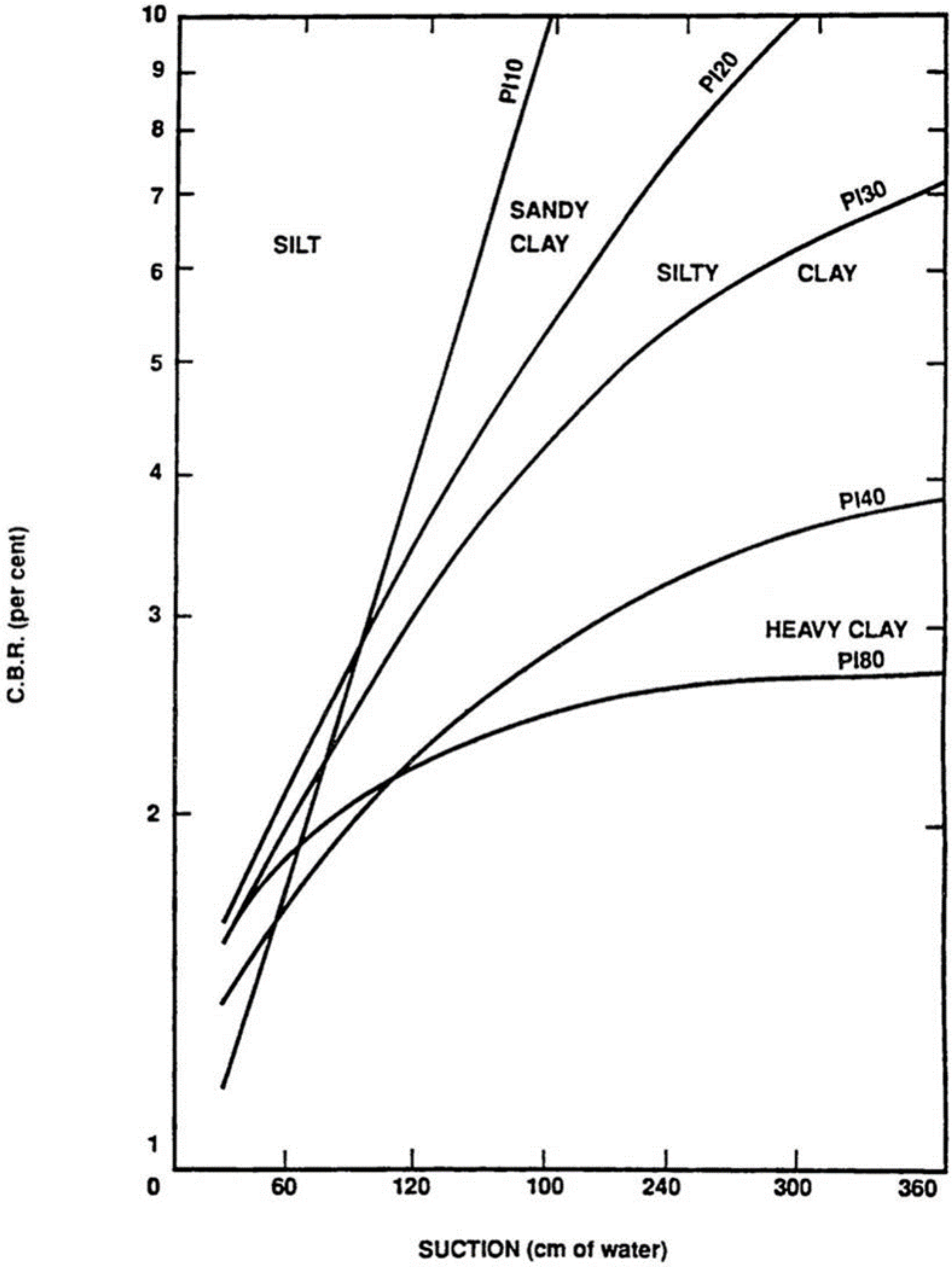
The height of capillary rise shown in Table 2.14.2 will increase, depending on:

- height of embankment
- environmental condition (for example, decreased humidity resulting in increased suction), and
- salt content and decreased humidity in arid environments (this has a double effect of increasing the capillary rise).

Table 2.14.2 – Typical height of capillary rise in different materials

Soil type	Height of capillary rise (mm)
Coarse Sand	20–50
Sand	120–350
Fine Sand	350–700
Silt	700–1500
Clay	2000–>4000

Figure 2.14.2 – Relationship between California Bearing Ratio value and suction for soils of various plasticities



Source: Gerke, 1982

2.14.3 Effects of moisture content changes on performance

Depending on the degree of saturation, failure of a pavement structure can occur in:

- rapid failure of an unbound pavement layer caused by shearing / bearing failure
- premature rutting of an unbound pavement layer or overlaying layers because of the decreased stiffness of the material
- lifting of the wearing course or asphalt layer caused by positive pore pressure, and/or
- embedment of the cover aggregate in the bituminous seal wearing course into the unbound granular base caused by a softening of the unbound pavement base.

The following factors are to be noted:

- The moisture effects are more significant on the subgrade than on the pavement materials.
- A 1% rise in the moisture content of a gravel base may cause CBR to fall from 100% to 40% (ARRB, 1984).
- Bituminous seals: Excessive water can lead to stripping; delamination; pressure-induced erosion; and modulus reduction in the underlying pavement materials and a loss of tensile strength in the underlying granular materials. Saturation can reduce the modulus of the underlying granular materials by 30% or more.
- Asphalt: Water can lead to stripping, delamination, pressure-induced erosion, modulus reduction and a loss of tensile strength. Saturation can reduce the modulus by 30% or more.
- Portland cement concrete: Moisture has only a slight effect on its modulus and strength, but it can influence curing and warping stresses in the slab. It can also lead to pumping and erosion of fine material from beneath the slab and, thus, removal of its support (lead to the formation of voids).
- Aggregate base and sub-base: Added moisture will result in a loss of stiffness for all unbound aggregate materials. Reductions in modulus values of more than 50% have been reported.
- Modified / stabilised bases: For bitumen stabilised bases, modulus reductions of up to 30% can be expected. For cementitiously and lime stabilised bases, modulus reductions are slight; however, these materials are susceptible to erosion.
- Subgrade: Though free-draining soils experience little reduction in modulus, those with low to very low permeability could experience modulus reductions of 50% or more.

2.14.4 Moisture control considerations in type cross-sections

Moisture can enter the pavement through its surface because of:

- unsealed shoulders
- inadequate pavement surface drainage during the construction phase
- exposure of the subsoil drainage during construction
- porous asphalt (for example, poor compaction)
- pavements left primed but not sealed for extended periods

- DGA base or base binder layers exposed to rain during construction (for example, AC20 base layers are relatively permeable if their characteristic value for compaction is less than 93%)
- inadequate residual bitumen (in a sprayed seal, including primer seals)
- the omission of seals under asphalt surfacing layers, especially OGA
- inappropriate maintenance practices (for example, cracks are not sealed promptly, surfacings are not renewed before they become permeable)
- joint seals are not maintained in jointed concrete pavements
- on superelevated pavements the 'high side' of unbound granular layers being left exposed and without subsurface drainage, and/or
- table drains not being maintained.

The following factors should be considered during the design and construction of moisture control systems:

- No subsurface drainage system can perform acceptably unless an adequate overall drainage system has first been provided (for example, adequate surface and stormwater systems).
- Beware of possible instability of drainage blankets.
- In Queensland, bituminous surfacings suffer from surface oxidation of the binder at about 7–12 years after initial placement, resulting in ravelling and increased permeability of the surface.
- Accumulated debris, silt and sand can slowly raise the level of the shoulders above the edge of the pavement, preventing surface water from shedding off the pavement.
- Deformation of the pavement (for example, rutting) can prevent or inhibit the shedding of surface water (cracks in ruts are particularly problematic).
- For all new pavements and in all rehabilitation designs, a SBS polymer modified waterproofing seal must be provided under asphalt surfacing layers until the asphalt contractor has demonstrated a history of compliance with the insitu air voids requirements (PDS).
- During construction, it is desirable asphalt layers below the final surface layer are not exposed to rain. A bituminous seal between construction layers would help in this regard.
- Placing OGA over a surface formed by cold milling can lead to accelerated stripping of the underlying asphalt. Grooves formed in the underlying DGA from the milling process may allow small water reservoirs to develop at the interface. In addition, poor level control during milling can result in depressions at the interface within the pavement which tend to allow the ponding of moisture on a larger scale. If OGA surfacing is to be used over a milled surface, a correction course of a small stone DGA such as AC10 must be placed on the milled surface to provide an interface to facilitate the shedding of water which permeates through the OGA. An alternative treatment is to use a fine-toothed mill, followed by a 7mm or 10mm seal. This may avoid the need for the DGA correction layer.
- OGA must be placed on a surface that drains (for example, is free of depressions that can pond water). The placement of an asphalt corrector course may be required to meet this requirement. OGA must always have a waterproofing seal placed underneath it with an AC10 or AC14 layer underneath the seal (PDS).

2.14.5 Evaluation of drainage system

The following are some of the major checks or tests required for evaluating a drainage system:

- check whether moisture is entering the pavement through the surface, shoulders or subgrade
- carry out trenching, drill probing and test pit sampling (also for materials evaluation)
- test pipes and subsurface drains by flushing them with water (the water may or may not contain dye)

  Note if water runs clear or not.

- test for moisture content variations in pavement materials – lightweight FWDs (for example, Clegg Hammer) may be used to test flexible granular pavements; additionally, GPR, deflection and SASW testing may indicate the depth to layers containing significant levels of moisture
- visually inspect table drains, cut-off drains and pipe outlets, and/or
- seek evidence of seepage from surface (stains, pumping, and so on) and embankments after rain. This may indicate the pavement drainage system is malfunctioning.

3 Selection of alternative rehabilitation options

3.1 Introduction

This section describes how the results of the pavement evaluation process, as outlined in Chapter 2, are used to develop a range of rehabilitation options. It also describes how appropriate treatments are selected from the developed alternatives.

As engineering judgement remains crucial to the selection process, the relationships between the defects and corresponding rehabilitation treatments are not presented in a prescriptive manner. The designer must, therefore use engineering judgement when applying this chapter to a project.

Other general design and construction considerations affecting the selection process are also discussed. These are not intended to be comprehensive and are simply presented to highlight the more significant issues. AGPT05 also contains discussion about what needs to be considered when developing options and selecting the appropriate ones. Reference must also be made to this document. This includes for subject matter or headings the same or similar.

Chapters 4 and 5 provide the specific technical detail or references needed to design the rehabilitation options developed.

3.2 Selection procedure

The method for selecting appropriate rehabilitation treatments is shown in Figure 3.2 and operates as follows:

- The designer identifies the purpose of the pavement investigation and pavement rehabilitation treatments, if required (refer Chapter 1).
- The designer gathers available pertinent information and determines an appropriate approach (for example, testing required, refer Chapter 2). This includes consulting the department's Asset Managers about maintenance.
- The designer identifies the existing pavement type for each section /subsection (refer Chapter 2).
- The designer evaluates each section / subsection using all available information (for example, historical records and deflection results) to determine its (functional and structural) condition (refer Chapter 2).
- The designer relates the condition obtained from the evaluation to the desired (functional and structural) performance (refer chapters 3 and 4). This is done to obtain a range of possible solutions or strategies.
- This range is narrowed by considering such aspects as the project purpose, project constraints, and relevant design and construction considerations (chapters 3 and 4).
- The options subsequently selected are designed (refer Chapter 5 and, for design, using deflections only, parts of Chapter 2).
- Alternative rehabilitation strategies are compared (refer Chapter 6). This usually includes determining the WOLC of each option.
- Recommendations about which options should be selected are made based on the results of these.

Figure 3.2 – Selection of rehabilitation options

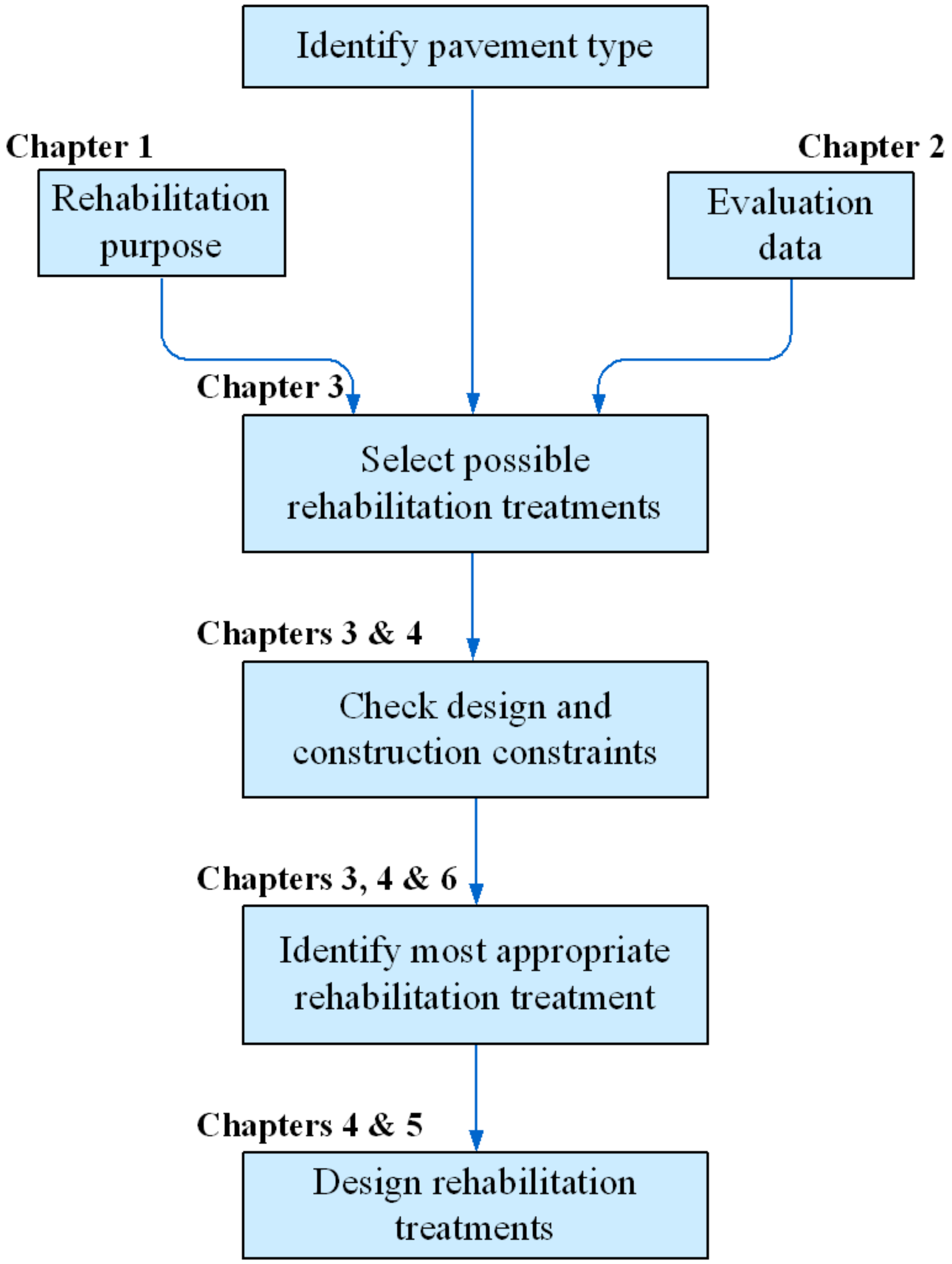


Table 3.2 outlines some preliminary options than can be considered after an initial assessment of the pavement. Table 3.2 is not exhaustive and is not intended to limit options that can be considered; therefore, it is expected the pavement rehabilitation designer may add, delete or develop other appropriate options as he/she goes through the pavement rehabilitation process.

Table 3.2 – Preliminary interpretation of state of pavement(s)

State of pavement (not exhaustive)	Possible options (not exhaustive)
Good pavement	Do nothing but monitor to determine if / when intervention is required
Sound pavement, but showing: • environmental or aging effects • use of possible poor construction or maintenance techniques.	• Do nothing but monitor to determine if / when intervention is required • Assess work performance against standards, specifications, and so on • Profile correction • Resurface (for example, with asphalt, bituminous seal or slurry seal) • Resurface after completion of deep patching (if distress is localised) • Crack sealing • For ageing, see also other sections in this table that discuss ageing
Deformation in pavement layer (for example, asphalt)	• Shape correction (for example, asphalt corrector with new asphalt surfacing, rut filling or profiling) • Nominal overlay with asphalt corrector where required • Resurface after completion of deep patching (if distress is localised) • Crack sealing in conjunction with shape correction and overlay
Weak or expansive subgrade Paving material with excessive plasticity	• Rehabilitation by overlay, stabilisation or a combination of both, investigation and design required but care needs to be exercised with regard to the use of stabilisation over expansive subgrades (for example, what is stabilised, subgrade versus base, and the type and dosage of binder used, or whether stabilisation is not used at all) • Rip / tyne, grade, trim, compact and place surfacing • Resurface after completion of deep patching (if distress is localised) • Crack sealing or place strain alleviating absorbing membrane interlayer (SAMI)

State of pavement (not exhaustive)	Possible options (not exhaustive)
Aging of seal or asphalt surfacing	• Seal enrichment or rejuvenation • Hot or cold in place asphalt recycling • Mill and replace asphalt surfacing • Place asphalt surfacing, reseal or slurry seal – the latter may not be suitable in high speed environments and/or be only a relatively short-term holding treatment • Resurface after completion of deep patching (if distress is localised) • Crack sealing, place strain alleviating membrane (SAM) or place SAMI with asphalt overlay
Pavement distressed and/or needing structural strengthening	• Structural interventions such as rehabilitation by overlay, stabilisation or a combination of both – investigation and design required • Crack control measures constructed under asphalt overlays (for example, SAMI)
Severely fatigued or distressed pavement	• Rehabilitation by overlay, stabilisation or a combination of both, investigation and design required • Reconstruction, investigation and design required • Crack control measures constructed under asphalt overlays (for example, SAMI)
Pavement failing due to, or partially due to, drainage problems.	Improve drainage (for example, reinstate / install table drains, install pavement drains) in conjunction with structural or functional treatments (refer Section 4.3 of Chapter 4)

Note: While the pavement rehabilitation designer will make recommendations, the option/s ultimately selected for construction will not usually depend solely on pavement factors (for example, geometry may dictate the options selected). Ideally the pavement rehabilitation designer will seek or be informed about all such constraints. Notwithstanding this, she/he may not always be aware of, or control, such constraints. Often, the Project Manager of the overall project and/or the relevant Department of Transport and Main Roads Region / District representatives will make the final decision.

When selecting options, the designer or customer may have preconceived ideas about the type(s) of treatment(s) that are the most appropriate solutions; however, such premature conclusions should be avoided, as they may preclude other options viable when the whole process described in this *Manual* is completed or non-financial factors are taken into account. Within a single project, a combination of different rehabilitation treatments may also prove to be appropriate, either within the pavement structure, over the extent of the proposed work or over the duration of the analysis period.

3.3 Rehabilitation options

AGPT05 provides guidance with respect to the causes of defects and associated structural and non-structural rehabilitation treatments applicable to addressing them. The details given in AGPT05 are a guide only. When selecting options, the designer can use options outlined in AGPT05 in the initial stages to determine likely rehabilitation options. In determining the options to be recommended, the designer needs to consider a range of factors and information. In some circumstances, treatments other than those outlined in AGPT05 may be appropriate (for example, use of an innovative technique developed after publication of AGPT05). In all cases, the designer should justify the selection of the treatment(s).

3.3.1 Generic versus proprietary

At the time of publication, the State Government purchasing policy discourages the specification of proprietary products for State Government works except in some limited circumstances or where no alternative is available. The use of the term 'or equivalent' can be used (added to a proprietary name) where appropriate. As a result, this *Manual* does not include discussion, information, and so on about proprietary treatments, materials, products, and so on. For works on state-controlled roads, only treatments built while complying with the purchasing policies of the State Government and Transport and Main Roads can be considered or specified.

3.3.2 Transport and Main Roads requirements

Except in limited circumstances (for example, small trials, special cases with a project specific supplementary specification), only treatments built while complying with departmental specifications, policies, manuals, standards, and so on can be considered or specified.

3.4 Pavement types

The PDS identifies a number of common pavement types:

- granular pavements with sprayed seal surfacings
- cemented granular bases with sprayed seal surfacings
- granular pavements with thin asphalt surfacings
- asphalt over granular pavements
- flexible composite
- deep strength
- full depth asphalt pavements
- concrete (rigid) pavements
- asphalt over cementitiously stabilised granular pavement, and
- plant-mixed foamed bitumen stabilised pavements

High Load Intensity, Low Intervention pavements are also included in the PDS. They comprise a limited set of some of the pavement types listed previously. Temporary pavements are also discussed in the PDS. Pavement rehabilitation designs can often be thought of as one of the listed pavement types, be it varied to suit rehabilitation rather than new construction.

Some of these pavement types use very different materials in their base and sub-base layers (for example, asphalt base and cementitiously stabilised sub-base in the case of a deep strength asphalt pavement). For such pavements, identifying an appropriate rehabilitation treatment may be more complex than for a pavement using the same material type for all base and sub-base layers (for example, granular from underside of the surfacing to the top of the subgrade). When evaluating an existing pavement with very different materials in its base and sub-base layers, the designer must determine whether the distress is caused by:

- one or more of the pavement layers reaching the end of their design life
- the poor performance of one or more of the materials, or
- the interaction between the different material types.

If the distress is of a type normally associated with a particular material, and tests reveal the pavement contains a substandard layer of that material, identifying an appropriate treatment should not be difficult.

If, however, the distress tends to originate at the interface between two materials, its cause and treatment may be less readily determined. In this case, problems may be the result of the relationship or interaction between the different material types, such as:

- differences in their relative stiffness, permeability or gradings
- how they chemically react with / to one another, and
- the degree to which the materials bond to one another, if at all.

Under these circumstances the processes of evaluation and rehabilitation design become less general, more case-specific and more difficult.

3.5 Design and construction considerations

Designers must consider a number of design and construction issues. The following sections discuss some of them (this section is not exhaustive).

3.5.1 Effect on public

Numerous issues related to the type of work, and how and when it is carried out, can have a direct bearing on the public and how the public perceives the relevant road authority (for example, Transport and Main Roads). These include:

- safety of road users and road workers
- nuisance and potential health effects such as dust (for example, from soil / lime / cement), noise, vibration, smell, and so on
- restriction of access or changes to access arrangements
- environmental considerations such as removal of trees for side tracks, soil erosion, production of smoke and other effluent(s)
- political sensitivities
- type of road users (for example, commercial heavy vehicles compared to private use vehicles)
- abutting land use (for example, commercial compared to residential)
- attitude of road users (for example, tolerance for inconvenience) and how this varies with time (for example, peak versus non-peak, holidays, when events are held)
- timely restoration where premature failure has occurred during or after works are completed
- duration of the works and/or each of the effects / inconveniences
- timing of works in relation to peak periods (for example, commuting), holidays, and so on
- community engagement should be considered before, during and after construction – discuss any community engagement requirements with the relevant district
- potential damage to buildings (for example, from vibrations originating from compaction equipment), and
- potential damage to vehicles (for example, from lime, from bitumen, windscreen damaged by loose stone chips).

Such issues need to be considered by the designer, overall Project Manager and the department's Region / District representatives.

3.5.2 Road geometry

When a rehabilitation investigation is being carried out, and associated designs are being developed, the effects of proposed treatments on the geometry of the present, or proposed, road(s) should be considered. Conversely, the geometry of the current or proposed road(s) may affect whether rehabilitation options are appropriate or feasible. Road geometry to be considered includes the horizontal and vertical alignments, crossfall, superelevation and the type cross-section(s).

Opportunities to improve road geometry may arise when rehabilitation is planned. Examples would be designing and constructing road widenings so as to increase the (horizontal) radius of sharp bends / curves and varying the transverse thickness of an overlay to correct crossfall / superelevation, application of superelevation or shape. Related to this are correction of these and other aspects to address surface drainage, refer Section 3.5.3. Similarly, works to correct the geometry of the road may present an opportunity to rehabilitate an existing pavement.

If geometry is not considered, undesirable effects may result. An example would be increasing the carriageway width (through widening) on a narrow formation, thereby increasing the slope of embankment or cut slopes to an undesirable level. Another is a geometric improvement including widening of the existing pavement by a (small) amount impractical to construct.

Chapter 2 also contains discussion about road geometry, including guidance about when an evaluation of the road geometry is warranted. The *Road Planning and Design Manual Volume 3* (RPDM3) provides detailed guidance with respect to road geometry.

3.5.2.1 Restrictions due to vertical geometry

The limitations imposed by restrictions on altering the vertical alignment (for example, raising the grade line) are often a major constraint. Situations likely to place such constraints on rehabilitation options include:

- at intersections and some accesses (driveways)
- where kerb or kerb and channel exists, this can be a particular problem in urban areas and often in cuts
- where gullies that drain the surface exist
- where raising the grade line will cause problems associated with afflux (for example, at floodways in rural areas)
- where road safety barriers exist, particularly if overlays have already been constructed since the barriers were first installed
- where existing road furniture would be affected adversely by a rise in levels (for example, where slip bases on signs, street lighting and the like will no longer function correctly); this can be more of a problem where overlays have already been constructed since the road furniture was first installed
- where vertical clearances under structures (for example, sign or Intelligent Transport System (refer Austroads Glossary AP-C87) gantries and pedestrian, road or rail overpasses) cannot be altered, and
- at bridges which cannot accommodate increased loading.

These limitations usually preclude the use of conventional overlay treatments and necessitate the use of alternative and, frequently, more costly solutions.

Such alternatives include:

- milling off or removing and replacing existing pavement layers
- milling off or removing existing pavement layers immediately adjacent to the feature and overlaying
- insitu stabilisation of existing materials
- full reconstruction
- use of asphalt with a PMB
- the use of warm mix asphalt, and
- combinations of these.

In some cases, it may be possible to adjust road furniture and so on (for example, raising the kerb and channel to place an overlay, replacing road safety barrier) instead; however, this option should be adopted with caution, not only because of the additional expense, but also because it restricts choices in the future.

3.5.3 Drainage

Rehabilitation treatments also have the potential to affect the drainage of the road.

3.5.3.1 Pavement drainage

The effect a rehabilitation intervention has on pavement drainage must be considered. Drainage of the existing and proposed pavements must not be impeded. An example of this is ensuring a new widening does not impede drainage of the granular base and sub-base layers of an existing pavement.

3.5.3.2 Road drainage

Drainage can be affected by rehabilitation treatments. An example would be a new widening, increasing surface flow paths and leading to water film thicknesses greater than those recommended by the department's RDM.

Pavement rehabilitation interventions may also present an opportunity to correct or improve any existing drainage issues.

Within the overall project, therefore, drainage needs to be considered. Ensuring it meets guidelines may, however, constrain which rehabilitation options can be chosen.

Chapter 2 also contains some discussion about road drainage. With the exception of pavement drainage, the RDM provides detailed guidance with respect to drainage.

3.5.3.3 Flooding

The level of flood immunity, and/or how flooding is catered for, along the road can also affect which rehabilitation options are viable; for example, the use of floodways and their level of flood immunity may limit the options used in these locations.

3.5.4 Safety

Safety requirements may also place constraints on the development of rehabilitation options – for instance:

- if a road safety barrier exists and is to remain unmodified, thick overlays may not be possible, and
- OGA surfacing may need to be used to address surface drainage concerns.

Conversely pavement rehabilitation treatments have the potential to improve safety – for example:

- road geometry may be improved (refer to Section 3.5.2 and Chapter 2)
- road drainage may be improved (refer to Section 3.5.3 and Chapter 2)
- skid resistance may be improved (for example, by providing a new surfacing), and/or
- the provision of sealed shoulders (refer to Section 3.5.6) and/or how and when pavement widening is applied can improve safety (refer to Section 3.5.2).

3.5.5 New pavement abutting an existing pavement

Rehabilitation of a road section may require the construction of a new pavement that abuts an existing or rehabilitated pavement. In such cases, the designer must consider differences in the strength, permeability, types and thickness of materials of the abutting materials, and of the pavement as a whole. Other issues to be considered include:

- road drainage, including:
  - surface drainage, and
  - subsurface drainage, including the installation of, or modifications to, subsoil drains
- how the joint between the pavements is to be constructed and sealed
- where the joint is located within the cross-section (in relation to wheel paths), and
- how the geometry of the new pavement fits with the existing / rehabilitated pavement and the project as a whole (for example, assess longitudinal transitions, crossfall, superelevation, horizontal alignment and vertical alignment).

In addition, the width of any pavement widening should be sufficient to ensure mechanical compaction equipment can access the area of widening and compact materials effectively and to the required standard(s).

The finished surface heights of the new and rehabilitated pavements must match at the joint(s). In addition, the options must be designed and constructed to provide a smooth riding surface across the junction. In some cases, it may be necessary to remove existing material(s) to enable this to occur.

Where it is necessary to taper the thickness of a layer to match surface levels, the layer should be keyed into the existing asphalt by methods such as cold milling and saw cutting. Further details of asphalt joints may be found at the AAPA and in AP-PWT00.

3.5.6 Shoulder sealing

Refer to AGPT05 Section 5.5.5 for a general discussion of shoulder sealing.

3.5.6.1 Structural considerations

Sealing the surface tends to make the shoulder more attractive to the drivers and riders of vehicles, including cyclists. Consequently, the use of the shoulder increases. This means the materials in the shoulder need to be of sufficient quality and thickness to withstand the loading imposed by the occasional trafficking of it by vehicles. The method to determine the design traffic loading for the shoulder design can vary between the department’s Regions / Districts and should consider local observations and experience related to the use of shoulders by road users.

3.5.6.2 Inhibiting moisture ingress

Partially or fully sealing the shoulder can help inhibit transverse moisture ingress into the pavement. In particular, it can help protect the OWP of the lane adjacent to the shoulder. To improve the pavement’s performance, the shoulder should be sealed to a minimum width of 0.5–1m; however, sealing the full width of the shoulder is desirable as it provides the greatest level of protection and so increases performance (other factors may require more of the shoulder to be sealed; for example, to improve safety). RPDM3-03 provides some guidance on the shoulder widths required while AGPT05 provides guidance with respect to the treatment of shoulders to improve pavement performance.

3.5.6.3 Other considerations

In addition to these considerations, the sealing of shoulders may lead to other effects or requirements for work. These include:

- the effects of the works on the operating speeds along the road, including adjoining sections, need to be considered and addressed in accordance with RPDM3-03
- the shoulder is to be designed and constructed in accordance with RPDM3-03
- the surfacing used on shoulders must be suitable for those expected to use it (for example, refer to Section 3.5.7.1
- sealing of the shoulder may:
  - require repositioning of road safety barriers and other road furniture, and
  - effect on drainage (refer to sections 3.5.3 and 3.5.5).

3.5.7 Pavement surfacing

Relevant guidelines must be consulted to select the pavement surfacing of pavement rehabilitation options. Relevant considerations are included in AGPT03.

3.5.7.1 Bicycles

On routes with relatively high bicycle volumes or future routes expected to cater for high bicycle volumes, consideration may need to be given to what surfacing is used on the shoulders as well as the through lanes; for example, if cycling is to be catered for by on-road means, then an asphalt surfaced shoulder, rather than a sealed one, may be appropriate. This is particularly important on the department’s current and future priority cycle routes in accordance with the department's *Cycling Infrastructure Policy* (CIP). Cycling should be catered for as required under the CIP.

3.5.8 Staged construction

Rehabilitation strategies may include:

- single treatment at the start of the analysis period, or
- range of staged treatments over a period of years (a staged approach).

In all cases:

- any proposal to use staged construction should be informed by WOLC of all options (see Chapter 6)
- the aim is to provide a serviceable pavement for the selected analysis period
- replacement of the surfacing (only) may also be required over the analysis period (replacement of the surfacing may not coincide with a structural intervention), and
- routine maintenance will also be required over the analysis period.

Note: In this *Manual*, routine maintenance is not classified as rehabilitation.

A staged approach to the rehabilitation may be adopted for a number of reasons, including those outlined in AGPT05 Section 5.5.6.

For a number of reasons, care should be exercised when adopting certain staged strategies.

If staged construction is being considered, departmental Asset Managers should be consulted when assessing solutions with staged construction against those without.

3.5.8.1 Issues related to timing and funding

Integral to the adoption of a staged approach is the need for interventions to occur as scheduled. It is essential this occurs, otherwise the rehabilitation strategy will likely fail (a serviceable pavement will not be provided for the selected analysis period). For interventions to occur as planned, the department's Region / District, and other relevant areas, must ensure funds are committed and provided to ensure the planned interventions take place as scheduled. The timing is related to the traffic loading rather than age (time passed since an intervention).

Deferral or cancellation of the planned later stages may lead to the need for unexpected and/or expensive remedial works, and dramatically alter the economics of the original strategy; for example, using a marginal material as a pavement base, with the expectation an overlay with a high-quality base material will follow, runs the risk of significant early failure occurring if the overlay is delayed.

Note: While this specific example may be suitable for lowly trafficked roads, its use is not recommended on other roads where the risk and consequences of failure are greater).

3.5.8.2 Traffic management

Structural interventions will often require more extensive traffic management than routine maintenance or replacement of the surfacing (only); further, increases in traffic volumes may make the implementation of future interventions impractical or increase the cost of traffic management. How future stages will be constructed, and how traffic will be managed during their construction, must be considered when developing staged options (Section 3.5.9 contains a more general discussion about construction under traffic).

3.5.8.3 Effects on road environment

The construction of later stages may also interfere with, or require alteration to, earlier works. This has implications for WOLC and the design of the overall road.

Things affected by staging include:

- road geometry
- signs
- drainage, including surface drainage, drainage gullies and drainage outlets
- cross-section elements (for example, shoulders become narrower, changes in crossfall or superelevation)
- vertical clearances
- bridges
- road safety barriers
- kerb
- kerb and channel, and/or
- batter slopes or stabilities.

The increase in surface heights over the analysis period can also be greater when staged approaches are used.

Where possible, the design and construction of a road should allow for future stages (for example, aquaplaning is checked for future widening, road safety barrier is placed to allow for a future overlay).

3.5.8.4 Whole-of-life costs

Staged approaches have lower capital costs, and this is part of the attraction of such an approach; however, before deciding on the option to be adopted, it is recommended the WOLC of each option be calculated. This will provide important information used in the selection process. Often, staged approaches have higher WOLC when compared to a single-stage option (for example, an option not requiring a structural intervention within its 40-year design life compared to an option with structural interventions at years 10, 20 and 30 during its 40-year design life). Chapter 6 provides guidance about how WOLC are calculated.

3.5.8.5 Replacement of surfacing layer(s)

Surfacing layer(s) may need to be treated or removed and replaced at times that differ from the structural interventions; the timing of resurfacing treatment or replacement may not align with the timing of structural interventions – for example, structural interventions may be required every 10 years but an OGA surfacing may need to be removed and replaced every seven years.

3.5.9 Construction under traffic

New construction, or reconstruction, work can often be carried out clear of (through)traffic, but this is rarely the case for rehabilitation work. This is because it almost exclusively occurs on in-service pavements.

AGPT05 Section 5.5.7 also details design and construction considerations associated with construction under traffic.

Trafficking while under construction may be of particular concern when the asphalt layer is to be overlaid with more asphalt (it is not the surfacing layer and is only exposed to traffic between the laying of different courses).

The designer should also consider the materials used in each option as these may affect to the degree to which traffic may affect construction.

3.5.10 Risk, design sensitivity, construction tolerances and degree of construction control

Certain rehabilitation options are more sensitive to variations in design parameters or construction factors than are others. These parameters and factors may be difficult to quantify or control and can significantly affect the quality or performance of the finished work.

AGPT05 Section 5.5.8 discusses this further.

3.5.11 Availability of resources

AGPT05 Section 5.5.9 discusses the availability of resources.

The availability of plant, labour and/or materials will be reflected in the cost of certain options; further, the cost of certain options may help decide what treatments are recommended.

3.5.12 Generic versus proprietary

Refer to the discussion in Section 3.3.1.

3.5.13 Transport and Main Roads requirements

Refer to the discussion in Section 3.3.2.

4 Technical details of specific rehabilitation treatments

4.1 Introduction

The typical process used to evaluate a pavement was discussed in Chapter 2. In Chapter 3, factors to be considered to develop appropriate rehabilitation treatments were outlined.

This chapter does not include the structural design of rehabilitation treatments (refer Chapter 5) but supplements AGPT05 and the reader must reference both documents. AGPT05 Section 5 contains specific technical details and references pertaining to the more common rehabilitation techniques while AGPT05 Section 8 discusses the economic comparison of treatments, where Figure 8.1 compares the typical relative cost of some of these.

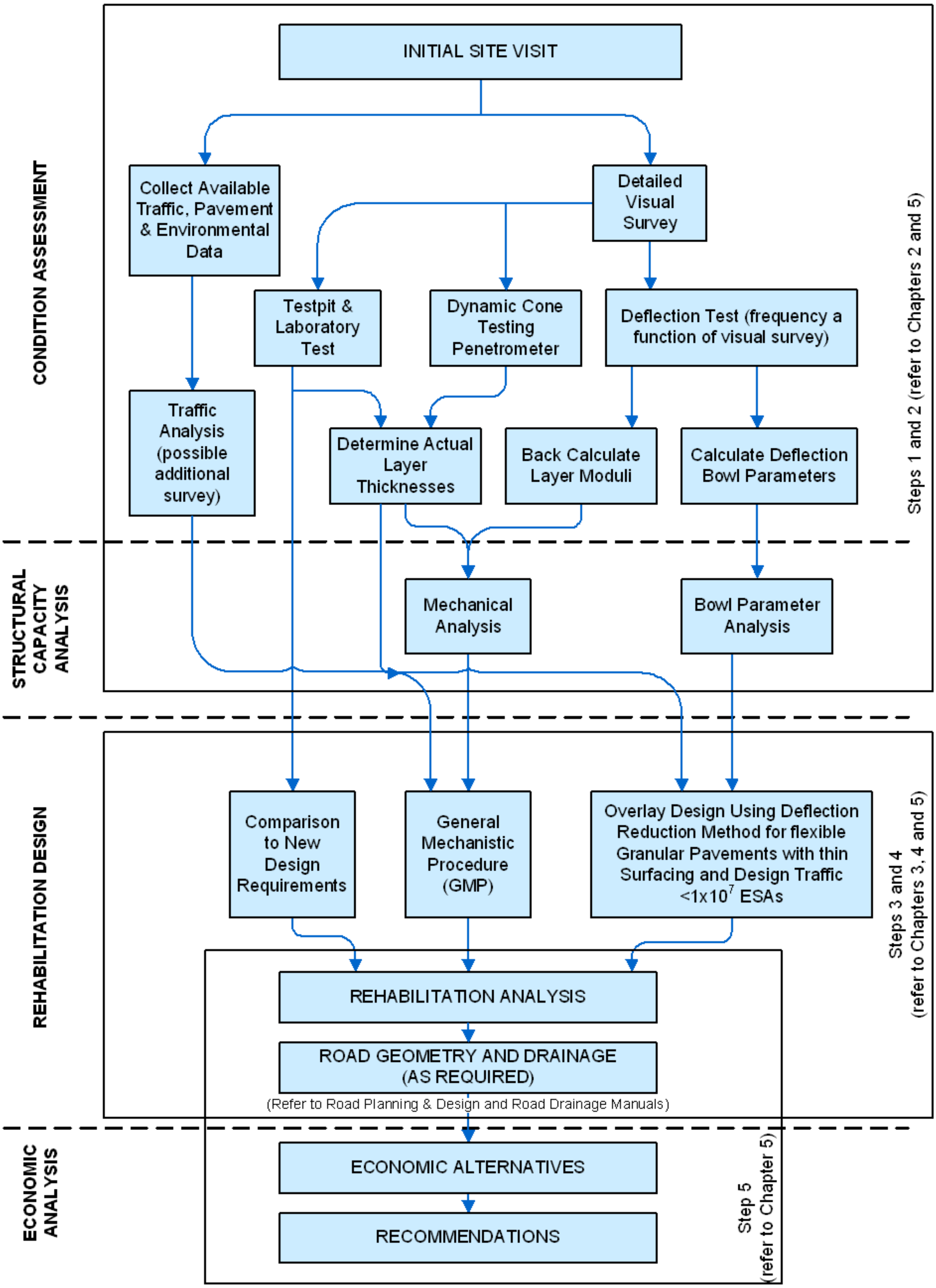
An overview of the typical process for the design of pavement rehabilitation treatments is outlined in Figure 4.2.

Note: While treatments are described discretely, a total pavement rehabilitation treatment may require treatments to be applied in combination (for example, improve drainage and resurface).

4.2 Materials

All materials, construction, and so on must comply with departmental policies, contracts, specifications and technical standards. Further details are given in the following sections.

Figure 4.2 – Typical pavement rehabilitation process



4.3 Drainage

Drainage treatments are required to correct problems associated with inadequate surface or subsurface drainage systems. In pavement rehabilitation investigations, this includes addressing deficiencies in pavement drainage and other drainage systems (for example, correct pavement surface drainage to avoid aquaplaning).

The main function of drainage treatments associated with rehabilitation is to overcome problems associated with adverse moisture conditions or inadequate drainage. These problems may be localised or extend over long sections of the pavement. AGPT05 Table 5.1 outlines some examples of drainage problems that may warrant treatment. Further examples and details are given in Table 4.3 of this *Manual*. Additional examples are:

- sheet water flow (water on running lanes leading to spray and splash and, where relevant, a subsequent loss of visibility, loss of friction or onset of aquaplaning), and
- water running along ruts which may, where conditions are right for it, lead to aquaplaning or loss of control.

AGPT05 Table 5.1, AGPT10 and Table 4.3 of this chapter provide guidance for selecting appropriate treatments.

Reference must also be made to the RDM for the assessment and design of road drainage.

Table 4.3 – Some drainage problems and possible solutions

Drainage condition	Description of problem(s)	Possible solution(s) to the problem(s)
Shallow table drains or table drains that have silted, or are silting, up	Water flowing or standing in shallow drains can weaken the shoulders and the subgrade – raising the road's level may not eliminate this effect	Deepen drain and/or remove silt and debris from drainage system or construct kerb and channel, gully pit and underground stormwater system Providing wide flat batters to shift water away from the pavement is recommended where possible –shallow table drains may be needed in expansive soil embankments
Blocked subsurface drainage	Subsurface drainage is blocked: widenings can produce inadvertent blockage of previously exposed drainage layers and any existing transverse drain outlets; while low permeability granular base sub-base course materials used in widening construction can block drainage from the existing granular base or sub-base courses	Proper design and inspection can reduce the potential for inadvertent blockages Remove blockage / install new bypass drain The granular base and sub-base courses under widened portions or pavements should have permeabilities exceeding those of the existing granular base and sub-base layers; or edge and lateral drains should be constructed to stop water accumulating in the existing base and sub-base layers – the latter may be the only viable solution where bound materials are used for binder, base or sub-base layers in the widening If a high-permeability granular material is used in the base or sub-base courses in widenings on the high side of horizontal curves, it must be placed with adverse crossfall on its underside to facilitate drainage and be protected by sealing – this sealing must be maintained to ensure it remains waterproof

Drainage condition	Description of problem(s)	Possible solution(s) to the problem(s)
Permeable shoulders or medians	Unpaved shoulders and medians, and shoulders and medians with inadequate drainage (for example, cracked concrete median without subsurface drainage) act as a potential source of water for the pavement structure	Seal or pave unpaved shoulders and unpaved portions of medians; however, ensure this construction does not block existing subsurface drainage (see *Drainage condition* in this table Where appropriate, install subsurface drainage at the edge of pavement closet to point(s) of moisture ingress Treat paved shoulders and median with inadequate drainage or are permeable (for example, seal cracks, install longitudinal subsurface drains)
Rigid pavements 'pumping'	Fines from the sub-base or subgrade are eroded, because of the softening (weakening) effect of water, and are flushed or pumped through a joint or crack in a rigid pavement Traffic passing over the joint produces a differential movement caused by load transfer, thereby pumping fines to the surface Cracks as well as joints may pump	Several possibilities exist: • slab undersealing (refer Section 4.8.1) • provide a SAMI in the form of polymer modified seal or geotextile seal with a thick asphalt overlay • other treatments to prevent reflective cracking and pumping are described in Section 4.8, and/or • improvement of internal drainage Other options include combinations of the solutions given here.
Impermeable pavement layers	Aggregate base and sub-base courses draining slowly (for example, have low permeability, have no or poor out letting of the courses / layers, or both) lead to prolonged wetting of the base, sub-base and subgrade, which allows softening (weakening) of the subgrade and possibly the migration of fines into the base or sub-base	For an existing pavement, only three options exist: • remove and replace with free draining materials • ensure surface of pavements is impermeable (for example, seal cracks promptly, resurface as required), and/or • provide positive outlets for granular base and sub-base layers by using longitudinal drainage systems (for example, strip drains)
Reduced drainage capacity of kerbed pavements caused by overlays	The cross-sectional area available for water flow along the kerb and channel is reduced because of the overlay In addition, the freeboard between the surface of the pavement and the top of the kerb is reduced	Raise kerbs in conjunction with overlay, install more gullies and/or replace drainage gullies

Drainage condition	Description of problem(s)	Possible solution(s) to the problem(s)
Water seepage into pavement adjacent to median	Water may seep through cracks and joints in concrete medians; sand infill placed under concrete medians acts as a reservoir of this infiltrated water which eventually seeps into the adjacent pavement, lowers its shear strength and causes failures in the wheel paths	Joints in the concrete should be sealed and either the sand drained by installing weepholes, or the sand and pavement drained by strip drains constructed along the edges of such medians
Pockets of unstable subgrade	These areas can be identified by localised indicators related to things such as peat pockets, localised springs and groundwater seepage, inoperative subsurface drainage systems, and so on; they may be produced by any one of these things or by a combination of some	Water-related problems can be mitigated or solved by the installation of localised drainage systems Removal and replacement of materials and/or insitu stabilisation of them are about the only effective means of improving a materials-related problem
Broken and clogged pipes and pipe outlets	Broken or clogged pipes or outlets act as obstructions or dams that retard or completely inhibit flow through the drainage system; the area affected may be localised and resemble unstable subgrade pockets as described previously	Removal and replacement or repair of pipes and outlets easily observed would be an initial step; back flushing and/or 'snaking' (that is, drain auguring with flexible metal wires) drain pipe and collector systems are another possibility
Pavement failures caused by impermeable shoulder pavement in boxed construction	Many pavements with box-type construction exhibit distress in the OWP caused by the low permeability of the shoulder pavement	Install a subsurface pavement (strip) drain at the interface between the shoulder and pavement Care is required to ensure other drainage problems are not introduced; for instance, strip drains may introduce water to the pavement in areas subject to inundation Remove and replace shoulder material(s) with a suitable material (for example, with suitable permeability) ensuring continuity of drainage

Drainage condition	Description of problem(s)	Possible solution(s) to the problem(s)
Water ponding on the surface caused by incorrect surface crossfalls and/or development of superelevation, and/or by surface depressions (for example, rutting) – these conditions may occur individually or together	Rutting, shoving or incorrect crossfall / superelevation collects water that eventually infiltrates the pavement or causes aquaplaning	If a sealed granular pavement, tyne, treat deficiencies, compact, trim and resurface, or pulverise or insitu stabilise, compact, trim and resurface – material may need to be imported (for example, when correcting crossfall) If asphalt surfaced granular pavement, remove asphalt, treat deficiencies and resurface Alternatively, use asphalt courses, including a corrector course as required, to correct surface irregularities, crossfall or superelevation and resurface; profiling / milling may also be used in conjunction with this treatment – a seal may be required between the profiled pavement and the overlying asphalt layer (for example, if OGA) In some cases, profiling / milling on its own may be considered for shape correction; however, the resulting surface should be checked to ensure adequate skid resistance and surface texture results
Water ponding against the edge of the pavement	Water ponding caused by incorrect levels, drainage design or crossfall / superelevation	Correct longitudinal grade, drainage deficiencies or crossfall / superelevation; this may require reconstruction (for example, regrade or reconstruct kerb and channel, install drainage gullies)
Moisture infiltration from a cutting	In cuttings, there are a number of combinations of soil or rock formations with varying permeabilities where subsoil water may present a problem	Provide deep table drain(s) or longitudinal subsoil drain(s) under and along the shoulder or under the kerb and channel / channel In the case of channel, an underground stormwater system may also be required A transverse subsoil drain under the road at the cut / fill interface may also be desirable Transverse pavement drains at permeability reversals are recommended A drainage blanket may be required
Moisture infiltration through cracks, joints and other discontinuities in the pavement	Initial sources of infiltration may be along the joints (for example, in concrete pavements), construction joints between paving runs or lots and through cracks, especially for a pavement with a cement treated base which often block cracks	Treat cracks – details of sealing treatments are covered in this *Manual* (sections 4.4.4 and 4.4.5)

Drainage condition	Description of problem(s)	Possible solution(s) to the problem(s)
Moisture infiltration through permeable surfaces and accumulation of moisture at changes of pavement type or thickness, or at patches occurring on longitudinal grades	Locations where permeability reversals occur on a longitudinal downgrade (for example, at ends of jobs or at bridge / culvert abutments) can cause moisture build-up	Reseal and/or provide transverse subsurface drains at locations of permeability reversal

4.4 Treatments applied to the surface and/or to the existing pavement

This section describes treatments either applied to the surface of a pavement, existing pavement to remain for pavement rehabilitation treatments, or applied to rehabilitate the existing pavement. They may then be covered as part of the overall pavement. Some treatments described in this section may be used in the new pavement layers (for example, a geotextile seal) or in combination.

4.4.1 Bituminous sprayed seals – sealing and resealing

Discussion of primes and primer seals is not included in this chapter. Further, the discussion in this chapter is general.

For further and more detailed guidance refer to:

- AGPT03 for the selection of an appropriate surface
- *Guide to Pavement Technology Part 4B: Asphalt* AGPT04B for an introduction to the nature of asphalt as a material and its application in road pavements
- AP-T68
- AP-T236
- *Guide to Pavement Technology Part 4K: Selection and Design of Sprayed Seals* (AGPT04K) for a guide to the selection of thin bituminous surfacings, and
- AP-PWT00 as a guide to good sealing / asphalt practice.

Note AGPT04K is expected to be updated to include the key findings of AP-T68 and AP-T236.

A bituminous sprayed seal consists of a thin film of sprayed bitumen onto which a layer of single-sized cover aggregate is spread and compacted. The seal may involve more than one application of aggregate and binder. This system of surfacing is used extensively on both lightly- and moderately-trafficked roads. The performance of the seal as a waterproofing layer is vital to the overall performance of the pavement itself.

Resealing is the application of a seal to an existing bituminous surface to maintain the existing asset (for example, to maintain integrity / 'waterproofness' of surface, restore surface texture and improve skid resistance). A single / single seal is used but, under some circumstances, multiple applications of binder and aggregate may be used (for example, double / double). It is not essential, nor necessarily more economical, to use the large stone size seals / reseals.

Where the seal or reseal is used as the final surfacing, it is vital it is also hardwearing with adequate surface texture and skid resistance. Together with good construction practice, use in appropriate circumstances and a detailed design, in accordance with AGPT03, is essential. The types of sprayed work alternatives usually within scope for a rehabilitation treatment are:

- seals and reseals using binders such as:
  - unmodified Class 170 bitumen
  - modified (for example, polymer) bitumen
  - emulsions with unmodified bitumen, and
  - emulsions with modified bitumen
- geotextile seals and reseals: often, these are designed and constructed to become a SAMI or SAM; refer to sections 4.4.5 and 4.4.6, and/or
- seals and reseals using modified bitumen designed and constructed to become a SAMI or SAM.

4.4.1.1 Functions

The functions of a seal coat include:

- to protect the underlying pavement and subgrade from water and other damaging effects of the environment
- where used as a surfacing, to also provide a wearing surface resistant to abrasion by vehicles with adequate skid resistance and adequate surface texture, and/or
- where used as a SAM or SAMI, to also mitigate the risk of cracks reflecting though the SAM or SAMI, but only where movements are not excessive, and the pavement is moving horizontally.

The functions of a reseal are the same as a seal but also include:

- to maintain and protect the existing asset, and/or
- to improve or restore skid resistance and surface texture to be adequate for circumstances.

4.4.1.2 Appropriate uses

Sealed pavements may exhibit block cracking, stripping or ravelling over time (caused by environmental effects) and loss of skid resistance or surface texture (for example, caused by polishing of the aggregate or wear). In appropriate circumstances, sealing or resealing treatments can be effective techniques to correct these deficiencies. Depending on the type, condition and extent of cracking, other treatments may be more effective or also be required (for example, refer sections 4.4.5, 4.4.6 and 4.4.7).

Resealing may be used as a preventative maintenance treatment to arrest surface deterioration and extend the service life of a pavement. In such cases, an effective seal is restored to the existing pavement to prevent the entrance of air and water and, thus, protect underlying pavement layers and the subgrade, and to reduce oxidation of the existing bitumen surface.

If a pavement shows distress caused by structural inadequacy, a reseal treatment may be an appropriate temporary measure to arrest further deterioration until an overlay or other rehabilitation measures are taken (placement of a 14 / 7 double / double seal used as a ‘holding measure’).

In tropical and temperate climates, the presence of a permeable surfacing is a critical factor in accelerating pavement deterioration. In arid climates, ravelling and surface break-up caused by oxidation are important factors leading to pavement deterioration.

The PDS requires a PMB seal at the bottom of all asphalt surface layers, except in areas subject to high shear forces. If the omission of this seal is proposed, specific advice is to be sought from the Pavements, Materials and Geotechnical Directorate and the decision is to be subject to a risk assessment. Refer to the PDS for further details.

4.4.1.3 Inappropriate uses

In heavily-trafficked sections or sections with 'tight' horizontal curves, the treatment may be inappropriate (for example, due to stripping). It may also be inappropriate to apply seals or reseals at intersections or the like with high turning movements. Up to a point, the use of a PMB can help overcome the stripping problem in some cases; however, in other cases, the use of an asphalt surfacing may be required.

A sealing or resealing treatment does not provide any additional structural strength to a pavement unless several layers build up to approach the thickness of an asphalt surface course; therefore, if a pavement exhibits load-associated crocodile, transverse, or longitudinal cracking, a seal or reseal will not remove the cause of the distress. SAM and geotextile seals and reseals have proven cost-effective in reducing reflective cracking and extending the life of pavements (refer sections 4.4.5 and 4.4.6). When cracks are being sealed or resealed, maintenance patching should be thorough and carried out well in advance of the sealing or resealing process.

A seal coat cannot correct shape deficiencies such as rutting, shoving or (high) roughness; however, a seal coat may be used in conjunction with other rehabilitation techniques.

It is essential sealing / resealing be done before cracking or ravelling becomes widespread and there is significant irreversible loss of pavement shape or degradation of pavement materials; for instance, resealing an asphalt surface that is extensively fatigue-cracked is not appropriate.

Seals and reseals generate the highest road tyre noise levels; therefore, this treatment is usually inappropriate wherever road-related noise levels exceed the criteria given in the TNMCP, to which reference must be made.

Seals should not be placed over lively asphalt patches. If asphalt patches are lively, options include either:

- removing, replacing them / patching with a suitable material other than asphalt, and sealed, or
- delaying enrichment until the youngest asphalt patch is at least six months old.

Note: For pavements with more than one layer of asphalt at locations subject to heavy braking and/or tight cornering (for example, intersections, roundabouts and approaches), excluding the waterproofing seal at the bottom of the asphalt surface layer can reduce the risk of shearing, but increase the risk of stripping of lower layers. Provision of a seal in these locations is not mandated. Refer to the PDS for further details.

Reference should also be made to Section 4.11 which deals with the use of polymer modified and multigrade binders.

Sealing over a pavement can trap moisture contained within it; therefore, care should be exercised when sealing or resealing over a pavement or asphalt that contains excess water.

4.4.1.4 Materials

Relevant departmental specifications include:

- Specification (Measurement) MRS11 (MRS11) and Technical Specification MRTS11 *Sprayed Bituminous Surfacing (Excluding Emulsions)* (MRTS11)
- Specification (Measurement) MRS12 *Sprayed Bitumen Emulsion Surfacing* (MRS12) and MRTS12
- MRTS17
- MRTS18
- Technical Specification MRTS19 *Cutter and Flux Oils* (MRTS19)
- Technical Specification MRTS20 *Cutback Bitumen* (MRTS20)
- Technical Specification MRTS21 *Bituminous Emulsion* (MRTS21)
- Specification (Measurement) MRS22 (MRS22) and Technical Specification MRTS22 *Supply of Cover Aggregate* (MRTS22), and
- Specification (Measurement) (MRS57) and Technical Specification MRTS57 *Geotextiles for Paving Applications* (MRTS57).

4.4.1.5 Design considerations

Seals and reseals must be designed in accordance with the documents listed in Section 4.4.1 and comply with departmental policies, contracts, specifications and technical specifications. AGPT03 provides additional guidance.

4.4.1.6 Construction considerations

Seals and reseals must be constructed in accordance with the documents listed in Section 4.4.1 and comply with departmental policies, contracts, specifications and technical specifications. AGPT03 provides additional guidance.

It is critical the aggregate adheres properly to the binder and the binder adheres to the existing surface; therefore, the preparation of the existing surface, close monitoring of the weather conditions, application of the bituminous material and aggregate, rolling and traffic management are all key considerations. Cleaning off the excess aggregate is critical in preventing whip-off and possible windshield damage. The appropriate amount of pre-coating and adhesion agents can be beneficial – in fact, absolutely necessary, in some circumstances – in preventing loss of aggregate.

4.4.1.7 Expected performance and comments

All sprayed bituminous treatments are susceptible to damage early in their life by weather and (construction or general) traffic, particularly during the first few hours. Traffic must be managed to avoid damage to them (for example, reduce the speed limit until the binder has set and the aggregate is well-embedded and adhering to the binder, use sidetracks). In addition, the following discussion should be noted.

4.4.1.7.1 Excess binder

The main forms of corrective treatment for removing excess binder are:

- If the binder is still soft enough, spread and roll more aggregate into the binder. The aggregate size used for spreading is smaller than the original aggregate. The most common aggregate used for this type of operation is 7mm aggregate.
- If the binder is not picking up, and the situation is not serious or likely to cause safety issues, wait for warm weather to soften the binder, then spread and roll more aggregate into it (alternatively, crusher dust could be spread).
- Apply a high-pressure water spray specially designed for the task and applied by a suitably qualified supplier / applicator.
- Use products made from gilsonite (a hard, natural rock asphalt with a softening point of 140°C). When applied, the gilsonite combines with bitumen to form a compound with a much higher softening point than straight bitumen. After application of the gilsonite, crusher dust or 7mm aggregate must be spread to absorb the excess but 'modified' binder.

If none of these works, a reseal, with the design adjusted accordingly, or removal of the seal in conjunction with the construction of a new seal, may be required.

4.4.1.7.2 Stripping

Stripping of aggregate can be caused either by loss of adhesion as a result of water being present on the aggregate, due to weather conditions (for example, it is cold); by a deficiency in the quantity of binder applied; or by incorrect seal design (for example, variable spray rates not used where they are required). It can also be caused by the action of traffic.

If the aggregate is stripping because of moisture, and it is possible to do so, trafficking should be delayed until all water has evaporated. Alternatively, while the seal is drying, the aggregate may be rolled and slow, controlled traffic allowed on it after some adhesion has occurred. Traffic management should remain in place until the seal has completely dried out and the binder has set up.

If there is a deficiency in binder quantity, then partial or total stripping may occur. For partial stripping, a surface enrichment or a light reseal may be applied. For total stripping, resealing with an appropriate design will be necessary.

4.4.1.8 Flushing

If sealing is carried out on cool days, followed by two or more days of continuous hot weather, the binder may flush up because of the amount of cutter required when the sealing work was undertaken. The corrective treatments for removing excess binder stated previously could be used (refer Section 4.4.1.7.1).

4.4.1.9 Further reading

Relevant publications include:

- AGPT03
- AGPT04B
- *Guide to Pavement Technology Part 4F: Bituminous Binders* (AGPT04F)
- AGPT04K

- *Guide to the Selection and Use of Polymer Modified Binders and Multigrade Bitumens* (AP-T235).
- AP-T68
- AP-T236
- AP-PWT00, and/or
- *Bituminous Roads in Australia* (Dickinson, 1984).

4.4.2 Surface enrichment of sprayed bituminous seals

Surface enrichment of a sprayed bituminous seal is a treatment involving the addition of a bituminous binder to a seal without the application of a cover aggregate. It is an economic way of extending the life of a sprayed seal from about three to five years, provided it is carried out before excessive oxidation of the binder in the original seal occurs.

Besides being used to enrich seals in which the binder has oxidised, this treatment may be applied to seals in which the initial binder application is low, to increase the depth of binder around the stones.

The surface of an enriched seal can be slippery initially; therefore, care is required during early trafficking (for example, implement reduced speed limits to improve safety).

4.4.2.1 Appropriate uses

Class 170 seals have one or more of the following conditions to benefit by bituminous enrichment:

- there is a lack of binder around the stones (it is ‘hungry’), and/or
- fine cracks of less than 1mm wide are present.

To facilitate the management of traffic, spraying can be carried out one lane at a time. The adjacent lane can be treated the following day. This approach enables traffic to be diverted around the work, and, if the emulsion takes longer to break because of unforeseen circumstances, it reduces the likelihood of having to put traffic on the sprayed area sooner than is desirable.

Enriching sprayed seals is most appropriate for lightly-trafficked roads. It can be used effectively on sealed shoulders in more highly-trafficked areas.

4.4.2.2 Inappropriate uses

Under no circumstances should traffic be allowed on the sprayed area until the emulsion has broken sufficiently to prevent pick-up by vehicles. Depending on prevailing weather conditions and the pavement temperature, bitumen emulsion takes between one or two hours to break fully. A surface temperature above 10°C for at least one hour before spraying commences is required.

Bitumen emulsion may be applied to a slightly damp surface but, in no circumstances, should it be applied if it is raining, if rain is imminent or if rain is likely to fall during spraying or before the emulsion fully breaks.

Under no circumstances should bitumen emulsion be heated above the recommended or specified temperatures as the emulsion may break prematurely.

Owing to the time required for bitumen emulsion to break fully, its use on heavily-trafficked roads is restricted.

It is also inappropriate to use the treatment to:

- prevent reflective cracking
- reduce road noise
- improve skid resistance
- act as a crack sealant for anything other than fine cracks, or
- treat PMB seals.

4.4.2.3 Materials

Either anionic or cationic emulsions are suitable for general use. Both types of emulsion have similar breaking times. Indications are the cationic emulsions tend to flow into and bridge fine cracks better than anionic emulsions.

The emulsion type to be used should be checked in the laboratory for compatibility with the local water supply. If required, either an acid or a surfactant, for cationic or anionic emulsions respectively, should be added to the water and dispersed thoroughly before the emulsion is added.

One disadvantage of rapid-setting emulsions is their tendency to commence breaking on contact with the stone, whereas the slow-setting emulsions flow down around the stones and into the voids before breaking. Premature breaking of rapid-setting emulsions results in a thicker bitumen coating on the stone, therefore reducing the amount bitumen in the voids where the additional binder is required most. Medium-setting emulsions seem to be the optimal choice.

4.4.2.4 Design consideration

The most important requirement for successful enrichment is for the seal to have adequate texture depth, especially in the wheel paths, and to eliminate the occurrence of bitumen flushing after treatment. If ruts exist in the wheel paths, there is an increased tendency for the applied binder to flow and pond in these areas, likely resulting in slick wheel paths.

On roads with steep grades or with high crossfall or superelevation, there is an increased tendency for diluted bitumen emulsion to run off the pavement. For resultant grades of up to 5%, the runoff is minimal. One way of classifying seals is shown in Figure 4.4.2.5.1. This is one way to describe seals.

Prior to a long length of road being treated, it is recommended a small trial be conducted to assess the suitability of the trial application rate for filling the voids and minimising runoff. If necessary, the application rate and the number of passes should be adjusted accordingly.

4.4.2.5 Construction considerations

Traffic management should consider the skid resistance may be lower initially. The department's Pavements, Materials and Geotechnical Directorate can be consulted for advice about what testing is needed, if required. To assess the effect of enrichment on skid resistance, a trial section should be enriched, and its skid resistance tested.

4.4.2.5.1 Pavement preparation

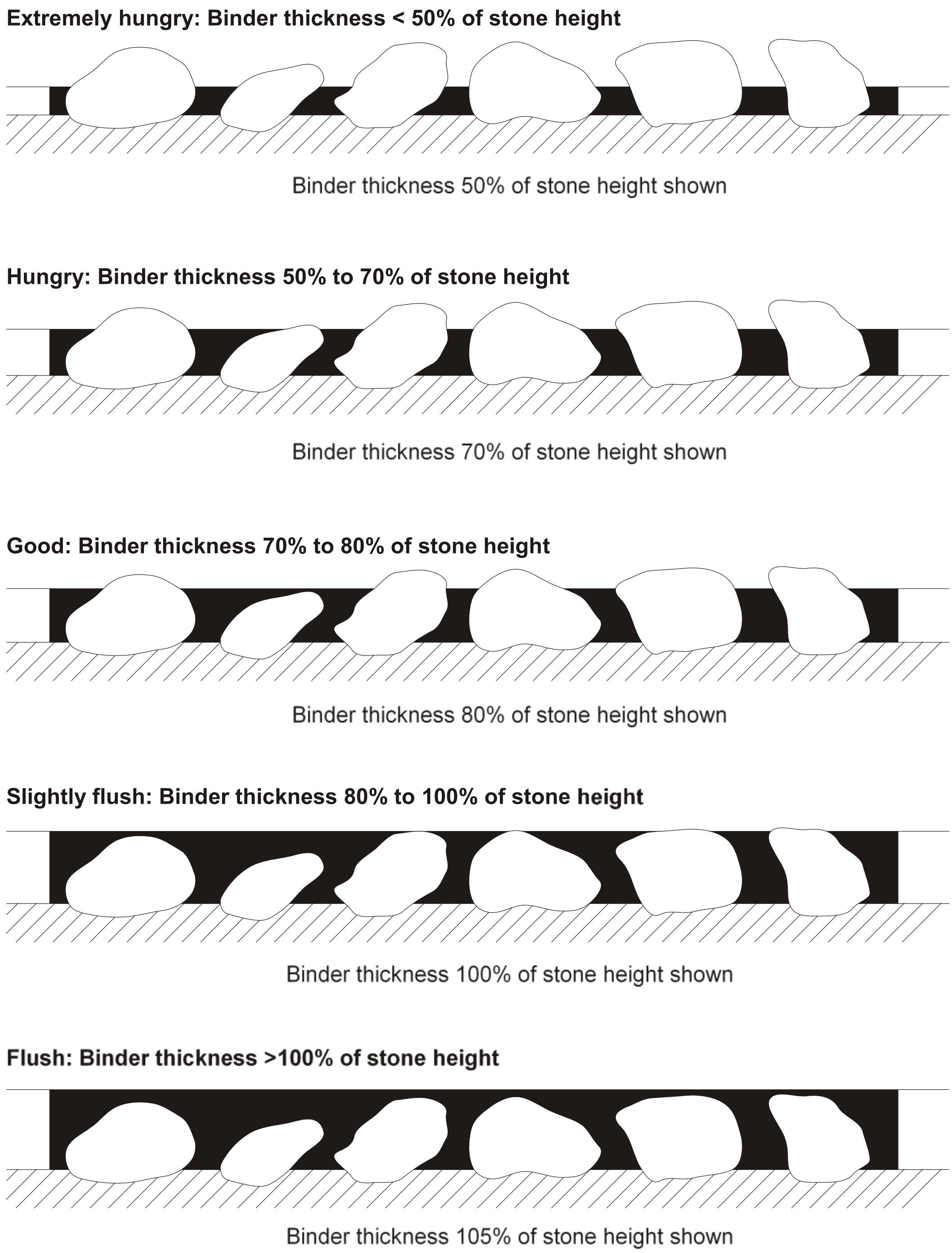
If necessary, the pavement should be cleaned (for example, by a mechanically-operated, rotary road-broom) to remove any dirt or foreign matter prior to spraying. Adherent patches of foreign matter may need to be removed by other means.

Seals should not be placed over lively asphalt patches. If asphalt patches are lively, options include either:

- removing, replacing them / patching with a suitable material other than asphalt, and sealed; or
- delaying enrichment until the youngest asphalt patch is at least six months old.

Any failures or wide cracks should be repaired prior to enrichment.

Figure 4.4.2.5.1 – Example of how seals can be described



4.4.2.5.2 Application of emulsion

A calibrated bitumen sprayer must be used for spraying the diluted-bitumen emulsion.

Bitumen emulsion is diluted at the rate of one-part water to one-part emulsion (a 1:1 mixture). To minimise runoff and achieve the required rate of application of residual bitumen, it is usually necessary to make more than one pass with the sprayer. Two to three passes are normally required. For each pass, the application rate of the mixture should not exceed 0.6 l/m². The direction of each pass should alternate so the emulsion surrounds the stones fully.

The time between each individual pass, where multiple passes are used, should be such the emulsion is broken fully before the subsequent pass. Depending on prevailing weather conditions, the time between passes can vary from about one to three hours. In some situations, it may be necessary to apply the subsequent application the following day.

4.4.2.5.3 Handling emulsion

The enrichment binder and agent shall be transported, stored and handled as per MRS12, MRTS12 and MRTS21. This includes sprayers.

4.4.2.6 Expected performance or comments

Important factors affecting the performance of a seal coat include:

- the surface texture and roughness of the exposed portion of embedded aggregate, and
- the ability of the binder to prevent aggregate particles from rotating or stripping under the horizontal shear forces produced by braking tyres.

By adding surface enrichment agents, the bond between the aggregate particles and binder is enhanced and/or stripping is reduced.

Polymer-modified emulsions may enhance this bond further, but may reduce the skid resistance initially, until trafficking wears the binder off the stone.

4.4.2.7 Further reading

Relevant publications include:

- AGPT03
- AGPT04F
- AGPT04K
- AP-T235
- AP-T68
- AP-T236
- AP-PWT00, and
- *Surface enrichment of sprayed seals using bitumen emulsion in New South Wales* (Walter, 1985).

4.4.3 Rejuvenation of sprayed bituminous seals

Surface-applied rejuvenating agents have been used for many years. The purpose of them is to rejuvenate aged and dry bituminous pavement surfaces without heating, scarifying or mixing the existing pavement. The degree of rejuvenation achieved is dependent upon the penetration of the agent and its ability to form a blend with the bitumen in the pavement's surface.

4.4.3.1 Functions

The major functions are to penetrate, rejuvenate and enrich aged bituminous pavements, including both seal and asphalt surfacings.

4.4.3.2 Appropriate uses

Use of the treatment is appropriate for Class 170 seals in the following cases:

- to lower the viscosity of bitumen in asphalt and sprayed seal surfacings – this extends the life in terms of aging; for example, it can be used where the bitumen binder has oxidised
- in bitumen seals where there is substantial loss of binder as well as embrittlement of existing bitumen (refer also to Section 4.4.2), and/or
- to seal pavements against intrusion of air and water, thereby slowing oxidation, and preventing stripping and ravelling.

4.4.3.3 Inappropriate uses

The treatment is inappropriate if used for any of the following purposes or in any of the following applications:

- to prevent reflective cracking
- to improve skid resistance
- to reduce road noise
- to act as a crack sealant
- to use on a road without sand being placed over it, and/or
- for use on PMB seals.

4.4.3.4 Materials

There are a number of proprietary products used commonly as rejuvenating agents for asphalt and spray seal surfacings. The user is advised to carry out trials and assess the performance of any product before using any product extensively. The department's Pavements, Materials and Geotechnical Directorate can be contacted for advice about how to assess such products.

The aim of the trial is to check the product selected for rejuvenation satisfies the following requirements:

- rate and depth of penetration into the surfacing should be significant
- permeability of the surface of the pavement should be reduced, and/or
- durability of the treated surfacing should be increased.

4.4.3.5 Design considerations

Bitumens used in Australia have two major components, namely asphaltenes and maltenes. The principal causes of pavement deterioration attributable to the bitumen alone are either insufficient amounts of bitumen or premature embrittlement. For a durable bitumen to meet rheological requirements and to have the proper balance of components, it should have a maltenes composition parameter in the range of 0.4–1.2.

The Maltene Composition Parameter is defined in Equation 4.4.3.5.

Equation 4.4.3.5 – Maltene composition parameter

$$\frac{\boldsymbol{N+A1}}{\boldsymbol{P+A2}}$$

where:

$\boldsymbol{N}$= Nitrogen Bases

$\boldsymbol{A1}$= 1st Acidaffins

$\boldsymbol{A2}$= 2nd Acidaffins

$\boldsymbol{P}$= Paraffins in the bitumen.

Asphaltenes and paraffins are the bitumen components most resistant to oxidation. The components most subject to oxidation, in descending order, are $\boldsymbol{N}$, $\boldsymbol{A1}$, and $\boldsymbol{A2}$. Since the fraction of asphaltene always increases at the expense of fractions of $\boldsymbol{N}$, $\boldsymbol{A1}$, and $\boldsymbol{A2}$ during the ageing process, these three components need to be added to age-hardened bitumen if it is to be restored to its original composition. If further improvement in composition is desired to retard deterioration after rejuvenation, then the rejuvenating agent should contain only minor fractions of nitrogen bases and asphaltenes.

Another factor affecting the bitumen performance, particularly in durability, is the molecular weight of the asphaltenes (the fraction insoluble in light asphaltic hydrocarbons, for example, petroleum ether). All other parameters being equal, the lighter the molecular weight of the asphaltenes fraction, the better the durability.

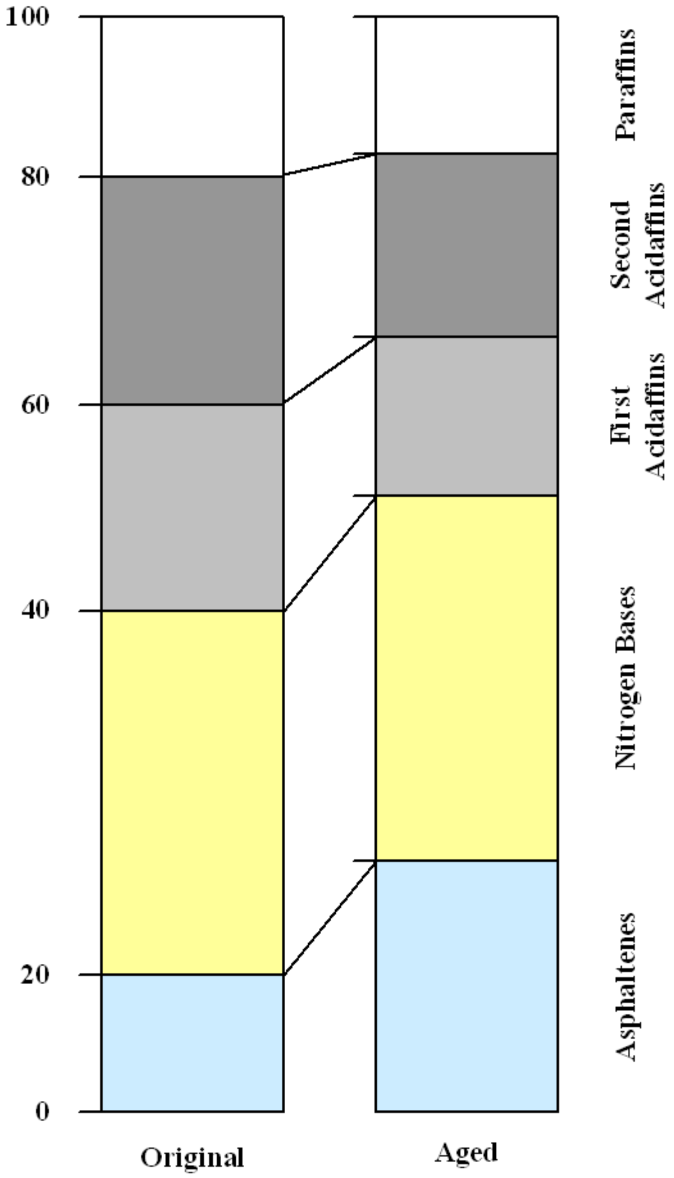
In most cases, adding an asphalt rejuvenating agent free of asphaltenes (a rejuvenating agent containing only maltenes) can re-plasticise and reverse the ageing process.

To rejuvenate asphalt surfacing, the rejuvenator is diluted at the rate of one part of agent to one part of water. The application rates are in the region of 0.3 l/m^2–0.4 l/m^2. Depending on the pavement conditions, dilutions and application rates may be varied. It is recommended the rate of sand cover (unwashed) be approximately $1m^3$ to $1000m^2/m^3$.

To rejuvenate bitumen seal coats, a similar dilution as this can be adopted. Application rates can be in the range of 0.35 l/m^2–0.5 l/m^2.

Figure 4.4.3.5 shows relative compositions of original and aged bitumen.

Figure 4.4.3.5 – Relative compositions of original and aged bitumen (not to scale)



4.4.3.6 Construction considerations

If necessary, the pavement should be cleaned (for example, by a mechanically-operated, rotary road-broom) to remove any dirt and other foreign matter prior to spraying. Patches of foreign matter adhered to the pavement may need to be removed by other means.

Rejuvenation should not be done when asphalt patches are lively. If asphalt patches are lively, options include either:

- removing, replacing them / patching with a suitable material other than asphalt, sealing and delaying rejuvenation an appropriate amount of time, or
- delaying rejuvenation until the youngest asphalt patch is at least six months old.

Any asphalt patches still lively should be removed, patched with suitable material, and sealed. Any failures or wide cracks should be repaired prior to rejuvenation.

Rejuvenating agents are applied using a conventional calibrated bitumen spraying unit fitted with calibrated special jets and recalibrated with them in place. Time should be allowed for the application to cure. This time depends on ambient and pavement temperatures and, where necessary, it should be covered with coarse, unwashed sand or grit and then reopened to traffic. The curing time varies with differing products.

A single-coat application of a rejuvenator with sanding is normally sufficient to restore an oxidised, but not heavily-crazed, bituminous pavement. For a badly-crazed or oxidised pavement, crack sealing needs to be completed prior to the application of two coats of a rejuvenator. Sanding can then be undertaken.

Traffic management should recognise the skid resistance may be lower initially. To assess the effect of rejuvenation on skid resistance, a trial section should be rejuvenated, and its skid resistance tested.

4.4.3.7 Expected performance

Experience shows one problem associated with the use of rejuvenating agents is a decrease in skid resistance. In some instances, rejuvenating agents can lead to flushing, have poor penetration, or fail to improve the physical properties of the bitumen.

Manufacturers claim skid resistance is not affected by the 'improved' rejuvenating agents presently available. More performance study will be required for confirmation.

4.4.3.8 Further reading

Relevant publications include:

- AGPT03
- AGPT04F
- AGPT04K
- AP-T235
- AP-T68, and
- AP-PWT00.

4.4.4 Crack filling and sealing – bituminous seals or asphalt

The discussion in this section relates to the treatment of individual cracks in bituminous seals or asphalt layers of pavements. Cracks and joints in concrete pavements will require a different approach (refer Section 4.8 and AGPT05).

Where cracking is widespread, the extensive use of a SAM or SAMI may be more appropriate (refer sections 4.4.5 and 4.4.6). In some cases, the filling and sealing of individual cracks using treatments described in this section may be required in conjunction with a SAM or SAMI (for example, seal individual wide cracks and then apply a SAMI to also deal with smaller cracks), or other treatment as described in sections 4.4.5 and 4.4.6.

Provided there is not excessive movement, the most common treatments for filling and sealing cracks involve the use of an appropriate proprietary or purpose-made crack filling / sealing product.

Reference should also be made to Section 4.11 which deals with the use of polymer modified and multigrade binders.

4.4.4.1 Function

Crack filling and sealing extends pavement life primarily by waterproofing the pavement surface to prevent softening of the structural layers by water from the surface. Other waterproofing measures, such as sealing granular shoulders and pavement edge drains, are separate treatments to prevent water ingress from other areas.

Crack filling and sealing of cracks to be overlaid may be undertaken in conjunction with treatments described in sections 4.4.5 and 4.4.6 to mitigate the effects of reflective cracking.

4.4.4.2 Appropriate uses

Appropriate uses for crack filling and sealing include:

- to treat transverse, reflective, longitudinal and diagonal cracks, and cracks around manholes, and so on
- to treat cracking of longitudinal and other construction joints
- for cracks of 'medium' to 'large' width; where required, a groove needs to be formed (routed) before the crack is filled and sealed (see Section 4.4.4.6), and/or
- as a preventative maintenance measure. A crack sealing program should be established and maintained following construction of every new pavement with a bituminous layer or seal for a period of at least five years. Cracks should be filled or sealed in a timely fashion, so the pavement's integrity is maintained.

4.4.4.3 Inappropriate uses

It is inappropriate to apply crack filler or sealant to a dirty and wet surface, as good adhesion cannot be achieved.

The treatment is inappropriate:

- where the cracking is widespread and extensive; its use is also uneconomical in such cases
- where cracking is caused by a structural pavement failure (asphalt fatigue, the use of a poor paving material, inadequate thickness, and so on)
- where there is excessive movement of cracks, especially if it is the only treatment proposed, and/or
- where the product used may cause skid resistance issues for road users (for example, cyclists and motorcyclists).

4.4.4.4 Materials

Proprietary products are typically used. These products and how they are applied should comply with the suppliers and/or manufacturer's requirements, except where these conflict with departmental specifications and technical specifications. Relevant departmental specifications include:

- MRTS17
- MRTS18
- MRTS19
- MRTS20
- MRTS21
- Specification (Measurement) MRS30 *Asphalt Pavements* (MRS30) and MRTS30, and
- MRS57 and MRTS57.

4.4.4.5 Design considerations

Crack filling and sealing materials should have the following design properties:

- low stiffness and high ability to deform
- waterproof integrity
- good resistance to oxidation and ultraviolet degradation (where they are left exposed)
- strong adhesion to bituminous surfaces
- safe to heat to high (application) temperatures
- softening point >80°C.

4.4.4.6 Construction considerations

Relevant departmental specifications include:

- MRS30 and MRTS30, and
- MRS57 and MRTS57.

4.4.4.6.1 Crack filling and sealing

Where a departmental specification or other contract document does not specify a treatment regime, it is best to classify the cracks according to their width as follows:

- Width from hairline–6mm: The most practical and economical means of sealing cracks of this size is to place an overband of sealer on the pavement surface, centring on the crack. The overband should be approximately 50mm–100mm wide. This can best be accomplished by using a pour pot and with a second operator with a 'V'-shaped squeegee. The crack filler should be hot enough to flow readily in front of the squeegee, which should be controlled to provide a thickness not greater than 2–3mm. One alternative to overbanding is to route the cracks; however, routing is a costly method and overbanding will achieve excellent results. Another alternative is to use strain alleviating fabric strips. These, too, are costly.

- Width from >6mm–12mm: Cracks of this size can be cleaned readily, dried to a minimum depth of 20–25 mm, and filled flush to the surface with the crack filler. Alternatively, a strain-alleviating fabric strip may be used.
- Width >12mm: Wide cracks in pavements can be filled completely with crack filler and sealed. Care must be taken excess moisture does not come into contact with a hot crack filler as rising moisture vapour will interfere with its bond with the sides. These cracks are filled level with, or just below, the pavement surface. Alternatively, strain-alleviating fabric strip may be used.

Where a specification, technical standard or other contract document does specify a treatment regime, the specified treatment regime shall be followed.

4.4.4.6.2 Sanding or dusting

Sanding or dusting is normally not necessary, provided traffic can be kept off the freshly filled / sealed crack for the curing period of the product; however, it is recommended in the following circumstances:

- any location where tyres are likely to be parked directly on the recently-poured crack filler
- in pedestrian areas subjected to high volumes of foot traffic, and/or
- areas where constant turning and stopping occurs, especially areas subject to trafficking by heavy equipment / vehicles.

Problems in these instances are aggravated by high ambient and pavement temperatures.

4.4.4.6.3 Priming

It is not essential to prime cracks in asphalt; however, priming does help to overcome minor inadequacies in the cleaning process.

4.4.4.6.4 Heating crack filler

Storage, heating and mixing shall be in accordance with the supplier's and manufacturer's requirements.

Hot crack fillers are typically heated to application temperatures of 160°C–180°C, depending on the width of crack or joint and the crossfall / superelevation and longitudinal grade of the road. Temperatures at the high end of this range are used to reduce the viscosity for improved penetration of narrow cracks.

A wide range of methods have been used over the years for applying hot joint and crack sealers, ranging from modified buckets and cans, to sophisticated pumping units with heat-traced hoses.

4.4.4.6.5 Crack and joint filling

As the hot sealer may contract 10%–15% when cooled to ambient temperatures, it is sometimes necessary to fill the crack to 90% of its depth, allow it to cool and then fill the crack to the final level.

With cracks expanding by more than 50% of the joint width, because of temperature or other causes, it may be necessary to apply a strain-alleviating fabric strip 100mm–150mm wide, depending on the joint size, to help maintain their waterproof integrity after sealing.

4.4.4.7 Expected performance or comments

There is a need for ongoing research and technical improvement of crack and joint filling and sealing. The research should look for ways to improve the effectiveness of sealing cracks, from both a technical and operational perspective.

4.4.4.8 Further reading

Relevant publications include:

- AGPT03
- AGPT04F
- AP-T235, and
- AP-PWT00.

4.4.5 Treatment of existing reflective and surface cracking for pavements other than concrete

The discussion in this section relates to treatments for existing or potential reflective cracking. It may also be applicable for other types of cracking (for example, surface cracking). If cracking is not widespread or extensive, then it is possible crack filling and sealing on its own may be appropriate (see Section 4.4.4). In some cases, the use of treatments described in this section may be required in conjunction with, and after completion of, crack filling and sealing (for example, crack fill and seal individual wide cracks and then apply a treatment described herein). They may also be used in conjunction with other treatments (for example, cementitious insitu stabilisation).

Cracks and joints in concrete pavements require a different approach (see Section 4.8 and AGPT05).

In Queensland, an examination of the overall network of declared roads suggests one of the most common deficiencies not necessarily obvious to the road user is cracking. The cause of reflection cracking in Queensland is often related to the use of cementitiously stabilised or CTB layers. These pavements have proven to be an economical alternative, in terms of capital cost, to conventional, flexible granular and full-depth asphalt pavement; however, significant proportions of pavements of this type are located in the wetter coastal areas and have displayed early distress in the form of regular transverse, longitudinal and, sometimes, block cracking. When this cracking is combined with seasonally wet conditions and high traffic volumes, fines are pumped to the surface from within the pavement structure. This leads to premature distress in the form of block and crocodile cracking. Prompt and effective remedial treatments need to be applied to seal such cracks, arrest pavement deterioration and ensure the pavement’s actual life is as close as possible to its design life.

Some of the major crack control treatments are:

- geotextile SAM (see also Section 4.4.6)
- geotextile SAMI overlaid with asphalt; a minimum thickness of asphalt is required over the geotextile (for example, 60mm) (see also Section 4.4.6)
- geogrid interlayer treatment overlaid with asphalt; a minimum thickness of asphalt is required over the geogrid (for example, 70mm) (see also Section 4.4.7)
- geotextile and geogrid composites overlaid with asphalt; a minimum thickness of asphalt is required over the composite (for example, 70mm)

- sprayed PMB SAM (see also Section 4.11)
- PMB SAMI overlaid with asphalt (see also Section 4.11)
- PMB asphalt overlay (see also Section 4.11)
- crack fillers and sealers, typically purpose-made proprietary products (see Section 4.4.4)
- strain-alleviating geotextile strips, including those incorporating PMB, and
- other proprietary products.

SAMIs and geogrids are not necessarily placed under the surfacing layer, they may be placed deeper in the overall pavement at a location most effective in mitigating the effects of reflective cracking. The designer must carefully consider whether a SAMI or the like is used close to the surface where shear forces are high (for example, on roundabouts or at intersections) as the SAMI may become a slip plane.

Pavement cracking can be attributed to a number of causes. These include surface oxidation of binder, fatigue due to an inadequate pavement thickness, environmental factors, use of poor-quality paving materials, poor construction and reflection of underlying cracks or joints. Transverse, longitudinal and/or block cracking can also be caused by presence of CTBs. A basic understanding of the cause of cracking in cement-treated bases and its effect on serviceable pavement life was sought via the Accelerated Loading Facility trial completed at Beerburrum.

A full-scale field trial of crack control products on a cracked CTB pavement was also set up on the Bruce Highway at Beerburrum in 1987 (near the Accelerated Loading Facility test site). All the previously-described crack control treatments were tried. Consistent trends of performance emerged from the trial. Internal reports (*Crack control trial on cement treated basis: Bruce Highway – Beerburrum Deviation* (RP 1200), *Beerburrum deviation crack control trial condition assessment report No. 1* (RP 1256), *Beerburrum deviation crack control trial condition assessment report No. 2* (RP 1369), *Beerburrum deviation crack control trial condition assessment report No. 3* (RP 2103), *Beerburrum deviation crack control trial condition assessment report No. 4* (RP 2374), *Pavement performance report: Beerburrum Deviation cement treated pavements* (RP 2383) and *Performance of unbound and stabilised pavement materials under accelerated loading: summary report of Beerburrum II ALF trial*, a joint Austroads and ARRB report (Austroads and ARRB, 1996)), provide details. For users outside Transport and Main Roads, contact the department's Pavement Rehabilitation Unit via email at ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au.

Further details of these treatments regarding materials, appropriate uses, inappropriate uses, and expected performance / comments are as shown in Table 4.4.5.

Table 4.4.5 – Treatments to mitigate the effects of cracking (for non-rigid pavements)

Treatments	Material(s)	Method of manufacture
Geotextile SAM or SAMI overlaid with asphalt (see also Section 4.4.6)	Must comply with departmental specifications including MRS11, MRTS11, MRTS17, MRTS18, and MRS57 and MRTS57, including: • for asphalt works polyester or polypropylene geotextiles • for sprayed bituminous works only polyester geotextiles can be used and emulsions cannot be used	Must comply with departmental specifications including MRS11, MRTS11, MRTS17, MRTS18, and MRS57 and MRTS57, including: • only non-woven, needle punched and formed from mechanically-entangled filaments shall be used for paving applications • geotextiles with calendaring on both sides not permitted • geotextiles with calendering on one side permitted where a trial on the actual project / job demonstrates its use is acceptable
Geogrid interlayer treatment overlaid with asphalt (see also Section 4.4.7)	• polymer • glassgrid	• polymer sheet is punched, heated and stretched to form a uniaxial or biaxial grid • bonded filaments
Geotextile and geogrid composites overlaid with asphalt	Must comply with departmental specifications including MRS58, MRTS58, MRS104 and MRTS104, including: • geogrids – as described previously • geotextiles – as described previously	• geotextile bonded or attached to a geogrid
Sprayed bituminous PMB SAM or PMB SAMI overlaid with asphalt (see also Section 4.11)	Must comply with departmental specifications including: MRS11, MRTS11, MRS12, MRTS12, MRTS17, MRTS18, MRTS19, MRTS20, MRTS21, MRS22 and MRTS22, including: • SBS • styrene-butadiene rubber (SBR) • polybutadiene • factory produced crumb rubber • field produced crumb rubber	Must comply with departmental specifications including: MRS11, MRTS11, MRS12, MRTS12, MRTS17, MRTS18, MRTS19, MRTS20, MRTS21, MRS22, and MRTS22, including: • blending polymer prior to supply

Treatments	Material(s)	Method of manufacture
PMB asphalt overlay (see also Section 4.11)	Must comply with departmental specifications including: MRTS17, MRTS18, MRTS19, MRTS20, MRTS21, MRS30 and MRTS30, including: • SBS • SBR • PBD • Ethylene vinyl acetate (EVA)	Must comply with departmental specifications including: MRTS17, MRTS18, MRTS19, MRTS20, MRTS21, MRS30 and MRTS30, including: • blending polymer prior to supply.
Crack filling and sealing (see also Section 4.4.4)	• typically proprietary products	These include: • pre-blended proprietary products • emulsion and sand.
Strain alleviating fabric strips, including those incorporating PMB	• typically proprietary products	These include: • bitumen impregnated geotextiles that adhere to the road surface – the bitumen incorporated may be a PMB

4.4.5.1 Materials

Relevant departmental specifications include:

- MRS11 and MRTS11
- MRS12 and MRTS12
- MRTS17
- MRTS18
- MRTS19
- MRTS20
- MRTS21
- MRS22 and MRTS22
- MRS30 and MRTS30, and
- MRS57 and MRTS57.

4.4.6 Geotextile strain alleviating membrane and strain alleviating absorbing membrane interlayer

Paving geotextiles are used in rehabilitation treatments over pavements that are cracked or have the potential to crack. They may also be used in the construction of new pavements (before any cracking occurs) where it is suspected cracking may occur in the future.

They are used to create a stress alleviating surfacing or interlayer between the new / existing pavement and the new surfacing treatment. The geotextile is impregnated with binder and this creates its stress alleviating ability.

The use of paving geotextiles in pavements is widespread throughout Queensland. They can be used in a wide range of applications, including:

- reseals that require a SAM
- SAMs on pavements with a CTB or a stabilised / modified base (for example, cement / fly ash blends)
- SAMIs on pavements with a CTB or a stabilised / modified base overlaid with asphalt
- within asphalt overlays as a SAMI, and/or
- geotextile-reinforced seals on clay pavements. These are not covered by this *Manual*.

Note: SAMs and SAMIs can use either a PMB alone or include a geotextile.

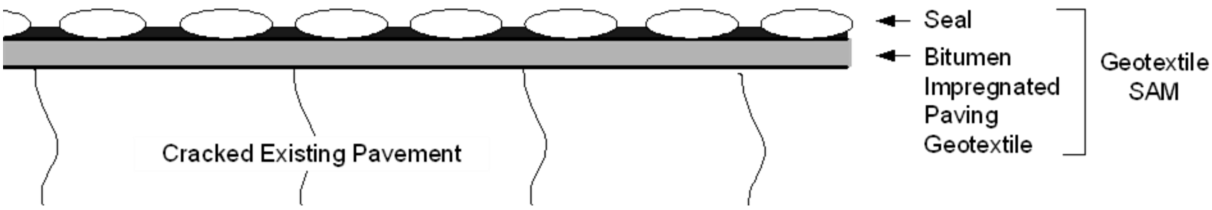
4.4.6.1 Functions

4.4.6.1.1 Strain alleviating membrane

The aims of a geotextile SAM (Figure 4.4.6.1.1) are to:

- provide a relatively impermeable membrane by using bitumen-impregnated geotextile which protects the underlying pavement by acting as a waterproof membrane (excess water can weaken a pavement, leading to premature failure; therefore, preventing water ingress can extend the life of the pavement)
- act as a road surfacing, and
- delay or inhibit cracks propagating from the substrate to the surface by mobilising the tensile strength of the geotextile and/or allowing a limited amount of slippage across a crack.

Figure 4.4.6.1.1 – Paving geotextile in a strain alleviating membrane over a cracked existing pavement



4.4.6.1.2 Strain alleviating absorbing membrane interlayer

The aims of a geotextile SAMI (Figure 4.4.6.1.2) are to:

- provide a relatively impermeable membrane, and
- delay or inhibit cracks propagating from the substrate to the surface.

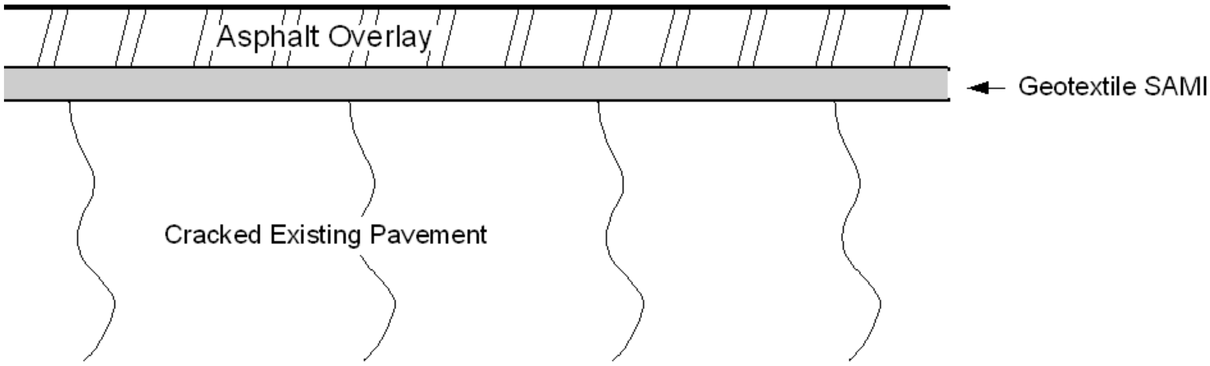
This is achieved in the same manner as for a geotextile SAM (see Section 4.4.6.1.1).

The design of a SAMI is different to a SAM. Geotextile SAMIs:

- are always covered by asphalt
- require a minimum thickness of asphalt to be placed over them, and
- are not designed to function as a road surfacing and so are not designed to be trafficked.

Trafficking of SAMIs by construction traffic should be minimised and, wherever possible, trafficking by general traffic should be avoided.

Figure 4.4.6.1.2 – Paving geotextile in strain alleviating absorbing membrane interlayer over a cracked existing pavement



4.4.6.2 Appropriate uses

Geotextile SAMs and SAMIs can be used on a variety of cracked flexible and flexible composite pavements (for example, pavements with a CTB / stabilised base). Like any technology, geotextile SAMs and SAMIs will only work if they are applied in appropriate situations.

SAMs and SAMIs are only effective where crack movements are, at least predominately, in the horizontal plane. Types of cracking that can be treated include:

- age cracking
- settlement cracking
- fatigue cracking
- cracking caused by expansive soils
- thermal cracking, and
- shrinkage cracking (for example, CTB pavements).

Geotextile SAMs and SAMIs will not effectively inhibit cracking migrating through them or overlying layers where the pavement is moving in the vertical plane (for example, rocking concrete slabs).

Geotextile SAMs and SAMIs are only effective if they are applied in a timely manner. If a cracked road is left for too long, water ingress will cause the road to fail. It is too late to apply a geotextile SAM or SAMI after a road has failed structurally or after water has entered the pavement.

4.4.6.3 Inappropriate uses

Geotextile SAMs and SAMIs should not be used in areas subject to high shear stresses (for example, within street intersections, on steep grades, or other areas where changes in vehicle speeds occur regularly).

Where significant shear forces will be applied to the surfacing by traffic (for example, turning movements associated with roundabouts and intersections), geotextile SAMIs (under asphalt) should be avoided. SAMs, like seals, may not be appropriate in such circumstances as well.

Geotextile SAMs and SAMIs are also not appropriate:

- where the underlying pavement is unsound or poorly drained
- where the underlying asphalt contains water
- under asphalt where its thickness is less than the minimum thickness required (for example, do not use under an ultrathin asphalt)
- for areas of fatigue-cracked asphalt, placement of a SAM or SAMI with a relatively thin asphalt overlay will only provide a short life
- over OGA
- over SMA unless it is proven to be in sound condition and its moisture content is no greater than 0.5% (the overlaying of SMA is discussed following)
- for treatment of cracks where the pavement has a vertical component to its current or potential movement
- where trafficking of a SAMI cannot be avoided
- where they are to be constructed in high stress areas, and/or
- where they are to be constructed on steep grades or tight bends.

The future rehabilitation of the pavement should be considered before applying a bitumen / geotextile interlayer treatment; for example, hot in-place asphalt recycling (HIPAR) will not be possible when a geotextile interlayer is present at the bottom of the asphalt layer intended for recycling.

Some earlier SMA surfacings were permeable and held water. Stripping of the existing SMA may, therefore, be an issue for older SMAs, particularly if they were constructed to the department's superseded MRS11.33 (12/99) specification for SMA, or earlier specifications; therefore, it is recommended the sealing over, or the overlaying of, an existing SMA is only considered where:

- it is proven to be in sound condition by an appropriate amount of coring and testing of it, and/or
- its moisture content is no greater than 0.5%.

4.4.6.4 Materials

Relevant departmental specifications include:

- MRS11 and MRTS11
- MRTS17 *Bitumen*
- MRS22 and MRTS22
- MRS30 and MRTS30, and
- MRS57 and MRTS57.

4.4.6.4.1 Geotextiles

Geotextiles must comply with MRS57 and MRTS57.

4.4.6.4.2 Raw material

Paving geotextiles are commonly manufactured from either polyester or polypropylene. Typically, polyester geotextiles have a melting point above 250°C and polypropylene geotextiles have a melting temperature of around 160°C.

In unmodified bitumen spraying operations conducted in accordance with MRS11 and MRTS11, geotextiles may be exposed to temperatures of up to 180°C (*The effective use of geotextiles in bituminous surfacings* (Pickering, 2000)). Consequently, only polyester geotextiles are permitted for use in sprayed bituminous seals (that is, for use in a SAM, refer to MRS57 and MRTS57).

For asphalt works constructed with unmodified binder in accordance with MRS30 and MRTS30, geotextiles may be exposed to temperatures of up to 145°C (Pickering, 2000). Because the geotextile will be exposed to relatively lower temperatures, both polyester and polypropylene geotextiles can be used for pure interlayers (not as a geotextile seal but as a bitumen impregnated geotextile interlayer with no bitumen sprayed on top) in asphalt work.

Often, as a construction expedient, a geotextile interlayer under asphalt is constructed as a geotextile seal rather than a pure interlayer. In these cases, only a polyester geotextile can be used.

Only UV-resistant geotextiles are permitted for use.

4.4.6.4.3 Types

Geotextiles may be manufactured in a variety of ways. They may be knitted, woven or non-woven and they can be bonded using different methods (for example, heat-bonded, resin-bonded or needle-punched).

Only non-woven, needle-punched geotextiles formed from mechanically-entangled filaments are permitted for paving applications. Other types shall not be used.

Heat- or resin-bonded geotextiles are unsuitable for paving applications because they do not readily absorb bitumen.

Only non-woven, needle-punched geotextiles are suitable because of their ability to absorb bitumen binder and because of their isotropic behaviour. It is difficult to predict the direction of applied load to a road's wearing surface. Given this, a geotextile with mechanical properties isotropic in nature in the horizontal plane are required. Non-woven geotextiles possess this trait.

The mass per unit area of a paving geotextile must be between 130 g/m² and 200 g/m² (MRTS57). Other recommended properties are given in Table 4.4.6.4.3. Typically, geotextiles with a mass per unit area of 140 g/m² are used in asphalt and seal applications. Heavier geotextiles can be used in some applications (refer Table 4.4.6.4.3).

Table 4.4.6.4.3 – Paving grade geotextile requirements modified from MRTS57

Property	Test Method	Application		
		Geotextile SAM or SAMI over a pavement without a soft or clay subgrade and without a soft or clay material within it	**Geotextile SAM or SAMI over a pavement with a soft or clay subgrade, or with a soft or clay material within it**	**Pure geotextile interlayer overlaid by asphalt**
Material	–	Polyester		Polyester or polypropylene
Calendering	–	Refer to clauses 6.1 and 8.3 of MRTS57		
Wide Strip Tensile Strength (kN) (refer Figure 4.4.6.4.3)	AS 3706.2	6.0	9.0	6.0
Elongation (%) (see Figure 4.4.6.4.3)	AS 3706.2	40%–70%		
G-rating	AS 3706.4			
Mass (g/m²)	AS 3706.1	130–160	170–200	130–160
Thickness (mm)	AS 3706.1	≥0.8	≥1.2	≥0.8
Melting Point (°C)	ASTM D276**	≥200	≥200	≥160
*Bitumen retention (loaded) (l/m²)	ASTM D6140***	≥0.9	≥1.1	≥0.9

*Note: Testing shall be completed in accordance with ASTM D6140 except Class 170 bitumen complying with MRTS17 shall be used for determination of bitumen retention. The manufacturer / supplier must be contacted to determine the actual bitumen retention so the design can be completed.

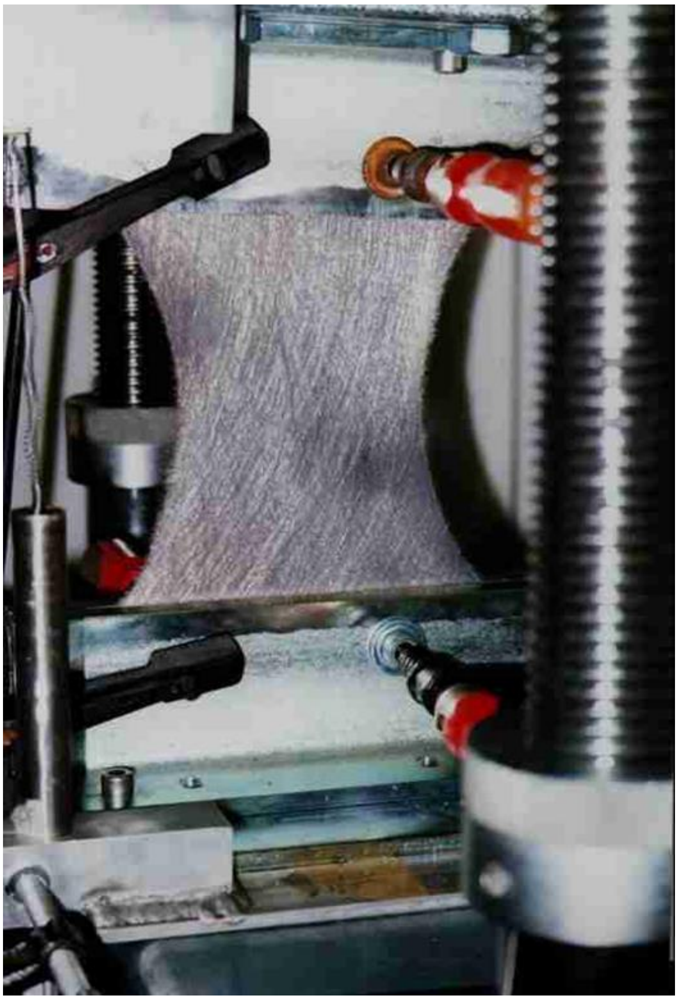
***Standard Test Methods for Identification of Fibers in Textiles* (ASTM D276)

****Standard Test Method to Determine Asphalt Retention of Paving Fabrics Used in Asphalt Paving for Full-Width Applications* (ASTM D6140)

The minimum recommended geotextile mass per unit area is 140 g/m². This is consistent with the US Task Force 25 recommendations, and it is doubtful the full benefits of a geotextile would be obtained in a paving application if a lighter-weight geotextile was used. Lighter-weight geotextiles may also not be robust enough to survive construction (for example, construction traffic).

Heavier grades of geotextile are available but there is a greater chance of a slip plane forming. In addition, heavier grades of geotextile absorb more bitumen, increasing bitumen quantities and subsequently increasing cost; therefore, the use of a geotextile with a mass per unit area of more than 200 g/m² is not recommended.

Figure 4.4.6.4.3 – Example of wide strip tensile test used to determine tensile strength and elongation



Source: Pickering, 2000

4.4.6.4.4 Calendering

Calendering is a process passing the geotextile through one or more heated rollers. This modifies the surface/s of the geotextile. Other properties may also be affected (for example, flexibility reduced).

Some non-woven, needle-punched geotextiles may be calendered on one or both sides. Calendaring can reduce the ability of the geotextile to absorb bitumen through the calendered side; therefore, those geotextiles heat-calendered on both sides are not permitted for any paving applications.

Non-woven, needle-punched geotextiles calendered on one side may only be used in some circumstances. MRTS57 defines the circumstances in which they can be used. Care must be exercised when planning the construction program where the use of a non-woven, needle-punched geotextile calendered on one side is used. In this case, sufficient tack coat must be sprayed to impregnate the geotextile fully and the side not calendered must be placed onto the tack coat.

Note: Calendaring reduces a geotextile's flexibility, and this can make them more difficult to lay.

4.4.6.4.5 Other types

Other geosynthetics (for example, geogrids, fibreglass materials, and so on) may be recommended by suppliers or manufacturers for pavement works. Composites may be similarly recommended. In all such cases, the Pavements, Material and Geotechnical Directorate should be contacted for advice before any decision is made regarding their use.

4.4.6.4.6 Binder

The bitumen binder used should allow the SAM or SAMI to fulfil its function while also providing adequate adhesive and shear strength between layers.

The binder recommended for:

- tack coats is hot Class 170 bitumen compliant with MRTS17 and constructed in accordance with MRS11, MRTS11, MRS57 and MRTS57, and
- top coats is hot Class 170 or cutback Class 170 bitumen compliant with MRTS17 or MRTS20 as relevant, and constructed in accordance with MRS11, MRTS11, MRS57 and MRTS57.

Bitumen used to spray the tack coat shall not be an emulsion or cutback bitumen. It shall be straight, unmodified, uncut Class 170 bitumen. Using cutter in the tack coat may trap cutter underneath the geotextile and lead to bleeding problems in the future. Similarly, the use of emulsions for the tack coat may trap water.

Note: Some geotextiles may be susceptible to damage from petroleum-based solvents.

If necessary, and the geotextile is not susceptible to damage from cutter, cutter can be used in the top coat (the balance of the bitumen applied to the top of the geotextile). Emulsions shall not be used for the top coat.

The use of bitumen emulsion is not permitted for any aspect of paving geotextile works.

4.4.6.4.7 Polymer modified binder

It is unusual to use a PMB with a geotextile interlayer.

The cost of both a geotextile seal and PMB would be relatively high and the additional cost is usually not warranted.

The use of a PMB with a geotextile also introduces construction difficulties (for example, rapid skinning of a PMB tack coat will inhibit absorption) which can lead to premature failure of the geotextile SAM or SAMI.

Further SAMIs and SAMs can also be constructed using a PMB without a geotextile.

Considering all of these, the use of PMBs with geotextiles is not recommended; when using a SAM or SAMI, the recommended choice is between:

- geotextile SAM or SAMI using unmodified binder with no cutter used in the tack coat and preferably none in the top coat, and
- PMB SAM or SAMI without a geotextile.

Reference should also be made to Section 4.11 which deals with the use of polymer modified and multigrade binders.

4.4.6.5 Design considerations

4.4.6.5.1 Tack coat

The tack coat is laid first, primarily to ensure there is an adequate bond between the paving geotextile and the existing pavement surface. It also partially impregnates the geotextile with bitumen.

Note: The tack coat forms part of the Total Bitumen Application Rate (TBAR).

The tack coat application rate is dependent on:

- the type of paving geotextile
- the mass per unit area of the paving geotextile
- the texture of the existing surface (surface texture)
- the ambient air temperature
- the pavement temperature, and
- the amount of rolling effort applied.

The tack coat rate should typically be between 0.6 l/m², which applies when the pavement temperature approaches 50°C, and 0.8 l/m², which applies for pavement temperatures between 15°C–50°C.

4.4.6.5.2 Sprayed strain alleviating membrane or strain alleviating absorbing membrane interlayer

Dry, pre-coated cover aggregate should be used, with a nominal size of 7mm–14mm.

As previously discussed, the bitumen binder used should be hot Class 170 bitumen:

- without cutter in the tack coat, and
- with or without cutter in the top coat as required.

Bitumen emulsions shall not be used for any part of a geotextile SAM or SAMI.

The design of a paving geotextile seal / reseal requires the designer to:

- obtain the geotextile's absorption from the supplier / manufacturer
- obtain the necessary inputs required for the Austroads seal design method (AGPT04K)
- design the seal using the Austroads seal design method (AGPT04K)
- determine the total binder application rate
- determine the tack coat binder application rate, and
- identify the binder application rate for the top coat (refer Table 4.4.6.5.3).

4.4.6.5.3 Total bitumen application rate

The total application rate of bitumen binder is equal to the sum of the binder required for:

- a sprayed seal / reseal design as per Austroads' design method (AP-T68 and AGPT04K), and
- the geotextile's absorption (for example, ≥0.9 l/m², refer Table 4.4.6.5.3).

The absorption of the geotextile should be obtained from the supplier / manufacturer of the paving geotextiles.

Table 4.4.6.5.3 shows how to calculate the total bitumen application rate.

Table 4.4.6.5.3 – Example of a geotextile strain alleviating membrane (single seal) design

Step		Example calculation (illustrative only)
No.	**Description**	
1	Determine the basic binder application rate (Bb) in accordance with the Austroads seal design method (AP-T68 and AGPT04K). This includes calculation of the design Voids Factor (VF) using the traffic volume for design, basic voids factor (Vf) and voids adjustment factors for the aggregate (Va) and for traffic effects (Vt).	Say Bb = 1.4 l/m²
2	Determine the design binder application rate (Bd) in accordance with the Austroads seal design method (AP-T68 and AGPT04K). This includes calculation of the allowances for surface texture (As), binder absorption (Aba) and embedment (Ae).	Say Bd = 1.8 l/m²
3	Determine the type of geotextile to be used (refer to MRS57 and MRTS57 (Table 6.2 in particular)).	Polyester geotextile compliant with MRS57 and MRTS57
4	Obtain the geotextile absorption rate (GAR) from the manufacturer / supplier (usually 0.9 l/m²–1.0 l/m² for a paving 140 g/m² geotextile).	Say GAR = 0.9 l/m²
5	Calculate the TBAR.	TBAR = Bd + GAR = 1.8 + 0.9 = 2.7 l/m²
6	Determine how to apply the TBAR.	Spray 0.7 l/m² as a tack coat, lay the geotextile, and spray 2.0 l/m² in the final seal.

4.4.6.5.4 Adhesion agents

The use of an adhesion agent in geotextile SAMs and SAMIs is recommended unless it is a pure interlayer (in which aggregate is not used).

4.4.6.5.5 Asphalt overlay

The minimum thickness of DGA to be applied over a geotextile SAMI using a geotextile with a mass of up to 140g/m² is 60mm (nominal) (typically, a DGA with a maximum nominal aggregate size of 14mm (AC14 mix) – for an AC14 layer, the thickness tolerance in MRTS30 is ±7mm so specifying a 60mm AC14 layer should produce an asphalt layer no thinner than 53mm). Where a geotextile with a mass higher than 140 g/m² is used, the minimum thickness of overlying asphalt will be greater than 60mm.

OGA surfacings have been trialled in conjunction with geotextiles to a limited extent. When OGA is used, there is a risk moisture entering it may tend to ‘strip’ the bitumen off the geotextile; therefore, OGA should only be placed on DGA which itself can be placed on a geotextile SAMI.

The pick-up and/or bleeding potential of geotextiles seem to be related, at least in part, to the uniformity of the geotextile. If the geotextile supplied does not appear to be uniform, the supplier should be notified and the matter rectified.

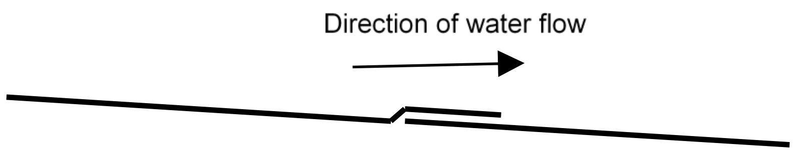
4.4.6.5.6 Floodways

Considerable hydraulic forces can be imposed on the surface of the pavement in floodway during major rainfall events.

Surfacing treatments incorporating a geotextile SAM or SAMI may be susceptible to separation of the interlayer from the pavement during major rainfall events. If geotextile SAM or SAMI treatments are likely to be exposed to such forces, a number of precautionary measures may be taken to reduce the risk of surfacing or other pavement failure. These measures include one or more of the following:

- adopt a type cross-section ensuring the edge of the geotextile on the most upstream side of the road is securely fixed (for example, by carrying the geotextile down the batter and covering with fill material or concrete, or by securing the geotextile into a longitudinal groove cut in the pavement)
- for carriageways with one-way crossfall, seal or concrete the upstream batter to prevent moisture entry and separation / lifting of the geotextile SAM or SAMI
- lap all longitudinal geotextile joints in the direction of water flow (refer Figure 4.4.6.5.6), and/or
- take particular care to ensure the underlying surface is clean and good adhesion is achieved.

Figure 4.4.6.5.6 – Diagram of longitudinal geotextile joints for a floodway



4.4.6.6 Construction considerations

4.4.6.6.1 Storage

Geotextiles must be stored under protective cover or wrapped with a waterproof, opaque, UV protective sheeting, including the ends of rolls, to avoid any damage prior to installation. Geotextiles shall not be stored on the ground or in any manner in which they may be affected by heat, sunlight or moisture. Additionally, the method of storage shall be in accordance with any recommendations set by the manufacturer. No chainsaw cut rolls shall be used.

4.4.6.6.2 Surface preparation and repairs

The area on which the geotextile is to be placed shall be prepared so all surface defects likely to cause tearing or damage to the geotextile are repaired or treated prior to placement of the geotextile. Isolated pavement failures should be repaired prior to the application of the geotextile SAM or SAMI. Geotextile SAMs and SAMIs should not be used in locations where a significant proportion of the pavement has failed badly. If geotextile SAM or SAMI treatments are used in these situations, they will be little more than short-term 'holding' treatments and recurrence of the pre-existing failures is likely. Structural failures cannot be treated by a geotextile SAM or SAMI (alone).

It is strongly recommended, if subsoil and/or pavement drains are to be installed as part of the remedial works, they are installed prior to application of the geotextile SAM or SAMI. They must also be effective in 'drying out' the pavement structure.

Existing cracks greater than 2mm wide should be filled and sealed and, where required, strain alleviating fabric strips installed as described in Section 4.4.4. Cracks should be sealed as described in Section 4.4.4. Filling cracks prior to installing the geotextile will prevent it from being starved of bitumen which can lead to a hungry seal which, in turn, leads to stone loss. Not filling cracks may also mean the geotextile will not be fully impregnated.

Surface depressions should be corrected and filled (for example, by applying an asphalt corrector course) prior to construction of the geotextile SAM or SAMI commences. If the existing pavement surface is badly deformed because of patching and existing failures, the application of an asphalt corrector course will greatly improve the performance of the rehabilitation treatment.

The surface of the pavement must be cleaned and swept prior to application of the tack coat and geotextile.

4.4.6.6.3 Placement of paving geotextiles

Unless otherwise approved by the Contract Administrator, geotextiles must be dispensed and laid using a purpose-built device specifically designed for such application (a geotextile applicator – mechanical applicators are usually available through suppliers.) It is important to stretch the geotextile, so it is always taut and wrinkle-free while it is being applied to the tack coat on the pavement surface.

The tack coat should be sprayed approximately 100mm wider than geotextile. The geotextile must then be immediately applied to the bitumen tack coat.

4.4.6.6.4 Wrinkles and folds

Geotextile shall be installed to avoid folds and wrinkles. Wrinkles formed during the laying process shall be removed by brooming and/or cutting. Folds and wrinkles in the geotextile large enough to cause laps and folds of greater than 25mm in length or height shall be cut, laid flat and lapped. Additional bitumen binder shall be sprayed over the lapped area at a rate equal to the bitumen absorption rate of the geotextile.

It is critical extra binder is sprayed between the geotextiles at the lap as there are two layers of geotextiles at it. Given this, extra binder additional to the TBAR and equal to the GAR is required (see Figure 4.4.6.6.6(a)).

4.4.6.6.5 Horizontal alignment

Curves in the horizontal alignment and around some other features (for example, intersections, traffic islands) are a particular problem when using geotextiles. Placing a straight roll of geotextile around a ‘tight’ curve will result in the formation of folds and/or wrinkles. These folds can be cut, the excess geotextile removed, and the remaining geotextile lapped with the cut treated as a joint.

Because of these difficulties, consideration should be given to using alternatives to geotextile SAMs or SAMIs where an alignment or other features with ‘tight’ curves exist. An alternative is to construct the SAM or SAMI with a PMB rather than a geotextile.

4.4.6.6.6 Joints

Joints must be of the 'cut and lap' type. Care is required during construction because they can cause surface failures. These failures may be caused by the application of excess or insufficient binder. Too little bitumen will result in a lack of bond strength, whereas too much can cause bleeding. In asphalt works, excess bitumen can also cause shoving or bleeding of the asphalt.

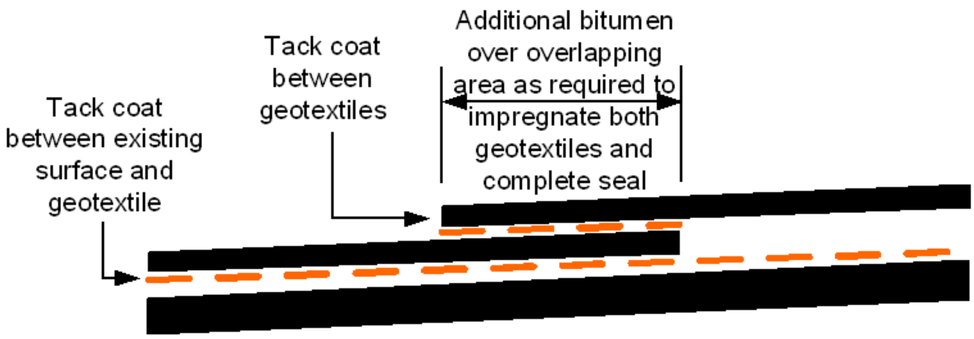
The application of excess binder may result in bleeding (of a seal or asphalt overlay) and/or the creation of a slippage plane. Asphalt overlays may shove and/or crack if a slippage plane is created. If insufficient binder is applied:

- a seal may become hungry, and/or
- an inadequate bond may be formed.

Should a seal joint become hungry, then stone loss (stripping) may result.

'Cut and lap' joints shall be overlapped a minimum distance of 150mm and must have a bitumen tack coat applied to the road surface. After application of the first (lower) geotextile, another tack coat should be sprayed on top of the lower geotextile over the overlapping area. Normally, this is done using a hand lance / hand-held sprayer, but it may be done by vehicular sprayer. Additional bitumen binder must be sprayed over the lapped area at a rate equal to the bitumen absorption rate of the geotextile. The second (upper) geotextile can then be placed (refer Figure 4.4.6.6.6(a)), followed by the final sprayed bituminous seal if this is required or specified. This will ensure proper bonding of the double layer of geotextile.

Figure 4.4.6.6.6(a) – Overlapping of geotextiles



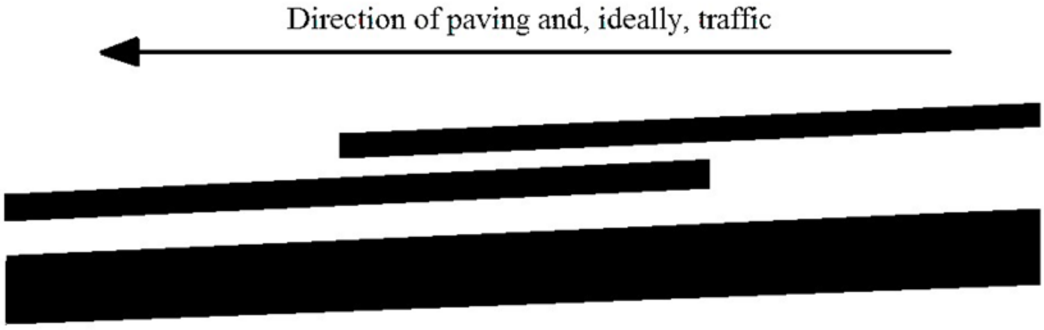
Geotextile widths shall be selected, and geotextile shall be laid so joints between parallel rolls (longitudinal joints) occur only at a centreline lane line, an edge line or in the shoulder. Joints shall not be located in any wheel path.

Transverse joints shall be lapped in the direction of paving to prevent edge pick-up by construction traffic and/or paver (refer Figure 4.4.6.6.6(b)). Further, the geotextile shall also be laid in the same direction as the traffic flow unless the Administrator determines it is not practical to do so.

Ideally, the direction of paving should be the same as the direction of traffic.

Note: Rolls of geotextile can be supplied in differing lengths. The longer the roll of geotextile the lesser the number of transverse joints. It is desirable the number of rolls be optimised so the number of joints are minimised.

Figure 4.4.6.6.6(b) – Transverse lap direction



4.4.6.6.7 Rolling

Rolling of the geotextile into the tack coat shall be carried out with a multi-tyred roller weighing less than 15 tonnes. The geotextile shall be rolled in a longitudinal direction only. Rolling shall commence along the centreline of the placed geotextile and gradually move towards the edge of the geotextile.

Rolling shall be sufficient to ensure the tack coat is absorbed into the geotextile whilst being less than required to cause bleeding of the bitumen through the geotextile or pick-up of the geotextile by rubber tyred plant.

4.4.6.6.8 Pick-up / bleeding problems

In general, pick-up and bleeding problems are closely related to:

- the amount of tack coat sprayed
- the ambient air temperature
- the pavement temperature
- the grade and uniformity of the geotextile
- the film thickness and viscosity of the bitumen binder, and
- the longitudinal grade of the road.

High ambient air and pavement temperatures are the usual cause of the problem. Typically, pavement temperatures in excess of 50°C have the potential to lead to bleeding / pick-up problems with geotextile placement operations. It is recommended geotextile SAMs and SAMIs not be placed in such conditions (refer Section 4.4.6.6.10).

In hot weather, the tack coat application rate may need to be reduced and the rate for the bitumen sprayed on the top increased by a corresponding amount. Alternatively, the works may need to be rescheduled for cooler periods to prevent pickup of the geotextile when trafficked by construction plant. If it is a pure interlayer, rescheduling is the only option.

4.4.6.6.9 Tack coat application rate

As discussed previously, it is critical the tack coat is applied at the correct rate.

To help overcome construction problems associated with over-application of the tack coat for a pure interlayer in asphalt works, consideration may be given to:

- minimising the number of vehicles trafficking the geotextile, and/or
- applying coarse sand in the wheel paths of the construction plant (the paver and trucks but excess sand must be swept off).

The second option is not desirable because the sand will absorb some bitumen, thereby robbing the geotextile, asphalt or seal of bitumen.

These options may solve construction difficulties associated with pick-up, but they will not necessarily solve the problem of excess bitumen, which can result in bleeding.

4.4.6.6.10 Ambient temperatures

Placement of geotextile shall cease when the pavement temperature exceeds 50°C unless otherwise directed by the Administrator.

4.4.6.6.11 Weather conditions

The spraying of bitumen or placement of geotextile must not take place until the road surface is dry. Further spraying of bitumen or placement of geotextile must not take place during periods of rain, nor if rain is likely to fall prior to the placement of geotextile, spreading and rolling of cover aggregate or the asphalt overlay, if specified, is constructed.

If the geotextile gets wet, it will retain water. If a sprayed seal is applied over a geotextile containing water, the bitumen will blister, leading to a failure of the seal.

4.4.6.6.12 Trafficking

Trafficking of the bitumen / geotextile shall be restricted to essential construction traffic only. The pavement shall not be opened to general traffic until after the final seal or asphalt overlay has been constructed over the bitumen-impregnated geotextile.

4.4.6.6.13 Construction of geotextile strain alleviation membrane and strain alleviating absorbing membrane interlayer

After the geotextile has been rolled into the tack coat, a seal can be constructed on top, so a SAM or SAMI is constructed. In this case, it must be designed as a SAM or SAMI as appropriate.

Hot bitumen binder is sprayed onto the laid bitumen / geotextile interlayer. The application rate for the bitumen sprayed onto the top is equal to the TBAR minus the tack coat application rate. Cover aggregate is then applied in the usual manner.

4.4.6.6.14 Laying of asphalt on geotextile strain alleviating absorbing member interlayer including pure interlayers

No tack coat for the asphalt should be applied to the top of the completed bitumen / geotextile SAMI.

Thin asphalt overlays (for example, 60mm thick) should be placed and compacted shortly after the application of the geotextile to maximise the bond between the existing pavement, the geotextile interlayer and the asphalt. This is not as critical with thicker asphalt overlays (for example, 100mm thick) where the heat and weight of the overlay promotes bonding. It is good practice to lay the asphalt overlay over the geotextile as soon as practicable in all cases.

Construction of the asphalt overlay must be completed in accordance with MRS30 and MRTS30.

Damage to the geotextile should be avoided by:

- minimising the amount of turning required by construction plant (for example, the paver and trucks)
- ensuring where turning of construction plant is required and cannot be avoided, it is done carefully and gradually, and
- avoiding parking construction plant on the geotextile for any period.

4.4.6.7 Expected performance or comments

When rehabilitating pavements incorporating a geotextile SAM or SAMI, their effect(s) on a treatment's 'constructability' and effectiveness need to be considered; for example:

- in-place recycling of asphalt cannot include a geotextile, and it will be difficult to recycle a surfacing course with a geotextile SAMI directly underneath it; for example, depth control would need to be consistently concise and an effective bond to the geotextile SAMI would also need to be produced
- geotextiles can be milled, although it may be difficult; any reclaimed asphalt pavement (RAP) will be contaminated with geotextile which may limit recycling options for the RAP
- milling of asphalt may leave less than the minimum thickness of asphalt over a SAMI, and/or
- a geotextile SAM would need to be removed prior to insitu stabilisation.

The future for geotextiles in paving applications is positive. The results from long-term performance trials at Beerburrum and the Cunningham Highway have reinforced the existing knowledge of, and experience with, using geotextiles in paving applications.

4.4.6.8 Further reading

Relevant publications include:

- AGPT03
- *Austroads Guide to Pavement Technology Part 4G: Geotextiles and Geogrids* (AGPT04G)
- AGPT04K
- *Standard Specification for Geosynthetic Specification for Highway Applications* (AASHTO M288)
- AP-T68
- AP-PWT00
- Australian Standard AS 3705 *Geotextiles – Identification, marking and general data* (AS 3705)
- Australian Standard AS 3706 *Geotextiles – Methods of test*, various Parts 0–13 (AS 3706.0, AS 3706.1, AS 3706.2, AS 3706.3, AS 3706.4, AS 3706.5, AS 3706.6, AS 3706.7, AS 3706.9, AS 3706.10.1, AS 3706.11, AS 3706.12, AS 3706.13)
- Pickering, 2000

- *Effective maintenance using geotextile reinforced spray seals* (Bullen, 1992)
- AGPT04G
- *Reflection cracking in asphalt overlays (with discussion)* (Caltabiano, 1991)
- *Thermal resistance of polypropylene geotextiles used in interlayers retarding the reflective cracking in pavements* (Rigo, 1990)
- *Durability and aging of geosynthetics* (Koerner, 1988)
- *Geotextiles and geomembranes in civil engineering* (text) (Zanten, 1986)
- *Fabrics in asphalt overlays – design, construction and specifications* (Button, 1984)
- *New Mexico study of interlayers used in reflective crack control* (Lorenz, 1987).

4.4.7 Geogrids in asphalt

A geogrid is a plastic fabric with relatively large openings, and is intended for use as reinforcement in soil, rock fill or pavements. There are several proprietary products available.

Note:

- Certain geogrids can be suitable for some applications but not others (for example, for use in embankments but not pavements), and
- only geogrids designed for use on pavements should be so used.

The discussion about geogrids in this *Manual* is limited to their use in asphalt layers in road pavements.

Geogrids must be covered by asphalt and a minimum thickness of asphalt is required over them (for example, 70mm). The requirements of the supplier / manufacturer in this regard should be followed. The department's Pavements, Materials and Geotechnical Directorate may be contacted for advice.

4.4.7.1 Function

The aim of a geogrid is to:

- delay or inhibit cracks propagating from the substrate to the surface, and/or
- reinforce the pavement.

This is achieved by mobilising the tensile strength of the geogrid (reinforcement). Unlike a SAMI, they do not allow slippage across a crack and do not provide a waterproof membrane to protect the underlying pavement (geogrid and geotextile composites may, however; refer to Section 4.4.8).

4.4.7.2 Appropriate uses

Geogrids are used to control cracks in pavements with asphalt layers, and to reinforce them. They can be used on a variety of cracked flexible and flexible composite pavements. They may also be used over repaired jointed concrete pavements with thick asphalt overlays. Like any technology, they will only work if they are applied in appropriate situations.

Geogrids are only effective if they are applied in a timely manner. If a cracked road is left for too long, water ingress will cause the road to fail. It is too late to apply a geogrid and asphalt overlay after a road has failed structurally, or after water has entered the pavement.

4.4.7.3 Inappropriate uses

Care should be exercised when using geogrids in areas subject to high shear stresses (for example, within street intersections, on steep grades, or other areas where changes in vehicle speeds occur regularly).

Uniaxial grids should not be used in pavement works.

Geogrids are also not appropriate:

- under asphalt where its thickness is less than the minimum thickness required (for example, do not use under an ultra-thin asphalt)
- for areas of fatigue-cracked asphalt, placement of a geogrid with a relatively thin asphalt overlay may only provide a short life
- over OGA surfacing
- over SMA unless it is proven to be in sound condition and its moisture content is no greater than 0.5% (the overlaying of SMA is discussed following)
- for treatment of cracks where the pavement has a vertical component to its current or potential movement
- where trafficking of a SAMI cannot be avoided
- where construction using them is difficult (for example, on steep grades), and/or
- on tight horizontal curves (for example, at intersections).

It is not known how effective geogrids are in treating cracks where the pavement has a vertical component to its current or potential movement.

The future rehabilitation of the pavement should be considered before applying a treatment that includes a geogrid; for example, HIPAR will be not possible when a geogrid is present at the bottom of the asphalt layer intended for recycling.

Some earlier SMA surfacings were permeable and held water. Stripping of the existing SMA may be an issue for older SMAs, particularly if they were constructed to the department's superseded MRS11.33 (12/99) specification for SMA or earlier specifications; therefore, it is recommended the sealing over or the overlaying of an existing SMA is only considered where:

- it is proven to be in sound condition by an appropriate amount of coring and testing of it, and/or
- its moisture content is no greater than 0.5%.

4.4.7.4 Materials

There are three types of geogrid, namely:

- welded mesh geogrids
- deformed geogrids, and
- high strength mesh geogrids.

Transport and Main Roads does not have any specifications or standards for geogrids.

It is recommended only biaxial grids be used in pavement works.

4.4.7.5 Design considerations

A minimum thickness of asphalt must be provided over geogrids (for example, 70mm). Where treatments or repairs are thinner than the width of a roll, it will be difficult to construct a treatment incorporating a geogrid, and alternatives may need to be considered.

The requirements of the supplier / manufacturer should be followed. The department's Pavements, Materials and Geotechnical Directorate may also be contacted for advice.

4.4.7.6 Construction considerations

Placing geogrids requires special equipment and construction control. The following aspects require special consideration before they are chosen as a treatment, before construction begins, and attention during construction:

- how the geogrid will be anchored within the pavement
- how the geogrid will be held in place during construction
- how adequate tension of the geogrid will be achieved and maintained
- the geogrid must not be damaged by construction plant
- damage to the geogrid caused by high paving temperatures must be avoided, and
- cutting and overlapping of geogrids, particularly on curves.

Where treatments or repairs are thinner than the width of a roll, it will be difficult to construct a treatment incorporating a geogrid and alternatives may need to be considered.

4.4.7.7 Expected performance or comments

Geogrids may be more effective than geotextile in reinforcing asphalt, if applied correctly. This is because they can lock into the aggregate and have superior strength / elongation properties; however, the bottom of the geogrid nodes may cause a slippage effect, resulting in shoving or early fatigue cracking of the asphalt overlay.

Geogrids placed near the bottom of the asphalt may improve the fatigue life of asphalt. When placed between asphalt courses, the geogrid may prevent or impede rutting.

To be effective, the geogrid needs to have a sufficiently high modulus to resist the formation or propagation of cracks within asphalt.

In place asphalt recycling cannot be carried out if geogrids are present, as they require high temperatures and forces to deform and break.

Experience from the crack control trial on cement treated base at Beerburrum Deviation (Bruce Highway) shows geogrids are difficult to place when used with a thin overlay. The techniques used during installation are critical. A flaw in any one of a number of key areas can create severe folding of the geogrid, thereby potentially negating any advantage of the system.

Note: Geogrids are expensive compared to other rehabilitation treatments addressing cracking (for example, SAMIs).

When rehabilitating pavements incorporating a geogrid, their effect(s) on a treatment's 'constructability' and effectiveness need to be considered; for example:

- in place recycling of asphalt cannot include a geogrid, and it will be difficult to recycle a surfacing course with a geogrid directly underneath it; for example, depth control would need to be consistently concise, and an effective bond to the geogrid and the underlying pavement layer would need to be produced
- geogrids can be milled, although it may be difficult; any RAP will be contaminated with geogrid which may limit recycling options for the RAP
- milling of asphalt may leave less than the minimum thickness of asphalt over a geogrid, and
- a geogrid would need to be removed prior to insitu stabilisation.

4.4.7.8 Further reading

Relevant publications include:

- AGPT03
- AGPT04G, and
- AP-PWT00.

4.4.8 Geogrid and geotextile composites

Some geogrids for use in pavements have a geotextile attached to them. The geotextile is usually attached to facilitate construction.

Geogrids by themselves are often stiff and can be difficult to lay. They often need to be firmly secured (for example, stapled) to the substrate; however, when a geotextile is attached to the geogrid, a tack coat is sprayed and the geogrid / geotextile composite is rolled onto it, adhering the geogrid to the substrate and avoiding the need for processes like stapling.

Note: The geotextile may not function as a paving geotextile as described in Section 4.4.6; rather, it may be included purely to facilitate construction – in some composites, the geotextile attached to the geogrid is not a paving grade geotextile and the intention of including it is not to waterproof or delay reflection cracking. In such cases, the geotextile is included merely to expedite construction. User expectations about the composite's performance should be moderated in these cases.

Most of the discussion in Section 4.4.7 pertains to geogrid / geotextile composites. There are some differences, including:

- A tack coat is required. The manufacturer / supplier should be contacted to ascertain the geotextile's absorption rate. As per Section 4.4.6, an emulsion or cutback bitumen should not be used in the tack coat. Straight Class 170 bitumen is recommended for the tack coat. Care must be exercised so excessive bitumen is not applied, and laps / joints are treated appropriately.
- A geotextile / geogrid composite may be thicker than a geogrid and may, therefore, have more potential for slippage, meaning the minimum thickness of asphalt to be used over them may be higher.
- Some of the discussion in Section 4.4.6 may apply, particularly where the geotextile attached to the geogrid is a paving geotextile as described in Section 4.4.6.

When rehabilitating pavements incorporating a geogrid / geotextile composite, its effect on a treatment's 'constructability' and effectiveness need to be considered. In addition, the future rehabilitation of the pavement should be considered before applying a treatment that includes a geogrid / geotextile composite. Some examples of such considerations include:

- In place recycling of asphalt cannot include a geogrid / geotextile composite and it will be difficult to recycle a surfacing course with a geogrid / geotextile composite directly underneath it; for example, depth control would need to be consistently precise. An effective bond to the geogrid / geotextile composite would also need to be produced.
- Geogrid / geotextile composites can be milled although it may be very difficult. Any RAP will be contaminated with geogrid / geotextile composite which may limit recycling options for the RAP.
- Milling of asphalt may leave less than the minimum thickness of asphalt over a geogrid / geotextile composite.
- A geogrid / geotextile composite would need to be removed prior to insitu stabilisation.

4.4.9 Open graded asphalt surfacing

OGA is porous; therefore, OGA must be placed on an impervious layer or layers to stop water penetrating underlying pavement layers. The PDS requires:

- the use of a PMB seal at the bottom of all surfacing layers, and
- 50mm thick AC14 layer immediately below the PMB seal under an OGA surfacing.

Reference should also be made to Section 4.11 which deals with the use of polymer modified and multigrade binders.

4.4.9.1 Functions

The main functions of OGA surfacings are:

- provide a surfacing with an appropriate level of skid resistance and surface texture
- provide a surfacing minimising the likelihood of aquaplaning
- provide a surfacing minimising water spray thrown up by vehicles
- provide a surfacing quieter than DGA, SMA, concrete or bituminous seal surfacings.

While it is not the main reason for selecting OGA, a possible ancillary benefit is it can also reduce water spray, and so improve visibility, in wet weather.

4.4.9.2 Appropriate uses

OGA can be used as a surfacing only where the underlying pavement is sound. In addition:

- OGA surfacings have also been used successfully on bleeding surfaces where excess bitumen was able to fill the voids without flushing the surface; however, this may compromise its functions (see Section 4.4.9.1) and, in this application, it may only be a short-term fix
- provision must be made to allow free flow of water through the layer (for example, free edge is proud of kerb and channel, transverse joints with DGA drain), and
- surfaces on which an OGA surfacing is placed must be reasonably smooth and free draining. Surface irregularities or depressions must be corrected prior to the construction of an OGA surfacing. This may include shape correction to milled surfaces.

4.4.9.3 Inappropriate uses

Inappropriate uses of OGA include:

- where the underlying pavement is unsound or poorly drained
- where the underlying asphalt contains water
- as anything other than a surfacing
- in areas subject to high shear forces (for example, tight curves, heavy braking areas)
- where it is not placed on a PMB seal, itself placed on a DGA layer (refer to the PDS for further details)
- as a treatment to improve structural capacity
- as a waterproof surfacing
- to correct the shape of the pavement
- where there is reflective cracking and it is not combined with appropriate treatments such as a SAMI or a minimum of 175mm (total thickness) of DGA (see also Section 4.9.6.3)
- over OGA surfacing.
- over SMA unless it is proven to be in sound condition and its moisture content is no greater than 0.5% (the overlaying of SMA is discussed following)
- over unsound asphalt: this should be removed or repaired as required before the overlay is constructed
- where cracks in underlying cement treated layers or concrete pavements are not treated (for example, refer sections 4.4.4 and 4.8)
- over surfaces that have depressions, rutting or other shape deficiencies corrected as these will pond water within the OGA layer (inhibit drainage through and out of the layer) and can lead to pavement failures – such shape deficiencies need to be corrected (for example, with a DGA corrector course) prior to overlay

- over milled surfaces without a DGA corrector course: grooves collect water and keep it within the OGA layer, even with a seal underneath the OGA surfacing (where the milling is very fine – for example, a fine milling drum with a tooth spacing not exceeding eight millimetre is used, this may be less of an issue), and
- for hardstand areas (for example, heavy vehicle parking areas).

Some earlier SMA surfacings were permeable and held water. Stripping of the existing SMA may be an issue for older SMAs, particularly if they were constructed to the department's superseded MRS11.33 (12/99) specification for SMA or earlier specifications; therefore, it is recommended the sealing over or the overlaying of an existing SMA is only considered where:

- it is proven to be in sound condition by an appropriate amount of coring and testing of it, and/or
- its moisture content is no greater than 0.5%.

4.4.9.4 Materials

Relevant departmental documents include:

- MRTS17
- MRTS18
- MRS30 and MRTS30.

4.4.9.5 Design considerations

OGA surfacings make a minor contribution to structural strength; however, they are used mainly due to their other properties (for example, porous nature).

Where the speed limit is greater than 80 km/h and asphalt surfacing is required, it is recommended an OGA surfacing be used.

Where the water flow paths are long, some of the desirable properties may be lost; for example, on a three-lane carriageway with one-way crossfall falling to the left, the leftmost lane may become saturated with water in wet weather, meaning there may be no or minimal spray reduction in the outer lane. To mitigate this, OG14 is recommended where there is a carriageway with three or more lanes with one-way crossfall.

4.4.9.6 Construction considerations

The following additional factors need to be considered:

- the construction requirements specified in MRTS30
- limited time of haulage, to reduce the chances of binder draining to the bottom of the truck bed
- OGA mixes not only are difficult to place and work by hand, but cool rapidly, so the time taken to spread it by hand may result in a lack of bond between the particles of aggregate
- a thin layer compresses very little under the roller and, as it cools quickly, it must be rolled as soon as possible
- evidence of binder drain down also includes blotches in the finished OGA surface, and
- placement of OGA prior to the mix cooling below the minimum rolling temperature, it is likely the OGA surfacing will ravel prematurely and require more frequent resurfacing.

4.4.9.7 Expected performance or comments

Based on the field experience with OGA surfacings, the following are points to note with respect to their performance:

- OGA surfacings should be laid in thin layers and with a consistent thickness. Any shape correction required should be done before a PMB seal and the OGA is laid (for example, with a DGA corrector course).
- It is porous, which means any liquid spilt on it will enter the matrix. Depending on the fluid, this can be detrimental to the OGA; for instance, a petroleum spill can cause rapid deterioration of the materials and, consequently, the surfacing. In such a case, the only solution is removal and replacement.

  Note: Patching is problematic (see following).

- It is difficult to undertake isolated repairs (patch) OGA surfacings; for example, a DGA patch cannot be used to repair an isolated failure in an OGA surfacing and it is difficult to get small quantities of OGA, especially if the repairer attempts to match the original OGA mix design.
- Line marking paint may change the properties of the OGA surfacing over the area of line marking (for example, reduced surface texture over line and/or may block pores).
- If the OGA surfacing is not carried across the full width of the carriageway (for example, shoulders not included) a ‘drop-off’ will result. Consideration needs to be given about whether this is an appropriate approach and, where appropriate, what maximum height of drop-off is acceptable for the project, where drop-offs are located and whether the portion not receiving an overlay requires any other sort of treatment (for example, reseal). This needs to include consideration of whether the use of the road by cyclists is significant and how the drop-off will affect them.
- Tapers, often in conjunction with cold planning, are usually required to match into road furniture that is not raised (for example, kerb and channels) or to match existing surface heights of the adjoining road(s) (for example, at each end of the works). Drainage of the OGA at joints with adjoining pavements needs to be addressed.
- Placement by hand and hand work often creates a different surface texture when compared to OGA laid by a paver.
- The expected service life of OGA is shorter than DGA surfacings.
- Early preventative maintenance is essential.
- The pavement remains wet longer.

4.4.9.8 Further reading

Relevant publications include:

- PDS
- AGPT03
- AGPT04B
- AP-T235, and
- AP-PWT00.

4.4.10 Stone mastic asphalt

Currently, the department does not have a general specification or standard for SMA; a project specific specification may be developed. The department's Principal Engineer (Road Surfacings) can be contacted for advice.

Reference should also be made to Section 4.11 which deals with the use of polymer modified and multigrade binders. Reference should also be made to the PDS.

4.4.11 Ultrathin asphalt surfacings

Currently, the department does not have a specification or standard specifically dealing with ultrathin asphalt surfacings. MRTS30 does have small-size DGA mixes that can be laid in a thin layer, but:

- AC7 mixes are not recommended as a surfacing layer for applications other than residential streets and pedestrian areas, and
- AC10 mixes are not recommended as a surfacing layer where the speed limit is 80 km/h or more, or on any motorway.

Any proposed ultrathin asphalt surfacing provides appropriate levels of skid resistance and surface texture. If such surfacings cannot provide appropriate levels of skid resistance and surface texture, they should not be used.

Given their thickness, ultrathin surfacings make virtually no structural contribution to the pavement. The underlying pavement must be in sound condition and, except for very minor ones, have no shape deficiencies.

4.4.12 High friction surfacings

The department's Principal Engineer (Asphalt and Surfacings) may be contacted for advice.

4.4.13 Slurry seals / surfacing and microsurfacing

Slurry seals, slurry surfacings and microsurfacings can be described as:

- aggregate mixes bound with bitumen emulsion and mixed insitu during placement by specialised plant using a dragged screed box are referred to as 'slurry surfacing'
- thin-layer systems using fine aggregates mixed with anionic-emulsion systems without polymer modification were called 'slurry seals' or 'slurries' previously, and/or
- Australia has seen the application of multiple stone-depth surfacings, using cationic emulsion systems, which incorporate a polymer in the binder to produce a quick-set surfacing able to be trafficked relatively quickly; these processes use more sophisticated mixing equipment – the

process is generically termed ‘microsurfacing’, and there are ‘microsurfacing’ proprietary products (AP-T235).

Reference should also be made to Section 4.11 which deals with the use of polymer modified and multigrade binders.

4.4.13.1 Functions

Slurry seals / surfacing and microsurfacing are a non-structural wearing course used as a surfacing to resurface a road and, to a very limited extent, correct minor rutting (10mm–15mm deep).

4.4.13.2 Appropriate uses

An advantage of seals or surfacings, or microsurfacing is they can be applied to damp surfaces.

The use of slurry seals or surfacings, or microsurfacing, is appropriate in the following cases:

- on roads with a low speed limit (for example, a speed limit of 80 km/h or less) because bituminous slurry seals / surfacings and microsurfacing have limited surface texture
- to correct minor shape deficiencies
- to surface a pavement or resurface a pavement with a bituminous sprayed seal surfacing where a bituminous sprayed seal surfacing is undesirable (for example, due to noise, to avoid windscreen damage)
- to surface a pavement or resurface a pavement where no or only a very small increase in surface heights is possible
- to prove a uniform surface texture, and/or
- as a pre-treatment for subsequent resurfacing with a bituminous sprayed seal surfacing; however, if done in wheel paths, for example, care will need to be exercised with respect to the seal design (different designs may be required for the areas with slurry as opposed to those without it).

4.4.13.3 Inappropriate uses

The use of slurry seals or surfacings, or microsurfacing, is inappropriate:

- where the slurry seal or surfacing will crack or fatigue quickly (for example, where the underlying pavement has inadequate structural strength)
- where the underlying asphalt contains water
- on roads with a low speed limit (70 km/h or less) because bituminous slurry seals / surfacings and microsurfacing have limited surface texture – they should not be used on motorways
- in areas likely to be subjected to repeated braking (for example, on approaches to signalised intersections)
- on pavements not stiff and sound as slurries tend to be stiff and susceptible to cracking
- to correct anything other than minor shape deficiencies as correction of ruts in excess of 15mm with one application will cause segregation of the mix and may lead to flushing of the surface (if the depth of shape correction is such the slurry seal is required to be laid in two layers, a thin asphalt overlay would be a competitive alternative resurfacing treatment)
- where any run-off of emulsion cannot be mitigated

- on newly-rejuvenated surfaces
- to prevent reflective cracking as slurry seals have no structural capacity, and/or
- on sites where the system cannot be used (for example, steep grades).

4.4.13.4 Materials

Relevant documents include:

- Specification (Measurement) MRS13 (MRS13) and Technical Specification MRTS13 *Bituminous Slurry Surfacing* (MRTS13), and
- MRTS21.

4.4.13.5 Design considerations

MRTS13 outlines mix design requirements. Table 4.4.13.5 outlines some accepted uses of some slurry mixes.

As slurry seals, slurry surfacings and microsurfacings are relatively thin and are not a structural layer, their structural contribution is ignored in the GMP.

Table 4.4.13.5 – Slurry mix types and their typical application(s)

*Mix	*Typically designed for (refer also to Sections 4.4.13.2 and 4.4.13.3):	Typical application rate (kg/m²)	Typical maximum thickness (mm)
QS10 (10mm mix (extra coarse mix))	Primarily corrects severe conditions, preventing hydroplaning and providing skid resistance under heavy traffic loads	16–25	13
QS7 (7mm mix (coarse mix))	Moderately to heavily trafficked roads to seal, correct moderate to severe ravelling, oxidation and loss of matrix and improved skid resistance	8–13.5	9
QS5 (5mm mix (general seal))	Asphalt and sealed pavements on minor and major roads, it provides an extremely good skid resistant surface while also correcting moderate to severe ravelling; this mix can be used for rut filling and in cape sealing	5.5–8	6
QS3 (3mm mix)	Low density / low wear pavements: this mix is used for maximum crack penetration, filling pavement voids and correcting moderate surface conditions – due to the fine texture of the mix, traffic noise is reduced	3–5.5	3.5

*Note: Not recommended for use on roads with a speed limit of 70 km/h or more or motorways.

4.4.13.6 Construction considerations

Construction considerations include:

- Emulsions can flow; the consequences of any runoff need to be considered and any mitigating measures necessary employed (for example, prevent run-off into watercourses, protect public utility plant) – whether the system can be applied to a particular site also needs to be considered (for example, may not be appropriate on a steep grade)
- prior to surfacing with a slurry seal, cracking and other major surface irregularities or structural deficiencies should be treated and the surface cleaned, and/or
- traffic should not be allowed onto the surfacing until it has cured sufficiently.

4.4.13.7 Further reading

Relevant publications include:

- AGPT03
- AGPT04F
- AP-PWT00
- AP-T235
- *Guidelines and Specifications for Microsurfacing* (AP-R569), and
- *Cold Overlay (Slurry Seal) Trials – Nerang Broadbeach Road – Gold Coast Springbrook Road* (RP 1143).

4.4.14 Hot in-place recycling of asphalt

HIPAR is an insitu process used to recycle existing asphalt in which the top asphalt layer is heated in place, scarified, remixed, re-laid and rolled. New binder, recycling additives, new asphalt mix, new aggregate or combinations of these may be added to obtain an end product with the desired characteristics. If HIPAR alone is proposed, and it is the final surfacing, then care must be taken to ensure the resultant product has appropriate properties for a surfacing (for example, skid resistance). Alternatively, the ‘Remix plus’ process described following or a two-stage process (that is, HIPAR plus new overlay) may be required. The discussion in this section is based on HIPAR plant and techniques used in Queensland on State Government projects.

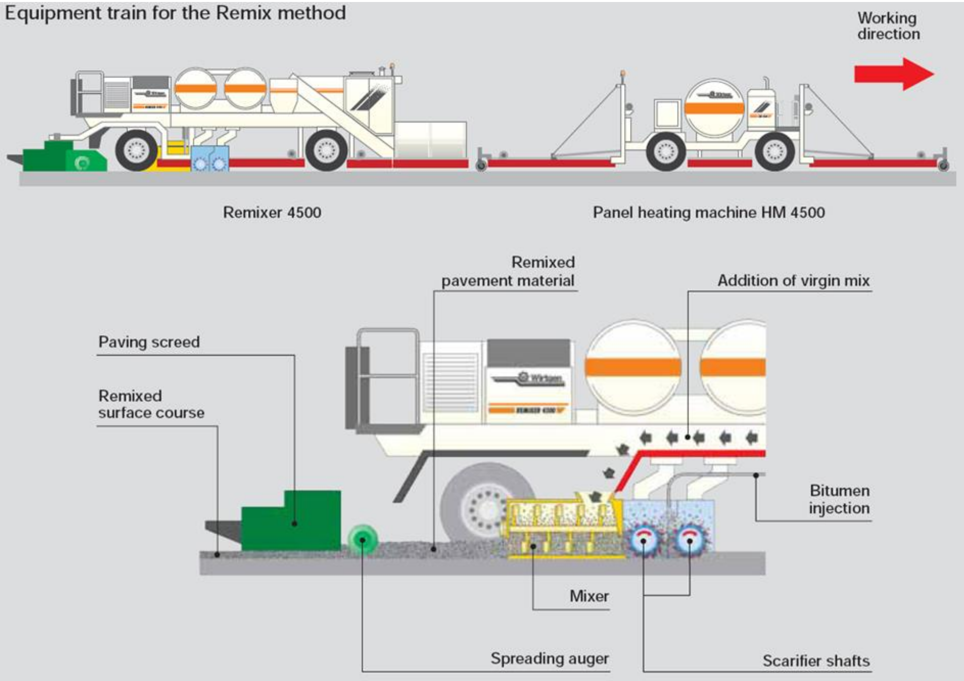
Currently, the department does not have a general specification or standard for HIPAR.

A fundamental advantage of HIPAR is the reduced consumption of virgin materials, the minimisation of waste and, in some cases, its ability to operate while minimising disruptions to traffic. It has the potential to be more economical than resurfacing with a new layer of asphalt.

HIPAR plant allows recycling of an existing layer (refer Figure 4.4.14(a)) which shows the ‘Remix’ process) while some other plant can do this and add a brand-new asphalt layer on top of the recycled layer (refer Figure 4.4.14(b) which shows the ‘Remix plus’ process). The latter process is useful where some limited strengthening of the existing pavement is required and/or the surfacing is deficient in some other respect (for example, contains aggregate with a low PAFV). An alternative to the use of the ‘Remix plus’ process is to HIPAR the existing asphalt and place a new asphalt layer on top as a separate process to provide a surfacing with the appropriate properties.

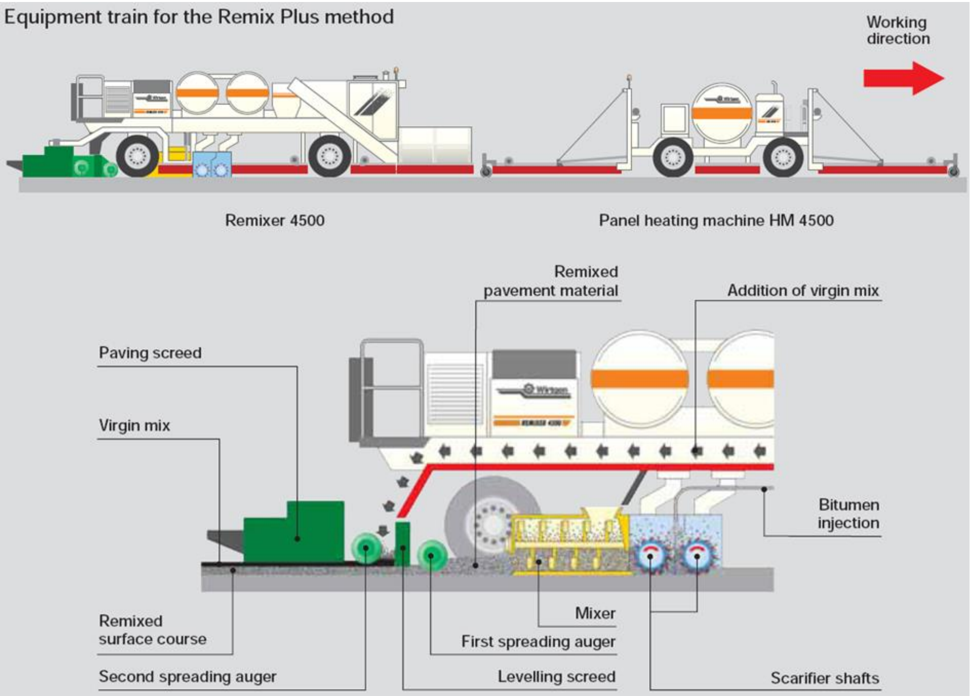
Any new constituents and the end product should conform to the relevant departmental specifications (for example, MRTS30). It is important the total compacted thickness of any asphalt layer, including construction layers, does not exceed the maximum values given in MRTS30 as relevant. This includes the 'Remix plus' process where the total compacted thickness (that is, compacted thickness of recycled asphalt plus compacted thickness of the new asphalt) does not exceed the maximum values given in MRTS30 as relevant (because the recycled asphalt and new asphalt are compacted together in the 'Remix plus' process).

Figure 4.4.14(a) – 'Remix' process



Source: (Wirtgen HR)

Figure 4.4.14(b) – 'Remix plus' process



Source: (Wirtgen HR)

4.4.14.1 Functions

HIPAR aims to restore, modify or correct an existing asphalt layer to an almost ‘as new’ asphalt layer. Some other properties may also be improved to a limited degree including roughness, shape and surface texture.

The bitumen in an asphalt layer exposed to air oxidises. This particularly affects the top 15–20mm of the layer. Oxidation leads to stiffening of the binder that, in turn, leads to age cracking of the surface. HIPAR helps to restore the aged or stiff binder to almost its original viscosity, provided suitable additives are used (for example, recycling agent). This rejuvenates the binder, helping to prolong the life of the asphalt layer, provided there are no structural or other deficiencies.

The ‘Remix’ process (see Figure 4.4.14(a)) is largely a rejuvenation process that does not enhance the structural capacity of the pavement; however, the ‘Remix plus’ process not only rejuvenates the existing asphalt layer but provides an additional new asphalt layer over the rejuvenated asphalt surface layer and is ‘hot bonded’ to it. The new asphalt layer may increase the structural capacity of the existing road pavements.

4.4.14.2 Appropriate uses

The department has no experience in relation to HIPAR of OGA or SMA asphalt.

HIPAR is appropriate in the following circumstances:

- all layers underneath the recycled layer are in sound condition and pavement drainage is adequate
- where adequate surface texture results: a DGA surfacing is not recommended on road sections with a speed limit of more than 80 km/h
- where projects are located far from DGA production plants
- where traffic volumes are not high and/or traffic management restrictions make its use practical; for example, traffic can be diverted around the area where HIPAR is to be done
- where existing levels cannot, or can only marginally, be increased
- where the individual lanes, but not all lanes, require treatment; for example, the left-hand (slow) lane of a rural motorway requires treatment but the right-hand (fast) lane does not
- to overcome surface problems such as surface irregularities and deformed surfaces, cracked, fretted and polished surfaces, subject to the following constraints:
  - deformed surfaces: cores must be taken across the pavement and the individual layer thicknesses compared – if variations in the surface thickness do not account for most of the visible deformation, it is possible the problem is in a lower layer and the surface layer will have to be milled off and the lower layer subject to HIPAR, while, if the deformation is limited principally to the surface course, it may be suitable for HIPAR
  - cracked surfaces: cores must be taken to determine the depth of cracking – if the cracks are limited to the surface course, HIPAR may be a suitable process, and
  - polished surfaces: HIPAR that adds a new layer (for example, the ‘Remix plus’ process) will normally be suitable for this maintenance requirement, provided the new surface layer meets the appropriate surface texture and skid resistance

requirements (for example, PAFV of aggregate used is greater than the minimum PAFV required) – refer to MRTS101 (including Annexure) for further details (a future Technical Note will provide further guidance), and/or

- to recycle a homogenous DGA surfacing. If there is extensive patching, for example, HIPAR may not be appropriate. As explained in Section 4.4.14.1, age cracking is caused by bitumen oxidation and hardening. Oxidation and hardening increase the viscosity of bitumen. When bitumen exceeds a critical viscosity of 7 x 106 Pa.s (6.85 log Pa.s) when tested in accordance with MRTS17, rejuvenation (HIPAR) is extremely difficult. This is due to a need for specialised chemical analysis to establish whether a recycling additive or additives, or a combination of additives and bitumen, will be able to restore the aged binder to an acceptable state. Any rejuvenation (or HIPAR work) should be carried out well before the bitumen reaches this critical viscosity.

4.4.14.3 Inappropriate uses

Transport and Main Roads has no experience in relation to HIPAR of OGA or SMA asphalt. Both these asphalt surface layers require polymer modified bitumen as a binder. HIPAR is not suitable for asphalt layers with PMBs as no research information is available about the effect of HIPAR on PMBs.

The recycling process is difficult, and can take a considerable amount of time, if there are public utility plant covers, drainage gullies or covers, and so on, present in the area to be milled. Such items are particularly prevalent in urban areas.

HIPAR should not be adopted:

- to recycle asphalts containing PMBs
- to recycle OGA or SMA
- to recycle a DGA surfacing that is not homogeneous (if there is extensive patching, for example, HIPAR may not be appropriate)
- so only a thin layer of an existing layer remains; for example, where there is a 50mm DGA layer, recycling to a depth of 40mm will leave 10mm of the original 50mm DGA surfacing layer – it would be best to recycle the surfacing entirely (its thickness may vary between tolerance limits)
- where any layer underneath the recycled layer is in unsound condition
- where changes in level are required (for example, to correct crossfall)
- where there are problems in the pavement underneath the layer to be recycled (for example, base or drainage problems exist, structurally deficient) – recycling of the top asphalt layer will not solve problems occurring lower down in the road formation
- when excessive hardening of binder has taken place (see Section 4.4.14.1.1)
- where the variations in the thickness of the asphalt layer to be recycled are excessive
- where the pavement structure cannot bear the recycling train and equipment (for example, because it is weak, cracking and/or rutting because of poor materials, inadequate thickness, and so on); the capacity of any structures (for example, culverts or bridges) to bear the recycling train will need to be considered as well
- when the pavement is excessively wet

- where there are geotextiles, geogrids or geogrid / geotextile composites within or close to the recycling zone
- where inadequate surface texture results; a DGA surfacing is not recommended on road sections with a speed limit of more than 80 km/h
- where traffic volumes are high and traffic management restrictions make its use impractical
- where HIPAR will cause unacceptable nuisance, adverse environmental or other adverse effects (for example, fumes released are a hazard), and/or
- where the recycled layer forms the surfacing of the completed pavement (for example, aggregate in existing asphalt may not comply with latest requirements for surfacing layers).

4.4.14.4 Materials

Any new constituents and the end product must conform to the relevant departmental specifications:

- MRTS17
- MRTS18
- MRTS19
- MRTS20
- MRTS21
- MRS30 and MRTS30, and
- MRTS101.

The recycling agent is required to bring the viscosity of the existing bitumen to the equivalent viscosity of new binder (for example, Class 320). It can be either a residual bitumen itself or a chemical rejuvenating agent. The recycling agents are in emulsified or oil-phased, non-emulsified forms. They are composed of selected maltene and asphaltene fractions derived from petroleum crude.

Transport and Main Roads has no experience with the inclusion of RAP external to the layer being recycled insitu being included in HIPAR works.

4.4.14.5 Design considerations

Any new constituents and the end product should conform to the relevant departmental specifications (for example, MRTS30). It is important the total compacted thickness of any asphalt layer, including construction layers, does not exceed the maximum values given in MRTS30 as relevant. This includes the 'Remix plus' process where the total compacted thickness (compacted thickness of recycled asphalt plus compacted thickness of the new asphalt) may not exceed the maximum values given in MRTS30 as relevant (because the recycled asphalt and new asphalt are compacted together in the 'Remix plus' process).

The maximum processing depth of the plant to be used must also be considered.

Testing additional to that given in MRTS30 to be performed for a good mix design comprise:

- indirect tensile strength
- immersion-tensile strength
- fatigue
- creep, and/or
- modulus of resilience.

The mix design process can be divided into five phases:

- characterising the existing materials
- determining the proportions of new aggregate
- determining the quantity and type of new binder to be added
- determining the quantity and type of recycling agent to be added, and
- mix preparation testing which includes tests for viscosity and Marshall properties.

Figure 4.4.14.4.1 summarises the mix design process.

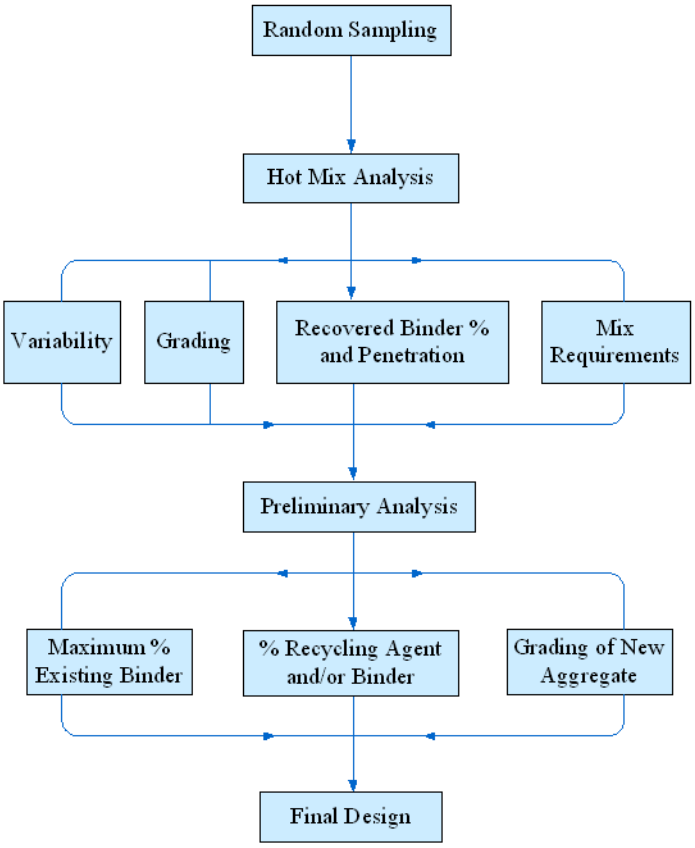
4.4.14.5.1 Characterising the existing materials

To characterise the existing materials, representative samples of the asphalt to be recycled must be obtained. The layer to be recycled must be defined clearly (for example, binder layer or surfacing). The asphalt samples taken should be asphalt slabs rather than cores. To characterise the samples of existing asphalt, establish:

- the layer to be recycled must be defined clearly
- binder type and content of the existing DGA to be recycled
- softening point and ductility of the recovered binder, or the apparent viscosity of bitumen
- grading and quality of the existing aggregate, and/or
- thickness and the degree of compaction of the pavement layers.

As explained in Section 4.4.15.1, age cracking is caused by bitumen oxidation and hardening. Oxidation and hardening increase the viscosity of bitumen. When bitumen exceeds a critical viscosity of 7×10^6 Pa.s (6.85 log Pa.s) when tested in accordance with MRTS17, rejuvenation (HIPAR) is extremely difficult. This is because there is a need for specialised chemical analysis to establish whether a recycling additive or additives, or a combination of additives and bitumen, will be able to restore the aged binder to an acceptable state. Any rejuvenation (or HIPAR work) should be carried out well before the bitumen reaches this critical viscosity.

Figure 4.4.14.4.1 – Flow chart for hot in-place asphalt recycling mix design



4.4.14.5.2 Determining the proportion of new aggregate

The final product must perform satisfactorily in terms of stability and its ability to withstand traffic and environmental stresses, ensuring the blend of new and existing aggregates produces an appropriate grading. The following information must be obtained to determine the proportion of new aggregate required:

- grading requirements of the type of layer to be constructed (refer MRTS30)
- grading of the existing aggregate.
- types, sizes and gradings of the available new aggregate, and/or
- the thickness to be recycled. The investigation needs to establish the depth to which oxidation and age hardening has taken place. The minimum depth of HIPAR should not be less than this depth; however, if this minimum depth means only a thin layer of the existing layer not recycled will remain, it is recommended the entire top asphalt layer be subject to HIPAR, provided the thickness remains within the limits of the mix.

This information will provide a good indication of the demand for new aggregate; however, the process of determining the most suitable proportions is largely iterative. MRTS30 should be referenced and various blends made up and evaluated against these standards to determine their suitability.

4.4.14.5.3 Determining the quantity and type of new binder

The quantity and type of new binder is determined by considering:

- the need to supplement the existing binder, and
- treating the aged binder so the properties of the reconstituted binder mirror those typically specified for new works (refer to MRTS17, MRTS18, MRTS30).

The first step is to obtain an estimate of the final binder content in the recycled mix by blending the existing aggregate (with binder extracted) and new aggregate in the proportions. The normal departmental mix design and testing procedure is then applied for each combination of new material / blend.

The binder content, type of binder, proportions of new aggregate, and the grading of the new aggregate can be adjusted to satisfy the desired mix-design criteria (for example, stability, flow and air voids).

4.4.14.5.4 Determining the quantity and type of recycling agent

There is a range of proprietary recycling agents available for use in the HIPAR process. These recycling agents can also be used on their own or in combination with soft, penetration-grade bitumen.

The process of determining the correct type and amount of recycling agent is iterative. Mixes should be checked with different dosages and the resulting reduction in viscosity and the Marshall voids will help to determine the optimum dosage. A significant reduction in viscosity may lead to a significant reduction in voids. A significant reduction in voids will result in a mix prone to rutting and flushing. The optimum dosage is determined by targeting a viscosity that satisfies the required Marshall properties.

4.4.14.5.5 Mix preparation and testing

The normal departmental mix design and testing procedure is applied for each combination of new material / blend.

The binder content, type of binder, proportions of new aggregate, and the grading of the new aggregate can be adjusted to satisfy the desired mix-design criteria (for example, stability, flow and air voids).

4.4.14.6 Construction considerations

The end product should conform to the relevant departmental specifications (for example, MRTS30). It is important the total compacted thickness of any asphalt layer, including construction layers, does not exceed the maximum values given in MRTS30 as relevant. This includes the ‘Remix plus’ process where the total compacted thickness (compacted thickness of recycled asphalt plus compacted thickness of the new asphalt) may not exceed the maximum values given in MRTS30 as relevant (because the recycled asphalt and new asphalt are compacted together in the ‘Remix plus’ process).

The following must also be considered:

- the production rate, working widths and workings depths of the plant to be used, and the location, treatment, and so on of joints – ideally, as many as possible of the construction requirements given in MRTS30 will be met
- the mass of the plant to be used, the stresses it imposes on the pavement and structures, and their ability to withstand these stresses

- suitable locations (for example, ground not soft, outside clear zone) where plant can be parked between work periods
- difficulty applying treatment at intersections and around tight curves
- treatment around structures, manholes, and so on
- the unlikelihood deck wearing surfaces can be recycled using HIPAR
- construction procedures and traffic management which account for the need for loading and filling operations (for example, gas) to take place clear of traffic
- Achieving a heating width approximately 200mm wider than the scarified width (100mm wider on each side) to ensure a hot joint with the existing asphalt
- including treatment (for example, removal) of raised pavement markers and line marking; contamination of the recycled asphalt is to be avoided
- consulting with the manufacturer and/or contractor to determine the characteristics of the plant and their proposed construction procedures
- extra, new asphalt to fill the screed and to correct undulations in the existing pavement, and/or
- temperature control, one of the most important factors affecting construction. Adequate temperatures must be achieved and maintained to achieve the required compaction and a good product.

4.4.14.6.1 Hot in-place asphalt recycling equipment

The features of the HIPAR equipment currently available in Australia and at sites in Queensland are similar to that depicted in Figure 4.4.14(a).

The panel heating machine is 12.7m long and weighs 20 tonnes. It preheats the pavement to 140°C with infrared heaters burning propane. A 6000-litre tank on this unit holds the heating gas.

The remixer is 15.2m long and weighs 42 tonnes. It continues to heat the pavement to 120°C, mills / scarifies it to the required depth, mixes in the required rejuvenating agent(s), additive(s), and so on in a pugmill, and then windrows the mixture for spreading and compaction by a double screed at the rear of the machine.

Both units have a variable working width of 2.4–4.2m. Heating panels may be turned off individually and the scarifier and screeds are infinitely variable via a hydraulic control. Scarifying depth is infinitely adjustable up to 80mm. The screed floats and has height controls for a string line or travelling beam if required. The tyres on the remixer are solid rubber because of temperatures to which the pavements are heated.

The working speed of the remixer is up to 5 m/min with mixing controls tied to speed of the machine. In a past job in the department’s Metropolitan Region, the machine travelled between 2–2.5 m/min for a 40mm average scarifying depth.

Rejuvenating agent is held in a 1500-litre tank on the remixer. It contains a heating unit and temperature sensors. A gear pump is used to pump the agent to the scarifier. A 5200-litre tank on the remixer holds the heating gas.

4.4.14.6.2 Operational aspects

The following aspects to HIPAR operations need to be considered:

- Transport: The remixer has a travel speed of only 7 km/h so allowance must be made for travel, from where equipment is stored outside working periods, to the job. Planks may be required, or a firm foundation may need to be constructed, to ensure the machines do not sink into soft material when stored beside the road overnight.
- Gas filling: Regulations require filling trucks to be 3.5m clear of any moving traffic. This should be borne in mind when deciding where the machines are to be stored outside working periods. Gas filling is only done once at the start of each work period.
- Adjustment of heating width: Front and rear panels of the panel heating machine can be adjusted in height and slewed independently to either side. Banks of heater units on both machines can be swung up out of the way for overall width-control while individual units have gas feed taps for fine control of the heating width. The total width of heating should be approximately 100mm outside the scarified width to ensure a hot joint with the existing pavement results.
- Firing of heating elements: Firing of the heating elements is done by the operators, using handheld flame torches.
- Removal of road studs and line marking: Road studs and line marking must be removed ahead of the panel heating machine.
- Addition of extra asphalt: Extra asphalt is required initially to fill the screed and to correct undulations in the existing pavement. It is added via a 3m³ hopper in the front of the remixer. From here, it travels via a heated, inclined conveyer to a batch bin on top of the machine. A heated horizontal conveyer then moves the asphalt to the pugmill where it is mixed with the recycled pavement material. The quantity to be added is controlled by the speed of the conveyer and the forward speed of the machine. If the two controls are not set properly, the pugmill can become clogged, forcing the whole operation to close down. This may cause the mix in the back section of the pugmill to cool and harden, making removal difficult and time-consuming.
- Scarification: The softened pavement is loosened by two rotating scarifier shafts with cutter teeth and augured into the pugmill with the help of levelling blades. The scarifier unit is suspended on rods positioned on both sides of the remixer and is positioned by two hydraulic cylinders to the desired depth of scarification. Automatic grade controls operate on each side to keep the selected depth of scarification. The measured depth immediately behind the scarifier varies depending on undulations in the unscarified surface. For an average scarification depth of 40mm, measured depths can vary from 30–50mm.
- Addition of rejuvenating agent(s): Rejuvenating agent is sprayed at a controlled rate onto the scarified material as it is augured into the pugmill. The rejuvenating agent to be used, and its incorporation rate, is determined as outlined in Section 4.4.3.5. Tank temperatures need to be checked regularly to ensure they stay at the correct temperature.
- Screed control: Having two screeds enables a different mix type to be placed on top of the recycled material (for the 'Remix plus' process).

- Temperature control: Temperature control is the most important factor in achieving compaction and a good surface. Temperatures immediately behind the rearmost screed vary from 100–120°C. Best results were achieved where this temperature ranges from 110–115°C. It is essential the asphalt not be over- or under-heated.
- Compaction: Compaction can be achieved using approved steel-wheeled rollers, vibratory rollers and pneumatic-tyred rollers.

4.4.14.7 Expected performance or comments

RAP can be used in the production of 'new' asphalt; for instance, MRTS30 permits a certain amount of RAP to be incorporated in 'new' asphalt. The alternative option of milling the layer and replacing it should also be considered. This overcomes some of the difficulties associated with HIPAR (for example, WH&S and environmental issues associated with fumes / gases released), although it may be more expensive.

There is a limited number of HIPAR plant / trains in Australia: competition between alternative suppliers may be limited and the procurement of HIPAR plant may be difficult, time-consuming or introduce time constraints.

The HIPAR process works extremely well when all parts are coordinated and working well, and the mix design has been done correctly. The train should be kept moving to minimise any possible overheating of the pavement or accelerated oxidation of the bitumen; it is important there are no delays in delivering materials, and so on to the train.

Ideally, to minimise the cost of the HIPAR:

- the number of hours the train is working in a work period should be maximised
- labour and supply of materials should be organised so there are no or minimal stoppages during each work period, and/or
- adjustment of the mix design in the field may be necessary to accommodate variations in the existing pavement.

4.4.14.8 Further reading

Relevant publications include:

- *Recycling of Asphalt Pavements* (Ramanujam, 2009)
- AP-PWT00
- *Hot Recyclers* (Wirtgen HR)
- AGPT04B
- *Hot In-Place Asphalt Recycling: Bruce Highway and Redcliffe Sub-Arterial* (RP 1568), and
- *Recycled asphalt wearing courses* (TRRL RR225).

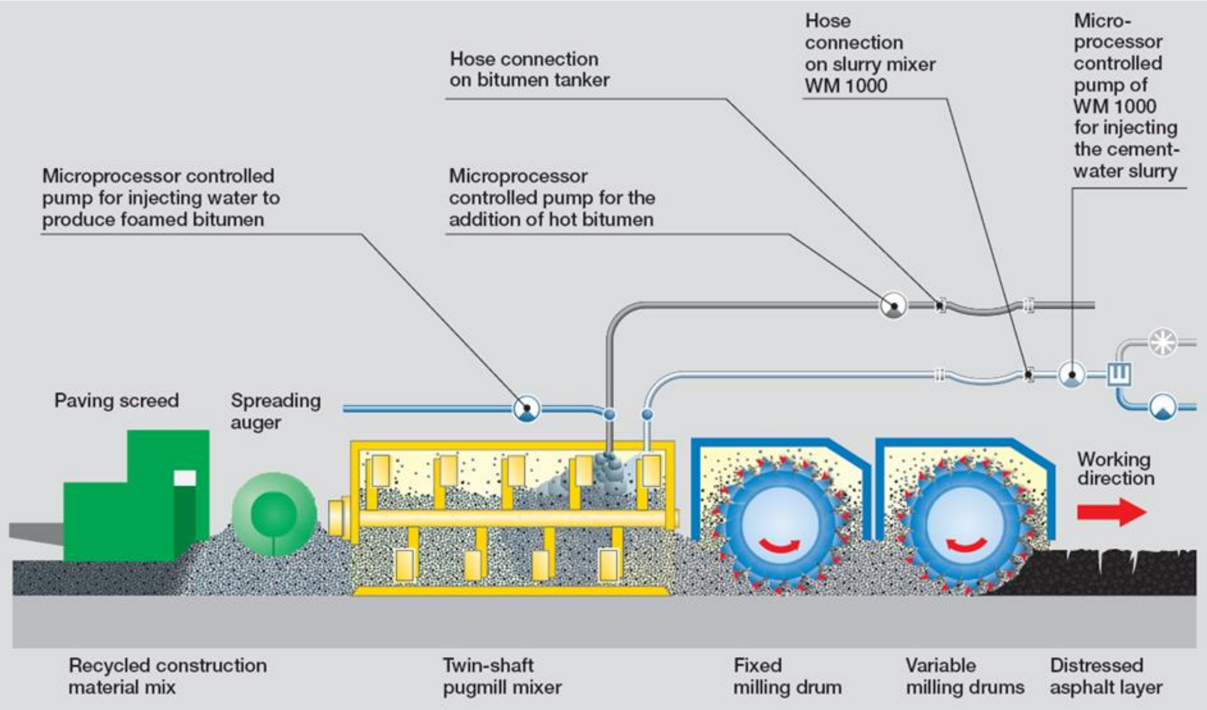
4.4.15 Cold in-place recycling of asphalt

In this *Manual*, cold insitu recycling is the process in which RAP is combined with new binder and/or recycling additives to produce cold-mix asphalt mixtures insitu. Plant to do this in one pass has been developed (for example, Figure 4.4.15). Other processes for recycling asphalt, such as stabilisation of it, exist but are not discussed in this *Manual*.

Fundamental advantages of cold insitu recycling are reduced energy consumption, the reduced consumption of virgin materials, the minimisation of waste and, in some cases, its ability to operate while minimising disruptions to traffic. It does require a new asphalt surfacing to be placed over it; a seal can be placed under the new surfacing. It is possible for the overall treatment to conform to the requirement given in the PDS a seal be provided under every surfacing layer.

Currently Transport and Main Roads does not have a general specification for cold insitu recycling.

Figure 4.4.15 – Cold in-place recycling



Source: Wirtgen CRSS

4.4.15.1 Function

Cold insitu recycling aims to reuse existing asphalt materials to produce a recycled material with physical properties which lie between hot mix asphalt and bituminous stabilised material (AGPT04B).

4.4.15.2 Appropriate and inappropriate uses

The department has very limited experience in Queensland in cold insitu recycling of asphalt.

Cold insitu recycling of asphalt cannot be used as a standalone treatment as it must be covered with a new asphalt surfacing. The recycled layer should not be used as a road surfacing.

Apart from these and the correction of shape and surface texture / skid resistance, constraints are similar to those of HIPAR (see section 4.4.14.1).

4.4.15.3 Materials

Any new constituents and the end product should conform to the relevant departmental specifications for asphalt.

The recycling agent is required to bring the viscosity of the existing bitumen to the equivalent viscosity of new binder (for example, Class 320). It can be either a residual bitumen itself or a chemical rejuvenating agent. The recycling agents are in emulsion form. They are composed of selected maltene and asphaltene fractions derived from petroleum crude.

Relevant departmental specifications include:

- MRTS17
- MRTS18
- MRTS19
- MRTS20
- MRTS21, and
- MRS30 and MRTS30.

4.4.15.4 Design considerations

Apart from the need to comply with MRTS30 and the normal departmental mix design procedures, design considerations are similar to those of HIPAR (see Section 4.4.14.5). Some of the more important additional tests performed to ensure a good mix design are:

- indirect tensile strength
- immersion-tensile strength
- fatigue
- creep, and
- modulus of resilience.

Selecting the right type of recycling agent is important. If the recycling agent has poor mixing characteristics with the aggregates, the bitumen binder could bleed to the surface of the pavement.

4.4.15.5 Construction considerations

Most considerations relevant to HIPAR are also relevant to cold insitu recycling of asphalt (see Section 4.4.14.5). In addition:

- recycling should be avoided during wet weather; in the case of cold in-place recycling, higher temperatures are needed so the cold mixture can cure and achieve higher strength values, and
- inadequate compaction stemming from the lack of heat applied to the mix is a potential problem.

4.4.15.6 Expected performance or comments

The recycling should be done when the climate is warm and dry. Cool or wet conditions will have an adverse effect on the recycling process. Higher temperatures are needed so the cold mixture can cure and achieve higher strengths earlier. Wet conditions are to be avoided because the recycled layers tend to be permeable.

Good quality control of emulsion dispersion and aeration of the recycled mix is important to achieve the required stability. Adjustment of the mix design in the field is sometimes necessary to accommodate variations in the existing pavement.

4.4.15.7 Further reading

Relevant publications include:

- AGPT03
- AGPT04B
- AP-PWT00
- *Cold Recyclers and Soil Stabilizers* (Wirtgen CRSS), and
- *Cold Recycling Manual* (Wirtgen CRM).

4.4.16 Cold milling, planing or profiling

’Cold milling’, also known as ‘cold planing’ and sometimes as ‘profiling’, involves the removal of a specified thickness of material mechanically. In most cases, it refers to milling off asphalt but may include the milling of any material used in pavements (for example, stabilised and granular materials). The plant used is designed to mill bound materials, different to stabilisers and reclaimer / stabilisers.

Currently the department does not have a specification or standard for cold milling.

4.4.16.1 Functions

Cold milling involves removal of existing materials by machine. It can be, and often is, used in conjunction with other rehabilitation techniques (for example, overlay). It can be used for smaller-scale patching or more extensive milling off of an existing surfacing before overlay.

4.4.16.2 Appropriate uses

Appropriate uses of cold milling include:

- removal of existing materials (for example, remove failed surfacing prior to overlay)
- shape correction (for example, to remove ‘bumps’ / decrease roughness, to restore pavement cross-section and provide adequate surface drainage), provided an acceptable surface texture / groove pattern is produced
- re-texturing of an existing DGA surfacing, provided an acceptable surface texture / groove pattern is produced, and/or
- when an overlay or resurfacing is required, and there is existing kerb and channel, milling can be used to:
  - prevent the loss of kerb and channel capacity (drainage)
  - avoid the need to raise kerb and channel, roadside furniture (for example, sign gantries, safety barriers) and the like, and
  - match levels of adjoining pavements (for example, tapers at ends of overlays).

The millings from asphalt only are known as RAP. This material can be reused in some other road construction materials or operations; for example, MRTS30 allows a certain amount of RAP to be included in the production of ‘new’ asphalt.

4.4.16.3 Inappropriate uses

The milling process is difficult, and can take a considerable amount of time, if there are public utility plant covers, drainage gullies or covers, and so on present in the area to be milled. Such items are particularly prevalent in urban areas. The presence of a geotextile or geogrid within the milling depth can also cause major problems during milling.

Cold milling is also not appropriate in the following circumstances:

- milling where there is a risk of the remaining layer delaminating or ravelling; for example, milling a thin layer to part of its thickness to restore texture may result in ravelling; therefore, an additional treatment such as a thin overlay may be required in conjunction with the milling, or the entire surfacing can be replaced
- milling where only a thin layer of an existing layer remains; for example, where there are three 50mm DGA layers milling to a depth of 40mm will leave 10mm of the original 50mm DGA surfacing layer (it is inappropriate to leave a thin 10mm layer in place (for example, for overlay) – it would be best to remove the surfacing entirely (its thickness may vary between tolerance limits) and is better to remove whole layers)
- if used for re-texturing or where the milled surface becomes the wearing surface, and the resultant texture is hazardous to road users (for example, cyclists or motorcyclists get caught in, and track along, deep grooves)
- re-texturing of an existing OGA or SMA surfacing, and
- where the asphalt layer overlying the milled surface is an OGA surfacing. If a DGA corrector course is placed first, and a seal constructed, then an OGA surfacing may be placed. Grooves collect water and keep it within the OGA layer, even with a seal underneath the OGA surfacing. Where the milling is very fine (for example, a fine milling drum with a tooth spacing not exceeding 8mm), this may be less of an issue.

4.4.16.4 Materials

No material is required for the milling operation.

4.4.16.5 Design considerations

There is minimal design involved in the milling project. The major decisions are about what the depth(s) of milling will be and the locations of the milling. This may vary (for example, where bumps are removed, over tapers to match existing levels, depth of unsound pavement varies).

4.4.16.6 Construction considerations

The finish of plant can vary considerably and this needs to be considered; it may, for instance, dictate the choice of plant where a fine finish is required for the re-texturing of a surfacing.

A variety of plant can be used, and the choice will depend on considerations like the depth, width and length of milling required. Small milling drum attachments can be fitted to bobcats or tractors (for example, for patching) and special drums fitted to them can mill to pre-set levels.

Larger dedicated machines are also available. These have higher production rates and usually:

- can mill deeper
- can adjust depth while milling is underway, and
- have better depth or level control (for example, in terms of accuracy and flexibility).

Often milled surfaces are trafficked (by general traffic) during construction; however, consideration must be given as to whether this is hazardous to road users (for example, cyclists or motorcyclists get caught in, and track along, deep grooves). Tracking will be a particular problem when changes of direction are attempted (for example, to avoid an obstacle, turn at an intersection or travel around a horizontal curve). If the surface is, or is likely to be hazardous, consideration should be given to use of plant producing a fine texture, alternative construction methods or alternative traffic management.

How materials will be removed in areas difficult to mill, or around structures difficult to mill against, must be considered; for example:

- the presence of public utility plant (for example, valve covers, manholes, drainage gullies) may require a variety of techniques such as milling plant of different sizes and removal by handwork
- tight curves at intersections may require a variety of techniques, and/or
- all material must be removed from the exposed vertical face of kerb and channel.

Other considerations include:

- saw cutting may be required (for example, at the ends of runs) to obtain a neat edge before asphalt is placed
- if the milling exposes a granular layer to remain, compaction and trimming may be required before any subsequent treatment is constructed; if asphalt (or a seal) is to be constructed on it, a prime or primer seal will be required
- reinstatement of detector loops (for example, for traffic signals or counters) may be required, and
- relieving slabs, public utility plant, kerb and channel, drainage structures and the like should not be damaged.

4.4.16.7 Expected performance or comments

It is essential the milling is performed within suitable tolerances to result in a smooth ride.

If a thin layer of materials is left in place, but appears likely to delaminate or ravel, the area should be re-milled and the entire layer of susceptible material removed.

4.4.16.8 Further reading

Relevant publications include:

- AGPT03, and
- AP-PWT00.

4.5 Asphalt overlay

Asphalt overlays can be a resurfacing or a structural treatment incorporating a surfacing layer. In an asphalt overlay treatment, the overall works may include:

- local repairs of any pavement failures (for example, cracks and potholes) prior to the overlay
- adjustment of manholes, drainage structures, drainage systems, road furniture (for example, kerbs, safety barriers) to suit the new surface heights
- improvements or additions to subsurface and surface drainage systems (for example, widen culverts)

- correction of the surface shape or cross-section of the road (for example, place asphalt corrector course(s) to correct rutting, crossfall or superelevation)
- construction of pavement and/or formation widenings
- works to match in at existing bridges or match existing surface heights (for example, reconstruction under an overpass rather than overlay)
- cold planning (milling) (see sections 4.6 and 4.4.16), and
- use of multiple asphalt types (for example, OGA surfacing, AC14 PMB layer and AC20 Class 600 base layer).

Reference should also be made to Section 4.11 which deals with the use of polymer modified and multigrade binders.

Asphalt overlays can be used in combination with other rehabilitation treatments; for example, an existing granular pavement may be stabilised with a SAMI constructed on top of the stabilised layer and an asphalt overlay constructed on the SAMI. The discussion in this section is for an asphalt overlay on its own but much of the discussion is also pertinent to combined treatments.

4.5.1 Functions

Where the primary purpose is resurfacing, the aim is to provide a surfacing with adequate properties (for example, skid resistance, surface texture, noise characteristics, riding quality, impermeable).

In the remaining cases, the purpose is to increase the structural capacity of the pavement as well as provide a surfacing with adequate properties.

In both cases, the shape of the surface of the pavement may also be corrected (for example, to remove ruts, improve ride quality, correct crossfall).

In some cases, the primary purpose is to correct the shape of the pavement surface or provide a surfacing with appropriate noise characteristics. Such treatments must include a surfacing with adequate properties

The PDS requires:

- the use of a PMB seal at the bottom of all surfacing layers, and
- AC14 layer immediately below the seal under an OGA surfacing.

4.5.2 Appropriate uses

Appropriate uses for asphalt overlays include:

- to correct the shape of the pavement (for example, to prevent water ponding in ruts)
- to improve the structural capacity of the pavement
- to reduce traffic noise
- to reduce roughness
- to improve skid resistance and surface texture
- to provide a less permeable surfacing when combined with an underlying seal
- to reduce road user costs, and/or
- to reduce maintenance costs.

4.5.3 Inappropriate uses

Inappropriate uses for asphalt overlays include:

- where the underlying pavement is unsound or poorly drained
- where the underlying asphalt contains water
- where there is reflective cracking and it is not combined with appropriate treatments, such as a SAMI or a minimum of 175mm (total thickness) of DGA (refer also Section 4.9.6.3)
- where there are restrictions with respect to increases in surface heights, and mitigation of these increases (for example, via milling) is not practical
- asphalt overlays are porous and should not be used solely to seal a pavement to prevent moisture ingress
- over OGA surfacing
- over unsound asphalt; this should be removed or repaired as required before the overlay is constructed
- where cracks in underlying cement treated layers or concrete pavements are not treated (for example, refer sections 4.4.4 and 4.8), and
- where the new surfacing layer does not include a PMB seal underneath it.

4.5.4 Materials

Relevant departmental specifications include:

- MRTS17
- MRTS18, and
- MRS30 and MRTS30.

4.5.5 Design considerations

Considerations for overlay design include:

- where the speed limit is >80 km/h, an OGA surfacing is recommended
- asphalt overlays can be designed using the GMP or by the deflection reduction method (refer to Chapter 5), relevant where a loading of 40 kN and tyre pressure of 750 kPa represents the road's experience
- adequacy of the existing pavement's structural capacity, roughness, surface texture and skid resistance should be evaluated prior to overlay design; if there is pavement distress resulting from a structural inadequacy and/or non-load associated factors are active (for example, degradation of pavement materials under environmental influences), then an overlay design based on deflection charts is inappropriate
- a proper survey of the existing conditions is required: based on this survey, the design may include base repairs, drainage improvements, crack filling, patching, joint repair and other treatments at some locations
- increases in surface heights, without associated pavement or formation widening, can narrow the formation or other cross-section elements (for example, shoulder); additional works may be required to ensure the end result is appropriate
- tapers, often in conjunction with cold planning, are usually required to match into road furniture that is not raised (for example, kerb and channels) or to match existing surface heights of the adjoining road(s) (for example, at each end of the works), and
- if the asphalt overlay is not carried across the full width of the carriageway (for example, shoulders not included), a 'drop-off' will result. The appropriateness of this approach should be considered and, where found appropriate, the maximum height of the drop-off acceptable for the project, the location/s of drop-offs and the requirement for the portion not receiving an overlay for any other sort of treatment (for example, reseal). This needs to include use of the road by cyclists, the significance of that use and how the drop-off will affect them.

In all cases, the PDS requires:

the use of a PMB seal at the bottom of all surfacing layers, and

AC14 layer immediately below the seal under an OGA surfacing.

4.5.6 Construction considerations

Key items in the construction of an asphalt overlay are proper preparation of the existing pavement, proper placement and compaction. Further traffic should not be allowed on the works while the recently-laid asphalt mat is still hot. Adequate planning must allow time for cooling before the works are opened to traffic.

Achieving an adequate level of compaction and avoiding segregation are two of the most important factors affecting the performance of an asphalt overlay. MRTS30 includes requirements aimed at achieving an adequate level of compaction and avoiding segregation.

4.5.7 Expected performance or comments

The performance of asphalt layers depends on:

- the composition of the asphalt mix and its appropriateness for the intended use
- the engineering properties of the compacted asphalt mix
- the appropriate specification of the job and mix requirements
- quality assurance during manufacture and construction to ensure they are achieved
- the properties of supporting pavement layers: their stiffness, and the bond between the layers
- the volume and characteristics of the traffic, and
- environmental and drainage conditions.

Each of these factors can be crucial in determining the effective life expectancy of the compacted mix and can determine the intensity of routine or special maintenance required to maintain the pavement to provide an acceptable level of service.

4.5.8 Further reading

Relevant publications include:

- PDS
- AGPT03
- AGPT04B
- AGPT04F
- AP-PWT00
- AP-T235
- *Overlays – Construction (A.C., Granular, CTB)* (Snow, 1981), and
- *Pavement Management Guide* (Hicks, 1977).

4.6 Asphalt inlay

A treatment incorporating the use of an asphalt overlay in combination with cold planning (milling) may sometimes be referred to as an asphalt inlay. Where the depth of milling equals the total thickness of the overlay (that is, existing surface heights will be matched), the combined treatment is sometimes called an inlay. Where the depth of milling is less than the total thickness of the overlay, it may be referred to as a partial inlay.

Much of the discussion pertaining to asphalt overlays (refer Section 4.5) applies to asphalt inlays. The chief difference is, inlays are used to minimise increases in surface heights and/or where the existing pavement materials are unsuitable for overlay (for example, remove extensively fatigued asphalt surfacing prior to overlay).

4.6.1 Construction considerations

A key issue for asphalt inlays is whether the surface height of the works at the end of a work period must match, or very nearly match, the surface heights of the adjacent road. It will be very difficult to do this where the total thickness of hot mix asphalt to be laid in one work period exceeds 100–150mm, because an asphalt layer must be allowed to cool sufficiently before the next layer is placed and before the works are opened to traffic. The use of the department's approved warm mix asphalt may help overcome some of these difficulties. The relevant Pavement Work Tips contain some further information about asphalt cooling (AP-PWT00).

Unsuitable and unsound material, including subgrade material if relevant, must be removed and replaced with suitable material and/or treated (for example, stabilised if suitable for stabilisation) before the new asphalt is constructed.

4.7 Granular overlay

Granular overlays are a viable form of pavement rehabilitation for flexible granular pavements, except for those that are or will be trafficked by high traffic volumes.

Granular overlays may be used in composite treatments. The discussion in this section is for a granular overlay on its own but much of the discussion is also pertinent to combined treatments.

An example of a composite treatment is a staged approach to the rehabilitation of a road. This may include the construction of a granular overlay, followed by stabilisation later. In such cases, the thickness of the granular overlay must be sufficient for stabilisation and the design must take account of all stages; if stabilisation is proposed, initially or in the future, representative samples of the material(s) to be stabilised must be tested prior to construction of the overlay to determine whether they are suitable for stabilisation and, if so, the mix design. This testing should include stabilisation testing and testing to determine the particle size distribution for the representative sample(s).

In a granular overlay treatment, the work involved may include:

- local repairs of any pavement prior to the overlay
- adjustment of manholes, drainage structures, drainage systems, road furniture (for example, kerbs, safety barriers) to suit the new surface heights
- improvements or additions to subsurface and surface drainage systems (for example, widen culverts)
- correction of the surface shape or cross-section of the road (for example, to correct crossfall or superelevation)
- tyning, scarification or removal of any existing surfacings (see Section 4.7.7)
- construction of pavement or formation widenings, and/or
- works to match in at existing bridges or match existing surface heights (for example, reconstruction under an overpass rather than overlay).

4.7.1 Functions

In general, granular overlays add structural strength to the pavement and/or improve its serviceability, for example, riding quality.

4.7.2 Appropriate uses

A granular overlay will only solve pavement deficiencies if the cross-section is designed to provide outlets for the various surface and subsurface drainage flow paths set up within the pavement structure and roadway proper. Deep table drains, subsoil or cut-off drains, adjustments to pavement and shoulder slopes, shoulder sealing and PMB waterproofing seal may be needed.

When using a granular overlay, the permeability of the granular overlay relative to underlying and adjacent materials must be considered to avoid trapping excess moisture in the pavement.

Appropriate uses for granular overlays include:

- to correct the shape of the existing pavement
- to improve the structural capacity of the pavement
- to reduce roughness
- to reduce road user costs
- when the existing pavement is surfaced by a bituminous seal or asphalt and the total thickness of bituminous seal and asphalt is thin (for example, <50mm); the surfacing may need to be scarified or tyned before construction of the overlay proper
- for overlaying flexible granular pavements
- where traffic volumes are low (for example, <2500 vehicles a day)
- where the resulting cross-section is adequate
- in combination with other appropriate treatments, such as insitu stabilisation of existing base and granular overlay to produce a thick stabilised layer, and/or
- where stabilisation is proposed after a granular overlay, the thickness of the granular overlay must be sufficient for stabilisation and the design must take account of all stages. If stabilisation is proposed, initially or in the future, representative samples of the material(s) to be stabilised must be tested prior to construction of the overlay to determine whether they are suitable for stabilisation and, if so, the mix design. This testing should include stabilisation testing and testing to determine the particle size distribution for the representative sample(s).

4.7.3 Inappropriate uses

Inappropriate uses for granular overlays include:

- where the existing pavement is saturated due to inadequate drainage, or drainage of the new pavement is inadequate
- where the existing pavement contains granular materials with poor gradations (for example, of the granular base) or are poorly compacted (excessive fines and/or low densities mean the granular material will be weak, particularly when its moisture content is high)
- where there is a wet, weak subgrade
- when the existing pavement is surfaced by a bituminous seal or asphalt and the total thickness of bituminous seal and asphalt is thick (for example, >50mm)
- where traffic volumes are high, and
- where the resulting cross-section is inadequate (for example, shoulder widths).

It may be inappropriate to overlay anything other than flexible granular pavements; for example, the construction of a granular overlay over a stabilised or other bound layer so a cementitiously-treated sub-base (CTSB) type of pavement results is not recommended. The reason for this is, should water enter the granular base, it often does not freely drain (water can, for example, enter through patches or oxidised (aged) seals where the pavement has not been resealed before the surfacing's impermeability is compromised). As a consequence, the degree of saturation increases beyond acceptable limits and the pavement fails. If a granular overlay over a stabilised or other bound layer so a CTSB-type of pavement results is proposed, it is essential a sound maintenance regime is implemented to ensure the impermeability of the pavement is maintained.

4.7.4 Materials

Relevant departmental specifications include:

- MRS05 and MRTS05
- WQ33 *Material Sources in Western Queensland*, and
- WQ35 *Paving Materials and Type Cross Sections for Roads on Expansive Soils in Western Queensland*.

The Western Queensland documents can be found under *Western Queensland Best Practice Guidelines* (WQBPG).

MRS05 and MRTS05 divide unbound granular pavement materials into four types. Some important points follow.

- Because of the minimal clay content, type one material has a very low unconfined strength. Consequently, a single / single seal may not provide adequate confinement, particularly under heavy traffic. Where a type one material is used for the granular overlay, a double / double seal, a DGA wearing course where such a surfacing is permitted, or a DGA base course with an OGA wearing course should be applied.
- Except in limited circumstances as permitted under MRTS05, type one materials must be constructed using self-propelled spreading machines purpose-built for this work.
- It is preferable type one material not be subjected to traffic without a wearing course, and it is subjected to traffic only for a very short period if it is covered only by a prime.
- Type one base will deteriorate rapidly if the wearing course fails.
- As significant amounts of clay can be present in a type two or type three material, there is the possibility unsatisfactory quantities of active clays will be present if the material is sourced from some quarries (particularly those with secondary mineral(s) present). Where this is likely, either a type one material should be specified or a type two or type three material specified with supplementary requirements to reduce the danger posed by the presence of active clay.
- The requirements for type one, two and three materials assume the pavement will have a wearing course or be overlaid with asphalt. If these are not applied, type one material should not be used, and type two and type three material should have an appropriate minimum plasticity index (PI) or linear shrinkage specified.

Reference should be made to MRS05 and MRTS05 for further details.

4.7.5 Design considerations

The minimum thickness of a granular overlay is 100mm, but a minimum of 150mm is highly desirable.

4.7.5.1 Overlay or reconstruct

The designer should consider whether to: leave the existing pavement intact; to scarify and spread it; or modify, stabilise or salvage it for use as a selected subgrade or sub-base layer. In addition to structural design aspects, the designer should consider the cost of any works associated with overlaying the existing pavement, such as:

- repairing localised weak areas in the existing pavement prior to overlay
- raising road furniture
- widening the pavement or formation
- widening or upgrading drainage systems (for example, cross-culverts, subsurface drainage), and
- correcting the shape of the existing pavement, including correcting roughness, rutting, crossfall, superelevation and cross-section. The latter can be particularly significant where the crown of the overlay does not align with the existing crown.

In some cases, construction costs may be increased because:

- the construction of narrow strips of widening beside the existing pavement may mean plant is restricted, making it difficult to excavate, compact and backfill materials adequately; sometimes, the formation / pavement must be widened more than the width given in cross-section standards to provide sufficient width for plant to operate in the widenings
- the 'corrector layer' over the existing pavement may require the placement and compaction of varying depths of material
- the widening and corrector-course construction operations may be affected by the need to maintain traffic flow through the job
- moisture control is critical when compacting material over the existing bitumen seal, and/or
- a project with compacting material over the existing bitumen seal but without adequate moisture control failed within eight hours. To control moisture, the moisture gradient (moisture contents with depth) and degrees of saturation (DOSs) with depth should be tested before trafficking or sealing of the granular layer(s). This is particularly important where the total thickness of granular material over a seal is thick.

If the existing seal is left in, intact:

- the depth of stabilisation must be at least equal to depth of bottom of seal, and
- shape deficiencies (for example, ruts, depressions) must be corrected with asphalt to avoid potential ‘bird baths’ trapping water within the granular layer.

4.7.5.2 Structural design

Granular overlays can be designed using the GMP or by deflection (refer to Chapter 5), relevant where a loading of 40 kN and tyre pressure of 550 kPa represents the road's experience.

4.7.5.3 Design of the geometry

Increases in surface heights without associated pavement or formation widening can narrow the formation or other cross-section elements (for example, shoulder). The appropriateness of additional works to ensure the end result should be considered.

Note: The overlay thickness derived by the pavement rehabilitation designer is the minimum thickness required. If shape correction is required, care must be exercised to ensure the minimum thickness is provided at all points.

Design geometry to:

- produce a design with the minimum overlay thickness specified by the pavement rehabilitation designer at all points
- minimise the quantity of overlay material needed to correct the shape (for example, alignment) of the existing pavement
- meet appropriate standards, and
- provide the minimum thickness required for the structural overlay.

To do this, a survey of the existing pavement is required: deficiencies and solutions can then be identified. This methodology minimises material requirements and enables an estimate of corrector layer quantities to be made in advance of construction.

4.7.6 Construction considerations

The minimum thickness of a granular overlay is 100mm but a minimum of 150mm is highly desirable.

4.7.6.1 Laying of granular corrector material

Corrector material should be laid as a separate course (correction occur) prior to construction of the overlay proper. An overlay course with a uniform thickness can then be constructed.

If the granular corrector is laid as a separate course, tyning of the existing pavement prior to placement of the corrector is required.

4.7.6.2 Locations and alignment of culvert headwalls

When placing pavement layers over the full formation width, the outer edge of the paver runs along the pavement edge. This may coincide with the location of culvert headwalls and prevent continuous operation of the paver. If continuous operation of the paver is not possible, surface irregularities will occur (roughness will increase) when the paver stops and on tapered sections.

Culvert headwalls should be located outside the line of the paver's operation to achieve continuous operation of the paver. Provided the cost is not excessive, culvert extensions should be considered where the continuous operation of the paver is not possible with the existing culvert widths. If a project is extending culverts, the widening of them should provide for any future widening envisaged.

4.7.6.3 Draining pavement layers at culverts

It may be necessary to seal the formation full width between headwalls over culverts to minimise ingress of surface water into the edge of the pavement. It may also be desirable to provide drainage holes through the headwalls or the outer sections of the culvert deck.

4.7.6.4 'Daylighting' pavement layers

It is common practice to extend base and sub-base layers over the full carriageway width, with edges trimmed to match the embankment batter slope. The design intention is to allow a free-draining outlet for excess moisture in the pavement; however, such exposed edges are difficult to maintain, especially if the pavement consists of crushed gravel with a minimum amount of binding soil fines.

4.7.6.5 Using double / double bitumen seals

When overlays are constructed over existing pavements with a bituminous surfacing left intact, the surfacing is a moisture barrier. Double / double seals on the overlay may be needed to minimise the risk of water infiltration into the overlay gravel. When constructing under traffic, it may be necessary to apply a primer seal instead of a priming coat to the finished overlay. This initial treatment is followed later by a seal.

Maintenance of surfacings to prevent the ingress of moisture is more critical where existing pavements with a bituminous surfacing left intact are overlaid.

4.7.6.6 Other aspects

The first step is to repair any localised weak areas in the existing pavement. These may be located by visual inspection and/or deflection testing. The repairs should be designed to ensure the strength of the resulting pavement equals, or exceeds, the surrounding pavement. Cementitiously treated granular material is often used to repair small areas. Care must be exercised to ensure cementitiously modified or stabilised materials used in repairs do not include too much binder.

The next step is to ensure the existing bituminous surfacing, if left intact, will shed water. It may be necessary to fill depressions and establish adequate crossfall.

If possible and required, the corrector layer should be constructed next. The heights on any widening can be related to the corrected profile, rather than the existing pavement profile, achieving uniform pavement thickness on the widening and avoiding the use of extra material as backfill. It may be difficult to maintain a relatively thin granular corrector layer under traffic while any widening is constructed. Consequently, on most jobs with widenings, widening is constructed first, followed by the laying of corrector over the existing pavement and widening. Care must be taken to ensure the widening backfill material is kept slightly below the existing bitumen surface to ensure adequate drainage of the pavement structure. Care must also be taken to minimise damage to the edge of the existing pavement while compacting the widening material.

When working under traffic, it may be more convenient to use a paver rather than a grader to spread a corrector layer, while one lane remains open to traffic; however, it may be difficult to correct surface distortions in one pass fully with the paver, and there may be localised areas where placement depths are excessive.

It is possible to use a grader to spread and compact the corrector layer progressively until an accurate profile is obtained.

The minimum thickness of corrector is 75mm with a greater thickness preferred (see MRTS05 as well). Thinner layers may segregate during spreading, loss of moisture is more rapid, and the material may lose stability under traffic. Thicknesses less than 75mm may work at isolated points, depending on the material, type of compaction equipment, and period under traffic. They may not comply with MRTS05.

The next step is construction of the granular overlay itself. It is placed to a uniform thickness where the surface shape has been corrected. A paver may be used successfully, provided the material is uniform (for example, moisture uniformly mixed to ensure uniform screed friction). The compacted layer thickness should not exceed 150mm. On a job where the thickness of granular overlay required is 185mm, one possible approach is to construct a nominal 95mm corrector layer (minimum depth 85mm with 10mm allowed to correct for shape deficiencies) followed by a nominal 100mm top layer.

When the job is constructed under traffic, it may be possible to apply a primer seal to each section as it is completed. Later, after a suitable time has elapsed, a bituminous seal or asphalt surfacing can be applied over the whole job.

Constructing a granular overlay under traffic may limit the length of sections to be worked at any one time. Excavations for widenings should be limited to a length able to be excavated, compacted and trimmed in one day where traffic management does not allow two-way traffic flow outside work periods.

Wet weather can affect job progress, especially when placing granular layers over existing bituminous surfacings left intact. A bituminous surfacing left intact is a moisture barrier impeding the dissipation of excess moisture in the overlay.

4.7.7 Expected performance or comments

There are at least two approaches to bituminous surfacings before construction of the granular overlay: to scarify or tyne bituminous surfacings; or to retain it. Appropriate pavement drainage must be provided in both options, including ensuring the crossfall at interfaces is adequate for the full width of the pavement.

Retention of any bituminous surfacing provides strength in the upper part of the pavement structure and, where the design is based on the deflection reduction method, the overlay thickness is usually determined by the bituminous surfacing remaining in place. If the seal is to be left intact, care must be taken to fill surface depressions that will not drain with some form of sealed corrector course prior to overlay; there are concerns about leaving any existing bituminous surfacing intact beneath an overlay. Many sections of old pavement surfaces barely shed water and gravel layers placed over them may not drain effectively.

Scarifying or tyning the existing bituminous surfacing avoids laminations and reduces the potential to trap water in the granular overlay in rutted areas or depressions.

While minimum thickness of a granular overlay is 100mm, the designer should consider the appropriateness of a granular overlay if its compacted thickness is less than 150mm.

The poor condition and narrow width of many existing pavements, coupled with design and construction difficulties encountered when constructing a granular overlay, mean alternative approaches need to be considered.

- Combine a granular overlay with insitu stabilisation of the overlay, or insitu stabilisation of a mixture of the overlay and some suitable existing pavement materials; if stabilisation is proposed, initially or in the future, representative samples of the material(s) to be stabilised must be tested prior to construction of the overlay to determine whether they are suitable for stabilisation and, if so, the mix design. This testing should include stabilisation testing and testing to determine the particle size distribution for the representative sample(s).

- Virtually reconstruct the pavement by removing the existing pavement to the subgrade level and constructing a new pavement. The design of the treatment would then be based on the subgrade of the existing pavement. The reclaimed existing pavement material could be salvaged for use in earthworks, as a select fill or as a sub-base. The end use will depend on the quantity and quality of the reclaimed materials, and the economy of the construction operation. To this end, testing will be required.

4.7.8 Further reading

Relevant publications include:

- *Guide to Pavement Technology Part 4A: Granular Base and Subbase Materials* (AGPT04A), and
- Snow, 1981.

4.8 Treating concrete pavements

This section includes discussion about treatments for concrete pavements used by Transport and Main Roads. AGPT05 and the following publications are useful references and contain additional information:

- AGPT04C
- AGPT07
- Technical Note TN143 *Rehabilitation Technique and Treatment Prioritisation Method of Plain Concrete Pavements* (TN143)
- *Concrete Pavement Base RMS QA Specification R83* (RMS R83)
- *Continuously Reinforced Concrete Pavement – CRCP, Standard Drawings – Pavement* (RMS MC)
- *Jointed Reinforced Concrete Pavement* – JRCP, Standard Drawings – Pavement (RMS MJ), and
- *Plain Concrete Pavement* – PCP, Standard Drawings – Pavement (RMS MP).

For any treatment of an existing rigid pavement, it is essential:

- slab and defect mapping be completed
- defective joint seals are repaired and sealed, and
- new joints in concrete be primed to ensure good adhesion.

It is essential joint seals in concrete pavements are maintained in an effective condition to preserve their function of preventing entry of silt, grit, stones and water.

4.8.1 Slab undersealing (grouting of voids)

The work consists of stabilising and undersealing concrete pavement slabs by drilling holes through the concrete base and pumping a grout mixture into voids beneath the slabs. It has also been used to correct dips successfully in relieving slabs in many locations. The department undertook trials of this treatment on Ipswich Road.

Currently, the department does not have any specifications or standards for rehabilitation treatments of concrete pavements.

4.8.1.1 Functions

The purpose of grout injection is to stabilise the slab by filling the existing voids below it with grout. Some treatments may also restore surface levels.

4.8.1.2 Appropriate uses

Cement grout undersealing is applied typically in areas where the support of the slab is compromised. It should be performed as soon as loss of support is noted. The four typical causes of the loss of support are pumping, consolidation of the base, subgrade failure and settlement.

Slab undersealing reduces deflections, particularly differential deflections at joints, reducing future pumping, slab cracking and reflection cracking through asphalt overlays.

For cement grout stabilisation to be an effective, long-term solution, it should be followed, as soon as practical, by an asphalt overlay to reduce impact loads. Alternatively, the raised edges of the existing slab joints could be ground back, following cement grout stabilisation, to reduce these impact loads. This is only an economic solution if the existing slabs are in good condition (for example, uncracked, adequate skid resistance) and it is not intended to resurface them.

Cement grout stabilisation is usually effective in improving load-transfer capacity of joints and reducing deflection levels. This can be confirmed by deflection testing.

4.8.1.3 Inappropriate uses

To be effective, undersealing should be performed before the voids become so deep or large, they cause pavement failure. Grout injection corrects depressions caused by voids but does not correct other deficiencies, increase the design structural capacity or eliminate existing faulting caused by moisture and temperature variations in the slab or traffic loading on the slab.

Where the loss of support is extensive, other options like cracking and seating need to be explored.

4.8.1.4 Materials

Typically, proprietary products are used. These products, and how they are applied, should comply with the suppliers and/or manufacturer’s requirements, except where these conflict with the departmental specifications.

4.8.1.5 Design considerations

The grout to be used should have:

- high early strength, particularly if the pavement will be subjected to trafficking within several hours of the initial set
- no shrinkage, as this will ensure the voids remain filled after the grout has set, and/or
- similar workability to a standard grout mix used by the contractor, so the contractor's equipment can handle the design grout mix.

4.8.1.6 Construction considerations

The first step in slab undersealing is locating the existence and position of voids under the concrete slabs. These are found by deflection testing, GPR survey and analysis, evidence of visual pumping and/or a coring program. The presence of slab rocking or dips over culverts can also indicate the presence of voids.

The actual process of pumping grout varies greatly. Close attention is required during the undersealing process, and care must be taken to ensure the slab is not raised above the surrounding surface heights or design geometry.

Selecting the hole pattern and depth for effective distribution of grout is not easy to do in advance and should be done in consultation with the contractor. It may be necessary to experiment onsite and arrive at a hole pattern optimising the undersealing before construction begins in earnest.

Undersealing must always be followed by joint sealing to prevent surface water or debris entering the pavement.

4.8.1.7 Expected performance or comments

Fluid grout is the most suitable for filling smaller voids under the slab. When applied under pressure, care is needed to ensure any public utility plant conduits, stormwater drainage pipes, and so on close to the slab are not damaged or filled as well. The consistency of grout used to fill small voids must be controlled carefully because, if it is too stiff, it may not penetrate the voids fully or may necessitate the use of excessively high pressure.

4.8.1.8 Further reading

Relevant publications include those given in Section 4.8 and:

- *A manual for the maintenance and repair of concrete roads* (Mildenhall, 1986)
- *Instructional Text C: Techniques for pavement rehabilitation: a training course* (NHI, 1993), and
- *Pavement Rehabilitation Manual* (FHWA, 1985).

4.8.2 Crack and seat with asphalt overlay

The method of cracking and seating concrete pavements is the process of cracking the pavement into approximately 0.5–1m square pieces in a controlled manner. The cracked slabs are then rolled to 'seat' them firmly on the underlying pavement layer. An asphalt overlay is applied over the cracked and seated slabs. This method of rehabilitation was used to treat sections of the Ipswich Road between Sandy Creek to Rudd Street.

A concrete pavement should be maintained through other treatments (for example, concrete patching, joint repair, undersealing, grinding) if this remains economically feasible; however, when deterioration and cost reach the point where this is not practicable, then cracking and seating becomes a viable alternative.

Currently, the department does not have a general specification for cracking and seating of concrete pavements.

4.8.2.1 Functions

The intent of cracking and seating is to create concrete pieces small enough to reduce horizontal slab movement so thermal stresses contributing to reflective cracking are reduced, yet still large enough (and still have some aggregate interlock between pieces) to retain the majority of the concrete pavement's original structural strength. Seating of the broken slabs after cracking is intended to re-establish support between the base and the slab (for example, where voids existed).

4.8.2.2 Appropriate uses

The use of cracking and seating is appropriate where a large area of concrete pavement is to be treated, other treatments are not economically feasible and one or more of the following exist:

- evident rocking caused by the presence of voids
- slab deterioration caused by reactive aggregates
- uneven slab settlement
- corner breaks
- faulted transverse joints
- faulted, mid-slab transverse cracks
- longitudinal cracks, and
- separated longitudinal joints.

The severity of deterioration is the criterion used to decide whether cracking and seating is needed. In making such a decision, consider:

- the proportion and amount of cracked slab, rocking slabs and other distressed slabs and how contiguous they are
- the effectiveness of load transfer at joints
- the condition of joints
- the support offered by the subgrade, and/or
- the proportion and number of slabs adversely affected by the presence of voids and pumping and how contiguous they are.

The crack and seat method is a good treatment for a concrete pavement with a substantial number of faulted cracks, a significant loss of load transfer with associated faulting, shearing of longitudinal tie bars or any combination of these distress manifestations.

4.8.2.3 Inappropriate uses

The crack and seat method of rehabilitation is not recommended for use on continuously reinforced concrete pavements.

The cracking and seating method is not to be undertaken where it is likely to cause damage to culverts, stormwater pipes, public utility plant or ducts.

Cracking concrete pavement reduces the structural strength of the concrete slab. The degree to which this strength is reduced will vary from project to project, depending on:

- characteristics of original pavement
- the size of the cracked pavement pieces
- the construction of equipment and methods used, and/or
- the existing subgrade or base support.

Unless the base is firm, there is a risk vertical movement between fragments will occur, leading to further pavement distress (for example, cracking of the asphalt overlay).

Concrete pavements should not just be cracked and seated. Cracking and seating must always be followed with an asphalt overlay.

Concrete overlays are not recommended.

4.8.2.4 Materials

There are no specific material requirements for the cracking and seating method itself. Any overlay must comply with relevant departmental specifications, including:

- MRTS17
- MRTS18, and
- MRS30 and MRTS30.

4.8.2.5 Design considerations

The crack pattern and size of pieces are the two most important parameters to be chosen. Continuous longitudinal cracks tend to reflect through asphalt overlays; it is advantageous to avoid creating such cracks. Normally, the concrete pavement is broken into 0.5–1m square pieces. The smaller the cracked pieces, the more the potential for reflective cracking and the greater the reduction in the structural strength of the pavement.

A detailed deflection survey, using either FWD or HWD testing apparatus, is recommended to assist with the assessment of the pavement and design of the treatment.

4.8.2.6 Construction considerations

The cracked and seated pavement should not be subjected to general traffic. Similarly, trafficking of intermediate asphalt layers by general traffic should be avoided. Trafficking of intermediate asphalt layers by construction traffic should be minimised.

The following aspects of construction should be considered.

- The drop height and spacing of the pavement breaker / hammer can be varied to obtain the desired crack pattern.
- Cores should be taken at selected locations over a crack to verify a full-depth cracking is taking place.
- A light spray of water should be applied to the lot after cracking to highlight the crack pattern.
- Certain precautions must be taken when cracking through an existing asphalt overlay; for example, strips of the asphalt overlay should be removed at regular intervals to verify the underlying concrete is cracked properly. These test strips should be located throughout the entire job, as pavement condition and overlay thickness may vary and cause the crack pattern to vary.
- Specialised plant is required. This may be difficult or time-consuming to procure. This needs to be considered well in advance of the commencement of construction.
- The pattern of cracks produced on the pavement can depend on the type of equipment used. Examples of typical cracking equipment are a pile driver with modified shoe, a guillotine hammer and a whip hammer.

- Many pavement problems can be attributed to insufficient attention to or deficient drainage. Rectifying any drainage problems or deficiencies prior to construction is essential if the treatment is to be effective.
- Failed areas and punch-throughs should be replaced with full-depth asphalt after completion of the rolling operations.
- The common device for seating is a heavy pneumatic-tyred roller, normally 35–50 tonnes. Steel-drum rollers and vibratory rollers tend to bridge over the cracked pieces, and so are not satisfactory for seating purposes.
- Any level differences arising from the seating process can be corrected (for example, by grinding).
- Consider the effect the treatment will have on road furniture, given the surface heights will increase.

4.8.2.7 Expected performance or comments

Improved performance can be expected if a SAMI is placed on top of the asphalt corrector layer.

The only apparent disadvantage in this method of cracking and seating slabs is the dedicated purpose-built plant to 'crack and not break up' the existing concrete slabs is not widely available in Australia, and, may have to be imported or developed locally. When cracking concrete slab sections of the inbound lanes of the Ipswich Motorway between Sandy Creek to Rudd Street, a relatively simple piece of equipment was developed using a diesel pile-hammer; the use of purpose-built plant is recommended.

4.8.2.8 Further reading

Relevant publications include those given in Section 4.8 and:

- *Cracking and Seating of PCC Pavements Prior to Overlaying with Hot Mix Asphalt* (NAPA, 1987)
- *Does PCC cracking and seating work* (NAPA, 1986)
- *Cracking and Seating of Jointed Portland Cement Concrete Pavements in Michigan* (Welke, 1984), and
- *Asphalt Overlays on Cracked and Seated Concrete Pavements* (NAPA, 1982).

Approved proprietary products may alternatively be used.

4.8.3 Full-depth concrete patching

The treatment relates to full and half slab replacement. It does not relate to pothole repairs.

Patching involves the careful removal and replacement of the full depth of distressed concrete pavement. It includes full-depth saw-cuts across the slab and along any longitudinal or transverse joints bounding the repair, and concreting. These full-depth repairs are small bays equivalent to the main slab in all respects. Irrespective of whether the main slab is reinforced or not, it is advisable to reinforce the repair.

Currently the department does not have a general specification or standard for concrete patching.

4.8.3.1 Functions

Full-depth concrete repairs replace failed areas (for example, where there is poor load transfer at a joint or crack). The repaired area should have adequate load transfer between new and existing slabs and minimise vertical movement at joints and cracks.

4.8.3.2 Appropriate uses

Jointed concrete pavements typically require far more patching at slab joints than at other points within slabs. The following distress types occurring at or near transverse joints may require full-depth patching, depending on severity of the distress.

- Blowup: Localised upward movement of the slab edges caused by thermal expansion.
- Corner break: Full-depth diagonal crack, usually beginning at the slab edge and intersecting the transverse joint, caused by loss of edge support beneath the slab and heavy traffic loading.
- Joint-load transfer-system associated deterioration: This is caused by inadequate dowel design, misalignment and/or corrosion or enlargement of the dowel socket under heavy traffic.
- Patch / adjacent slab deterioration: Cracking of existing patching and/or adjacent pavement caused by inadequate load transfer.
- Concrete spalling.

4.8.3.3 Inappropriate uses

Full-depth patching alone is inappropriate when the following conditions exist:

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

4.8.3.4 Materials

Relevant departmental specifications include:

- Specification (Measurement) MRS39 *Lean Mix Concrete Sub-base for Pavements* (MRS39) and MRTS39
- Specification (Measurement) MRS40 *Concrete Pavement Base* (MRS40) and MRTS40
- Technical Specification MRTS42 *Supply of Wax Emulsion Curing Compound for Concrete* (MRTS42)
- Specification (Measurement) MRS70 *Concrete* (MRS70) and Technical Specification MRTS70 *Concrete* (MRTS70), and
- Specification (Measurement) MRS71 *Reinforcing Steel* (MRS71) and Technical Specification MRTS71 *Reinforcing Steel* (MRTS71).

Alternatively, approved proprietary products may be used.

4.8.3.5 Design considerations

The designer must determine, based on the severity and causes of the observed distresses, whether full-depth patching, or an alternative rehabilitation treatment, is the most appropriate solution for a specific project.

A critical issue to be resolved is the concrete mix design to be used. In some cases, a special and/or proprietary mix will need to be used. One reason such mixes may be required is the need for early trafficking of the patch. Advice should be obtained from relevant specialists and the concrete supplier.

Load transfer at the transverse joint of concrete pavements is the most critical design feature affecting the performance of the full-depth patch. Adequate load transfer is achieved by the proper design and construction of joints.

Two basic methods have been developed to achieve load transfer across transverse joints, namely the aggregate interlock (Figure 4.8.3.5(a)) and dowelled joint (Figure 4.8.3.5(b)) systems. The aggregate interlock method has proven to be particularly unreliable in heavy traffic situations with spalling, corner breaks, and so on common. In addition, partial-depth sawing is required as part of the system to induce a rough-faced joint. Frequently, this results in damage to the adjacent slab.

The dowel-bar load-transfer system is preferred for its superior performance. Adequate load transfer is achieved by installing dowel bars of sufficient size and number properly. The system provides the necessary load-transfer across the joints and allows full-depth sawing, which minimises damage to the adjacent slabs.

Figure 4.8.3.5(a) – Aggregate interlock (rough-faced) joint

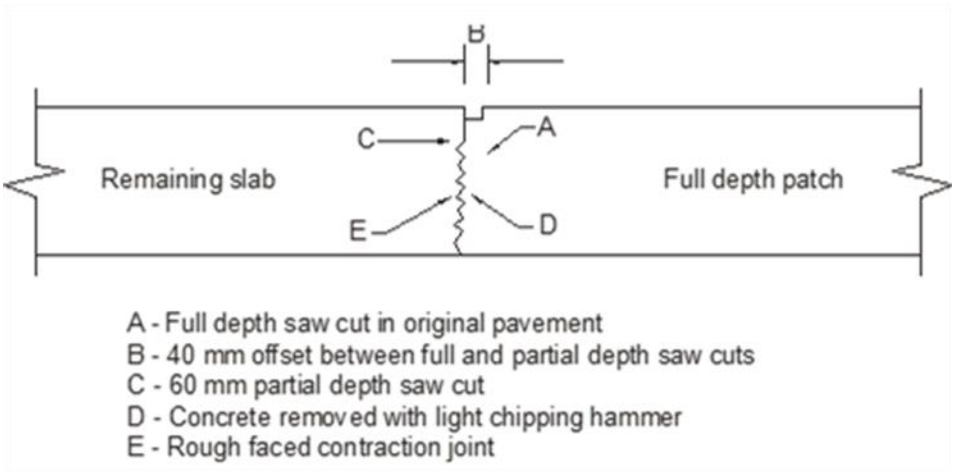
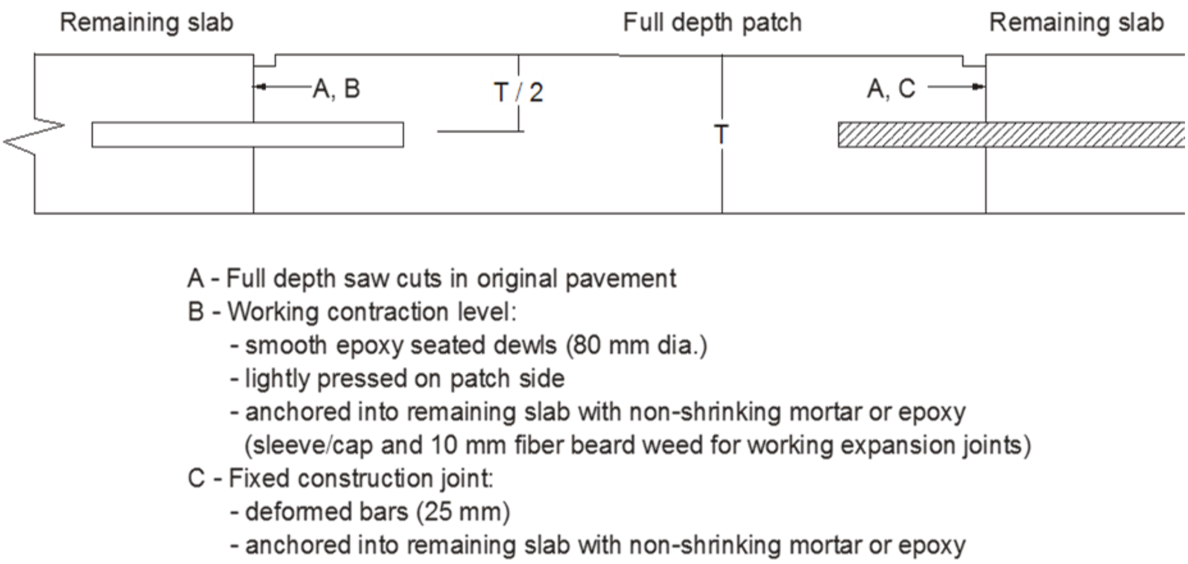


Figure 4.8.3.5(b) – Dowelled (working and fixed) joints



4.8.3.6 Construction considerations

To obtain good performance of full-depth concrete repairs, attention must be paid to:

- identifying the mode(s) of failure properly
- locating patch boundaries properly
- selection of minimum patch dimensions
- ensuring the depth of all saw-cuts is equal to the thickness of the concrete
- optimisation of methods for concrete removal, placement and curing, and
- sealing joints properly.

Identifying what the actual locations and boundaries of full-depth patches should be, and any alternate site-specific rehabilitation techniques available, are perhaps the most difficult aspects of (design and) construction. Even when detailed plans have been prepared, the project should be resurveyed just prior to construction to identify any changes.

Some other major considerations regarding full-depth patching are:

- confirmation is required of whether full-depth patching or an alternative rehabilitation treatment is the most appropriate treatment for the specific location
- boundaries of patches falling within 2m of existing transverse cracks or undowelled joints, or within 1m of existing dowel joints, should be extended to include the crack or joint – recommended to minimise the chances of rocking and pumping, and ultimately cracking, of the patch or adjacent slab
- a boundary falling at an existing dowelled transverse joint should extend at least 300mm into the adjacent slab, so the existing joint is included; attempts at salvaging the existing dowel system frequently result in damage to the dowel bars and adjacent concrete slab
- on multi-lane highways, the condition of adjacent lanes should be reviewed concurrently with the lane being marked for repair; if distressed areas in adjacent lanes are similar to those in the lane to be repaired, it is desirable to align patch boundaries in the transverse direction wherever possible to avoid small offsets and maintain continuity
- saw cutting during hot weather sometimes results either in cracking ahead of the saw-cut or binding of the saw blade; restricting saw cutting to early morning hours when the slabs have not fully expanded may avoid these problems, and/or
- boundaries of patches must be selected so all deteriorated pavements or materials are removed, including underlying ones. Saw-cut locations should be marked at least 300–400mm away from the edge of the observed distress(es). Once concrete removal has commenced, the vertical faces should be checked for any deterioration. The size of the patch may then be increased if required.

4.8.3.7 Expected performance or comments

In addition to full-depth concrete patching, there are a number of repair techniques intended to restore the rigid pavement to as close as possible to its original condition.

These include:

- for surface defects:
  - diamond grinding
  - thick asphalt overlay
  - concrete overlay
  - transverse grooving
  - mechanical roughening
  - thin-bonded surface repairs, and/or
  - shrinkage-crack sealing

- for structural repairs:
  - crack sealing for hairline to medium-width cracks
  - cross-stitching of longitudinal cracks
  - bay-replacement repair for wide cracks
  - slab undersealing (Section 4.8.1), and/or
  - cracking and seating (Section 4.8.2), and
- for joint repairs:
  - spall repair
  - slab stabilisation
  - diamond grinding
  - joint and crack cleaning and sealing, and/or
  - joint rehabilitation.

4.8.3.8 Further reading

Relevant publications include those given in Section 4.8 and:

- AGPT04C
- Mildenhall, 1986, and
- FHWA, 1985.

4.8.4 Joint sealing and joint repairs

It is essential joints are maintained so the entry of silt, grit, stones, water, and so on is prevented.

Reference should be made to AGPT05 for guidance about joint sealing and joint repairs in rigid pavements.

4.8.4.1 Joint sealing

Joints in a concrete pavement to be sealed should be cleaned thoroughly to remove any foreign or latent material. This can be accomplished using one of the following methods:

- The use of a high-pressure water blaster. This is suitable for critical areas where an extremely clean joint is needed. A major disadvantage is the joint must be allowed to air-dry for at least 24 hours or be dried using a joint heater. It may also introduce more water into the joint and/or pavement.
- The use of a compressed-air blowpipe. This is the most commonly used method.
- The use of a compressed / LPG heater blower. It is useful to use this equipment for drying joints after they have been ‘water blasted’.

4.8.4.2 Priming

New joints in concrete should be primed to ensure good adhesion.

4.9 Insitu stabilisation or modification of granular materials and soils

Selecting the appropriate stabilising or modifying agent is a decision based largely on the material to be stabilised or modified. In this respect, Table 4.9 is a useful guide for selecting an appropriate stabilising or modifying agent; however, it is indicative only and assessment of the site conditions, existing materials and testing of a range of stabilising or modifying additives should be completed to determine the most appropriate treatment in any one case. Laboratory testing is essential for assessing the amenability of a specific material to this process.

Table 4.9 – Guide indicating suitability of stabilising agents for use with different soils

Stabilising agent (or primary stabilising agent)	Material with particle size distribution with:					
	More than 25% passing the 75µm sieve and:			Less than 25% passing the 75µm sieve and:		
	PI ≤10	10<PI<20	PI≥20	PI ≤6 and PI x% passing 75µm ≤60	PI ≤10	PI ≥10
Cement and cementitious blends	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 suitable	Usually suitable	Usually unsuitable
Bitumen and cement blends	Usually suitable	Doubtful	Usually unsuitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually unsuitable	Usually unsuitable	Usually suitable	Usually suitable	Doubtful
*Polymers	Usually suitable	Usually suitable	Usually unsuitable	Usually suitable	Usually suitable	Usually unsuitable
*Miscellaneous chemicals		Usually suitable	Usually suitable		Doubtful	Usually suitable

Source: AGPT05 Table 5.3

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

Note: The above forms of stabilisation may be used in combination (for example, lime stabilisation to dry out materials and reduce their plasticity, making them suitable for other methods of stabilisation).

Insitu stabilisation or modification is only practical where the existing pavement is granular with a thin asphalt or bituminous seal surfacing. In the case of the former, it is feasible to remove the asphalt prior to modification or stabilisation, whereas in the case of the latter, provided the total thickness of bituminous seals is thin, it is often possible to include the seal(s) in the modified / stabilised layer / material. Alternatively, it can be removed, and this is preferred.

Insitu modification or stabilisation is undertaken by purpose-built plant (that is, stabiliser or reclaimer / stabiliser). The most common type of stabilisers and stabiliser / reclaimers have a single mixing rotor mounted either between the front and rear wheels or behind the rear wheels. Pulverisation and/or mixing runs undertaken by the same type of plant may be required prior to the addition of modifying or stabilising agent(s).

Most insitu modifying or stabilising equipment has some difficulty working hard up against some road structures and furniture (for example, safety barriers, kerbs and manholes). This needs to be considered when planning construction. This can be overcome by using a grader, which should be onsite in any case for trimming / scarifying work.

The department has specifications for insitu stabilisation using lime, cement, cementitious binders and foamed bitumen. These are:

- Specification (Measurement) MRS07A *Insitu Stabilised Subgrades using Quicklime or Hydrated Lime* (MRS07A) and Technical Specification MRTS07A *Insitu Stabilised Subgrades using Quicklime or Hydrated Lime* (MRTS07A)
- Specification (Measurement) MRS07B *Insitu Stabilised Pavements using Cement or Cementitious Blends* (MRS07B) and Technical Specification MRTS07B *Insitu Stabilised Pavements using Cement or Cementitious Blends* (MRTS07B)
- Specification (Measurement) MRS07C *Insitu Stabilised Pavements using Foamed Bitumen* (MRS07C) and Technical Specification MRTS07C *Insitu Stabilised Pavements using Foamed Bitumen* (MRTS07C)
- *Guide to Pavement Technology Part 4D: Stabilised Materials* (AGPR04D)
- *Guide to Pavement Technology Part 4L: Stabilising Binder* (AGPT04L), and
- AGPT05.

4.9.1 Appropriate uses – general

The cases when it is appropriate to modify or stabilise granular materials and soils include:

- where the materials to be stabilised are suitable and the dosage rate required is economical
- for existing pavements with a thin asphalt surfacing or where the total thickness of bituminous seals is small
- for existing pavements with an adequate granular / soil thickness for stabilisation, or where it is practical to import and place suitable material to obtain an adequate granular / soil thickness for stabilisation
- in old or unsealed shoulders, provided any pavement drainage issues are addressed
- where the stabilised pavement is to receive a bituminous seal or asphalt surface
- where the stabilised pavement is to receive an asphalt overlay, and/or
- where traffic volumes are not high and traffic management restrictions make its use practical; for example, alternating one-way traffic flow is permitted on a rural two-way, two-lane road.

4.9.2 Inappropriate uses – general

The cases when it is inappropriate to modify or stabilise granular materials and soils include:

- where the materials to be stabilised are unsuitable or the dosage rate required is uneconomical
- where the support for the stabilised or modified layer is weak (for example, stabilised base layer lying directly on a weak subgrade)
- where weak or saturated subgrades (for example, caused by leaking mains or long-term ingress of subsurface moisture) exist; these need to be treated prior to the stabilisation of materials above the subgrade
- where there are problems in the pavement underneath the proposed stabilised layer: stabilisation of the upper layer(s) of the pavement will not solve problems occurring lower down in the road formation (for example, may not overcome problems of a weak or saturated subgrade, especially if the stabilised layer rests directly on such a subgrade)
- for existing pavements with a thick asphalt surfacing or where the total thickness of bituminous seals is not small
- for existing pavements with an inadequate granular / soil thickness for stabilisation, or where it is impractical to import and place suitable material to obtain an adequate granular / soil thickness for stabilisation
- the presence of public utility plant may make stabilisation impractical, cause delays or lead to abandonment of work
- where traffic volumes are high and traffic management restrictions make its use impractical; for example, where one-way traffic flow is not permitted on an urban two-way, two-lane road
- where its use blocks pavement drainage; for example, its use, without longitudinal pavement drains along the joint, to widen an existing granular pavement that remains untreated
- where application of the binder or construction will cause unacceptable nuisance, adverse environmental or other adverse effects (for example, binder dust floats into nearby houses, vibration from heavy compaction equipment causes damage to nearby buildings, modifying / stabilising agent leaches from stabilised layer and enters surface or ground waters and causes problems). This means insitu stabilisation is undertaken in rural areas rather than urban ones.

It may be inappropriate to restabilise a pavement already stabilised using any type of binder or where early failures have occurred. The department has not undertaken any large-scale works including restabilisation using any type of binder.

It may also be inappropriate when 'heavy clays' or subgrades will be incorporated into the modified or stabilised layer.

4.9.3 Construction considerations

In urban areas, it is usually not practical to stabilise to a depth of more than 200mm as compaction of thicker layers can damage structures, plant, and so on.

4.9.4 Expected performance or comments

Not all materials may be suitable for stabilisation. To achieve good results from the insitu stabilisation process, a number of things must be done.

- Undertake a site investigation: This will establish the forms and causes of pavement distress, the thickness of pavement layers and the composition of pavement materials. Samples for testing can also be taken.
- Laboratory testing and analysis: In the absence of an inventory of successful regional experience with particular materials, laboratory testing is required to evaluate whether existing materials are suitable for stabilisation. Testing is also needed to determine the mix design.
- Pavement rehabilitation design: The ability of the stabilised material to meet the required design criteria should be checked.
- Construction: The success of insitu stabilisation depends on the use of adequate construction procedures producing a stabilised layer meeting the intent / requirements of the design. This includes the use of purpose-built equipment, proper use of equipment and good construction practice.

4.9.5 Granular (mechanical) stabilisation

Improvement of the existing pavement material by blending it with one or more other granular materials or soils (sands, quarry products, clay, and so on) is referred to as granular or mechanical stabilisation. The particle-size distribution of the end product will be different to the existing material. The plasticity may also be different.

Granular stabilisation may require:

- removal of some material prior to stabilisation, and/or
- the addition of some extra suitable material prior to stabilisation (for example, add granular material where the thickness of the existing granular material is insufficient).

4.9.5.1 Functions

By producing an end product with a suitable particle-size distribution and/or with suitable plastic properties, mechanical stabilisation can improve the pavement material's strength, 'compactability', impermeability or abrasive resistance markedly.

4.9.5.2 Appropriate uses

Some of the conditions requiring mechanical stabilisation to be used in rehabilitation works are:

- segregation of aggregate in the original construction, causing a lack of uniformity in load-bearing capacity
- deficiencies in quality or properties of insitu material(s) (poor particle-size distribution, plasticity, and so on)
- increasing traffic requires higher strength materials, and/or
- if the existing construction is a gravel road maintained by the yearly addition of aggregates, a blend of the remaining aggregate and the underlying soil may produce a sufficiently stable base for a wearing surface.

Except for some selected differences, the materials requiring mechanical stabilisation are usually similar to those normally used to construct granular pavement bases and sub-bases.

If mechanical stabilisation is impractical, or the improvement in properties to be achieved is insufficient, it is necessary to consider modification or stabilisation using either lime, cementitious or other modifying / stabilising agents. This is usually cheaper than importing higher-grade granular material from another area.

Reference must also be made to Section 4.9.

4.9.5.3 Inappropriate uses

Mechanical stabilisation is inappropriate if it is uneconomical, or if the combined material mixture is shown not to possess the required qualities or properties.

Reference must also be made to Section 4.9.2.

4.9.5.4 Materials

Typical materials used include natural gravels, silty sands, sand clays, silty clays, crusher-run quarry products, waste quarry products, dune-and-river deposited sands.

While the department has no specifications for mechanical stabilisation, some relevant departmental documents include:

- MRS05 and MRTS05
- WQ33 *Material Sources in Western Queensland*, and
- WQ35 *Paving Materials and Type Cross-Sections for Roads on Expansive Soils in Western Queensland*.

The Western Queensland documents can be found under *Western Queensland Best Practice Guidelines* (WQBPG).

4.9.5.5 Design considerations

The design of a mix should produce something to satisfy MRS05 and MRTS05.

Preliminary mix design will be based on the particle-size distribution and plastic properties. Strength tests should be carried out to verify the required improvement has been achieved. When unconventional materials are used, more detailed testing and investigation will often be needed and may include modification of the accepted design or specification criteria.

AGPT04D provides further guidance about the selection of materials and mix design. The selection of suitable criteria should consider local experience, especially relating to the performance of local materials. When considering local experiences, care must be taken to ensure factors such as traffic and drainage are comparable.

4.9.5.6 Construction considerations

The spreading of materials for mechanical stabilisation is normally done by a grader or self-propelled spreader to a given depth over the full width to be stabilised. The use of box or mechanical spreaders usually provides effective control over depth and width. Alternatively, the use of windrows of pre-determined and uniform cross-section may assist in accurate metering of materials.

4.9.5.7 Expected performance or comments

In addition to carrying out adequate investigation and design, good construction and control-testing techniques are essential for a satisfactory road pavement. This involves careful proportioning and a thorough mixing of the constituent materials to produce a uniform (not segregated) end product which can be compacted and finished in accordance with the specification.

4.9.5.8 Further reading

Relevant publications include:

- AGPT04L
- AGPT04D
- Wirtgen CRM, and
- *Principles and Application of Soil Stabilisation* (Metcalf, 1981).

4.9.6 Stabilisation and modification using cement or cementitious binders

The treatment is to recycle all or part of the existing material by changing its properties using cement or cementitious binders. This may require:

- removal of unsuitable material prior to stabilisation (for example, asphalt surfacing, patches), and/or
- the addition of suitable material prior to overlay (for example, add granular material where the thickness of the existing granular material is insufficient).

It may be used in conjunction with an asphalt overlay or overlaid in the future as traffic loads increase.

Many urban and rural road-recycling projects involve granular pavements with a bituminous seal surfacing. In these cases, the existing pavement materials and bituminous surfacing are usually scarified, pulverised and mixed insitu with cement or cementitious binders (modified / stabilised). After mixing has been completed, the modified / stabilised material must be compacted and trimmed to profile quickly and thoroughly. The completed layer must be cured immediately and properly. A SAM, SAMI or asphalt surfacing is then placed on the completed layer. Alternatively, a SAMI followed by an asphalt overlay can be constructed.

Cement and cementitious stabilising agents can be used to produce a ‘modified’ material (with a 7-day curing UCS of 1.0–2 MPa with a target of 1.5 MPa) or ‘bound’ material (with a UCS of 2 MPa or more). The amount of modifying / stabilising agent added determines which is produced. The dosage for modified materials typically ranges from 1–2% of the dry mass of the untreated material. Further information and requirements for modified materials are given in Section 4.9.6.5.

To produce more highly-bound material, the dosage is typically from 3–6% by weight of untreated material (the binder content is determined through a mix design process including laboratory testing of the existing materials and consideration of experience with the same or similar materials in particular localities).

4.9.6.1 Functions

Insitu stabilisation using cement or cementitious binders offers considerable benefits in many instances. These include:

- the reuse of existing pavement materials
- the option of constructing under traffic where traffic volumes are not high and traffic management restrictions make this possible
- high rates of output are usually achieved
- improvement in the structural capacity of the pavement for stabilised materials and, in some cases, for modified materials; for example, a stiff stabilised pavement layer providing support to an asphalt surfacing leading to a relatively long asphalt fatigue life, and/or
- reduction in the moisture sensitivity of the pavement (by virtue of the modified or stabilised layer).

Modification results in a material with improved properties when compared to untreated material; for example, its strength is higher, it has a lower plasticity, it has a higher shear strength and it has higher resistance to penetration. It is not as likely as a bound material to crack but the improvement in structural capacity is not as great.

A bound material provides the greatest increase in structural capacity but is more likely to crack.

4.9.6.2 Appropriate uses

Appropriate uses for stabilisation using cement or cementitious binders include:

- where high-quality granular materials are not readily or economically available, and/or
- for treatments including a SAM or SAMI over the modified or stabilised layer, particularly if it is stabilised.

In most situations where the existing granular material (gravel / crushed-rock material) has degraded because of long-term weathering, and the surfacing is a bituminous seal, the existing surfacing is likely to contain the highest-quality aggregates in the whole pavement structure. Every effort should be made to retain the surfacing and incorporate it into the recycled pavement. Unless the total thickness of bituminous seals is thick, there is no reason to remove the bituminous surfacing to waste; however, one or more pulverisation passes may be required.

Some granular materials non-compliant with MRTS05 can be modified by adding cement or cementitious binders to a point where they are acceptable for use as a sub-base or base course material under MRTS05.

Reference should also be made to Section 4.9.1.

4.9.6.3 Inappropriate uses

Inappropriate uses for stabilisation using cement or cementitious binders include:

- where maintenance budgets or practices are such, cracks will not be filled and sealed in an effective and prompt manner
- where unacceptably high levels of sulphates are present
- for treatments excluding a SAM or SAMI over the modified or stabilised layer, particularly if it is stabilised, and/or
- for treatments excluding a minimum of 175mm of DGA (total thickness) or SAMI / SAM over the stabilised layer if asphalt lies directly on the stabilised layer.

Stabilisation may also be inappropriate when subgrade materials will be incorporated into the modified or stabilised layer.

In some cases, a treatment including a layer of material modified or stabilised using a cementitious binder but without at least 175mm of DGA (total thickness) may be considered; however, if adopted, it must be recognised there may be an increased risk of cracking, and higher maintenance costs and frequencies are likely. Provision must be made for increased monitoring, risks and maintenance frequencies if such treatments are adopted (for example, include additional costs in future maintenance budgets and programs).

Reference should also be made to Section 4.9.2.

4.9.6.4 Materials

Definitions are contained in Chapter 1 of this *Manual*.

Relevant departmental specifications include:

- MRS05 and MRTS05, and
- MRS07B and MRTS07B

Cement or cementitious binders used for stabilising granular pavement materials include:

- type GP (general purpose) cement
- type LH (low heat) cement
- type GB (general blend) cement
- blends with 30% of Portland cement and one or more of ‘fine grade’ fly ash, ground granulated blast furnace slag or hydrated lime, and
- blends of hydrated lime and fly ash.

Whatever the binder, it is important to undertake laboratory testing to determine the mix design and resultant properties.

4.9.6.5 Requirements for modified materials, including testing and design methodology

Cementitiously modified materials shall be granular materials modified with:

- cementitious additives (for example, GP cement, GB cement, cement / slag blend, and so on), or
- slow setting additives (for example, lime / fly ash blend, lime).

Further modified materials shall have a UCS between 1.0–2 MPa with a target of 1.5 MPa when tested as follows:

- testing is conducted in accordance with *Test Method Q115: Unconfined compressive strength of stabilised materials* (Q115)
- for cementitiously modified material, 7-day UCS is used
- for material modified with slow-setting additive, 28-day UCS is used
- samples are air-cured at 230°C ± 20°C and at least 95% relative humidity
- samples are prepared at 100% of standard compaction, and/or
- samples are prepared at OMC.

When modified materials are used, the following requirements shall be met:

- the minimum thickness and strength of unbound granular pavement material specified in Table 4.9.6.5 shall be provided under the modified layer
- there shall be only one modified layer
- the thickness of the modified layer shall be between 200–300mm
- must complete direct measurement of the characteristic modulus as per AGPT02, and
- multiple layers of modified material shall not be constructed.

Table 4.9.6.5 – Minimum thickness and strength of unbound granular pavement material required below the modified layer

Minimum thickness of unbound granular material required below the modified layer (mm) (see note)	Minimum CBR strength of unbound granular material required below the modified layer (%)	Design subgrade strength CBR (%)
300	35	≥3–<5
150		≥5–<15
0		≥15

Note: Where there is a subgrade with a design CBR <3%, a capping layer is required in accordance with the PDS.

4.9.6.5.1 Testing

Representative samples of the materials proposed to be modified shall be tested to determine their suitability for modification and the mix design if they are suitable. This includes checking the target UCS can be achieved. Where the materials proposed to be modified are not suitable, modification shall not be used.

4.9.6.5.2 Characterisation for design

Modified materials are considered to behave as unbound granular materials with improved stiffness. For the purposes of mechanistic design, they shall be modelled with the following properties:

- be cross-anisotropic (Degree of Anisotropy of 2)
- have a Poisson's Ratio of 0.35
- be sublayered, and
- have a maximum potential modulus less than the characteristic modulus determined by direct measurement as per AGPT02 and 600 MPa.

4.9.6.5.3 Design

The steps for design are best illustrated through an example as shown in the steps following.

Step 1 – Develop a trial design

The first step is to develop a trial design by selecting a thickness for the modified layer of between 200–300mm. An example is shown in Table 4.9.6.5.3(a).

Table 4.9.6.5.3(a) – Trial pavement for the design example for pavement with modified material

Material type	Thickness (mm)
Sprayed seal surfacing	Ignored in GMP
Modified material	250mm (for this example)
Minimum thickness of unbound granular pavement material required below the modified layer	Refer Table 4.7; for this example, 150mm is chosen based on the design subgrade CBR of 7%
Design subgrade CBR = 7% (for this example)	Semi-infinite

The GMP (refer Chapter 5) is then applied to determine the allowable traffic loading, using steps 2–7.

Step 2 – Characterisation of the subgrade

For this example, the subgrade for this example has design CBR of 7% and is modelled as having:

- a vertical elastic modulus (E_v) of 70 MPa
- a horizontal elastic modulus (E_H) of 35 MPa
- a Poisson's Ratio of 0.45, and
- a shear modulus of 48.3 MPa.

Step 3 – Characterisation of the top layer of the required unbound granular pavement material

The unbound granular material required below the modified layer shall be modelled and sublayered as per AGPT02. The maximum potential modulus of this material shall not exceed the values given in Table 4.9.6.5.3(b).

The maximum potential modulus of the top sublayer of the unbound granular pavement material required below the modified layer is the minimum of:

- the limiting modulus based on supporting conditions, and
- the relevant value given in Table 4.9.6.5.3(b).

The unbound granular pavement material for this example has a design CBR of 35%; for example:

- the maximum potential modulus based on supporting conditions is:

$$E_{V \cdot top \cdot granular} = E_{V \cdot subgrade} \times 2^{\left(\frac{total \cdot granular thickness}{125}\right)} = 70 \times 2^{\left(\frac{150}{125}\right)} = 161 \cdot MPa$$

- the maximum potential modulus from Table 4.9.6.5.3(b) is 150 MPa.

For this example, the unbound granular pavement material is modelled as having:

- a vertical elastic modulus for the top layer of (E_V top granular) of 150 MPa
- a horizontal elastic modulus (E_H) of 75 MPa
- a Poisson's Ratio of 0.35, and
- a shear modulus of 111 MPa.

Table 4.9.6.5.3(b) – Maximum modulus of unbound granular pavement layer / sublayers below modified layer

Laboratory CBR of the existing granular layer or sublayer (%)	Maximum vertical modulus (MPa) for the:			
	Topmost granular sublayer	Granular layer / sublayer second from the top of the granular layers	Granular layer / sublayer third from the top of the granular layers	Granular layer / sublayer fourth from the top of the granular layers
80	250	200	150	120
50	225	200	150	120
45	200	150	120	120
40	150	120	120	120
35	150	120	120	120

Note: Bound material is modified material, stabilised material and asphalt. The total bound thickness is calculated by adding the total thickness of asphalt, if any, to the total thickness of any stabilised and modified materials.

Step 4 – Characterisation of the other four sublayers of the required unbound granular pavement material

The remaining sublayers of the unbound granular pavement material required below the modified layer need to be characterised next, as per AGPT02 for this example:

- the total depth of the unbound granular pavement material is divided into five sublayers of equal thickness which equals 30mm in this case (= 150 divided by five), and
- the ratio of the moduli of adjacent layers is calculated:

$$R = \left(\frac{E_{top of \cdot granular}}{E_{top of \cdot subgrade}}\right)^{\frac{1}{5}} = \left(\frac{150}{70}\right)^{\frac{1}{5}} = 1.164$$

Table 4.9.6.5.3(c) summarises the results for this example.

Table 4.9.6.5.3(c) – Properties of granular sublayers and top of subgrade design example for pavement with modified material

Material type	Thickness (mm)	Vertical elastic modulus (MPa)	Poisson's Ratio	Shear modulus (MPa)
Granular sublayer 1 (top)	30	150	0.35	111.1
Granular sublayer 2	30	129	0.35	95.5
Granular sublayer 3	30	110	0.35	81.5
Granular sublayer 4	30	95	0.35	70.4
Granular sublayer 5 (bottom)	30	82	0.35	60.7
Subgrade (top)	Semi-infinite	70	0.45	48.3

Step 5 – Characterisation of the top sublayer of the modified layer

The maximum potential modulus of the top sublayer of the modified granular layer is assumed to be 600 MPa for this example.

The maximum potential modulus of the top sublayer of the modified granular layer is modelled as having:

- a vertical elastic modulus for the top layer of (E_v top granular) of 600 MPa
- a horizontal elastic modulus (E_H) of 300 MPa
- a Poisson's Ratio of 0.35, and
- a shear modulus of 444.4 MPa.

Step 6 – Characterisation of the other four sublayers of the modified layer

The remaining sublayers of the modified layer required below the modified layer need to be characterised next. As per AGPT02 for this example:

- the total depth of the modified layer is divided into five sublayers of equal thickness which equals 50mm in this case (= 250 divided by five), and
- the ratio of the moduli of adjacent layers is calculated:

$$R = \left(\frac{E_{top \cdot of \cdot granular}}{E_{top \cdot of \cdot subgrade}} \right)^{\frac{1}{5}} = \left(\frac{600}{150} \right)^{\frac{1}{5}} = 1.319$$

Table 4.9.6.5.3(d) summarises the results for this example.

Table 4.9.6.5.3(d) – Properties of sublayers of modified material for design example

Material type	Thickness (mm)	Vertical elastic modulus (MPa)	Poisson's Ratio	Shear modulus (MPa)
Modified material sublayer 1 (top)	50	600	0.35	444
Modified material sublayer 2	50	454	0.35	336
Modified material sublayer 3	50	344	0.35	254
Modified material sublayer 4	50	261	0.35	193
Modified material sublayer 5 (bottom)	50	198	0.35	146

Note: Automatic sublayering should not be used in CIRCLY for the modified layer.

Step 7 – Calculate the allowable traffic loading

The GMP is used to calculate the allowable loading using the appropriate traffic information and characteristics.

Step 8 – Iterate as required to arrive at the final design

If required, iterate through steps 1–7 to determine a final design (for example, by varying the thickness of the modified layer).

4.9.6.6 Design considerations

Reference must also be made to Section 4.9.6.5 which contains some design-related information and requirements for modified materials.

Typically, tests applied to cement or cementitiously stabilised materials include UCS (for determining binder content), moisture / density relationships (for controlling compaction), capillary rise testing (to assess moisture susceptibility) and PI (to assess moisture and shrinkage susceptibility).

MRTS05 requires the PI of unbound pavement materials be: ≤6 for type 2.1, 2.2 and 3.1 materials; ≤8 for type 2.3 and 3.2 materials; ≤2 for type 2.4, 3.3 and 3.4 materials; and ≤14 for type 2.5 and 3.5 materials. Usually, PIs can be reduced to these levels via cement modification.

Deflection measurements (for example, using a FWD), in conjunction with back analysis techniques, are used widely to evaluate the structural condition of pavements, including cement or cementitiously stabilised layers.

The actual thickness to be stabilised depends on the strength of the subgrade, design traffic loading and proposed construction procedure(s); however, the maximum thickness of a single insitu stabilised layer is 250–300mm. It is difficult to achieve adequate compaction for layers with a greater thickness. The design may need to account for the reduced level of compaction at the bottom of thick layers.

A stabilised bound material must be resistant to erosion. To achieve this, a minimum amount of cement or cementitious binder must be added. While not directly applicable, MRTS08 provides some guidance with a minimum of 2% by mass required for plant- mixed pavements stabilised using cement or cementitious binders.

Any prime, primer seal, seal, SAM or SAMI overlying a stabilised layer must be selected and designed carefully. It is important the type of materials used are compatible with and have an affinity for the stabilised material (for example, to bond with it adequately).

4.9.6.7 Construction considerations

Reference must be made to Section 4.9.6.5 which contains some construction-related information and requirements for modified materials.

MRTS07B applies for modification and stabilisation of clay subgrades and has many requirements including:

- preliminary pulverisation to allow identification of unsuitable materials to be removed and replaced with suitable materials prior to spreading of the cement or cementitious binder
- minimum plant requirements
- a site survey of public utility plant, drainage systems, and so on before works start; construction procedures must nominate how such items will be protected during construction
- final compaction and trimming of the stabilised or modified material completed within the allowable working time, measured from the start of mixing
- existing surface must be shaped, compacted and trimmed
- for blends incorporating hydrated lime, the actual dosage of binder applied during construction must be adjusted, based on the available lime index of the hydrated lime being used for construction and the available lime index of the hydrated lime used in laboratory testing
- the maximum spread rate for any one spreading run is 20 kg/m^2
- after each spreading run, the material must be trimmed and/or compacted
- there must be at least two mixing passes
- trimming must take place before compaction as well as after
- stabilised and modified material is to be trimmed to waste only
- construction joints require care and there are particular requirements for them
- curing must commence immediately upon completion of final trimming and compaction until the stabilised layer is sealed or covered by the next pavement layer
- there are construction constraints related to weather conditions
- a maximum time between spreading and mixing is nominated (for example, 30 minutes)
- process control for compaction can be used, provided a trial demonstrates compliance successfully with MRTS07B– a trial is required for each unique combination of materials, plant, methodologies or processes, using the same plant, processes, methodology and materials in the trial as in construction
- removal and disposal of unsuitable materials(for example, asphalt patches)
- water must be free from oil, acids, organic matter and any other matter which could be deleterious to the mixture, be potable and contain less than 0.05% of sulphates.

Pulverisation of the existing materials to be modified or stabilised is necessary to break up a bituminous seal, if it is left in place, to break up the clay clods (if they exist), reveal unsuitable materials to be removed and replaced (for example, patches) and to distribute the materials within the depth of stabilisation more evenly. Pulverisation must be well-controlled to ensure the materials immediately below the depth of stabilisation remain as undisturbed as possible and do not pollute the material to be modified or stabilised. At this stage, some minor shape correction to the pavement can be carried out either by adding (suitable) material or moving materials across the pavement.

It is essential:

- management of traffic management be considered before construction begins
- allowable working time be determined before construction begins
- modified / stabilised material receives the full compactive effort as soon as possible after final mixing
- modified / stabilised material is compacted fully and trimmed within the allowable working time
- material is trimmed to waste (for example, not reused in the stabilised or modified layer), and
- completed layer is cured (to maximise hydration and strength gain).

Slower-setting cementitious binders have longer allowable working time and provide greater flexibility during construction. This may be at the expense of other desirable properties (for example, lower strength).

4.9.6.8 Expected performance or comments

Reference should be made to Section 4.9.4.

4.9.6.9 Further reading

Relevant publications include:

- AGPT04L
- AGPT04D
- Wirtgen CRM, and
- *Recycling Pavements by Cement Stabilisation* (CCAA, 1988).

4.9.7 Insitu treatment of clay subgrades using lime

Insitu treatment of clay subgrades using lime involves mixing lime, which has been either hydrated previously or hydrated on site by adding water (slaking), into a subgrade to increase subgrade strengths. This may be done to modify the subgrade (usually achieves a temporary strength gain) or it may be added to stabilise it (achieves a permanent increase in strength). In the case of the latter, sufficient lime must be added to ensure these strength gains are permanent.

It is essential the purpose of the treatment is clearly identified and clear to all concerned.

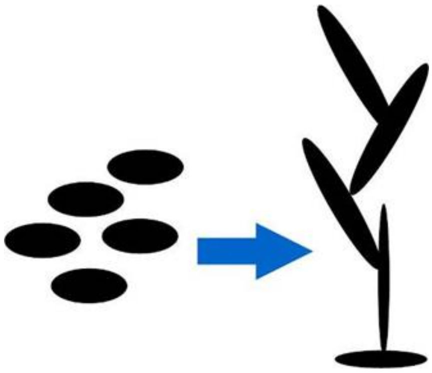
4.9.7.1 Functions

In the context of this *Manual*, a differentiation is made between lime modification and stabilisation. The following sections define the differences.

4.9.7.1.1 Modification

Modification involves the addition of relatively low amounts of lime. The calcium introduced causes an initial ionic exchange, which results in flocculation or edge-to-face reorientation of the clay plate-like particles (Figure 4.9.7.1.1).

Figure 4.9.7.1.1 – Flocculation: reorientation of clay particles



This reaction can have a dramatic effect on the soil in terms of improved workability and shear strength; indeed, modification can often be a most economical construction expedient because it permits the use of heavy construction machinery for placement and compaction of pavement layers over subgrades, which otherwise are too wet and/or too weak to support construction traffic. Notwithstanding this, it must be possible for the plant used to mix the lime to traverse it. While it may aid construction, modification can also lead to problems.

Modified subgrades can have a higher permeability and, so, may be more susceptible to moisture entry. The extent of moisture entry can be so great, the initial strength gains are reversed through leaching of calcium from the subgrade. Essentially, the strength gained from modification is most likely to be temporary.

Extensive research, conducted by McCallister and Petry in 1990 (*Property Changes in Lime Treated Expansive Clays Under Continuous Leaching* (McCallister, 1990)) demonstrated the permeability of soils increased significantly after small amounts of lime was added, then decreased at higher dosages. These effects seem to be caused by flocculation of the clay plates and the development of some pozzolanic reaction products. The resultant soil structure is an open matrix. Such effects have been confirmed by testing in Queensland and are suspected to be the main cause of some unsatisfactory historical experiences.

It is therefore recommended modification be used with caution, particularly if subgrades are likely to be exposed to water entry (for example, in low-lying areas). Successful modification has often been associated with well-drained embankments, or subgrades protected from moisture ingress by extensive subsoil-drainage systems.

4.9.7.1.2 Stabilisation

The correct use of lime stabilisation can help avoid or minimise problems associated with expansive subgrades (for example, refer figures 4.9.7.1.2(a)), 4.9.7.1.2(b) and 4.9.7.1.2(c)).

Figure 4.9.7.1.2(a) – Longitudinal cracking caused by an expansive soil subgrade

Figure 4.9.7.1.2(b) – Longitudinal cracking of a cementitiously treated base on an expansive soil embankment

Figure 4.9.7.1.2(c) – Typical shrinkage cracking of expansive soils

To achieve long-term strength gain using lime, a pozzolanic reaction between the silica and alumina in the clay minerals and the calcium hydroxide in the lime must occur. A pozzolanic reaction occurs between finely-divided siliceous or aluminous material, calcium hydroxide and water. The reaction produces calcium-silicate hydrates and calcium-aluminate hydrates: essentially, the same as the cemented products responsible for strength and durability of conventional concrete. These reactions are illustrated by equations 4.9.7.1.2(a) and 4.9.7.1.2(b).

Equation 4.9.7.1.2(a) – Pozzolanic reaction showing formation of calcium-silicate hydrate

$$Ca^{2+} + OH^{-} + SCC \rightarrow CSH$$

where:

SCC = Soluble Clay Silica

CSH = Calcium-Silicate Hydrate

Equation 4.9.7.1.2(b) – Pozzolanic reaction showing formation of calcium-aluminate hydrate

$$Ca^{2+} + OH^{-} + SCA \rightarrow CAH$$

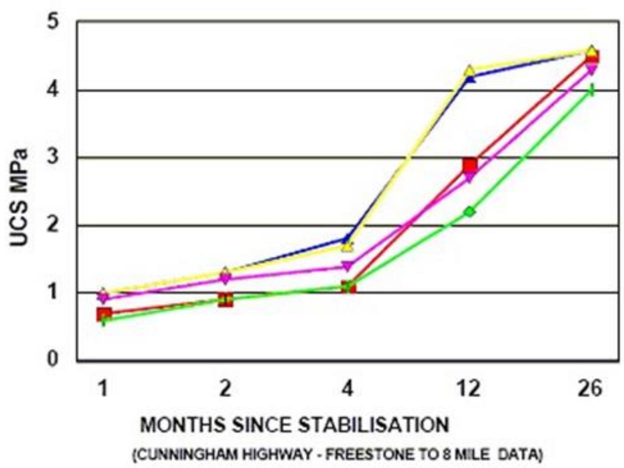
where:

SCA = Soluble Clay Alumina

CAH = Calcium-Aluminate Hydrate

With the presence of sufficient lime, it is possible for pozzolanic reactions to occur in many soils. Provided the material is suitable and enough lime is added, strength gains arising from these pozzolanic reactions are permanent and ongoing (Figure 4.9.7.1.2(d)).

Figure 4.9.7.1.2(d) – Example of strength development over a period of time for a lime stabilised subgrade



In the case of montmorillonite clays, which are present in ‘black soil’, the pozzolanic reaction does not provide a permanent effect until enough cation exchange has occurred to saturate the layers between the clay minerals. Furthermore, as a precondition for the pozzolanic reaction to be effective, the pH of the soil environment must be sufficiently high to dissolve the silica and alumina clay minerals, so they are available to react with the added calcium hydroxide.

In summary, for lime stabilisation to be successful, sufficient lime must be made available to:

- provide for the initial soil modification
- ensure a pozzolanic reaction between the silica and alumina in the clay minerals and the calcium hydroxide in the lime occurs
- make the pH of the soil environment sufficiently high to dissolve the silica and alumina clay minerals so they are available to react with the added calcium hydroxide, and
- realise an appropriate strength gain.

4.9.7.2 Appropriate uses

Reference should also be made to Section 4.9.1.

4.9.7.2.1 Modification

It is only appropriate to modify a clay subgrade with lime where it is required as a construction expedient and long-term strength gain is not required; however, when used, the permeability of the modified material may be greater than the insitu materials. This can lead to problems or failures. Successful modification has often been associated with well-drained embankments, or subgrades protected from moisture ingress by extensive subsoil-drainage systems.

4.9.7.2.2 Stabilisation

Appropriate uses for stabilisation of clay subgrades using lime include:

- where the materials to be stabilised are suitable (for example, have enough suitable pozzolans, the amount of organic carbon is not excessive) and the dosage rate required is acceptable, and/or
- for treatment of subgrades under existing pavements, only where removal and replacement of the existing pavement can be tolerated (for example, in terms of traffic management, cost).

4.9.7.3 Inappropriate uses

Lime stabilisation can be affected adversely by any one of:

- lack of suitable pozzolans
- the presence of excessive organic carbon
- the presence of soluble sulphates, or
- the presence of highly-weathered soils with high ferric-oxide levels (for example, some lateritic soils) – this can actually interfere with the pozzolanic reaction.

Stabilisation of unsuitable soils can lead to serious problems which, at times, can only be rectified by removing and replacing the treated materials. If such problems occur, the overlying pavement layers must be removed. This can be a very expensive consequence of simply not undertaking adequate and appropriate testing.

It is crucial adequate testing is conducted before the decision is made to stabilise a subgrade with lime. Advice should be sought from the Principal Engineer (Pavement Rehabilitation) (email ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au) to ascertain appropriate testing protocols.

Reference should also be made to Section 4.9.2.

4.9.7.3.1 Modification

It is recommended modification be used with caution. It is inappropriate to undertake lime modification of clay subgrades in any of the following circumstances:

- for any purpose other than to expedite construction
- where subgrades are likely to be exposed to water entry (for example, in low-lying areas) unless they are protected from moisture ingress by extensive subsoil-drainage systems
- where the modification plant cannot traverse the subgrade to mix the lime effectively and efficiently
- for modification of subgrades under existing pavements where removal and replacement of the existing pavement can be tolerated (for example, in terms of traffic management, cost), or
- where unacceptably high levels of sulphates are present.

4.9.7.3.2 Stabilisation

Inappropriate uses for stabilisation of clay subgrades using lime include:

- where the stabilisation plant cannot traverse the subgrade to mix the lime effectively and efficiently
- for stabilisation of subgrades under existing pavements where removal and replacement of the existing pavement cannot be tolerated (for example, in terms of traffic management, cost), and/or
- where unacceptably high levels of sulphates are present.

4.9.7.4 Materials

Definitions for the limestone, agricultural lime, quicklime and hydrated lime are contained in Chapter 1 of this *Manual*.

Relevant departmental specifications include:

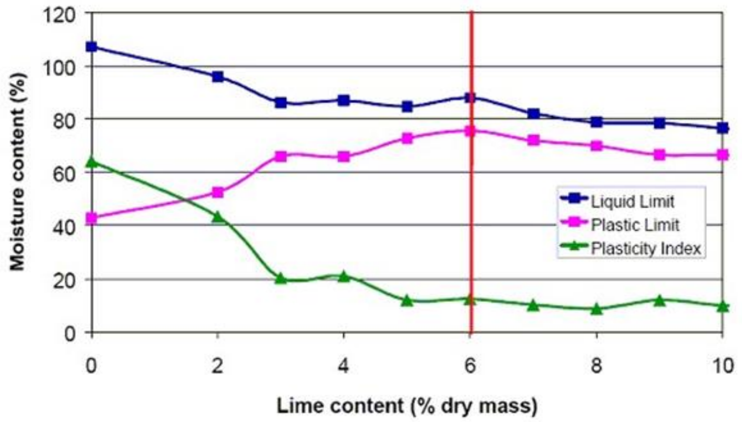
- MRS05 and MRTS05
- MRS07A and MRTS07A, and
- Technical Specification MRTS23 *Supply and Delivery of Quicklime and Hydrated Lime for Road Stabilisation* (MRTS23).

For materials without adequate natural pozzolans, fly ash or slag can be added to support the pozzolanic reactions.

4.9.7.5 Design considerations

For modification, sufficient lime should be added to reach the 'lime fixation' percentage which occurs when there are no further changes in PI for further additions of lime; for example, the lime fixation percentage is 6% by mass in Figure 4.9.7.5(a).

Figure 4.9.7.5(a) – Example of the effect of lime on the plastic limit, liquid limit and plasticity index of a typical clay



For stabilisation, the recommended design method involves lime fixation testing plus the use of pH testing to determine the 'lime demand', whether a soil is 'reactive' to lime and estimate the required lime content. Figure 4.9.7.5(b) is an example showing the required dosage of lime is 6% by mass. This testing is augmented by 28-day UCS testing to establish the optimum lime content known as the lime stabilisation optimum (LSO). The LSO is the dosage determined from a plot of UCS results versus the corresponding lime content. The dosage where the UCS attains a maximum is the LSO (Figure 4.9.7.5(c)). The greater of the lime content required to achieve lime fixation, the lime content determined from the pH testing and the LSO is taken to be the required dosage to achieve a permanent effect. An additional 1% lime should then be added to allow for variations caused by losses and uneven mixing in the field.

Figure 4.9.7.5(b) – Lime demand test results

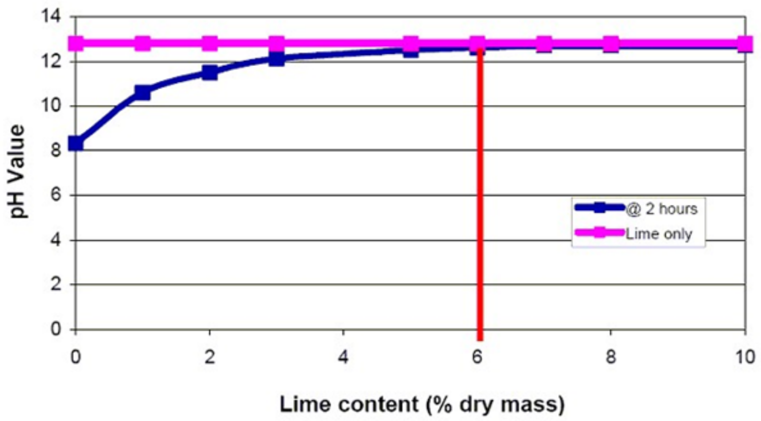
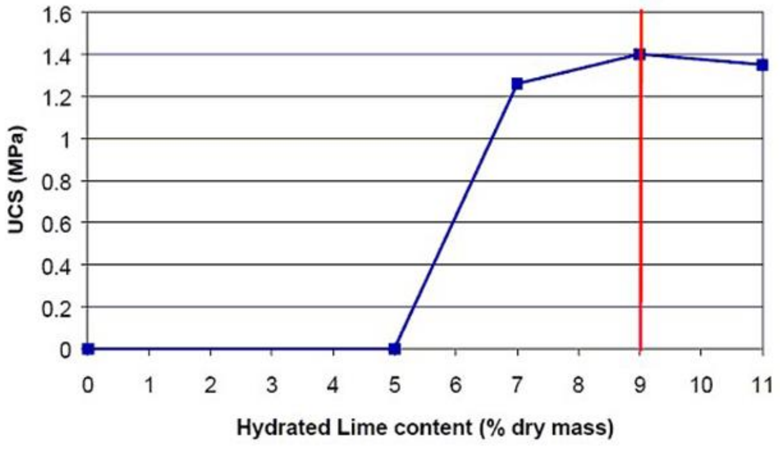


Figure 4.9.7.5(c) – Unconfined compressive strength test results to determine the lime stabilisation optimum



Research indicates accelerated curing (at higher than field temperatures) can alter the mechanism of normal pozzolanic reaction and, for this reason, curing for less than 28 days before testing UCS cylinders is not recommended.

Lime produced by different manufacturers is of varying quality, depending on the purity of the limestone deposit and the type of manufacturing kiln used. Lime must conform to the requirements of MRTS23.

Note:

- Laboratory testing is always carried out using hydrated lime, and
- quicklime contains impurities, the level of which depends on the source.

If quicklime is used in the field, the dosage determined through laboratory testing will need to be adjusted:

- so the amount of quicklime added in the field equates to the dosage of hydrated lime determined from the laboratory testing, and
- to account for the level of impurities present in the particular quicklime used on the project.

4.9.7.6 Construction considerations

Figures 4.9.7.6(a), 4.9.7.6(b), 4.9.7.6(c), 4.9.7.6(d) and 4.9.7.6(e) show some aspects of lime stabilisation.

Figure 4.9.7.6(a) – Spreading of quicklime

Figure 4.9.7.6(b) – Slaking of quicklime

Figure 4.9.7.6(c) – Slaking of quicklime

Figure 4.9.7.6(d) – Steam cloud generated during slaking of lime

Figure 4.9.7.6(e) – Lime stabilisation of an expansive-soil subgrade in the outer region of a pavement

As noted in Section 4.9.7.5, it is essential, where quicklime is used, the dosage used in the field is adjusted to take account of the impurities present in the quicklime used and the fact laboratory testing was undertaken using hydrated lime.

MRTS07A applies for modification and stabilisation of clay subgrades and includes many requirements, including:

- minimum plant requirements
- a site survey of public utility plant, drainage systems, and so on before works start; construction procedures must nominate how such items will be protected during construction
- modification and stabilisation carried out over two days: on the first day, up to two-thirds the required amount of lime is added and mixed, while, on the second day the balance of the lime is added and mixed with a minimum of two mixing passes required
- final compaction and trimming of the stabilised or modified material completed within the allowable working time, measured from the start of wet mixing on the second day
- when quicklime is used, mixing can only commence once slaking is complete
- the maximum spread rate for any one spreading run is 10kg/m^2
- after each spreading run, there must be at least one mixing run and the material trimmed and/or compacted as required to enable construction to meet the requirements of MRTS07A
- mixing must continue until the modified or stabilised material has a suitable particle size distribution and the lime is incorporated uniformly
- the last mixing run is to 100% of the stabilisation or modification depth, but all other mixing runs are to 90% of the stabilisation or modification depth
- quicklime cannot be used when a reclaimer / stabiliser with an integrated spreader is used.
- trimming must take place before compaction as well as after it
- stabilised and modified material is to be trimmed to waste only

- construction joints require care and there are particular requirements for them
- curing must commence immediately upon completion of final trimming and compaction until sealed or covered by the next pavement layer
- there are construction constraints related to weather conditions
- a maximum time between spreading and mixing is nominated (for example, 30 minutes) as is a maximum time for incorporation of all lime (six hours)
- the actual dosage of lime applied during construction must be adjusted based on the available lime index of the lime being used for construction and the available lime index of the hydrated lime used in laboratory testing
- process control for compaction which can be used, provided a trial successfully demonstrates compliance with MRTS07A, is conducted for each unique combination of materials, plant, methodologies or processes and uses the same plant, processes, methodology and materials as in construction
- removal and disposal of unsuitable materials, and
- water must be free from oil, acids, organic matter and any other matter which could be deleterious to the mixture. It must also be potable and contain less than 0.05% of sulphates.

Any attempt to correct the shape after modification or stabilisation of the material will result in an uneven thickness of the modified or stabilised material remaining.

As well as laboratory testing, an assessment must be made to determine the materials' amenability to modification or stabilisation. Embedded rocks in the clay can damage construction plant and add significantly to the cost. The depths of cover over public utility plant, stormwater pipes, culverts and the like should also be checked before attempting pulverisation, modification or stabilisation so they will not be damaged during construction by them being within the mixing depth or by construction plant travelling over them when the amount of cover is insufficient to prevent damage.

The use of quicklime can save money when compared to hydrated lime, and results in less dust in the air; however:

- it is essential it is fully slaked in the field before mixing commences, and
- it raises additional WH&S issues.

Quicklime is extremely aggressive desiccant; therefore, care must be exercised so contact with the skin and, particularly, the eyes, does not occur. This may be difficult to prevent, especially for members of the public.

Lime from different sources can have different gradings or other properties. Consequently, some products require more slaking than others to achieve full hydration. The field test for slaking is to test areas for evidence of further reaction by adding more water. The literature recommends enough water be added to the material to be stabilised, so its OMC is exceeded; however, the OMC of the stabilised material is a moving target which varies with time. The simplest way to control moisture practically is by a time-proven squeeze test conducted in the field by trained staff. The recommended approach is to prepare reference samples at OMC and monitor the moisture content in the field by squeeze testing to maintain consistency.

Experience suggests up to three days might be required before construction of the next pavement layer commences. It is necessary to plan for this in the construction program (for example, by working on longer areas to allow areas stabilised previously time to cure).

Most equipment used for modification or stabilisation is 2.4m wide and this should be considered during design. Pavement widening on one side may be more economical than narrow widths of widening on both sides. A suitable pavement cross-section must also result (for example, consider the location of the crown and whether any correction of crossfall or superelevation is required).

As significant quantities of water are required for the process, responsibility for the supply of water should be established clearly in the contract.

4.9.7.7 Expected performance or comments

A pavement design methodology for lime-stabilised material as pavement layers is still currently being researched, and no recommendations are provided at this stage; however, a pavement design methodology was discussed by Transfund, New Zealand in 1988 (*Mechanistic Design of Pavements Incorporating A Stabilised Subgrade* (TNZ RR127).

There may be cases where stabilisation of the subgrade with lime is not enough to deal with expansive soils. One alternative or additional treatment is the injection of lime (refer figures 4.9.7.7(a), 4.9.7.7(b), 4.9.7.7(c), 4.9.7.7(d) and 4.9.7.7(e)).

Reference should also be made to Section 4.9.3.

Figure 4.9.7.7(a) – Example of equipment used to inject lime into expansive soils

Figure 4.9.7.7(b) – Effect of lime injection into a railway embankment

Figure 4.9.7.7(c) – The creation of a 'lime curtain' formed via injection of lime

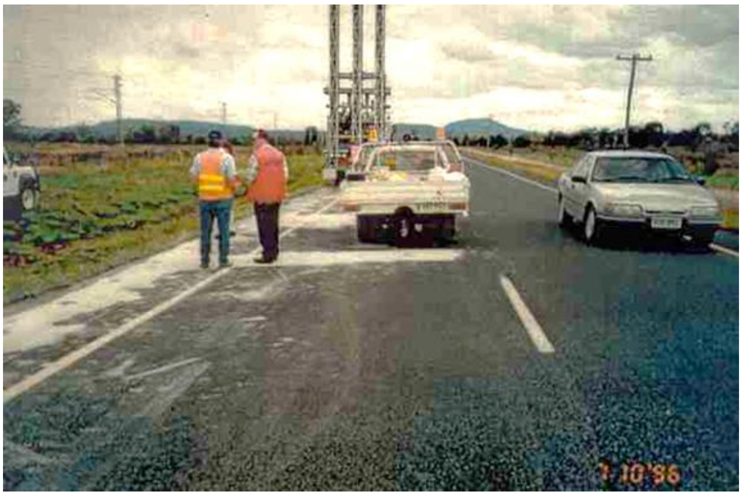


Figure 4.9.7.7(d) – Example of lime injection pods

Figure 4.9.7.7(e) – Injected lime under pressure exiting through cracks in the pavement

4.9.7.8 Further reading

Relevant publications include:

- AGPT04L
- AGPT04D
- Wirtgen CRM
- *Structural design procedure of pavements on lime stabilised subgrades guideline*
- *Materials Testing Manual* (MTM) *Part 2: Application Section 7 Testing of materials for lime stabilisation*
- *Soil Stabilisation with Cement and Lime* (Sherwood, 1993)
- McCallister, 1990
- TNZ RR127
- *Lime Stabilization: Reactions, Properties, Design and Construction* (TRB, 1987)
- *Soil Stabilisation for Roadways and Airfields* (Little, 1987), and
- *Suggested Method of Mixture Design Procedure for Lime Treated Soil* (ASTM STP479).

4.9.8 Foamed bitumen stabilisation of pavement materials

The foamed bitumen technique involves insitu recycling or plant-mix stabilisation using bitumen and hydrated lime as the binding agents to improve the strength and stiffness properties of unbound granular materials. Bitumen, when in a foamed or expanded state, allows more efficient coating of fines, resulting in a foamed bitumen stabilised gravel more resilient to flooding with significantly less shrinkage cracking, compared to cement stabilised pavements. Foamed bitumen pavements also exhibit excellent fatigue properties. The purpose of the bitumen (in conjunction with the secondary stabilising agent) is to achieve a strong, yet flexible, pavement layer compared to other stabilisation treatments (compared to those using cement).

Queensland projects typically use hydrated lime as the secondary stabilising agent. Further, MRTS07C and *Technical Specification MRTS09 Plant-Mixed Heavily Bound (Cemented)*

Pavements (MRTS09) only allow the use of lime for the secondary stabilising agent. Cement has been used in some projects and in other countries. If cement is used, a project-specific specification and/or standard will be required. Care must be exercised when using cement as the secondary stabilising agent to ensure the advantages of, and reasons for, using foamed bitumen (for example, greater flexibility and working time) are not nullified through its use.

Foamed bitumen stabilisation may require:

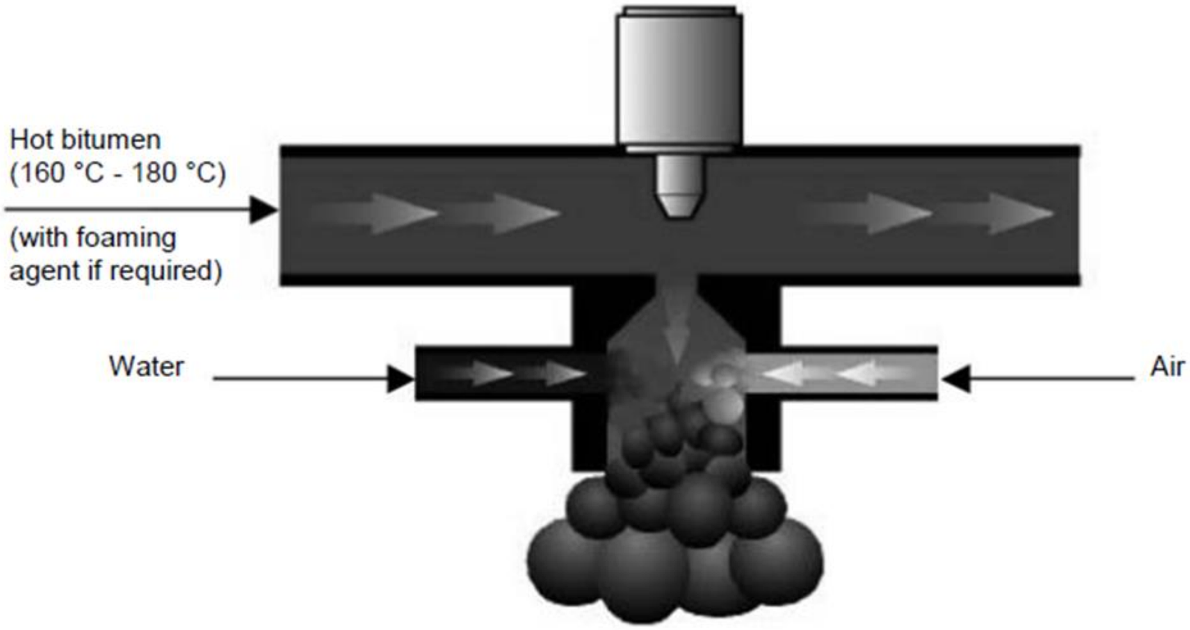
- removal of unsuitable material prior to stabilisation (for example, asphalt surfacing, patches, concrete, cement treated patches), and/or
- the addition of suitable material prior to overlay (for example, add granular material where the thickness of the existing granular material is insufficient).

Where material is added, a representative sample must be tested to determine whether the materials are suitable for stabilisation, the mix design, if so, and the correct grading.

4.9.8.1 Mechanics of foamed bitumen

Foamed bitumen is a mixture of air, cold water and hot bitumen. Currently, a foaming agent is also typically used in Queensland to achieve the required foaming characteristics (see Section 4.9.8.4.4). Injecting a small quantity of cold water into hot bitumen, with a foaming agent where required, produces an instantaneous expansion of the bitumen up to 15 times its original volume to form foam. The concept of foamed bitumen is illustrated in Figure 4.9.8.1. When the bitumen is in a foamed state, it is ideal for mixing with fine materials. The foam collapses very quickly and, therefore, rapid mixing is required to disperse the bitumen adequately throughout the material. During the mixing process, the foamed bitumen preferentially coats the finer particles, thereby forming a mortar which binds the mixture together effectively. Unlike cement and cementitious binders, the bitumen does not react with the materials; rather, it acts as an adhesive (or glue) to bind particles of the material together.

Figure 4.9.8.1 – Example of foamed bitumen production



Source: Wirtgen CRM

Table 4.9 is a useful guide with respect to material types and their suitability for stabilisation using foamed bitumen. Sections 4.9.8.4.1 and 4.9.8.4.2 provide additional guidance.

4.9.8.2 Appropriate uses

Foamed bitumen is suitable for a wide range of pavement materials; however, notwithstanding the information presented in Table 4.9 and sections 4.9.8.4.1 and 4.9.8.4.2, laboratory testing is essential for assessing the amenability of a particular material stabilisation using foamed bitumen.

Foamed bitumen stabilised pavement layers are suitable for use:

- as a foamed bitumen stabilised base covered by less than 100mm (total thickness) of DGA or with a sprayed seal surfacing for other than High Intensity, Low Intervention pavements (refer to Transport and Main Roads PDS foamed bitumen stabilised pavement with sprayed seal surfacing and asphalt over foamed bitumen stabilised base pavement typical pavement structures) or
- as a foamed bitumen stabilised sub-base covered by 100mm (total thickness) or more of DGA for High Intensity, Low Intervention pavements (refer to Transport and Main Roads PDS asphalt over foamed bitumen stabilised sub-base pavement typical pavement structure).

The PDS is an important reference and designers will need to refer to it to complete pavement rehabilitation investigations and designs. For greenfield sites, a guide to selection of foamed bitumen stabilised pavement types is provided in Table 2.2.1 of the PDS.

Reference should also be made to Section 4.9.1.

4.9.8.3 Inappropriate uses

Stabilisation of inappropriate pavement materials can lead to serious problems which, at times, can only be rectified by removing and replacing the stabilised materials. These problems can include excessive rutting, cracking and bleeding of the stabilised layer, which may lead to eventual failure of the pavement.

Typically, foamed bitumen is only used for stabilising pavement materials, not subgrades.

Inappropriate uses for stabilisation using foamed bitumen include:

- where the materials to be stabilised are unsuitable (see Section 4.9.8.4) or the dosage rate required is uneconomical; this includes:
  - for materials containing highly plastic fines, and/or
  - when ‘heavy clays’ will be incorporated into the stabilised layer
- when straight bitumen or bitumen dosed with foaming agent with the required foaming characteristics is not available, and/or
- where unacceptably high levels of sulphates are present.

Re-stabilisation of a pavement already stabilised using any type of binder or where early failures have occurred is also inappropriate because the fines will have been modified and may not be available in the initial stabilisation process, as the bitumen and lime have modified the fines (particularly the plastic fines) so these fines would not be available for coating with bitumen during re-stabilisation.

Reference should also be made to Section 4.9.2.

4.9.8.4 Materials

Definitions are contained in Chapter 1 of this *Manual*.

Relevant departmental specifications include:

- MRS05 and MRTS05
- MRS07C and MRTS07C
- MRTS17, and
- MRTS23.

MRTS07C only allows the use of lime for the secondary stabilising agent. Cement has been used in some projects and in other countries. If used, a project-specific specification and/or standard will be required.

4.9.8.4.1 Suitability of materials to be stabilised – general

A wide range of pavement materials is suitable for stabilisation with foamed bitumen. The most prominent criteria to determine a material’s suitability include:

- its source rock mineralogy
- its particle size distribution
- its fines content (for the percentages passing the 0.075mm and 2.36mm sieves), and
- plasticity.

Based on the department’s experience to date, materials that:

- comply with the 'C grading' or ‘modified C’ grading envelope in MRTS05 are suitable for foamed bitumen stabilisation
- have a particle size distribution that falls outside of the limits given in Table 4.9.8.4.1 are unsuitable for foamed bitumen stabilisation, and
- have relatively low plasticity fines (4% <PI <10%) are suitable for foamed bitumen stabilisation.

Experience from interstate projects indicates adequate performance may also be achieved with materials containing non-plastic fines.

The interaction of bitumen with the finer material (the material passing the 2.36mm sieve) is an important but complex phenomenon influencing the suitability of this treatment. It is essential an adequate amount of fines is available for the bitumen to coat. Without sufficient fines, the bitumen may not be dispersed effectively throughout the material, possibly resulting in a lower standard of performance. Coarsely graded materials are considered unsuitable for foamed bitumen stabilisation.

Table 4.9.8.4.1 – General grading limits for foamed bitumen stabilisation

Sieve size (mm)	**Percentage passing by mass**	
	Grading C	**Grading 'Modified C'**
75.0	100	100
53.0	100	100
37.5	100	100
19.0	80–100	87–100
9.5	55–90	67–88
4.75	40–70	50–65
2.36	30–55	38–50
0.425	12–30	16–26
0.075	5–20	8–16

Source: Modified from *Interim Technical Guidelines: The Design and Use of Foamed Bitumen Treated Materials* (CSIR), 2002

Testing is required to determine the suitability of materials for stabilisation.

4.9.8.4.2 Testing to determine suitability of materials

Representative samples are required to undertake testing to determine suitability of materials. A representative sample is required for each different material or combination of materials.

The strength, or more specifically the resilient response, of foamed bitumen stabilised material has typically been determined using either the:

- indirect tensile strength test, or
- indirect tensile resilient modulus test.

Consideration can be given to supplementary tests as required (for example, capillary rise testing to determine the susceptibility of the stabilised material to the ingress of water).

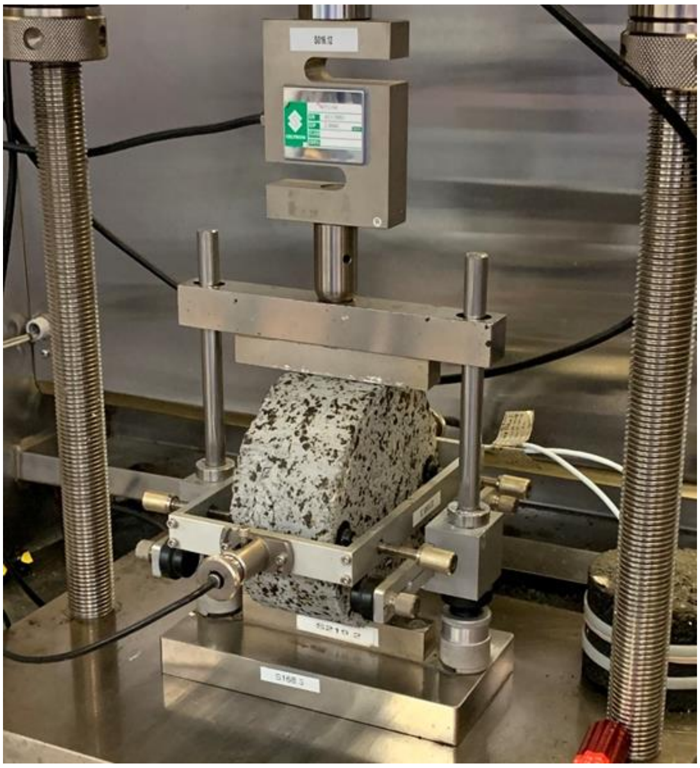
In Australia:

- the MATerial Testing Apparatus (MATTA) machine is often used to determine the indirect tensile resilient modulus so the testing is often referred to as MATTA testing, and
- MATTA (indirect tensile resilient modulus testing) is the usual method of testing foamed bitumen stabilised materials.

Specialised foamed bitumen testing equipment is required to undertake relevant laboratory testing. It includes the laboratory bitumen foaming apparatus, for example, the Wirtgen WLB 10 of Figure 4.9.8.4.2(a) or WLB 10 S (*Ancillary Equipment* (Wirtgen AE)), laboratory mixing apparatus (for example, Wirtgen, WLM 30 (Wirtgen AE)) and the MATTA test machine (Figure 4.9.8.4.2(b)). Currently, the department's Material Services Laboratory, located in Brisbane, conducts mix design testing for foamed-bitumen stabilisation using this equipment.

Figure 4.9.8.4.2(a) – Laboratory foaming process

Figure 4.9.8.4.2(b) – MATerial Testing Apparatus



During the early to mid-1990s, Emoleum (Australia) Ltd developed a test procedure for testing materials stabilised with foamed bitumen which is discussed in detail in *Pavement Recycling Using Foamed Bitumen* (Maccarrone, 1994). The department has since simplified and modified the procedure to suit Queensland conditions. The department's testing procedures (Q138A and Q139) are used to test materials for their suitability for foamed bitumen stabilisation for departmental works.

The department's mix design test procedures involve mixing the material (to be tested for stabilisation) with hydrated lime and foamed bitumen at an appropriate moisture content. The selected moisture content will depend on many factors, including the material's properties and moisture content. Prior to foamed bitumen testing, samples are typically prepared to have a moisture content equal to 70% of the OMC of the unstabilised material. If the insitu moisture content of the unstabilised material exceeds 70% of its OMC, and it is not possible to dry the material back during construction, the material used for laboratory testing should be prepared at its insitu moisture content.

After the foamed bitumen and secondary stabilising agent is mixed into the pavement material using the laboratory foaming apparatus, multiple specimens are prepared from each representative sample. Stabilised material is compacted into 150mm diameter Marshall specimens using 50 blows per face. Specimens are then extruded from the moulds. Three hours after completion of compaction, the specimens are tested to determine their initial 'MATTA' modulus. This provides an indication of the rut susceptibility and initial strength of the material immediately after stabilisation and compaction. Ultimately, this provides an indication of whether the stabilised materials can be trafficked early in their life (for example, at the end of a construction work period). The applicability of the initial modulus tests, and any curing required prior to testing, will need to be assessed based on the traffic conditions of, and the construction process to be adopted for, each individual project. Guidance is given in Table 4.9.8.5.1(a). Further testing is subsequently required.

After initial modulus testing, the specimens are then cured at 40°C for three days prior to testing at 25°C. This curing regime is aimed at simulating the curing over the medium term (three to six months). Each specimen is then tested in a 'dry' state to determine their 'dry MATTA' modulus. Subsequently, all specimens are soaked in water for 10 minutes under a vacuum (95 kPa less than atmospheric pressure). At the end of the 10 minutes, the specimens are removed and tested in a 'soaked' state to determine their 'soaked MATTA' modulus. This provides an indication of the stabilised material's susceptibility to weakening caused by water penetration (for example, if the pavement becomes inundated). The suitability of a material for foamed bitumen stabilisation is largely determined considering the 'initial', 'dry' and 'wet' moduli.

The guidance provided in tables 4.9.8.5.1(a), 4.9.8.5.1(b) and 4.9.8.5.1(c) were developed by the department and are based on extensive experience in Queensland. They are appropriate for typical road pavement moisture and traffic conditions in Queensland.

It is crucial adequate testing be conducted before deciding whether to stabilise or not.

4.9.8.4.3 Bitumen and foaming agent

MRTS07C requires Class 170 bitumen complying with MRTS17 be used for foamed bitumen stabilisation. Even though it complies with MRTS17, the foaming properties of Class 170 bitumen can vary (for example, with time), even if the bitumen is from the same supplier or source. As a consequence, each batch of bitumen used for foamed bitumen stabilisation must be tested for bitumen foaming properties prior to use. AGPT-T301 test procedure is used for measuring foaming properties of bitumen used for this stabilisation process.

Ideally the binder to be used as the primary stabilising agent will be, prior to its use as the primary stabilising agent:

- subjected to laboratory testing to determine its foaming and mixing characteristics, and
- used in the laboratory testing to manufacture test specimens.

If anything other than standard uncut Class 170 bitumen is proposed, completion of this testing is essential. A project-specific specification and standard will also be required.

The addition of a foaming agent is required. Ideally, the foaming agent used in the laboratory testing is the same as on the project.

If the required foaming and strength criteria are not achieved, the binder and/or foaming agent, where it is needed, must not be used.

The foaming characteristics of bitumen are influenced by:

- the bitumen class / type
- the temperature of the bitumen at the time of foaming
- the amount of water added to induce foaming
- the refining process used to extract the bitumen from crude oil
- the chemical composition of bitumen
- the type of foaming agent and the amount of it added to assist foaming, if any, and/or
- how long the bitumen has been stored at a high temperature.

Testing conducted at the department's Brisbane laboratory with the foaming agents currently available in Queensland shows:

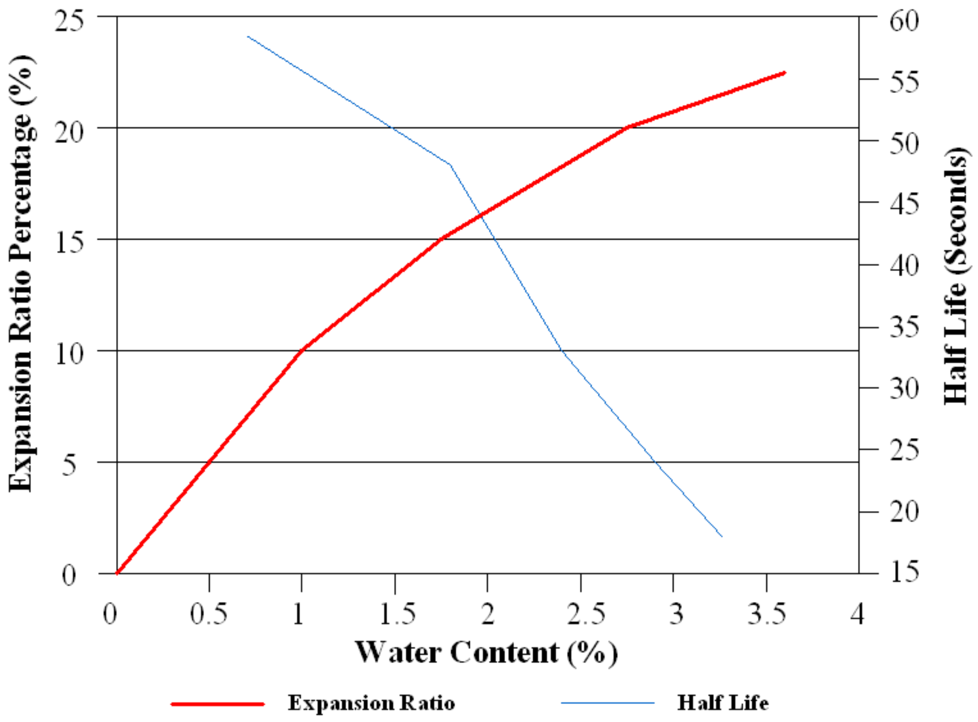
- foaming characteristics improve as temperature increases (noting, as per MRTS17, the maximum temperature to which Class 170 bitumen can be heated is 190°C), and/or
- a small percentage of foaming agent (for example, 0.5%) added to the bitumen can improve its foaming characteristics considerably.

4.9.8.4.4 Foaming water content

The influence of foaming water content on the bitumen's foaming characteristics is shown diagrammatically in Figure 4.9.8.4.4. As the water content increases, so, too, does the expansion ratio but the half-life decreases (the expansion ratio and half-life counteract each other). A compromise must be made between the amount of expansion achieved and the half-life of the foamed bitumen.

Based on experience with British Petroleum, Class 170 bitumen optimal foaming properties are achieved with a foaming water content of 2.5% by mass; however, slight variations in foaming water content can affect the expansion ratio and/or half-life of the bitumen actually used. Testing in the laboratory should always be completed to confirm the required foaming water content, whether foaming agent is required and, if so, what amount. The foaming characteristics required are an expansion ratio of at least 10 and a half-life of at least 20 seconds (see MRTS07C). The minimum expansion ratio should be 12 and the minimum half-life 45 seconds (*Design, construction and performance of insitu foamed bitumen stabilised pavements* (DFBSP)).

Figure 4.9.8.4.4 – Example of influence of water content on bitumen foaming properties



4.9.8.4.5 Lime

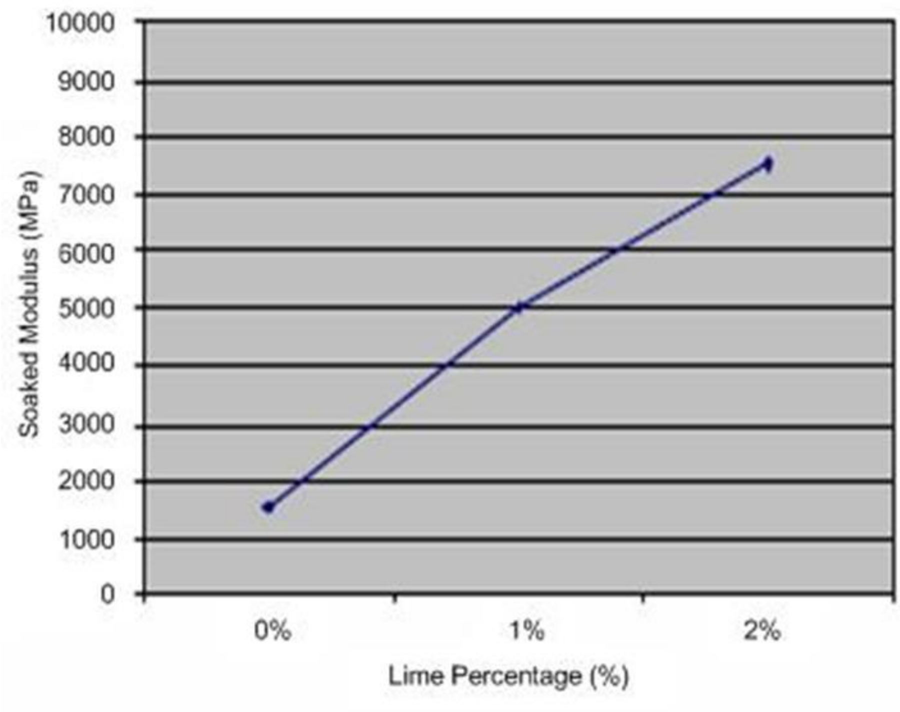
Lime is used in the foamed bitumen stabilisation process:

- to stiffen the bitumen binder
- as an anti-stripping agent
- to assist dispersion of the foamed bitumen throughout the material
- to improve the initial stiffness and early rut resistance of the stabilised material (Figure 4.9.8.4.5), and/or
- to reduce moisture sensitivity of the stabilised material, compared with stabilisation undertaken only with bitumen.

Lime should be mixed into the material at the same time as the bitumen. If lime is added to the pavement material before or after the bitumen, it can affect the early performance of the stabilised layer adversely. MRTS07C requires lime mixed into the pavement material not more than four hours before the bituminous stabilising agent is mixed into the pavement material.

Either hydrated lime or quicklime can be used as the source of lime for the stabilisation process; however, quicklime must be slaked fully prior to mixing into the pavement material and is not usually suitable for urban areas.

Figure 4.9.8.4.5 – Influence of lime on MATerial Testing Apparatus test results



4.9.8.4.6 Properties of foamed-bitumen stabilised pavement materials

With foamed bitumen stabilisation, it is predominantly the finer particles coated with bitumen. This differs from asphalt where all the particles are coated. As a consequence, the stabilised material has a mottled appearance (Figure 4.9.8.4.6(a)).

Figure 4.9.8.4.6(a) – Foamed bitumen stabilised specimen after curing for MATerial Testing Apparatus testing

When the uncompacted and stabilised material is squeezed in the hand, specks of bitumen from the bitumen-coated fines are retained on the hand. In addition to this, the material should feel 'spongy' under foot prior to compaction.

In most instances, where suitable material has been stabilised with foamed bitumen, the road has been able to be trafficked once the stabilised layer passes proof rolling. The stabilised material continues to develop strength over time and becomes a fully-bound, but flexible, pavement layer: for example, Figure 4.9.8.4.6(b) illustrates the reduction in deflection response over time and Figure 4.9.8.4.6(c) shows the strength of the stabilised layer increases with time for a past departmental project. The pavement has gained significant strength in the first year of its life.

Based on back analysis of deflection-testing data, it is considered the short-term flexural modulus of foamed-bitumen stabilised pavements is 1000–2000 MPa and it acts similar to a granular layer; however, the long-term modulus of the material increases to become a fully-bound layer and it remains in this state for years. Modulus values to 5000 MPa can be achieved with certain host materials, but no transverse shrinkage cracks have developed on departmental projects with lime as the secondary stabilising agent. Some of these projects are 10 years old.

Figure 4.9.8.4.6(b) – Example of reduction in deflection of a foamed bitumen stabilised pavement

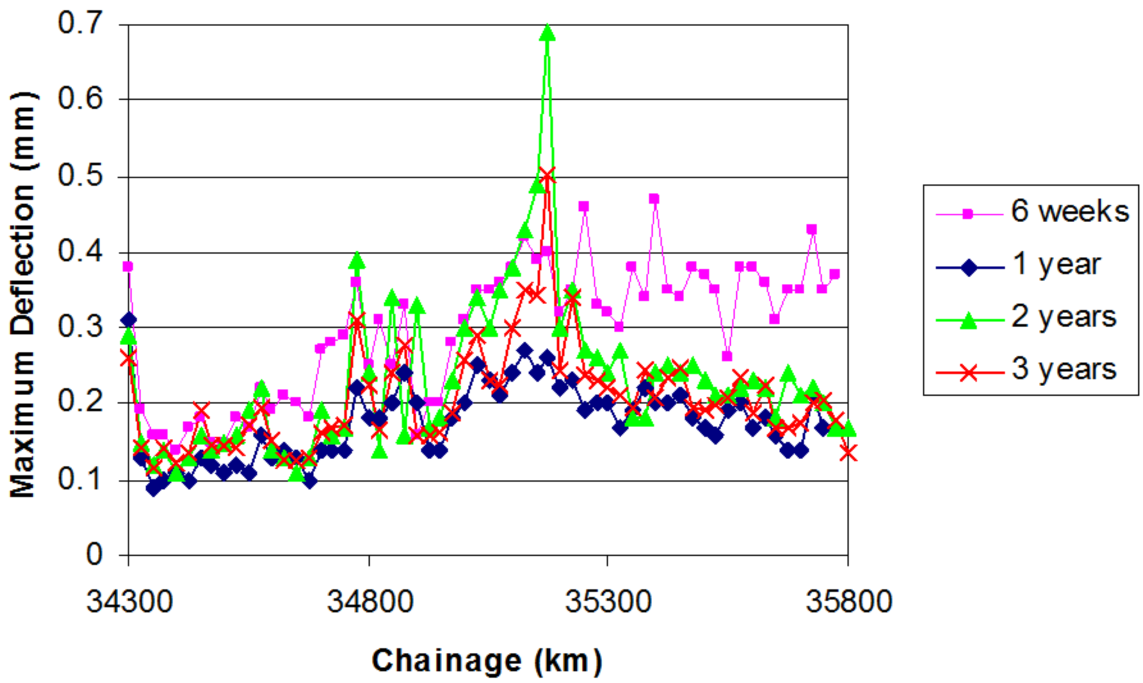
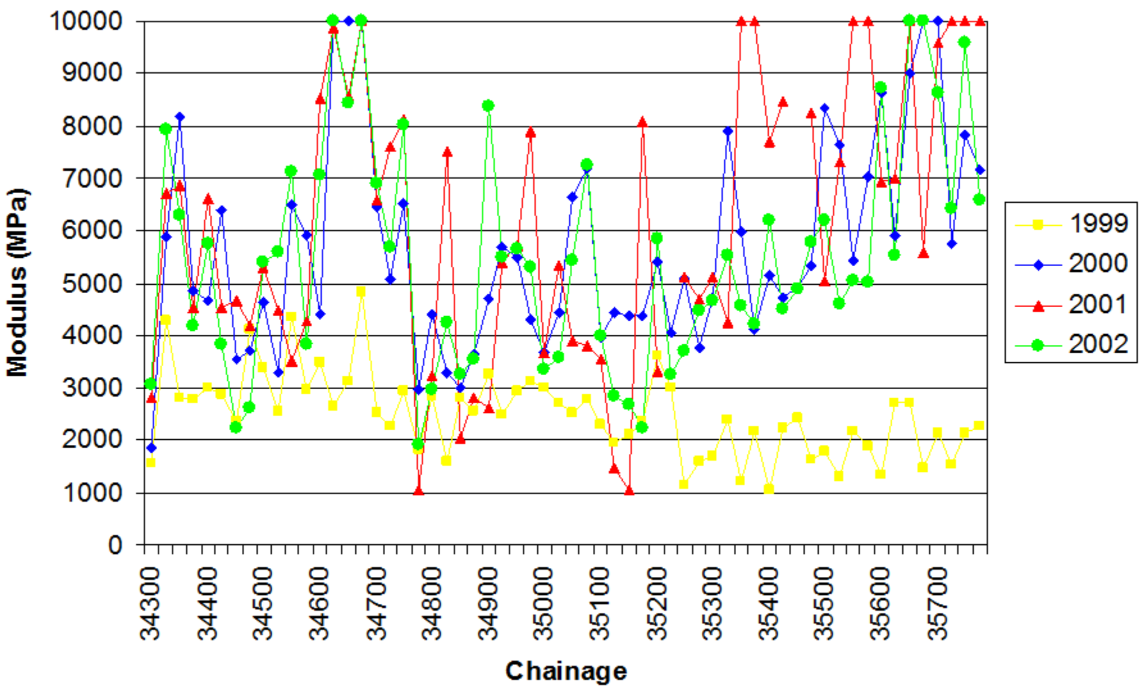


Figure 4.9.8.4.6(c) – Example of increase in back-calculated moduli of a foamed bitumen stabilised pavement



4.9.8.5 Design considerations

Any prime, primer seal, seal, SAM, SAMI or asphalt overlaid on a stabilised layer must be selected and designed carefully. It is important the type of materials used are compatible with and have an affinity for the stabilised material (for example, to bond with it adequately).

4.9.8.5.1 Design of stabilising additives

MATTA testing should be completed for a range of additive contents. Testing over a range of additive contents indicates the variations in properties with slight changes in additives. The optimum bitumen content occurs when the plot of soaked MATTA modulus versus bitumen and/or lime percentage attains a maximum (figures 4.9.8.4.5 and 4.9.8.5.1). In addition, for the material to be deemed suitable, the stabilised material must meet minimum strength criteria.

For the purposes of mix design, tables 4.9.8.5.1(a), 4.9.8.5.1(b) and 4.9.8.5.1(c) outline the recommended strength criteria used to determine whether a suitable mix can be achieved. The. department developed the guidance provided in tables 4.9.8.5.1(a), 4.9.8.5.1(b) and 4.9.8.5.1(c), based on extensive experience in Queensland. This guidance is appropriate for typical road pavement moisture and traffic conditions in Queensland. In addition to the guidelines provided in this *Manual*, prior to performing the mix designs, reference shall be made to the MTM (Part 2 *Application*), MRTS07C and MRTS09.

Mix designs for insitu foamed bitumen stabilisation shall satisfy the requirements of tables 4.9.8.5.1(a) and 4.9.8.5.1(b).

Mix designs for plant-mix foamed bitumen stabilisation shall satisfy the requirements of tables 4.9.8.5.1(a) and 4.9.8.5.1(c).

The department's experience with bituminous mixes has indicated higher bitumen contents result in superior fatigue performance, but excessive bitumen can lead to poor rut resistance. For most suitable materials, 3–3.5% bitumen and 1–2% hydrated lime, both by dry mass, has been found appropriate.

The bitumen content determined via laboratory testing should be adopted in construction.

Table 4.9.8.5.1(a) – Initial modulus mix design limits for foamed bitumen stabilised materials

Average daily ESA in design year of opening	*Minimum initial modulus (MPa)
<100	500
≥100	**700

*Initial sample curing time of three hours at 25°C ± 5°C required prior to completed initial resilient modulus testing.

Table 4.9.8.5.1(b) – Cured modulus mix design limits for foamed bitumen stabilised materials

Average daily ESA in design year of opening	Minimum '3-days cured' modulus (MPa)	Minimum 'soaked after 3-days cured' modulus (MPa)	*Minimum retained modulus ratio
<100	2500	1500	0.4
100–≤3000	3000	1800	0.45
>3000	4000	2000	0.5

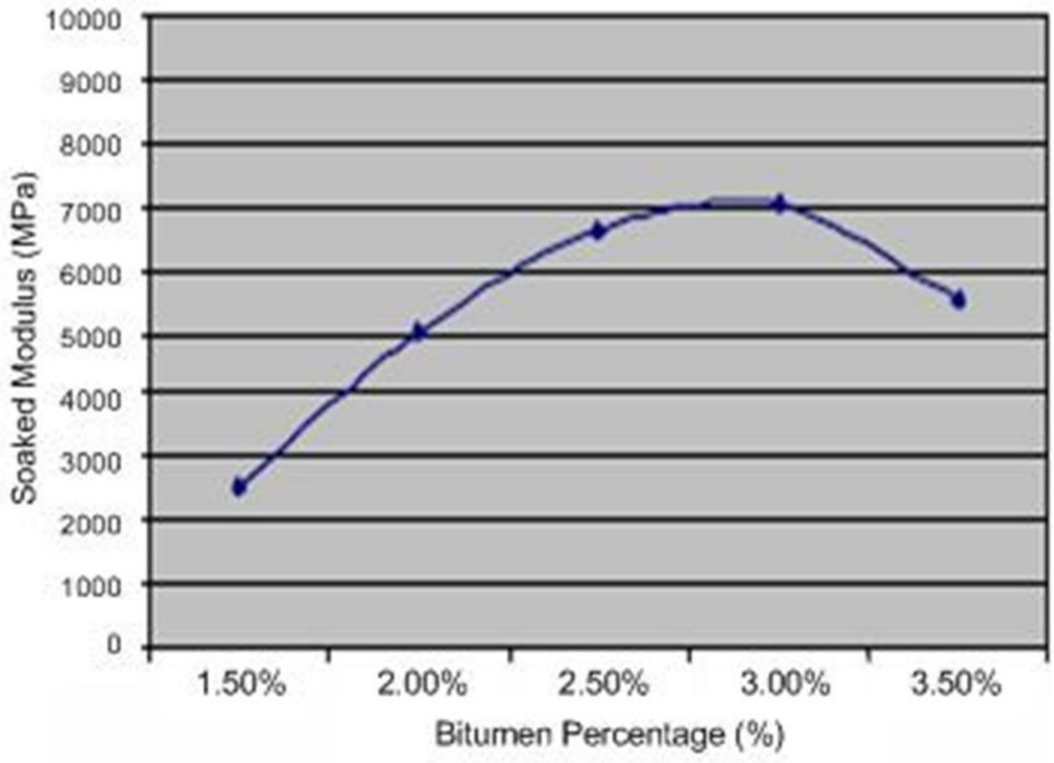
*Retained modulus ratio = 'soaked after cured' modulus divided by the 'cured' modulus.

Table 4.9.8.5.1(c) – Cured modulus mix design limits for production plant-mix foamed bitumen stabilised materials

Average daily ESA in design year of opening	Minimum 'soaked after 3-days cured' modulus (MPa)	Minimum 'soaked after 7-days cured' modulus (MPa)	Minimum 'soaked after 14-days cured' modulus (MPa)	*Minimum retained modulus ratio
All	1000	1400	1800	0.45

*Retained modulus ratio = 'soaked after cured' modulus divided by the 'cured' modulus.

Figure 4.9.8.5.1 – Example of influence of bitumen content on 'soaked' cured MATerials Testing Apparatus modulus



4.9.8.5.2 Pavement rehabilitation design

There is no published, broadly-accepted design procedure for foamed bitumen stabilised pavement layers in Australia. Research is underway to determine an appropriate design methodology for foamed bitumen stabilised pavement layers; therefore, guidance using only one method is described in this *Manual*.

Current performance monitoring of properly designed and constructed foamed bitumen stabilised pavements throughout the country indicate failure of these pavements may be either through fatigue or permanent deformation of the stabilised layer.

Permanent deformation of the stabilised layer can be minimised by ensuring:

- the stabilised materials comply with the minimum stiffness characteristics provided in tables 4.9.8.5.1(a), 4.9.8.5.1(b) and 4.9.8.5.1(c)
- good construction practices are followed, and
- materials used in laboratory testing are representative of the materials used in construction.

With respect to fatigue, as an interim measure, it is recommended foamed bitumen stabilised layers be designed as asphalt pavement layers in accordance with the mechanistic methodology outlined in AGPT02. It is assumed the layer will:

- eventually fail due to fatigue, and
- exhibit isotropic material characteristics with a Poisson's Ratio of 0.40.

When designing foamed bitumen stabilised layers, the percentage by volume of bitumen in the stabilised layer is typically assumed to be 7%.

The decision about the design modulus to adopt for foamed bitumen treated material is influenced by:

- the stiffness of the stabilised material, influenced by:
  - properties of the untreated material
  - binder types (for example, bitumen and lime) and contents
  - foaming characteristics of the bitumen used
  - curing conditions (for example, temperature and moisture), and
  - insitu pavement moisture conditions
- the environmental conditions it and the whole pavement encounter over their lives (for example, temperature and moisture)
- the rate of loading, and/or
- the age of the stabilised layer.

Due to the complexities involved in estimating the long-term stiffness of a foamed bitumen stabilised layer, the recommended method for determining the design modulus is:

- determine the indirect tensile resilient modulus of the mix during the laboratory mix design stage using samples and materials representative of what will be encountered during construction, and
- adjust the modulus or moduli obtained for temperature.

The design modulus, E_f, used for foamed bitumen stabilised materials subject to fatigue is given in Equation 4.9.8.5.2.

Equation 4.9.8.5.2 – Foamed bitumen design modulus

$$E_f = F_t \times M_s \leq 2{,}500$$

where:

E_f = Foamed bitumen design modulus (MPa)

F_t = Temperature correction factor from Table 4.9.8.5.2

M_s = For insitu foamed bitumen stabilisation , the soaked modulus determined from laboratory testing after 3 days' curing (at 40°C) for the selected bitumen / secondary stabilising agent content combination

For plant mix foamed bitumen stabilisation, the soaked modulus determined from laboratory testing after 14 days' curing (at 40°C) for the selected bitumen / secondary stabilising agent content combination.

Laboratory modulus testing of test specimens is determined at a temperature of 25°C. For pavement design, the moduli determined in the laboratory testing must be corrected to reflect the WMAPT of the site. The temperature correction factors to be applied are provided in Table 4.9.8.5.2.

The following points also apply in relation to determination of the design modulus of the foamed bitumen stabilised material:

- The full thickness of the foamed bitumen stabilised material is assigned the same design modulus (the design modulus is mix-dependent and is between 1800–2500 MPa at 25ºC prior to temperature correction).
- Where the overlying asphalt thickness is greater than or equal to 100mm, temperature correction does not apply.
- Where the overlying asphalt thickness is less than 100mm, temperature correction applies using the factors in Table 4.9.8.5.2

Table 4.9.8.5.2 – Temperature correction factors for determination of foamed bitumen stabilised material design modulus based on the weighted mean annual average pavement temperature

WMAPT (ºC)[1]	Temperature correction factor (F_t)	
	<50mm overlying asphalt	**50–<100mm overlying asphalt**
≤25	1.00	1.00
30	0.90	0.95
35	0.80	0.90
40	0.70	0.85

Notes:

1. For intermediate temperatures, linear interpolation is used to determine the temperature correction factor.

4.9.8.6 Construction considerations

MRTS07C applies for stabilisation using foamed bitumen as the primary binder. It has many requirements including:

- preliminary pulverisation to identify and remove unsuitable materials, replaced with suitable materials prior to spreading of the cement or cementitious binder
- minimum plant requirements
- a site survey of public utility plant, drainage systems, and so on before works start: construction procedures must nominate how such items will be protected during construction
- completing final compaction and trimming of the stabilised material within the allowable working time, measured from the commencement of the first mixing run with a stabilising agent
- shaping, compacting and trimming the existing surface to comply with MRTS07C
- adjusting actual dosage of lime applied during construction, based on the available lime indices of the lime being used for construction and the hydrated lime used in laboratory testing (hydrated lime is used for laboratory testing; if quicklime is used in construction, the dosage must be adjusted to give the same effective dose as the design hydrated lime dosage determined through laboratory testing)
- the maximum spread rate for lime in any one spreading run is 10 kg/m²

- after each spreading run, lime incorporated, and material trimmed and/or compacted as required to enable construction to meet the requirements of MRTS07C
- measuring of the bulking height and determination of the target depth during the final bitumen incorporation pass
- measuring the actual spread rates of the stabilising additives
- trimming before compaction as well as after it
- stabilised material trimmed to waste only
- measuring the actual stabilised layer thickness
- addressing particularly requirements for construction joints
- commencing curing immediately upon completion of final trimming and compaction until sealed or covered by the next pavement layer
- construction constraints related to weather conditions
- where the secondary stabilising agent is incorporated into the material(s) to be stabilised prior to the incorporation of the bituminous stabilising agent, mixing the secondary stabilising agent into the pavement material to a depth 50mm less than the final stabilisation depth and not more than 1.5 hours before the bituminous stabilising agent is mixed into the material to be stabilised
- process control for compaction, provided a trial demonstrates compliance with MRTS07C successfully – a trial is required for each unique combination of materials, plant, methodologies or processes and requires the use of the same plant, processes, methodology and materials in the trial as in construction
- removal and disposal of unsuitable materials (for example, asphalt patches), and
- water must be free from oil, acids, organic matter and any other matter which could be deleterious to the mixture. It must also be potable and contain less than 0.05% of sulphates.

The use of quicklime can save money when compared to hydrated lime, and results in less dust in the air; however:

- it is essential it is fully slaked in the field before mixing commences, and
- it raises additional WH&S issues.

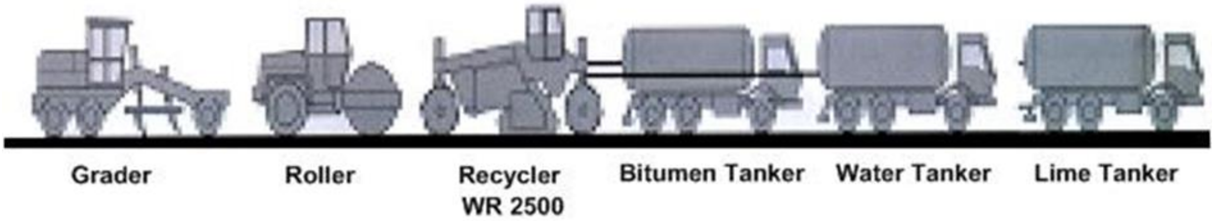
Quicklime is extremely aggressive desiccant; therefore, care must be exercised so contact with the skin and, particularly, the eyes does not occur. This may be difficult to prevent, especially for members of the public.

Lime from different sources can have different gradings or other properties. Consequently, some products require more slaking than others to achieve full hydration. The field test for slaking is to test areas for evidence of further reaction by adding more water. The literature recommends enough water also be added to the material to be stabilised, so its OMC is exceeded; however, the OMC of the stabilised material is a moving target which varies with time. The simplest way to control moisture practically is by a time-proven squeeze test conducted in the field by trained staff. The recommended approach is to prepare reference samples at OMC and monitor the moisture content in the field by squeeze testing to maintain consistency.

4.9.8.6.1 Construction process

Figure 4.9.8.6.1(a) shows, diagrammatically, a typical foamed bitumen stabilisation train.

Figure 4.9.8.6.1(a) – Diagram of typical construction process for foamed bitumen stabilisation



Figures 4.9.8.6.1(b)–(l) show different parts of the construction process.

In addition to laboratory testing, assess:

- the extent of particle breakdown during the stabilisation process
- the ease with which the existing pavement can be pulverised or stabilised: this includes assessment of the likelihood of damage being caused to the stabilising machine and, if so, its frequency, severity, and so on, and/or
- the depth of cover over public utility plant, culverts and the like so construction plant does not come in contact with them and cover over them is adequate so they will not be damaged when construction plant traverse over them. This may include public utility plant such as mains, valve covers and sewer and stormwater access chambers.

Pulverisation and shape correction of the insitu material must be completed prior to stabilisation because:

- it allows for the deficiencies to be corrected before the stabilising additives are incorporated and provides a more even surface and more uniform stabilisation depth, and/or
- it allows wet spots, patches (for example, of asphalt or cement treated material) and other unsuitable material to be identified and removed and replaced prior to stabilisation.

The pulverising pass must be 50mm less than the stabilisation depth so no 'unstabilised' or loose material ends up underneath the final stabilised material.

Figure 4.9.8.6.1(b) – Preliminary pulverising

Figure 4.9.8.6.1(c) – Grading to receive additive (before spreading or addition of any stabilising agent)

Figure 4.9.8.6.1(d) – Tray test to check the additive content of quicklime

Note: The secondary stabilising agent used in this case was quicklime.

Figure 4.9.8.6.1(e) – Spreading of hydrated lime

Figure 4.9.8.6.1(f) – Spreadrate test in operation

Notes:

1. This does not show practices compliant with current WH&S requirements.
2. Total area of mat = 1m².

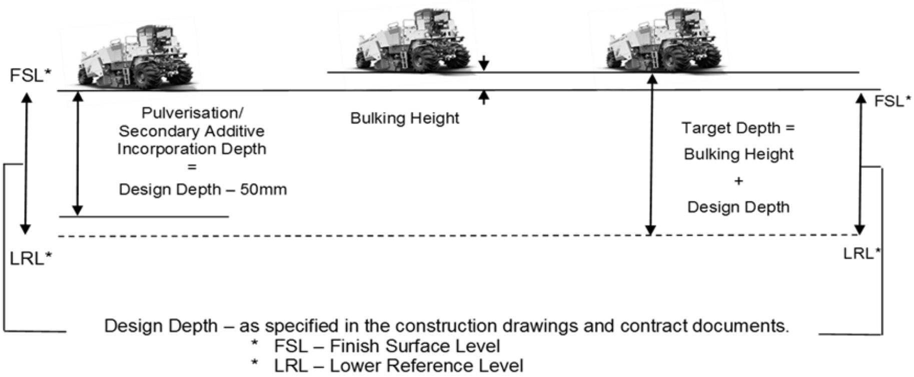
Figure 4.9.8.6.1(g) – Incorporation of secondary agent (hydrated lime) including moisture

Figure 4.9.8.6.1(h) – Light compaction (smooth drum) of bulked material

Figure 4.9.8.6.1(i) – Grading / shaping prior to the final foamed bitumen incorporation pass

Figure 4.9.8.6.1(j) – Target depth needs to account for the bulked height above finish surface level

Pulverisation/ Secondary Additive Incorporation	Bulking after secondary additive incorporation	Target Depth/ Final Foam Bitumen Pass
Preliminary pulverisation and incorporation of secondary agents is mulched to 50mm less than the design depth	After surface compaction and trimming, volume of materials change due to incorporation of the secondary agents. The bulking height above FSL needs to be identified and added to the design depth.	Cutting depth required on the final pass by the reclaimer/ stabiliser to reach the LRL will need to account for the additional bulking above the FSL.



Source: Transport and Main Roads Technical Specification MRTS07C

Figure 4.9.8.6.1(k) – Bitumen tanker coupled to recycler

Figure 4.9.8.6.1(l) – Compaction with pad foot rollers

In circumstances where the stabilising machine is capable of injecting lime directly into the mixing chamber, hydrated lime is added to and mixed with the material prior to the incorporation of the bitumen. In situations where lime cannot be injected directly into the mixing chamber, lime should be spread onto the prepared surface of the pulverised material and mixed into the pavement at the same time as the run to incorporate the bitumen. This will help disperse bitumen and allow the optimal strength properties of the stabilised layer to develop. If quicklime is used, it must be slaked fully prior to being incorporated into the stabilised layer. To determine whether the lime slaking process is complete, a thermometer can be inserted into the lime and the surrounding lime squirted with water to establish whether an exothermic reaction occurs. If the temperature rises significantly, further slaking is required. This can be done at a number of locations. This is illustrated in Figure 4.9.8.6.1(j).

Other points to note are:

- attention must be paid to the control of moisture
- the working time for foamed bitumen is longer than for cement stabilisation; MRTS07C stipulates the allowable working time is 6.5 hours, but compaction should be completed within two hours of the material being stabilised with foamed bitumen, and/or
- roughness can be improved significantly by stabilising shoulders and the running lanes, so the final trimming process will not be affected significantly by existing shoulder heights or shape.

4.9.8.6.2 Construction equipment

Foamed bitumen stabilisation requires the use of a stabilising machine with a purpose-built foaming spray bar within its mixing chamber. The spray bar nozzles have foaming chambers where water is injected into the hot bitumen.

Some stabilising plant have an integrated spreader which can be used to add / spread the lime. Quicklime cannot be used where such plant is used, and its integrated spreader is used to add / spread the lime.

Because the stabilisation process is completed in a single pass, a relatively powerful machine is required to mix and distribute bitumen adequately throughout the pavement material. MRTS07C outlines the minimum requirements for this and other plant.

4.9.8.7 Expected performance and comments

Foamed bitumen stabilised pavements, when compared to CTB pavements, has much less shrinkage cracking (reflection cracking requires continuous maintenance; therefore, the more there is, the more problematic it is). Cracking, although not necessarily reflection cracking, may have developed with some foamed bitumen stabilised pavements because:

- the bitumen content in the bitumen / lime matrix is low (for example, due to incorrect design or poor construction control)
- excess lime or cement was used in the stabilisation process (for example, due to incorrect design or poor construction control)
- movements of the highly expansive subgrade materials caused longitudinal cracking in the stabilised layer because of changes in its moisture
- the stabilised layer becomes fatigued, and/or
- support of subgrade is poor.

Reference should be made to Section 4.9.4.

4.9.9 Exsitu / Plant-mixed foamed bitumen

Plant-mixed foamed bitumen (as illustrated in Figure 4.9.9(a)) is a cold recycling technique performed using a mobile pugmill exsitu plant. Fed from a stationary silo, hydrated lime is delivered to the plant through an auger and a bulk bitumen tanker or auxiliary bitumen storage tanker supplies the C170 binder. A front-end loader feeds stockpiled source gravel into the plant's material hopper. The exsitu plant produces foamed bitumen agent into the twin shaft pugmill via a series of evenly-spaced nozzles, injecting a metered flow of water and compressed air into the hot C170 bitumen. The foamed bitumen stabilised product is produced from continuous homogeneous pugmill mixing of all agents (bitumen (foamed), hydrated lime and construction water) into the granular material.

Figure 4.9.9(a) – Wirtgen KMA 220 exsitu foamed bitumen plant

As illustrated in Figure 4.9.9(b), the final foamed bitumen stabilised product exits the plant from a slewing conveyor and can be stockpiled up to five hours prior to transporting to site as prescribed in MRTS09 for placement and compaction. The foamed bitumen stabilised stockpiles are loaded into trucks with a front-end loader and transported to the jobsite for the placement phase. The maximum allowable working time in MRTS09, measured from when the foamed bitumen is produced until final trimming and compaction is completed, shall be no longer than eight hours. Plant-mixed foamed bitumen can be either grader placed, or paver placed. Due to haulage costs to transport the plant-mixed foamed bitumen material to site, the exsitu / plant-mixed process is often a more expensive option in terms of price per cubic metre than insitu (in-place and does not require transport). Despite this, Transport and Main Roads has increased the use of plant-mixed foamed bitumen on projects in Queensland, particularly in environments where the expansive nature of the clay subgrade is very high. Plant-mixed foamed bitumen offers the ability to place a pavement geosynthetic reinforcement layer directly under the plant-mixed foamed bitumen as part of the construction process. This beneficial geosynthetic reinforcement layer is used to retard cracking caused from expansive clay subgrades and is not possible with an insitu foamed bitumen stabilisation construction process.

Figure 4.9.9(b) – Ammann exsitu foamed bitumen plant

4.9.9.1 Further reading

Relevant publications include:

- AGPT04L
- DFBSP
- AGPT04D
- Wirtgen CRM
- MTM parts 2 and 5
- PDS
- *Foamed Bitumen Stabilisation: the Queensland Experience* (Ramanujam, 2001)
- *Characterisation of Foamed Bitumen Stabilisation* (Ramanujam, 2000)
- AP PWT00, and
- Maccarrone, 1994.

4.9.10 Stabilisation using chemical stabilising agents

A number of chemical soil and aggregate stabilisers are available. Many are proprietary. Chapter 3 discusses the use of proprietary products and this should be referenced where the use of any proprietary product is being considered.

Unlike traditional stabilising agents (for example, lime, cement and bitumen), there are no 'standard' laboratory tests to assess chemical binders or develop mix designs or design procedures available to predict the performance of chemically stabilised materials. Indeed, these tests and design procedures may be peculiar to a particular chemical stabilising agent.

The department has very limited experience with stabilisation using chemical binders, mainly through limited trials.

Currently, the department does not have specifications for modification or stabilisation using chemical stabilising agents.

4.9.10.1 Functions

Chemical binders typically aim to:

- improve workability to achieve the required compaction with relative ease
- achieve soil structure stability
- reduce the swelling characteristics of clay soils
- reduce the permeability of a soil
- increase a soil's resistance to moisture absorption, and/or
- increase the shear strength and bearing capacity of a soil.

Chloride salt retards evaporation, attracts moisture from the air, and is effective as a dust palliative.

4.9.10.2 Appropriate uses

When considering chemicals for stabilisation, it is essential each be evaluated in light of the benefits derived versus the cost of the chemical admixture, as well as the care, time and effort involved with their use. Laboratory testing is required, and field trials should be conducted and evaluated.

Reference should also be made to Section 4.9.1.

4.9.10.3 Inappropriate uses

The use of chemicals for stabilisation may be restricted to isolated cases and special conditions. The cost of the chemical itself may be so great, other means of stabilisation are more economical.

The justification for using a proprietary product in lieu of a generic product needs to be considered (see Chapter 3).

Chlorides are not suitable in arid areas because of low humidity.

Reference should also be made to Section 4.9.2.

4.9.10.4 Materials

The department has no specification or standard for stabilisation using chemicals.

Calcium-chloride solutions are better lubricants than those of sodium chloride in the compaction process; however, sodium chloride may assist indirectly by dispersing clay binder material partially.

4.9.10.5 Design considerations

Any prime, primer seal, seal, SAM or SAMI overlaid on a stabilised layer must be selected and designed carefully. It is important the type of materials used are compatible with and have an affinity for the stabilised material (for example, to bond with it adequately).

The department has no design methodology for chemical binders. The design methodology may depend on the stabilising agent used.

4.9.10.6 Expected performance or comments

Given many of these products are proprietary, the supplier and/or manufacturer should be contacted for relevant information (for example, construction procedures, WH&S and handling requirements).

Ideally, trials under conditions similar to those encountered on state-controlled roads (for example, materials, supporting, environmental) would be conducted and evaluated rigorously before any particular chemical binder is used widely.

As the department has no specifications or standards for stabilisation using chemical binders, project-specific ones will be required where such binders are used. The department's Principal Engineer (Pavement Rehabilitation) should be contacted (email ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au) if the use of a chemical binder is being considered.

Reference should also be made to Section 4.9.4.

4.9.10.7 Further reading

Relevant publications include:

- AGPT04D
- AGPT04L, and
- *Trials with Proprietary Product Stabilising Agents* (Jameson, 1990).

4.10 Plant mixed stabilisation of granular materials and soils

The focus of this *Manual* is on pavement rehabilitation. While it is possible plant-mixed products may be used in pavement rehabilitation treatments, it is less common, so they are not discussed in this *Manual*.

4.11 Selection and use of polymer modified and multigrade binders

AP-T235 describes the types and classes and the Austroads classification system. It also describes the polymers commonly used in Australia.

- The department uses PMB different to Austroads classes; binder properties and nomenclature are different – refer to MRTS18.
- Currently, the department does not have a specification for multigrade bitumens.
- PMBs can minimise binder drain-down.
- While PMB joint and crack sealants may cope better than unmodified ones, they still may not reduce significantly the risk of reflection cracking where cracks are moving in the vertical plane.
- Weather can affect the choice of PMB and/or when construction incorporating a PMB takes place.

4.11.1 Materials

Relevant departmental specifications include:

- MRS11 and MRTS11
- MRS12 and MRTS12
- MRS13 and MRTS13
- MRTS17
- MRTS18
- MRTS19
- MRTS20
- MRTS21
- MRS22 and MRTS22, and
- MRS30 and MRTS30.

4.11.2 Further reading

Relevant publications include:

- AGPT03
- AGPT04K
- AGPT04F
- AP-PWT00
- AGPT04B, and
- AP-T235.

5 Design of pavement rehabilitation treatments: Mechanistic and deflection reduction methods

5.1 Introduction

This chapter outlines the methodologies to be adopted for designing appropriate pavement rehabilitation treatments for pavements excluding one or more concrete layers (for pavements that are not rigid). The design methods are:

- the deflection reduction method (see Section 5.3) which can only be used for the design of overlays over flexible granular pavements with a thin wearing course, with the overlay comprising either:
  - a granular overlay with a thin wearing course (see Section 5.3.2), or
  - a DGA with Class 320 binder asphalt overlay (see Section 5.3.4), and/or
- the GMP (see Section 5.4) which can be used to design any rehabilitation treatment for any pavement or treatment excluding a concrete layer.

The overlay design procedures included in this *Manual* for the deflection reduction method have been developed specifically for Queensland conditions. Their use is recommended for roads in Queensland for overlays designed using the deflection reduction method.

See Section 1.12 for detail on the pavement rehabilitation design process.

5.2 Work activities

The work activities required to carry out a pavement rehabilitation investigation and arrive at appropriate rehabilitation treatments may include:

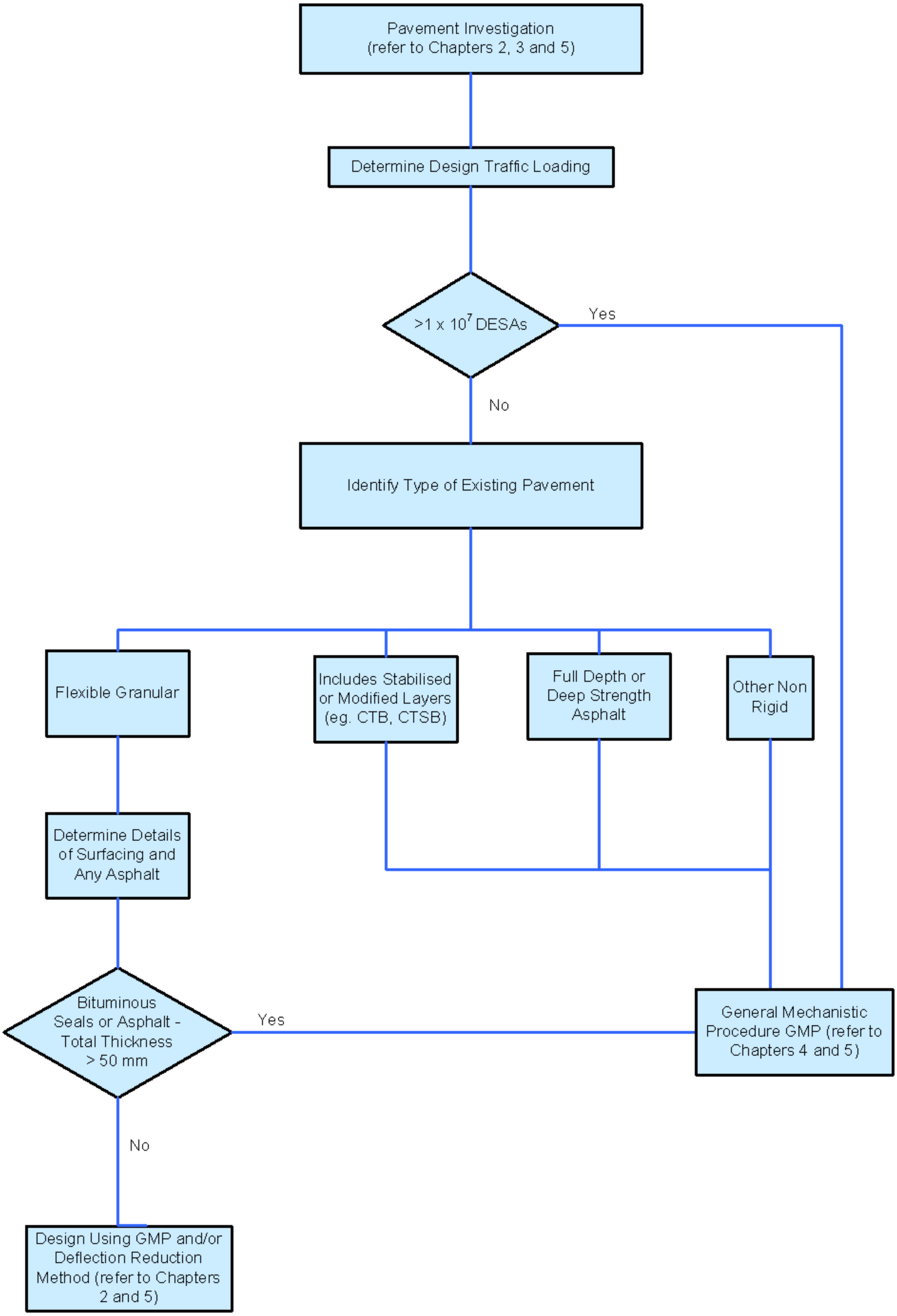
- mapping site defects to ascertain the actual pavement condition at the time of the pavement investigation (undertake a visual survey)
- testing using a non-destructive deflection survey: analysis of the results will help the designer divide the carriageways into representative sections and determine the structural capacity of the existing pavement layers (for example, undertake back analysis to inform a structural assessment)
- testing using a non-destructive GPR survey: analysis of the results will help the designer divide the carriageways into representative sections and help determine the pavement configuration(s) within the extents of the investigation
- sampling of the existing pavement and subgrade materials via the excavation of trenches, test pits or cores: these samples can be subjected to laboratory testing, fieldwork can include field testing and can be used to help determine the pavement configuration at each location and to ‘calibrate’ the GPR results
- characterising the existing pavement materials based on the laboratory and other test results
- formulating, analysing and designing pavement rehabilitation options using the deflection reduction method or the GMP, and/or
- making recommendations: these need to consider factors including the anticipated future traffic volumes, the existing pavement’s structure and condition, the characteristics of the existing subgrade, ‘constructability’ and WOLC.

This chapter provides guidance in relation to the following design methodologies:

- overlay design procedures using the deflection reduction method, and
- GMP.

These methods cannot be used where the existing pavement or proposed treatment includes one or more concrete layers. Figure 5.2 guides the user in selecting one of these methods, based on the type of the existing pavement and design traffic levels.

Figure 5.2– Selection of pavement rehabilitation methodology for non-rigid pavements



5.3 Overlay design procedure using deflection reduction method

The department's methodology for the use of surface deflections follows the philosophy outlined in AGPT05; however, the details are different (for example, the deflection figures, design of overlays using deflection). This *Manual* takes precedence over AGPT05 for departmental projects and roads controlled by Transport and Main Roads.

Deflection data must be representative of the current pavement condition when the data are used to characterise or evaluate the current condition of a pavement. An important aspect to consider, when deciding about whether the data can be so used, is age. A general rule of thumb is deflection data more than two years old are unlikely to be representative of the current condition of a pavement; however, this not always the case and the decision must be reviewed on a case-by-case basis by the designer and the client.

The deflection reduction method (overlay design procedure), using tolerable deflections presented in design figures (refer Figure 5.2), is considered a low-level analysis procedure currently. It should only be used as a first pass approach or to check the treatments developed using the more versatile deflection back analysis and GMP design approach. This method is based on a standard axle with a tyre pressure of 750 kPa. The methodology presented in AGPT02 adopts a tyre pressure of 750 kPa for design. This value is adopted for design of most Queensland state-controlled roads.

This section outlines the current departmental procedure to be used for determining the thickness of granular or Class 320 DGA-only overlays for existing flexible pavements. It is a standalone procedure and must not be used in combination with other design methods. As shown in Figure 5.1, the procedure is only applicable where:

- the design traffic loading is less than 1 x 10^7 design equivalent standard axles (DESAs), and
- the existing pavement is a flexible granular one with a thin wearing course, either:
  - a sprayed bituminous seal, or
  - thin asphalt (total asphalt thickness is ≤50mm).

For all other pavements, except for those with one or more concrete layers, the GMP must be adopted (refer Section 5.4).

When using the deflection reduction overlay design procedure, the various functional, environmental and other relevant influences must also be considered so a cost-effective and acceptable rehabilitation strategy can be developed.

The figures for the overlay design using the deflection reduction method are included in Appendix C.

Deflection testing and designs should be completed:

- at a minimum:
  - for each different pavement configuration (the pavement configuration may vary across the road, such as when a carriageway has been widened)
  - for roads with a single carriageway, in the most heavily-trafficked lane, and/or
  - for roads with multiple carriageways, in the most heavily-trafficked lane of each carriageway, and/or
- ideally for each OWP adjacent to a pavement shoulder / edge of each carriageway and one IWP with a minimum of one wheelpath per lane.

Typically, the results are examined and rationalised so, for each representative section, one overlay thickness, the greatest, is applied across all lanes. How investigations and designs for shoulders are completed may require further consideration.

When designing any treatment, consideration must be given to the final location of the lanes and wheel paths (after the treatment is applied, these may be different to the locations of the existing ones).

5.3.1 Basis of design procedures

The deflection response of pavements subjected to an applied load is explained in Chapter 2 and AGPT05.

The development of several overlay (rehabilitation) design methods followed the development of the Benkelman Beam in the 1950s. This instrument provided a simple, low-cost and accurate method of measuring rebound deflection under a specific load. The non-destructive nature of the test meant it was used for project-level investigations. Some deflection testing equipment can test at a higher speed (for example, deflectograph which is, effectively, a modified, automated version of the original Benkelman Beam, and TSD). They are often used for network surveys and, sometimes, project-level investigations. The FWD and HWD are later developments used by Transport and Main Roads for both network- and project-level testing. Only FWD and HWD results, using a 40 kN load correctly normalised, can be used in this method. Overlay design methods using deflections without any further analysis use a response-analysis approach. The response of the pavement, in terms of the rebound deflection measured under an applied load, is used to assess the performance of the pavement.

The elastic rebound deflection D_0, induced from an applied load, measured at any one point, is the sum of the elastic vertical strains in each of the pavement layers. The rebound deflection on its own gives no indication of the contribution each layer makes to the total deflection. Such information can only be obtained from measuring the entire deflection bowl in addition to the rebound deflection.

In deflection reduction methods, deflection is related to accumulated traffic loading. The pavement behaviour is quantified by distress parameters such as the plastic deformation at the subgrade level. By measuring the deflection, an overlay can be designed. The remaining life of the pavement can also be determined if the past traffic is known.

Overlay thicknesses are determined, based on controlling:

- rutting of the subgrade, and
- for DGA overlays only, cracking caused by fatigue of DGA layers.

In this design method, the critical level of distress is defined to be when the pavement has reached the end of its structural life. A 'tolerable deflection', below which the pavement is not expected to reach the critical level of distress for the accumulated traffic, is then determined. For DGA overlays, a thickness is also determined so fatigue cracking of the asphalt does not occur before the end of the design life and the greater thickness of the two is taken as the required thickness.

The normal (first) design standard for calculating the tolerable deflections for deflection tests carried out with a Benkelman Beam or FWD with a 40 kN loading is shown in Figure C1.1(a) in Appendix C. This corresponds to a terminal condition (distress level) of rutting approximately 20mm deep (and the associated roughness).

The second design standard for calculating the tolerable deflections for tests carried out with a Benkelman Beam and FWD with a 40 kN loading is shown in Figure C1.1(b) in Appendix C. This corresponds to a terminal condition of rutting approximately 30mm deep (and the associated roughness).

Similarly, figures C1.1(c) and C1.1(d) in Appendix C provide the design standards for calculating the tolerable deflections for deflection tests carried out with deflectograph.

Design using the normal design standard can be done for any road; however, design using the second design standard should only be undertaken with discretion. It may be considered for:

- special maintenance works subsequently overlaid
- roads with low traffic volumes, or
- pavements where environmental effects dictate performance.

Correction factors for FWD / HWD and deflectograph were considered when the figures included in Appendix C were developed; therefore, no correction needs to be applied when using 40 kN FWD and deflectograph deflections and the figures in Appendix C.

5.3.2 Initial steps of thickness design procedures

This section presents the initial steps of the procedures to be used for the design of overlays using the deflection reduction method. All steps must be completed for:

- all designs irrespective of the type of overlay, and
- for each representative section.

The remaining steps for granular overlays and asphalt overlays are presented in sections 5.3.3 and 5.3.4 respectively. The relevant remaining steps must also be completed to determine a design thickness (refer Chapter 2 for details and for an explanation of the terminologies used in defining deflection bowl parameters).

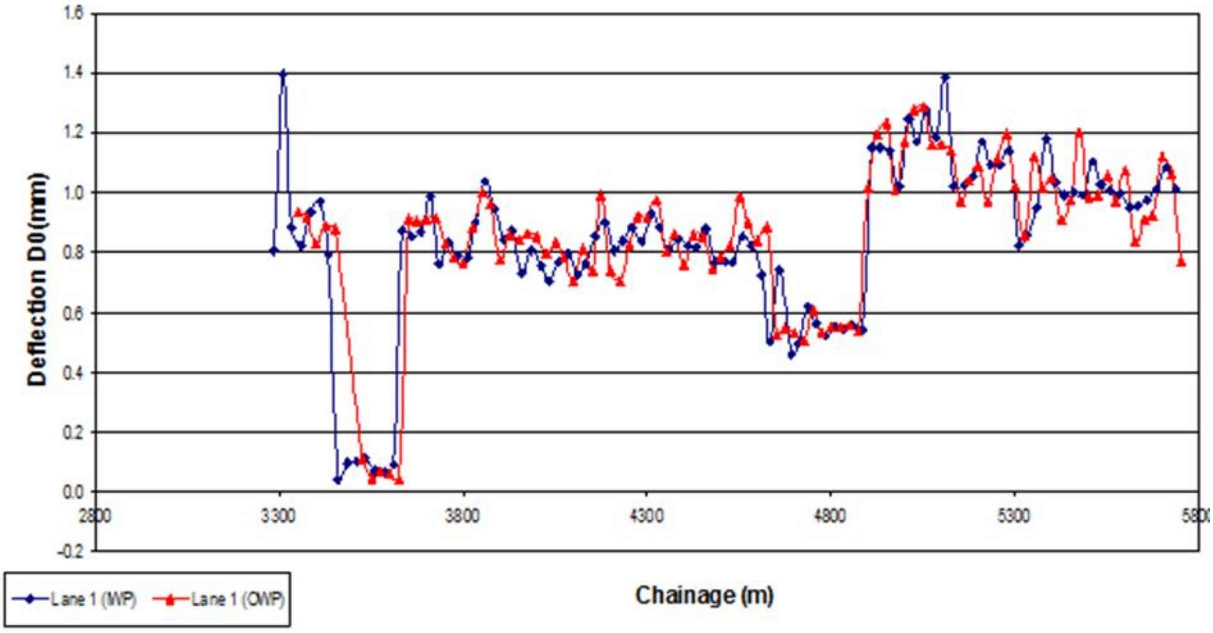
5.3.2.1 Step 1: Design traffic

Calculate the design traffic (DESAs) as per the PDS and AGPT02.

5.3.2.2 Step 2: Representative sections

Analyse the deflection data and subdivide the pavement(s) into relatively homogeneous sections (for example, sections with the same pavement structure and with relatively uniform deflection, refer to Chapter 2). These representative sections are used for design. An example of a road section subdivided into representative sections is shown in Figure 5.3.2.2.

Figure 5.3.2.2 – An example of a D_0 deflection profile with representative sections shown



5.3.2.3 Step 3: Uncorrected representative deflection, D_r

Arrive at a representative, uncorrected deflection, D_r, as described in Chapter 2 for each wheel path tested for each representative section.

5.3.2.4 Step 4: Uncorrected representative curvature function and representative deflection ratio

Obtain the representative CF before overlay ($CF_{b\ r}$) and DR_r for each representative section. As explained in Chapter 2, for a representative section, the mean value of the CFs in the representative section is adopted as $CF_{b\ r}$ and the 10th percentile lowest of the DRs is adopted as the DR_r.

5.3.2.5 Step 5: Uncorrected representative $D_{900\ r}$

Determine the representative uncorrected $D_{900\ r}$ value for each representative section as described in Chapter 2.

5.3.2.6 Step 6: Moisture correction

Correct D_r and $D_{900\ r}$ values for moisture as detailed in Chapter 2. When applying moisture corrections:

- the moisture correction factor depends on the subgrade type, rainfall, location of water table, pavement type, and so on, and
- correction is applied only to deflections measured in wheel paths not adjacent to shoulders or edges of pavement.

5.3.2.7 Step 7: Temperature zone

Determine a WMAPT appropriate for the location of the pavement to be rehabilitated (see Chapter 2).

5.3.2.8 Step 8: Temperature correction

If the wearing course of the existing pavement is asphalt, apply a temperature correction factor to all D_r and $CF_{b\ r}$ values as described in Chapter 2.

5.3.2.9 Step 9: Correction of deflection parameters for speed of loading

If the wearing course of the existing pavement is asphalt, all D_r and $CF_{b\ r}$ values may need to be corrected to take account of the speed of loading during deflection testing as described in Chapter 2.

Corrections to take account of the speed of loading during testing are only applied if the rate of testing the deflection response of a pavement is much slower than the speed of loading experienced under ‘real’ traffic. The Benkelman Beam is one type of testing equipment requiring such a correction; the deflectograph is another. The rate of load application for the FWD and HWD, compared to the loading experienced under ‘real’ traffic, is not well understood or researched; refer Section 2.13.10.4.9.2 for recommendations.

5.3.2.10 Step 10: Corrected representative deflection D_r

Obtain the corrected representative deflection D_r to be used for design for each representative section using the following guide:

- If the corrected deflection/s are higher than corresponding OWP deflection/s, then the corrected deflection/s should be used to determine D_r.
- If the corrected deflections are lower than the corresponding OWP deflection/s, a check should be made to ensure other factors are not controlling the OWP deflection (for example, 'box type' construction trapping moisture or providing inadequate lateral restraint). If there are no obvious defects in the OWP to cause high deflections, then the OWP deflections should be used to determine D_r.

5.3.2.11 Step 11: Subgrade (response) California Bearing Ratio

Determine the subgrade (response) CBR using the moisture corrected $D_{900\ r}$ described in Chapter 2.

5.3.2.12 Step 12: Tolerable deflection

Tolerable deflections for various scenarios are shown in figures 5.3.4.1.4(a), 5.3.4.1.4(b), 5.3.4.1.4(c) and C1.1(a) in Appendix C. These must be used to determine the tolerable deflection, D_{tol}, for each representative section. Selection of the appropriate figure to use will depend on the design standard, design traffic loading and testing device used.

5.3.3 Granular overlays only

This section presents the design procedure for a granular overlay with only a thin bituminous seal or asphalt wearing course (total asphalt thickness is ≤50mm) and not subsequently modified or stabilised during its design life. For any other scenario with the subsequent addition of a granular material, treatment, modification or stabilisation during its design life, the GMP must be used to design the treatment (see Section 5.4).

5.3.3.1 Thickness design procedure

In addition to steps 1 to 12 given in Section 5.3.2, the following steps must be completed to arrive at the granular overlay design thickness. These steps must be completed for each representative section (refer Chapter 2 for details and for an explanation of the terminologies used in defining deflection bowl parameters).

5.3.3.1.1 Step 13: Determine whether an overlay is required and, if so, its thickness

DR_r and $CF_{b\ r}$ are not used.

Check whether the corrected $D_r > D_{tol}$. If not, an overlay is not required. If so, an overlay is required to lower the deflection to an acceptable level (a deflection no greater than D_{tol}).

If an overlay is needed, the thickness of it required to reduce D_r to D_{tol}, T_s, is determined from the following figures contained in Appendix C:

- Figure C1.2(a) for material compliant with MRTS05 and has a subgrade CBR of 3%, and
- Figure C1.2(b) for material compliant with MRTS05 and has a subgrade CBR of 15%.

Irrespective, the absolute minimum thickness for a granular overlay is 100mm.

Circumstances may dictate a non-standard or marginal material must be used in a granular overlay; this shall only occur subject to the approval of the relevant departmental Regional Director. The development of such provisions should be based on:

- documented performance history of the proposed material
- the moisture sensitivity and relative permeability of the underlying layer
- the quality and uniformity of the materials to be used, as demonstrated by regular laboratory testing
- the proposed traffic loading, and/or
- the climatic environment of the site.

As many of these materials are moisture-sensitive and not free-draining, added emphasis should be given to the effect of the overlay on the moisture control aspects of the existing pavement and formation; for example, a low permeability overlay enveloping the formation (for example, extending over the edge of a free-draining granular material) may trap moisture within the pavement.

Typical applications for the use of non-standard materials are in areas with an annual rainfall less than 500mm and with:

- a design with a traffic loading less than 1×10^6 DESAs for unproven materials, or
- a design with a traffic loading less than 1×10^7 DESAs for materials with proven performance histories for such traffic loadings.

Figure 5.3.4.1.4(a) summarises Step 12 (see Section 5.3.2.12) and this Step 13.

A typical example of the results obtained by applying all the design steps is shown in tables 5.3.4.1.4(a) and 5.3.4.1.4(b).

5.3.4 Asphalt overlays only

This section presents the design procedure for a Class 320 DGA overlay only, including a DGA asphalt wearing course and not subsequently overlaid or otherwise rehabilitated during its design life. For any other scenario, the GMP must be used to design the treatment (refer Section 5.4).

All new asphalt must comply with the requirements of MRS30 and MRTS30.

5.3.4.1 Thickness design procedure

In addition to steps 1 to 12 given in Section 5.3.2, the following steps must be completed to arrive at the asphalt overlay design thickness. All steps must be completed for each representative section (refer Chapter 2 for details and for an explanation of the terminologies used in defining deflection bowl parameters).

For asphalt overlays, the designs must be based on controlling subgrade rutting and fatigue cracking of asphalt layers.

The asphalt overlay process is summarised in Figure 5.3.4.1.4(b).

5.3.4.1.1 Step 14: Operating speed

Determine the operating speed for each geometric element of the road; refer to RPDM3-03 and AGRD03. Where there is more than one operating speed for one representative section, further subsectioning is required. The design figures appropriate for the operating speed and temperature zone must be used to develop the design for each section.

5.3.4.1.2 Step 15: Asphalt thickness to control subgrade rutting

For the relevant temperature zone and operating speed, obtain the thickness of Class 320 DGA overlay (T_s) required to reduce the corrected D_r to D_{tol} from the appropriate figure (from figures C1.3(a), C1.3(b), C1.3(c), C1.3(d), C1.3(e), C1.3(f), C1.3(g) or C1.3(h) in Appendix C). The required thickness determined is the minimum thickness required to control subgrade rutting for the analysis period; however, this thickness may not be sufficient to prevent fatigue cracking of the new asphalt.

5.3.4.1.3 Step 16: Asphalt thickness to control fatigue cracking

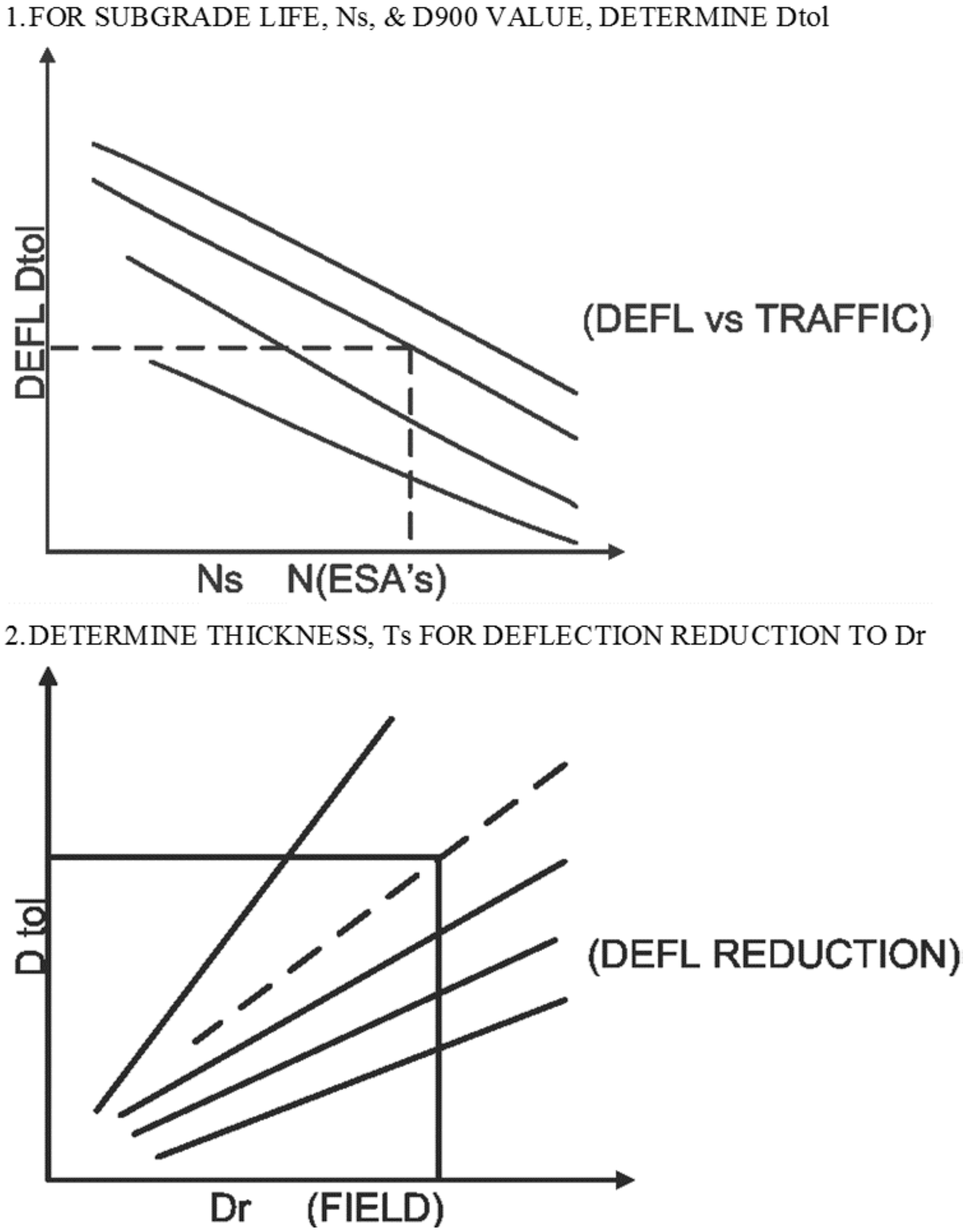
For the relevant temperature zone and operating speed, and using the corrected $CF_{b\ r}$, obtain the thickness of Class 320 DGA overlay (T_d) required to control fatigue cracking of asphalt from the appropriate figure (from figures C1.5(a)–C1.5(p) in Appendix C).

5.3.4.1.4 Step 17: Determine asphalt thickness for overlay

The greater of the thicknesses required to control subgrade rutting (Section 5.3.4.1.2) and fatigue cracking of the asphalt (Section 5.3.4.1.3) is the value to be adopted for the overlay.

A typical example of the results obtained by applying all the design steps is shown in figures 5.3.4.1.4(c) and 5.3.4.1.4(c).

Figure 5.3.4.1.4(a) – Summary of granular overlay design process for design using the deflection reduction method



Note:

N_S Design traffic loading for subgrade rutting

D_{tol} Tolerable deflection

D_r Corrected D_r = corrected representative deflection

T_S Required thickness of overlay to control rutting.

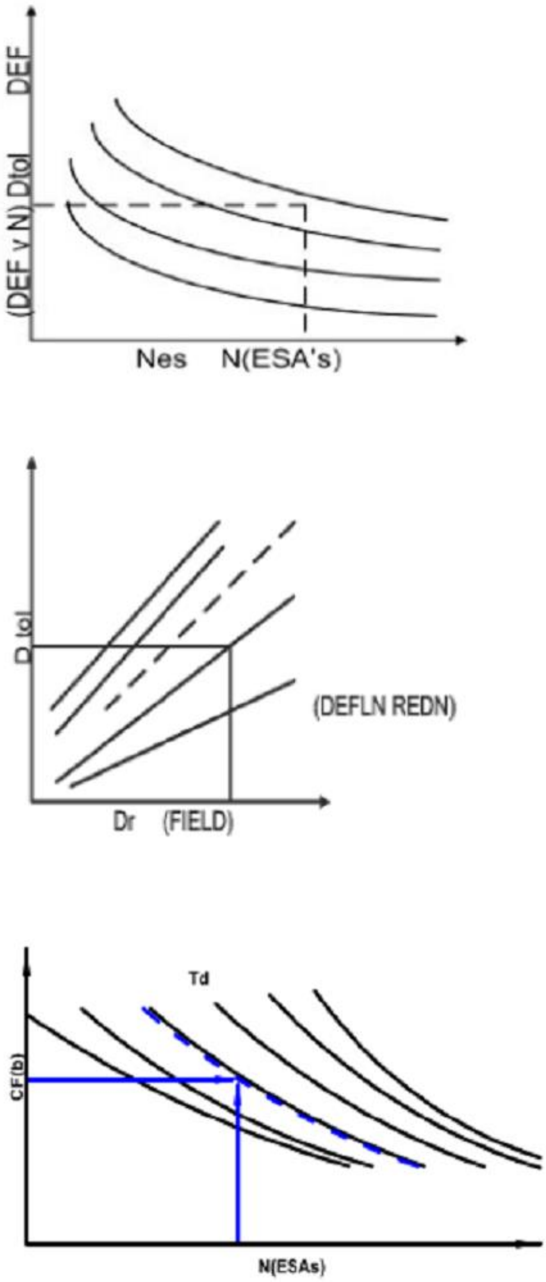
Table 5.3.4.1.4(a) – Example summarising the falling weight deflector deflection results to design a granular overlay using the deflection reduction method for a pavement without asphalt

Section					Rebound deflection													
No.	**Start chainage (m)**	**End chainage (m)**	**Lane No.**	**Wheel path**	**Average / Mean D_0**	**Standard Deviation (mm)**	**CV**	**Uncorrected D_{rep} (mm)**	**Seasonal moisture correction factor**	**Seasonal moisture correction factor X D_{rep}**	**Seasonal moisture corrected D_{rep} (mm)**	**Uncorrected D_{900} (mm)**	**Seasonal moisture correction factor X D_{900}**	**Adopted D_{900} (mm)**	**Representative subgrade CBR from**	**Average curvature function - $D_0 - D_{200}$**	**Adopted curvature function – $D_0 - D_{200}$**	**D_{200}/D_0 – 10% low**
1	3525	3625	1	OWP	0.036	0.012	33.4	0.050	1.0	0.050	0.089	0.045	0.045	0.100	17	0.00	0.00	0.739
	3460	3610	1	IWP	0.061	0.011	18.5	0.074	1.2	0.089		0.083	0.100			0.00		0.911
2	3650	4625	1	OWP	0.623	0.064	10.2	0.699	1.0	0.699	0.842	0.082	0.082	0.097	17	0.17	0.17	0.594
	3635	4610	1	IWP	0.629	0.061	9.6	0.702	1.2	0.842		0.081	0.097			0.17		0.603
3	4650	4875	1	OWP	0.430	0.024	5.6	0.459	1.0	0.459	0.557	0.022	0.022	0.028	25	0.17	0.17	0.457
	4635	4886	1	IWP	0.430	0.028	6.6	0.464	1.2	0.557		0.023	0.028			0.24		0.481
4	4900	5775	1	OWP	0.768	0.088	11.4	0.873	1.0	0.873	1.049	0.107	0.107	0.120	13	0.22	0.24	0.599
	4910	5760	1	IWP	0.780	0.078	10.0	0.874	1.2	1.049		0.100	0.120			0.24		0.556

Table 5.3.4.1.4(b) – Example summarising the thickness design of a granular overlay using the deflection reduction method for a pavement without asphalt using the data from Table 5.3.4.1.4(a)

Design life = 20 years								
DESAs = 3 x 10^6								
Section				**Use values from Table 5.3.4.1.4(a)**		**Tolerable deflection (mm) from Figure 5.9 in Appendix 5A**	**Thickness of granular material required (mm) from Figure C1.2(a) in Appendix C**	**Comments**
No.	**Start chainage (m)**	**End chainage (m)**	**Length (m)**	**Seasonal moisture corrected adopted D_{rep} (mm)**	**Adopted Subgrade CBR%**			
1	3525	3625	100	0.089	17	0.58	Not required	Very strong pavement. Possibly stabilised. There is no need for an overlay to improve structural strength. Note: Sometimes there may be other reasons for an overlay, e.g. to correct shape.
	3460	3610	150					
2	3650	4625	975	0.842	17	0.58	300	Alternatives include constructing overlay in stages or reconstruction. Either of these alternatives require a separate design using the GMP.
	3635	4610	975					
3	4650	4875	225	0.557	25	0.50	<50	Minimum thickness is 100mm. Provide 100mm thick granular overlay.
	4635	4886	251					
4	4900	5775	875	1.049	13	0.64	>300	Alternatives include constructing overlay in stages or reconstruction. Either of these alternatives require a separate design using the GMP. The requirement for such a thick overlay usually indicates the need for full, detailed pavement and subgrade investigations.
	4910	5760	850					

Figure 5.3.4.1.4(b) – Summary of asphalt overlay design process using the deflection reduction method

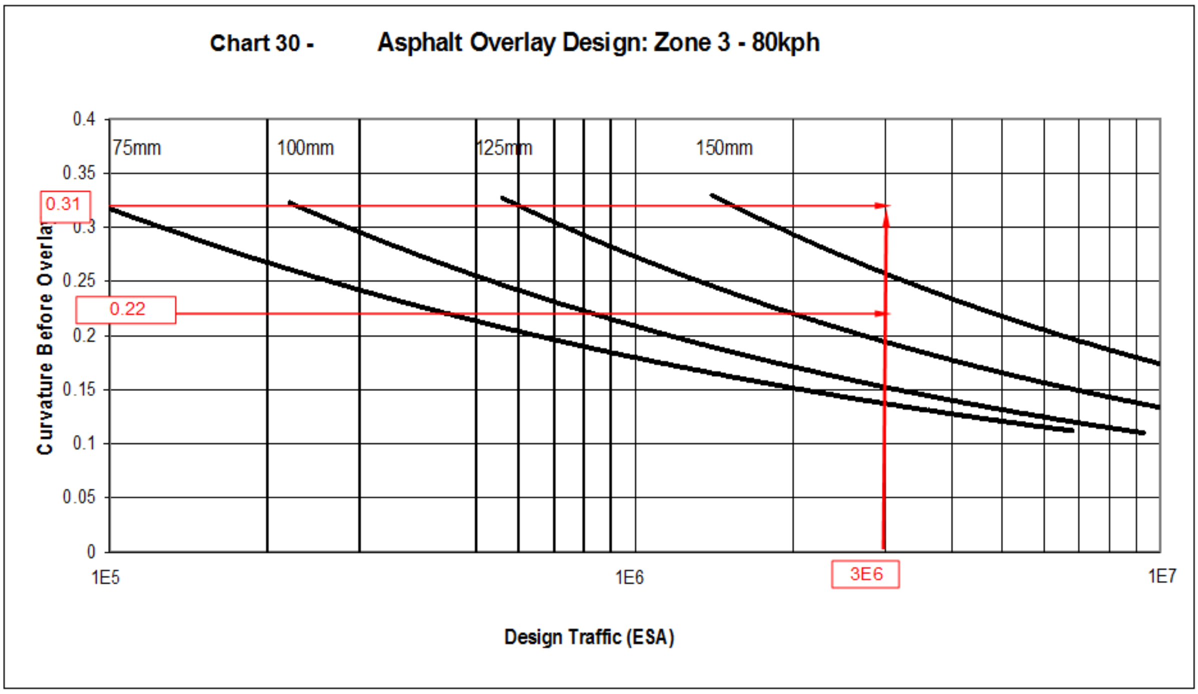


1. **Determine D tol using N(ESAs) and CBR of Subgrade.**
2. **Check thickness of asphalt required to protect Subgrade.**
3. **Using CFb and N(ESAs) obtain asphalt thickness for fatigue life.**

Table 5.3.4.1.4(c) – Example of a partial summary of asphalt overlay design using falling weight deflector deflection results and the deflection reduction method

Design life = 20 years														
DESAs = 3 x 10^6														
Existing 50mm asphalt wearing course														
Operating speed is 80 km/h for all elements														
WMAPT = 32°C, Temperature Zone 3														
Pavement temperature measured for IWP for sections 1, 3 and 4 = 17°C. For Section 2 it is 18°C														
Temperature correction for D_r: 1.09 for all cases														
Temperature correction for C_F: 1.29 for all cases														
Representative section				Season-al moisture corrected D_r (mm)	D_r temperature correct-ion factor	Temperature corrected D_r. (mm)	Adopted subgrade CBR (%)	D_{tol} (mm)	Overlay thickness required to control subgrade rutting (mm)	Seasonal moisture corrected $CF_{b\ r}$, before overlay (mm)	$CF_{b\ r}$ temperature correction factor	Temperature corrected $CF_{b\ r}$ before overlay (mm)	Overlay thickness required to control fatigue cracking (mm)	Thickness of overlay to be adopted (mm)
No.	Start chainage (m)	End chainage (m)	Length (m)											
1	3460	3610	150	0.089	1.09	0.10	17	0.58	<50	0.00	1.29	0.00	<50	50
2	3635	4610	975	0,842	1.09	0.92	17	0.58	110 or 115	0.17	1.29	0.22	135	135
3	4635	4886	251	0.557	1.09	0.61	25	0.50	<50	0.17	1.29	0.22	135	135
4	4910	6875	1965	1.049	1.09	1.14	13	0.64	140	0.24	1.29	0.31	>150	Cannot be determined with this method

Figure 5.3.4.1.4(c) – Example of how Figure C1.5(m) in Appendix C is used for the example presented in Table 5.3.4.1.4(c)



5.4 General mechanistic procedure

The GMP for pavement rehabilitation design is, in principal, identical to new pavements as given in the PDS and AGPT02, save for the differences described in this *Manual*. Chapter 4 of this *Manual* contains design considerations and design guidelines for foamed bitumen and lightly bound cementitious materials.

The properties of the insitu pavement materials need to be determined first. These properties are then used to characterise the strength of each of the existing pavement layers, using the methodologies outlined in this *Manual*.

The department's pavement rehabilitation methodology, which must be used for departmental projects, differs from AGPT05. The departmental method considers the strength of the existing layers (bound or granular) but limits the maximum moduli to be adopted for design based on the actual field conditions (for example, presence of significant cracking or minor cracking), laboratory test results and the results of back analysis derived from deflection survey results and other relevant information. Approved pavement design software, such as CIRCLY, is then used to simulate the response of the pavement, providing critical strains and determine:

- the allowable axle repetitions for the existing and new pavement layers; and/or
- the thickness of materials required for a given traffic spectrum.

The results of back analysis are an important input; it is important the designer understands the strengths and limitations of the technique.

The GMP cannot be used for pavements that do or will include a concrete layer; however, provided suitable and sufficient data are available, the GMP can be used for the thickness design for all other types of pavement rehabilitation treatments.

The GMP relies on 'linear layer elastic theory', an approximation of the extremely complex conditions of real pavement structures. Pavement materials are often inhomogeneous, anisotropic (Degree of Anisotropy) and have non-linear, stress-strain relationships (modulus is stress-dependent). The conditions at the interfaces are also not well understood.

In addition to elastic deformation, most pavement materials will experience viscous, visco-elastic and/or plastic deformations under stress.

The modulus of bituminous materials, such as asphalt and foamed bitumen treated material, are dependent on the rate of loading (AGPT02) and referred to as a dynamic (frequency-dependent) modulus. Traditional forward calculation methods, such as used by CIRCLY software, assume the load applied to the pavement is static (does not vary with time). For bituminous material, this approach tends to predict deflections significantly larger than would actually occur for a moving wheel load travelling at the operating speed of the pavement; if back calculation is carried out on the deflections produced by a dynamic load (FWD and TSD), incorporating a forward calculation assuming a static load, the resulting moduli will be overestimated.

One method of compensating for this is to adjust deflections based on the speed of the device used to create and measure the deflections (refer Section 2.13.10.4.9.2).

Alternatively, a dynamic loading model can be used for the forward calculation. The result of the dynamic forward calculation is a deflection time series (sometimes referred to as a deflection time history). For the FWD, a time series can be recorded for each geophone at sub-millisecond intervals, while a single time series at intervals of several milliseconds would be available from the TSD.

3D-Move software developed by the University of Nevada at Reno provides an example of dynamic forward calculation; however, the dynamic forward calculation usually takes a significant amount of time (*3D-Move* takes several minutes) to produce a single time series. This makes it unsuitable for direct use in dynamic back calculation of moduli, although research suggests the results for various pavement configurations can be accumulated and used to train machine learning algorithms, significantly reducing the time taken.

3D-Move was developed for a moving wheel load (for example, TSD) and is therefore not suited to dynamic simulation of the FWD device. Additionally, while allowing the user to specify values for the Poisson Ratio of each layer, the Degree of Anisotropy cannot be specified, and it is assumed an isotropic model has been used for all materials, making *3D-Move* incompatible with Austroads guidelines.

All FWD devices available in Australia can produce time series information; however, in most cases, this option must be activated prior to testing, slows down the survey and adds to costs.

For these reasons, dynamic back calculation has yet to be widely adopted.

5.4.1 Back analysis methodology

The back analysis of a pavement is an iterative process, requiring for each representative section:

- a deflection survey (deflection results) with testing conducted at an appropriate interval and in appropriate locations (for example, OWP), and/or
- a representative model of the insitu pavement (for example, material type, layer thickness, Poisson's Ratio and Degree of Anisotropy for each layer).

A stress level of 750 kPa is considered the appropriate level to use for design; however, to achieve this for a FWD or HWD, an 'odd' loading of 53 kN is required. When a FWD or HWD is used, a stress level of 707 kPa is normally targeted by using a load of 50 kN; however, by testing pavements using 40, 60 and 80 kN loads, there will be sufficient data to allow non-linear interpolation of back calculated moduli for a stress level of 707 kPa. For heavy duty pavements, including tests at 80 kN or 100 kN will increase the deflection values, improving the accuracy of the back-analysis results. Corrections to back-calculated moduli of granular materials may be required (see Section 5.4.1.3).

This information can be input into back analysis software to obtain estimates of the moduli of insitu materials and layers. Currently, Transport and Main Roads uses the ARRB *EfromD3* software package which includes an embedded version of CIRCLY; however, other software packages may be used if their compliance with Austroads guidelines (AGPT05) can be demonstrated. The software then calculates theoretical deflections with the aim of producing results matching field measured deflections (calculates theoretical deflections within a specified tolerance); there may be more than one combination of parameters to achieve this aim and there may not be a unique solution. The outputs include estimated elastic moduli for each pavement layer modelled. These estimates must then be adjusted or moderated for use in the design of the rehabilitation treatments. Once completed, the pavement model developed can be provided as input to forward calculation software such as CIRCLY so rehabilitation treatments can be designed.

The basic rule is always to review the back analysed data critically and apply engineering knowledge and judgement to the interpretation of the results.

5.4.1.1 Depth to bedrock

'The accuracy of FWD back calculated moduli depends heavily on knowledge of bedrock depth.' (*Determination of Bedrock Depth from Falling Weight Deflectometer Data* (Chen, 1999)) and both forward (for example, CIRCLY) and back calculation (for example, *EfromD3*) software provide for the entry of the depth to bedrock.

Apart from borehole logs, which are typically available only for bridge and tunnel locations, the depth to bedrock is seldom recorded in the department's pavement inventory database.

A variety of literature on the subject tends to suggest bedrock less than three metres from the surface of the pavement can significantly influence back calculated moduli.

Chen (Chen, 1999) suggests shallow bedrock (<1.5m) is likely to be present where the 40 kN deflection at an offset of 900mm from the centre of load (D_{900}) is less than 25.4 microns, and this should be used to identify sections of the pavement where the depth to bedrock should be further investigated.

5.4.1.2 Data irregularities

The need for experience in analysing materials and deflections, to ensure the back-analysis process yields the most accurate set of moduli for a given deflection bowl, cannot be overemphasised. Most deflection bowls are 'well behaved', forming a smooth sigmoidal (S-shaped) curve whose deflection decreases monotonically from the centre of load to the farthest offset from the load. Back analysis of such bowls will pose no problem; however, a certain number of irregular bowls will be encountered which may not be strictly monotonic (often referred to as non-decreasing) or contain a sudden change in deflection.

Irregular bowls are often caused by conditions contravening the continuity assumed in the partial differential equations of the forward calculation; for example, if deflection bowls appear more 'Z-shaped' than 'S-shaped', this can often be traced to poor longitudinal load transfer associated with transverse cracks or joins. If irregular bowls are back calculated, the results are likely to be unreliable and this tends to be indicated by a high fit error. The SCA method of Appendix A attempts to separate bowls into groups based on the similarity of their shape and magnitude, and irregular bowls are often the constituents of small bowl groups, groups that contain a single bowl.

Data irregularities may have several causes, including:

- localised pavement distress(es)
- variations in layer thicknesses
- changes in pavement configuration (for example, unidentified asphalt patches hidden under an overlying seal)
- the presence of bedrock (refer Section 5.4.1.1) or other stiff layers, and/or
- variations in moisture.

Effects of data irregularities can be addressed by recognising probable causes and adjusting during back analysis or, providing it can be justified, by omitting spurious or atypical results (for example, omit data obtained from tests conducted over a culvert close to the surface). The objective of applying adjustments to the pavement model is to yield values for the layer moduli consistent with the type and properties of the material, not just to fit the deflection bowl more closely.

The type of layers and their thicknesses and quality must be determined from extensive field investigations, coupled with field and laboratory testing. That is why a detailed pavement and subgrade investigation for each pavement rehabilitation project is vital.

While original design or historical information may provide some guidance on the likely nature of the existing pavement, such records should not be used as the sole indicator of pavement composition. Design details may have been altered during construction and not recorded or been affected by traffic and other environmental factors. Even ‘As Constructed’ plans may contain errors.

5.4.1.3 Stress dependency

The modulus of any unbound granular material is stress dependant. If an adjustment for stress dependency is required, the process of modulus adjustments is (*Analyses of a heavy-duty granular pavement using finite element method and linear elastic back-calculation models* (Vuong, 1988)):

1. Determine the in-service mean principal stress for each granular layer and sublayer by modelling the existing pavement in CIRCLY. Input data required to do this include the existing pavement configuration; design moduli based on the elastic characteristics for each pavement layer and sublayer (for example, moduli derived from consideration and moderation of test and back analysis results); the type of deflection device; and the deflection test load. Principal stresses are determined directly under the load at the mid-depth in each layer and sublayer. The in-service mean principal stress for granular layer and sublayer is calculated using Equation 5.4.1.3(a). View the ‘*.CLO’ file from within CIRCLY to obtain the principal stresses. In calculating the in-service mean principal stress, if the value of any stress is negative (tensile), its value is set to zero for the computation.

Equation 5.4.1.3(a) – Calculation of the mean principal stress

$$Mean\ Principal\ Stress = \frac{\sigma_{xx} + \sigma_{yy} + \sigma_{zz}}{3}$$

2. Determine the measurement mean stress for each granular layer and sublayer by modelling the rehabilitated pavement in CIRCLY. Input data required to do this include the pavement configuration of the rehabilitated pavement; design moduli based on the elastic characteristics for each 'new' (for example, stabilised or modified virgin material) and existing pavement layer and sublayer (to remain); the in-service loading; the in-service moduli of the existing asphalt layers (to remain); and the in-service moduli for the existing granular and subgrade layers and sublayers (to remain). Determine the measurement mean stress for all granular layers and sublayers as described previously.
3. Adjust the moduli of granular layers and sublayers using Equation 5.4.1.3(b) which is based on results of dynamic triaxial tests on undrained samples.

Equation 5.4.1.3(b) – Adjustment to determine in-service modulus of granular layers and sublayers

$$In - service\ modulus = modulus\ from\ back\ analysis\ \times A$$

where

$$A = \left[\frac{In - service\ mean\ stress}{Measurement\ mean\ stress}\right]^K$$

with a value of K being selected from within the range of 0.3 (for low-quality granular sub-base material) to 0.5 (for high quality granular base material).

5.4.1.4 Adjustment of deflection parameters for speed of loading

Refer sections 5.3.2.9 and 2.13.10.4.9.2.

5.4.1.5 Adjustment of deflection parameters for moisture

In the strictest sense, the correction of deflection parameters for moisture should be made before back analysis commences. It can be assumed the same correction factor can be applied to each bowl parameter (for example, to D_0 through to D_{900}) as every part of the bowl is affected; otherwise, the adjustment is applied as described in Section 5.3.2.6 and Chapter 2.

5.4.1.6 Adjustment of deflection parameters for temperature

Refer Section 2.13.10.4.9.1.

5.4.2 Characterising existing pavement and subgrade layers

The department's method considers the strength of the existing layers (bound or granular) but limits the maximum moduli assigned to pavement layers for design based on the actual field conditions (for example, whether cracking is significant or minor), test results and the results of the back analysis. Part of the method to be used to moderate moduli, selected elastic characteristics and the maximum moduli to be adopted for design are shown in figures 5.4.2.1.1(a) and 5.4.2.1.1(b). Additional factors to be considered are discussed in the following sections.

5.4.2.1 Assigning layer modulus values

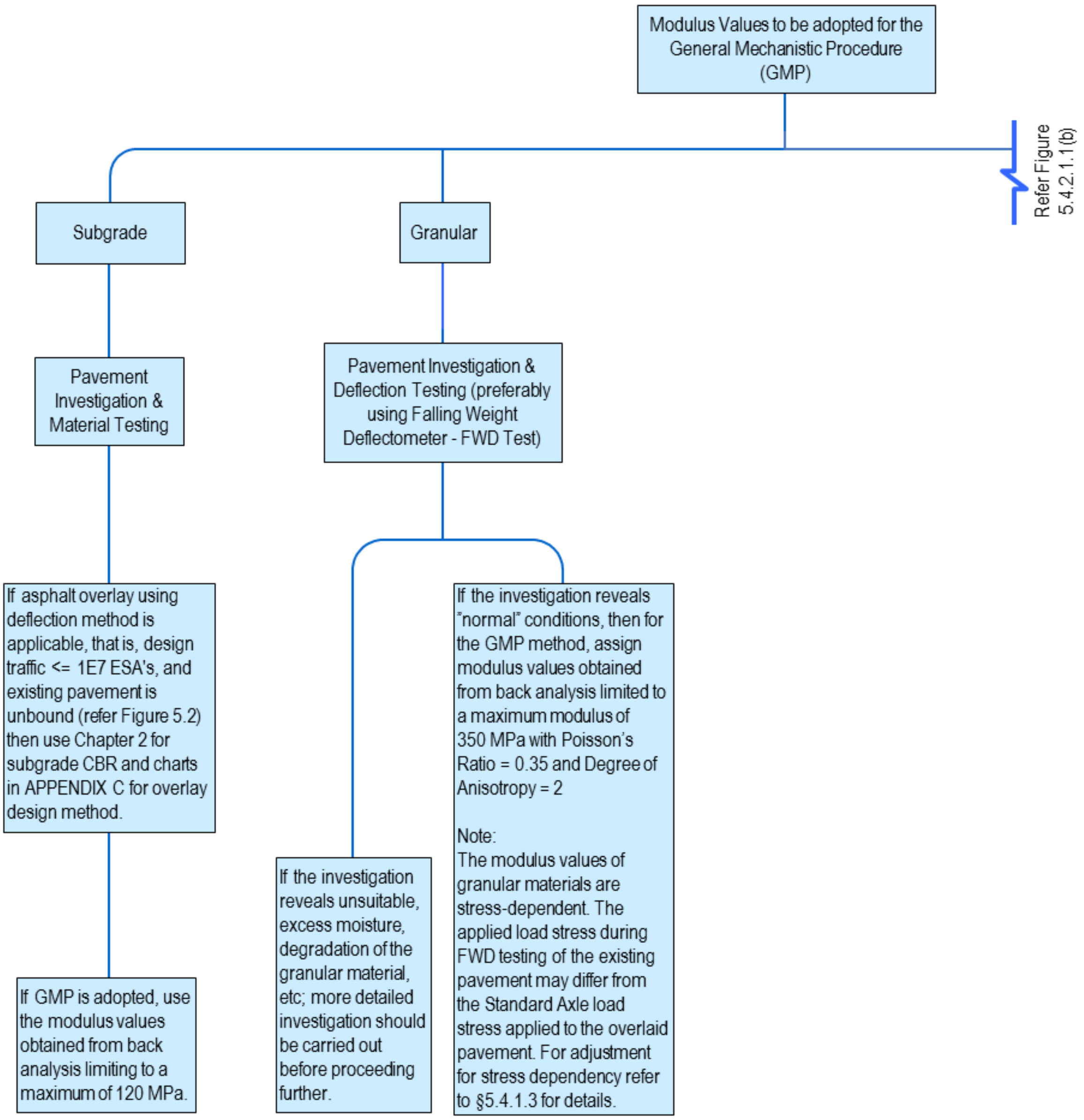
The following guidelines, in addition to those given in figures 5.4.2.1.1(a) and 5.4.2.1.1(b), are to be used when assigning design moduli to pavement layers. This must be done before design of the rehabilitation treatment commences.

5.4.2.1.1 Subgrade

The subgrade shall be modelled as per the PDS and AGPT02, except the maximum value to be adopted for the subgrade is 120 MPa.

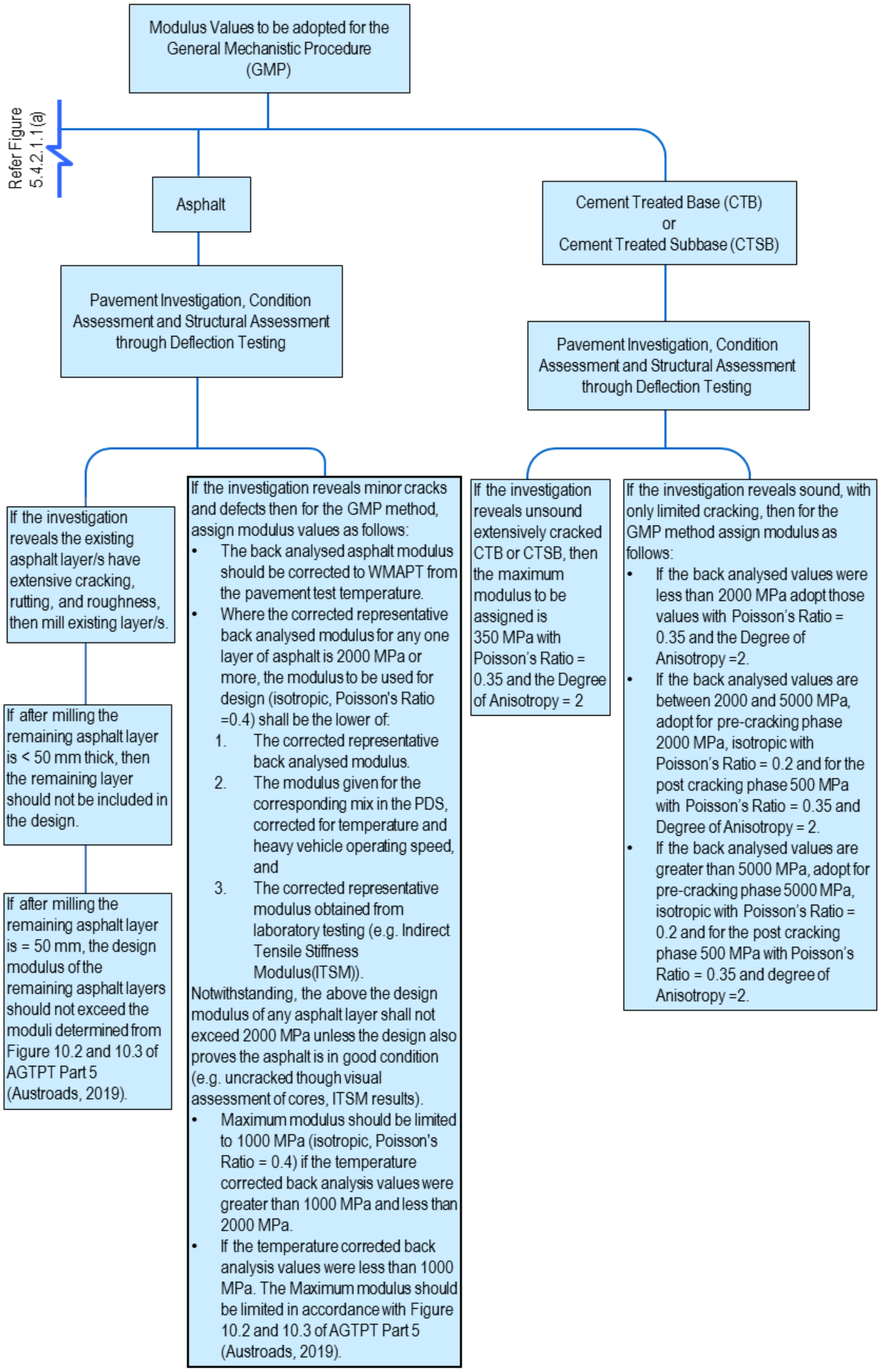
Note: The PDS takes precedence over AGPT02.

Figure 5.4.2.1.1(a) – Method of moderating moduli, selected elastic characteristics and maximum moduli for design, Part 1



Note: Refer also to Figure 5.4.2.1.1(b))

Figure 5.4.2.1.1(b) – Method of moderating moduli, selected elastic characteristics and maximum moduli for design, Part 2



Note: Refer also to Figure 5.4.2.1.1(a)

5.4.2.1.2 Existing unbound granular layers

The moduli of granular materials have been observed to be stress-dependant. Reference must be made to Section 5.4.1.3 for the procedure to obtain an adjustment factor and figures 5.4.2.1.1(a) and 5.4.2.1.1(b) for how adjusted moduli are to be moderated. For each layer, the design modulus is to be determined:

- where the total thickness of overlying bound material is less than 100mm, the lower of the following is to be adopted for design:
  - the corrected, moderated modulus, and
  - the maximum modulus calculated from supporting conditions and sublayering, and
- where the total thickness of overlying bound material is greater than 100mm, the lower of the following is to be adopted for design:
  - the corrected, moderated modulus
  - the maximum modulus calculated from supporting conditions and sublayering, and
  - the limit given in Tables 5.4.2.1.4(a) or 5.4.2.1.4(b), whichever is applicable.

5.4.2.1.3 Existing cementitiously stabilised layer

The design modulus for any cementitiously stabilised layer (for example, CTB or CTSB) should be determined as given in figures 5.4.2.1.1(a) and 5.4.2.1.1(b).

If the cementitiously stabilised layer is in sound condition with only limited cracking, a pre-cracked life can be included. In this case, the post-cracked modulus for any cementitiously stabilised layer should be limited to the lower of the back-analysed modulus or 500 MPa, with a Degree of Anisotropy of 2.0, no sublayering and a Poisson's Ratio of 0.35.

5.4.2.1.4 Existing asphalt layer

In addition to the guidelines given in figures 5.4.2.1.1(a) and 5.4.2.1.1(b):

- The asphalt modulus should be moderated for the pavement temperature at the time of the testing as described in Section 5.4.1.6.
- If the back-analysed modulus of an asphalt layer is less than 2000 MPa but greater than 1000 MPa, referring to figures 5.4.2.1.1(a) and 5.4.2.1.1(b), it is recommended the actual back analysed modulus be adopted for design. The Poisson's Ratio is 0.4 and the layer is considered isotropic; this means, for example, if the back analysed modulus is 1500 MPa, then the value of S_{mix} is taken to be equal to 1500 MPa. The volume of bitumen V_b is assumed to be 11% if the existing asphalt layer has a polymer modified bitumen and 10% if the existing asphalt layer has a Class 320 binder.
- If the back-analysed modulus of an asphalt layer is below 1000 MPa, the design modulus value should be determined using Figure E.12 in Appendix E of AGPT05. It is modelled as an asphalt layer, except no performance criteria are used for it.

Table 5.4.2.1.4(a) – Maximum vertical moduli for unbound granular layers where the total thickness of overlying bound material is from 100mm to less than 250mm

Laboratory CBR of the existing granular layer or sublayer (%)	Maximum vertical modulus (MPa) for the:			
	Topmost granular sublayer	Granular layer/sublayer second from the top of the granular layers	Granular layer/sublayer third from the top of the granular layers	Granular layer/sublayer fourth from the top of the granular layers
80	250	200	150	120
50	225	200	150	120
45	200	150	120	120
40	150	120	120	120
35	150	120	120	120

Bound material is modified material, stabilised material and asphalt. The total bound thickness is calculated by adding the total thickness of asphalt, if any, to the total thickness of any stabilised and modified materials.

Table 5.4.2.1.4(b) – Maximum vertical moduli for unbound granular layers where the total thickness of overlying bound material is 250mm or more

Laboratory CBR of the existing granular layer or sublayer (%)	Maximum vertical modulus (MPa) for the:			
	Topmost granular sublayer	Granular layer/sublayer second from the top of the granular layers	Granular layer/sublayer third from the top of the granular layers	Granular layer/sublayer fourth from the top of the granular layers
80	150	150	150	120
50	150	150	150	120
45	150	150	120	120
40	150	120	120	120
35	150	120	120	120

Bound material is modified material, stabilised material and asphalt. The total bound thickness is calculated by adding the total thickness of asphalt, if any, to the total thickness of any stabilised and modified materials.

6 Comparison of alternative rehabilitation strategies

6.1 Introduction

Often more than one option is a viable solution. In such cases, options need to be compared (for example, by identifying and comparing costs and benefits). This comparison includes:

- the capability and/or capacity of industry
- financial considerations or constraints
- the technical merits of each option (as discussed in previous chapters), and
- an economic comparison (refer to Section 6.3).

Where appropriate, other relevant factors (for example, environmental impact, refer to Section 6.4) should also be considered.

The comparison of rehabilitation options should be undertaken as discussed in the PDS and AGPT05: Chapter 6 of this *Manual* takes precedence.

There are many factors that may influence the choices made for projects: the pavement cannot always be considered discretely or in isolation (for example, refer to Section 6.4). The pavement rehabilitation designer may not:

- have knowledge of, or access to, information about all such factors
- have the expertise to evaluate factors not related to pavement rehabilitation design, and/or
- be responsible for considering all factors.

On a large project with a multi-disciplinary team, the pavement rehabilitation designer may not have access to information about the cost of upgrading structures / bridges to maintain vertical clearances, the adequacy of the surface drainage of the existing or rehabilitated pavement, or local issues such as those related to noise and vibration.

Given these, the customer needs to determine the appropriate level and type(s) of analysis and consider the alternatives in a holistic fashion (for example, by using multi-criteria analysis (MCA), see Section 6.4.1).

6.2 Scope

This *Manual* is focused on pavement rehabilitation investigation and design; this chapter is limited to providing guidance about how to compare pavement rehabilitation options, based primarily on pavement rehabilitation factors (a simplified method of analysis is presented). This is limited to comparing:

- the technical merits of each option (refer to previous chapters), and
- the WOLC of each option, calculated using the present worth of costs (PWOC) method for pavement-related costs (as described in this chapter).

Note: The WOLC is calculated over the same assessment period for all options.

Given the simplicity of the method of economic comparison presented in this chapter, designers and the customer should consider carefully the merit of the results obtained from its use. It may not be appropriate to use any such results as the only determinant of the most appropriate option. Advice should be sought and/or relevant literature should be referenced if an analysis more detailed than described in this chapter is warranted (for example, for major projects or projects involving high traffic volumes).

Some other methods are discussed briefly for the reader's information (for example, sections 6.3.3.1, 6.3.3.2.1 and 6.4.1); detailed guidance with respect to them is beyond the scope of this *Manual*.

6.3 Economic comparison

A prime consideration when comparing various alternative rehabilitation options is their WOLC, or lifecycle cost. It is used, in one form or another, by most road authorities to compare pavement rehabilitation options. The WOLC ownership and user costs necessary to provide a serviceable pavement over the assessment period include the costs associated with construction, maintenance, rehabilitation, traffic disruption / road user delay costs; and the salvage value. The option(s) with the lowest WOLC are often preferred; however, a decision cannot be made based on the results of a WOLC analysis alone. The decision must also take other relevant factors into consideration (for example, Section 6.4), including:

- the relative importance of initial capital expenditure compared to future expected expenditures
- maintenance requirements
- road safety considerations
- the scale of the project
- noise and spray effects
- the potential for differential settlement
- traffic management during construction
- the method of rehabilitation, and/or
- the option which best suits the requirements of the road authority / owner.

An alternative that had a low WOLC but required unavailable equipment or personnel would be a poor choice. An alternative with a moderate overall WOLC beyond the project's budget would be an equally poor choice. The alternative chosen must be a viable one.

The discussion in Section 6.4 provides more examples of factors to be considered.

6.3.1 Key principles

Four basic principles of economic analysis are:

- economic analysis is not intended to provide a decision, it is merely a tool to aid the decision-making process
- any economic analysis must consider all viable alternatives
- the same assessment period must be used for all alternatives (see Section 6.3.2), and/or
- where possible, costs, including road user costs, as well as benefits, should be included in the economic analysis.

Modelling and costing road user costs is complex and beyond the scope of this *Manual*.

6.3.2 Assessment period and design period

The assessment period is the length of time for which comparisons of the WOLC are to be made. It should be the same for all options evaluated and should not be less than the longest effective life of the options considered; for example, if option A has a 20-year design life and option B is a staged 10-year design, the assessment period would be 20 years for all options. If option C with a 40-year design life was included, the assessment period would be 40 years for all options

The assessment period does not always equal the design period; for example, rehabilitation designs may be developed for design lives of 10 and 20 years but, in this case, the assessment period, to compare all options, must be 20 years.

For new pavements, the PDS recommends a minimum analysis period of 40 years or the life of the alignment, whichever is the lesser. For rehabilitation projects, assessment periods of 10 (for example, for low-volume roads) to 40 years may be more appropriate. The designer must confirm the assessment period to be used with the customer.

When selecting the assessment period, the expected life of the road should be considered; for example, if a complete realignment is expected 15 years after opening, the assessment period should not exceed 15 years from opening.

6.3.3 Methods

Keeping the key principles outlined in Section 6.3.1 in mind, there are a number of methods available for performing an economic analysis. These include:

- equivalent annual cost which divides all costs into equal payments over the analysis period
- PWOC which discounts future costs to present costs (this is different to determining a net present value (NPV))
- internal rate of return which determines the rate at which costs, and benefits are equal
- cost effectiveness, which is useful when significant non-monetary outputs are involved
- benefit-cost analysis (see Section 6.3.3.2): this may include the calculation and consideration of one or more of the following items:
  - a benefit-cost ratio (BCR), which is the ratio of present worth of benefits to PWOC, and/or
  - the NPV (see Section 6.3.3.2.1).

These methods are further described in the *Australian Transport Assessment and Planning Guidelines* (ATAPG).

Typically, a simplified economic comparison is used by the pavement rehabilitation designer. This entails calculating WOLCs via the PWOC method to compare pavement rehabilitation options (where the only costs considered are those for constructing, rehabilitating and maintaining a pavement; road user costs, for instance, are ignored). It is a relatively simple approach and most relevant where the effect of various pavement rehabilitation options on other aspects (for example, on road safety) is the same or very similar. More complex analyses may be warranted (refer to Section 6.2).

6.3.3.1 Whole-of-life costs using the present worth of cost method

The purpose, in the economic comparison described in this section, is to evaluate alternatives primarily according to the criterion of minimum total WOLC, calculated using the PWOC method described in this chapter.

The PWOC method described in this chapter only considers the construction, rehabilitation and maintenance costs of pavement rehabilitation options / treatments. The method allows for both routine and periodic maintenance and rehabilitation costs which will occur during the assessment period.

Ignoring any maintenance costs between the present and the year of opening, the WOLC can be calculated using the PWOC formula given in Equation 6.3.3.1 and including all costs during the assessment period (in most cases, the maintenance costs would be the same for each option and, in this case, ignoring them would not lead to a different conclusion). When evaluating options using their WOLCs, the lower the WOLC, the better, in economic terms.

Equation 6.3.3.1 – Present worth of costs formula

$$PWOC = C(1+r)^{-x} + \sum_{i} M_i(1+r)^{-(y_i+x)} - S(1+r)^{-(z+x)}$$

where:

$PWOC$ =	WOLC when PWOC of all costs within assessment period are included
C =	cost of initial construction
x =	number of years from the present to the year of opening (provided by the customer), zero if the year of opening is the present year
M_i =	cost of ith maintenance and/or rehabilitation work for all such work which takes place within the assessment period
r =	real discount rate expressed (for example, for a real discount rate of 3% per annum, 0.03 is used in the formula, see Section 6.3.9)
y_i =	number of years from the year of opening to the ith maintenance or rehabilitation work, within the analysis period
S =	salvage value of pavement at the end of assessment period expressed in terms of present value
z =	assessment / analysis period measured from the year of opening, in years.

To estimate the WOLC of options using the methodology described in this chapter, the principle elements to be considered are the:

- opening (or base or evaluation) year
- pre-construction costs
- initial construction costs
- staging costs, if any
- routine and periodic maintenance costs
- (structural) rehabilitation costs
- salvage value
- real discount rate, and
- assessment / analysis period.

Calculation of the WOLC of pavement rehabilitation options can be completed using spreadsheet software.

6.3.3.2 Benefit-cost analysis

An alternative to the use of the PWOC method is the use of benefit-cost analysis, or cost-benefit analysis. It is used in the department for road infrastructure projects and is an input to the evaluation and comparison of projects. It involves the calculation of BCR and, where this is >1, there is economic benefit and merit in undertaking the project. Options for pavement rehabilitation are mutually exclusive (if option A is constructed, options B and C are not constructed). This means the use of an absolute BCR can be misleading when comparing pavement rehabilitation options for the same section of road (incremental BCR analysis, however, may be used to rank options; for further details, refer to the ATAPG). BCR can be used as an input when deciding which independent projects are undertaken (for example, when comparing the benefit of doing works on road section X as opposed to section Y).

6.3.3.2.1 Net present value

NPV analysis is a form of benefit-cost (economic) analysis. It requires the calculation of costs and benefits accruing from each option.

> *Net present value is an absolute measure equal to discounted benefits (user + non-users) over the life of the project less discounted costs* (ATAPG).

A positive NPV indicates an option or project has an economic benefit and, hence, economic merit.

Based solely on NPV analysis, the option with the highest NPV would be selected.

NPV is not used in this *Manual*, however, partly because quantifying benefits can be difficult and may be outside a pavement rehabilitation designer's area of expertise.

6.3.4 Preconstruction costs

Preconstruction costs include the costs associated with any design, development of contract documentation, tendering and contract administration. This may be particularly important for staged options where new contracts, designs (for example, for changes to drainage) and so on will need to be developed for future interventions.

6.3.5 Capital (initial construction) costs

Unit costs for alternative pavement rehabilitation designs will vary widely, depending on factors such as degree of competition, locality, availability of suitable resources (for example, plant, personnel and material), types of resources employed and how they are employed (for example, product standards required in standards and specifications) and the scale of the project. The department has records of unit costs from past Transport and Main Roads (and Queensland Transport and Queensland Main Roads) road infrastructure projects. For departmental works, these can be used to estimate the costs of the options being compared. If this information is not available, costs may be assessed from experience. There are other less obvious costs which warrant consideration.

Some alternatives will require more excavation (cut) or more fill (for example, where surface levels are fixed by extraneous constraints), may interfere with public utility plant (services), or require more shoulder material than others; for example, a significant saving in shoulder material (for example, with respect to excavation and supply) can be made, for equivalent performance, by using a full-depth asphalt or pavement, with a modified or stabilised layer (these will be thinner, compared to an unbound granular pavement). Consequently, comparing the cost per square metre of the pavement alone is often misleading. Comparative costs should be expressed as a cost per kilometre for the treatments for the full carriageway formation (cross-section) or as a total cost per project. The costs should include all ancillary works, such as shoulder improvements and the relocation of public utility plant (services). Alternatively, the total cost of the project, including overheads, can be used. Where costs associated with overheads and 'non-pavement' activities differ between options, these should also be included.

Overheads and other costs, which are not necessarily included in the pavement unit costs, may vary with each option. Such costs include those associated with:

- provision for traffic/traffic management
- wet weather
- establishment and disestablishment
- supervision
- overheads
- relocation of public utility plant (services)
- testing, and
- economies of scale.

These are discussed further in the following sections.

In many cases, designers may not have sufficient information to make an accurate comparison of the construction costs of various alternatives and, in such cases, it may be desirable to call for both conforming and non-conforming (alternative) tenders.

6.3.5.1 Provision for traffic / traffic management

Alternatives which take longer to construct, have more complicated traffic management plans, or have to be constructed under traffic usually have a higher cost. Some alternatives can be built under traffic while others may require a side track. In some road corridors, side tracks may not be possible which will affect the cost and practicality of alternatives.

6.3.5.2 Wet weather

The costs associated with dealing with wet weather will need to be considered (for example, to rework or dry materials, providing bituminous seals between asphalt layers). In wet climates, these costs can be very significant and often unpredictable. Alternatives using bound material tend to be quicker to build and may help avoid costs associated with reworking; however, the use of bound materials may introduce other costs (for example, the cost of providing bituminous seals between asphalt layers likely to be exposed to rain, refer to the PDS).

6.3.5.3 Establishment and disestablishment

Establishment and disestablishment costs, such as those for the transport and set up of plant and so on, can vary between alternatives. Specialised equipment (for example, for producing, mixing, laying or compacting) may be required for some options and not others. Options involving a number of different types of pavement / materials are likely to involve higher plant establishment and disestablishment costs.

6.3.5.4 Supervision and overheads

Supervision and overhead costs are time-dependant; alternatives which take longer to construct tend to cost more to supervise. Longer construction periods involve higher overheads costs, such as those for camp maintenance, power, phone and project cost management.

6.3.5.5 Relocation of public utility plant (services)

Pavement alternatives with different total thicknesses may involve different costs in relocation of public utility plant (services) such as electricity or communications cables, sewerage pipelines, water or gas supply pipelines and drainage installations. These relocation costs may be extremely high, particularly in urban areas. In such situations, the use of full depth asphalt, special high-strength cementitiously stabilised materials or concrete may be required to keep the pavement as thin as possible.

6.3.5.6 Testing

Different pavement types will differ in their demand for testing, contributing to cost differentials. The use of new or unfamiliar materials may require an extensive pre-construction testing program, also resulting in cost differences between options. In the latter case, a considerable expense may be incurred (it also requires an adequate lead time to allow the testing required to be completed during design and/or before construction).

6.3.5.7 Economies of scale

There are significant economies of scale in some pavement operations. Economies of scale may also be realised by combining projects.

6.3.6 Staging and other costs associated with pavement rehabilitation

As mentioned in Chapter 3, a staged approach may result in a lower initial (capital) cost but result in a higher WOLC.

Note: A staged approach is one in which at least one of the future interventions is structural).

This is because of the future costs associated with:

- constructing structural interventions during the analysis period
- traffic management during future interventions / works
- reconstruction of bridges and/or pavements to provide the required vertical clearances
- raising or reconstructing road safety barriers
- raising or reconstructing kerb and kerb and channel, or other roadside furniture
- modifying or reconstructing drainage systems and structures, and/or
- correcting the surface shape to satisfy geometric requirements (for example, for widening where crossfall and superelevation may need to be changed and/or a crown may need to be introduced or changed).

Where possible, these costs should be minimised; for example, some road safety barriers can be constructed to allow, to a limited extent, for future overlays.

When calculating the WOLC for each option, **all** future costs that can be estimated practically need to be estimated and included; for example, the cost of raising or reconstructing road safety barriers can be included but the costs associated with amount of greenhouse gases generated may be difficult to estimate and include).

Reference should be made to Chapter 3 for a more detailed discussion on staged construction.

6.3.7 Maintenance and rehabilitation costs

Refer to AGPT05 Section 8.3.2 for a discussion about subsequent maintenance and rehabilitation costs.

6.3.7.1 Typical maintenance requirements

6.3.7.1.1 Granular pavements with bituminous seals

Granular pavements with bituminous sprayed seals require routine maintenance from early in the life of the pavement. Resealing to maintain a competent seal is required every five to 10 years. Some reduction of the reseal frequency, particularly on roads with lower traffic volumes, can be achieved by applying periodic surface enrichments. Double (two)-coat bituminous sprayed seals typically perform adequately for six to 12 years.

6.3.7.1.2 Asphalt surfaced pavements

Asphalt surfaced pavements require less routine maintenance during their life than do granular pavements with bituminous sprayed seals, but unit costs for repairs may be higher. Because of oxidation of the bitumen in the asphalt surfacing layer and the resultant wear or cracking, the milling and replacement of the surfacing layer or resurfacing is normally required every seven to 12 years.

For full-depth asphalt and deep-strength asphalt, High Intensity, Low Intervention-type pavements without staging milling and replacement may be preferred as surface heights will not be affected.

6.3.7.1.3 Pavements with cementitiously modified / stabilised layers

Pavements with cementitiously modified / stabilised layers close to the surface require increased maintenance to deal with reflective cracking (for example, seal cracks). Unsealed cracks will allow water to ingress into the pavement and reduce pavement life.

6.3.7.1.4 Pavements with concrete layers

Pavements with concrete layers and a concrete surfacing require less maintenance than other pavement types; however, concrete pavements are more sensitive to inadequate or incorrect design, or unanticipated increases in traffic loading. Where this is the case, maintenance costs can be significant and unanticipated.

6.3.8 Salvage or residual value

The salvage value of unbound granular and bitumen stabilised materials is high while that of asphalt (unless recycled) and cementitiously stabilised materials may be low if they are severely cracked or otherwise distressed. In cases where the existing pavement must be removed, the salvage value can be negative (there is a cost rather than a salvage value).

Where asphalt fatigue in a flexible granular pavement is the dominant distress mode, the other pavement layers may be quite sound. Provided the asphalt is in reasonable condition, the salvage value is high. In this case, rehabilitation using an asphalt overlay is one appropriate option.

Sound layers within a full-depth asphalt or deep-strength asphalt High Intensity, Low Intervention-type pavement will be high, provided all underlying layers are in sound condition.

The salvage value can vary between options.

Refer also to AGPT05 Section 8.3.3 for further discussion about salvage costs.

6.3.9 Discount rate

Interest rates and discount rates are different. Interest rates are associated with borrowing and lending of money; discount rates are used to express future costs to costs in present-day terms.

An appropriate (real) discount rate must be selected and used to calculate the PWOC of future expenditure. This rate is expressed in real terms (excludes inflation). For departmental projects, the discount rate should be obtained from Queensland Treasury before analysis begins. Real discount rates as low as 4% have been used when evaluating alternative strategies. It is desirable to carry out a sensitivity analysis with the discount rate varying between 4–10%.

Refer to AGPT05 Section 8.3.4 for further discussion about real discount rates.

6.3.10 Road user costs

Road user costs are the most complex of the costs considered in a WOLC analysis and are excluded from the simple economic comparison methodology described in this chapter.

Different pavement rehabilitation options or strategies may result in different roughness profiles (for example, initial roughness plus increases in roughness over time). This can affect road user costs.

Refer to AGPT05 Section 8.4 for further discussion about road user costs.

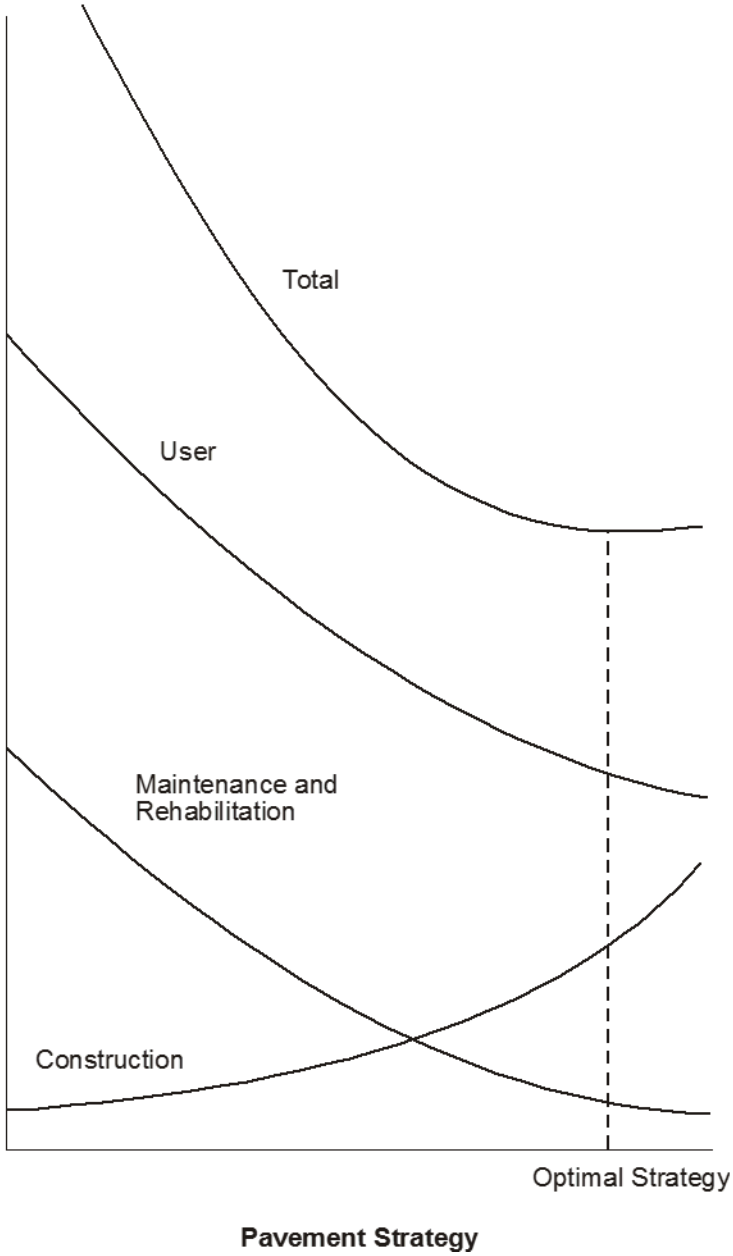
6.3.11 Optimal strategy

The optimal strategy can depend on a number of factors.

6.3.11.1 Economics

In economic terms, the optimal pavement strategy is one with the lowest WOLC (see Figure 6.3.11.1). This is the case when making comparisons in isolation on a particular project. AGPT05 Appendix H provides an example of an economic comparison; however, the economics of options may be only one of the aspects considered when choosing an option (see Section 6.3 of this *Manual*).

Figure 6.3.11.1 – Analysis of pavement strategies using discounted costs



Source: Derived from *Optimal Rehabilitation Frequencies for Highway Pavements* (Markow, 1985)

6.3.11.2 Network considerations

The method of economic comparison described in this chapter focuses on selecting the option with the lowest WOLC. Using this criterion alone means an option with a relatively high capital cost is identified as the best; however, the effect of constructing such options on the state of the road network as a whole should be considered.

Adopting options with a relatively high capital cost may not be the most effective way to improve the state of the network as a whole. Doing so would compromise the capacity to improve other / more links in the network and the ability of the road authority to raise the overall standard of the network. This applies even if such options have the lowest WOLC.

The overall state of the network can deteriorate when low cost capital works, requiring repeat interventions at relatively short intervals, are undertaken.

A balance must be sought between what is best at the project and network levels, while considering the available road funding. The use of 'what if' scenario analyses, using a pavement management system, may be used to inform decisions.

6.3.12 Sensitivity analysis

When comparing options, a sensitivity analysis may be warranted. It helps assess the effect of uncertainty and identify the variables most critical in determining the outcome of an evaluation. This may include sensitivity to changes such as discount rates (as described in Section 6.3.9), the timing and cost of future interventions, and changes in traffic growth. Austroads report *Economic Evaluation of Road Investment Proposals: Risk Analysis* (AP-R203) and ATAPG provide guidance on the factors and limits to be considered.

A sensitivity analysis must be done in a systematic way with the results presented clearly.

6.3.13 Risk

Risks should be managed in accordance with departmental polices and documents (for example, the Transport and Main Roads *Risk Management Framework* (RMF) and *OnQ Project Management Framework* (OnQ). Depending on the mitigating measures employed, how risk is managed may influence the cost of an option.

6.4 Other considerations

Interrelationships or interdependencies need to be considered (for example, effect of geometry on surfacing choice) and may be more important than the economics of the option(s). Factors to be considered include:

- effect of options on future road operation and vice versa (for example, with respect to geometry and vertical clearances)
- risks associated with options and mitigating measures (for example, with certain pavement materials or techniques, accuracy of traffic forecasts, settlement)
- types of road users, including cyclists, pedestrians and motorcyclists, and the effects of options on them
- project constraints (for example, structural capacity of existing bridges, budget)
- availability of plant, personnel and materials
- road safety (such as performance / effect on crash rate during and after construction)
- environmental effects, such as:
  - use of scarce recourses
  - use of recycled water or materials
  - noise (during and after construction)
  - effects on flora, fauna and (aboveground and underground) water quality
  - vibration (during and after construction), and
  - air quality (during and after construction).
- geometry effects on pavement rehabilitation design or choices (for example, OGA may be required where aquaplaning is an issue due to superelevation)

- availability of current and future funding
- the degree of certainty the required amount of future funding will be committed at the required time (in terms of DESAs), particularly for staged options
- competing network priorities
- community opinion / feedback, including that of adjacent or adjoining landholders
- effect on amenity, and/or
- political aspects.

Some of these may be able to be costed in part or in total and included in an economic analysis.

As discussed in Section 6.2, detailed guidance about how to consider all relevant factors is beyond the scope of this *Manual*; however, for the reader's information, one tool that may be used to consider multiple factors is discussed briefly.

6.4.1 Multi-criteria analysis

One tool that allows comparison of alternatives is MCA, sometimes also known as multi-criteria assessment or multi-criteria evaluation. Its use may be warranted in some cases; however, its results may not be the only input into the decision-making process. MCA is a decision support tool rather than a decision-making one.

There are various MCA methods. The one described here is a simple weighting method. For detailed guidance with respect to MCA, relevant literature should be referenced and/or specialist advice should be sought.

When using the MCA approach, assessment criteria are developed. This may be on a project-by-project basis. Each criterion is given a weighting, based on their importance on influencing the final outcome:

- a value (weighting) is assigned to each criterion so the total of the weightings equals one (or 100%)
- each option is given a score, rating or rank for each of the criterion
- for each option, the weighting is multiplied by the score, rating or rank, and
- the total for each option is determined.

Options can then be ranked and/or compared. A simplified, illustrative example is shown in Table 6.4.1, using rankings.

Qualitative and/or quantitative measures can be used to assign scores, ratings or rankings. Often, there is some subjectivity involved in determining the criteria and assigning the weightings and scores. The acceptance of the framework, weightings, scores, and so on can be enhanced by including stakeholders in the process of determining them (for example, through facilitated workshops, including internal and external stakeholders).

Table 6.4.1 – Simplified, illustrative example of multi-criteria analysis calculations using ranking

[1]Criteria	**Weighting**	**Options**					
		1		**2**		**3**	
		[2,3]Rank	**[2]Score**	**[2,3]Rank**	**[2]Score**	**[2,3]Rank**	**[2]Score**
Economics (lowest WOLC)	0.6 (60%)	2	1.2 (= 0.6 x 2)	3	1.8 (= 0.6 x 3)	1	0.6 (= 0.6 x 1)
Minimising increase in road surface height	0.2 (20%)	2	0.4	1	0.2	3	0.6
Minimising road traffic noise	0.1 (10%)	1	0.1	3	0.3	2	0.2
Minimising impact on community during construction	0.1 (10%)	1	0.1	2	0.2	2	0.2
Total	1 (100%)		1.8		2.5		1.6
[4]Overall rank		2		1		3	

Notes:

[1]Criteria are for illustrative purposes only. Criteria used in any MCA should be determined on a case-by-case basis.

[2]Values are for illustrative purposes only. Values used in any MCA should be determined on a case-by-case basis.

[3]In this example, 3 is highest and 1 is lowest.

[4]Based on the MCA, Option 2 would be selected for this example.

Appendices

Appendix A – Statistical cluster analysis of deflection bowls

A1 Introduction

Data will be summarised as an element of a decision support framework.

A significant proportion of summarisation procedures involve separating data into groups.

Where univariate (single variable) data are involved, it may be a simple task to separate the results into groups, categories or bins based on preselected threshold levels or discriminants; however, the example presented in this document deals with the more challenging case where groups are required to be identified from multivariate data (many variables).

This document describes how project-level FWD bowls can be separated into groups but the principles used are applicable to data from any deflection measurement device, such as Benkelman Beam, deflectograph, TSD or process that produces multivariate data.

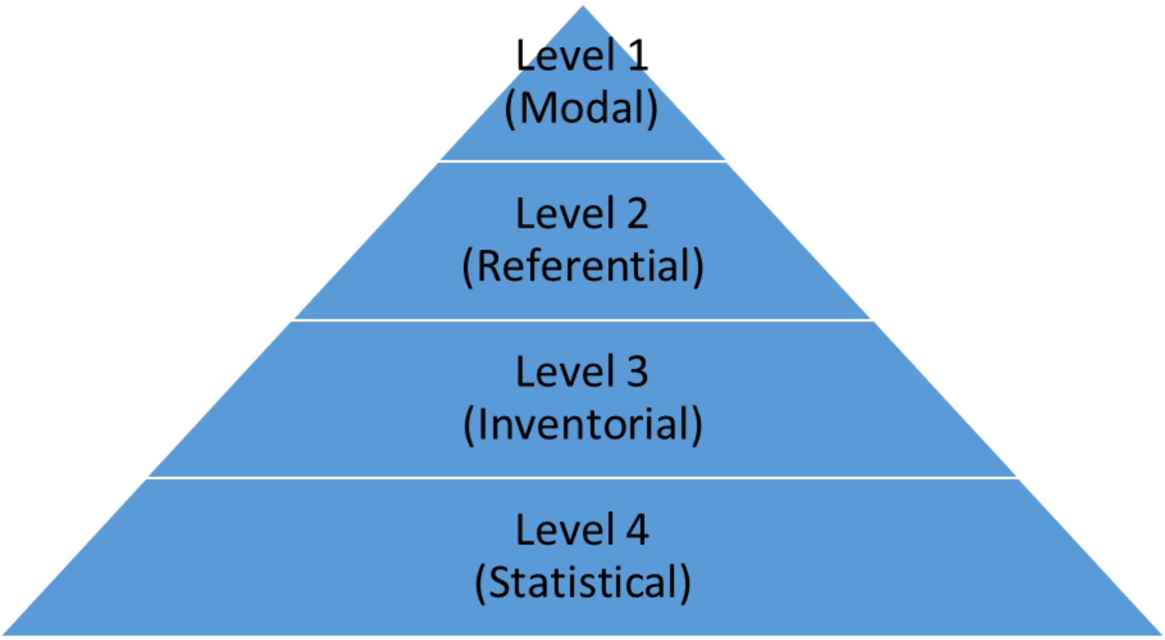
A2 Background

Although this document describes a statistical classification method, it is not the only classification applied to pavement condition data.

Within the department, a hierarchy of classification procedures is employed as indicated in Figure A2.1. These procedures are undertaken in the sequence indicated and statistical classification is subordinate to other processes.

A2.1 Classification

Figure A2.1 – Classification procedure hierarchy



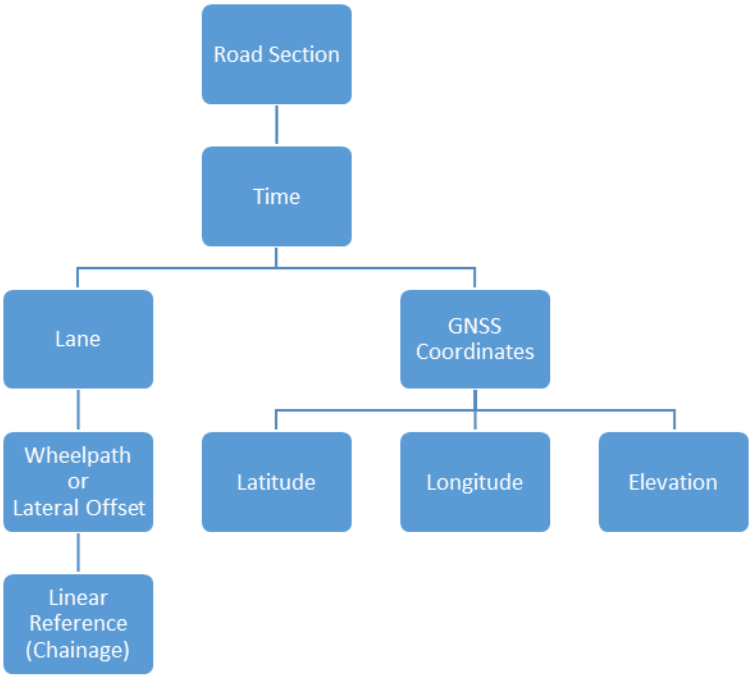
A2.1.1 Level 1, modal classification

Modal classification determines the types of data and their delivery format. The types of data presented in this document are related to the measurement of project-level pavement deflection. Departmental upload standards for network-level deflection data are an example of modal classification of data. In 1998, the American Association of State Highway and Transport Officials (AASHTO) attempted to address delivery standards for deflection data by publishing a pavement deflection data exchange specification; however, in practice, a variety of project-level delivery formats (predominantly spreadsheets) are encountered which tend to be vendor-specific and some effort is often required to obtain a common format for subsequent processing.

A2.1.2 Level 2, referential classification

Referential classification attempts to assign a reference to information in time and space. Figure A2.1.2 presents the essential variables of the departmental referential classification system. GNSS coordinates offer a complementary alternative to the linear referencing system.

Figure A2.1.2 – Departmental referential classification variables



The statistical classification method in its current implementation is not influenced directly by the reference classification.

A2.1.3 Level 3, inventorial classification

Inventorial classification is used to select lanes, wheel paths and chainages for independent statistical classification. If inventorial classification variables are not used, the statistical classification process will group data from the entire road section with no consideration of lane, wheel path or chainage. If the statistical classification process can process all deflection bowls within the project, the results are helpful in identifying similar pavement structures to determine the optimal number of subsurface sampling locations required.

As its name suggests, inventorial classification relies on some knowledge of the pavement inventory which allows for differences in layer thickness, material, construction date, geometry, drainage, climate and so on. This information may originate from the departmental road inventory database (*Chartview*), augmented with GPR information as required.

The two inventorial classification variables referenced in this document are 'site' and 'section'.

The site is defined typically by a gap in chainage between one set of results and another or to identify portions of the pavement with a differing construction history. The site variable may also be used for lateral classification of lanes and wheel paths for independent statistical analysis where structural differences have been identified.

The section variable is intended for longitudinal classification only, to identify a range of chainages within a site that indicate relatively uniform properties (homogeneous section) which differ from those of adjacent sections. The AASHTO Cumulative Difference method, described in Chapter 3 of the *Guide for the Design of Pavement Structures* (AASHTO, 1993), assists with longitudinal sectioning.

The section variable is subordinate to the site variable.

Sections were subdivided based on deflection (maximum deflection D_0 and D_{900}) and derived (for example, deflection ratio) values. These measures were an early attempt to categorise bowl magnitude and shape, a strategy adopted and formalised in the statistical classification procedure.

A2.1.4 Level 4, statistical classification

The statistical classification process attempts to form meaningful groups from sections produced by the levels 1–3 classification processes.

Although the data in these groups have greater homogeneity than data at higher levels of classification, they are geospatially amorphous, having irregular boundaries. For this reason, it is difficult to use statistical classification to identify homogeneous sections with distinctly rectilinear (longitudinal and lateral) boundaries, particularly where the pavement has been subject to varying traffic and environmental influences over a significant period.

A3 Traditional classification

The first step of traditional manual processing of deflection bowls involved separating a site into 'homogenous' sections, based on departmental structural profile records (ARMIS) and GPR reports.

Such sections were further subdivided based on deflection (maximum deflection D_0 and subgrade response D_{900}) and derived (for example, curvature and deflection ratio) values. These measures are an early attempt to categorise the bowl magnitude and shape respectively, a strategy adopted and formalised in the statistical classification process with the improvement the entire bowl is now considered in the classification process. This approach is likely to satisfy those who have argued, although the entire bowl was measured, only isolated portions were used in subsequent classification.

A4 Back analysis

Back analysis attempts to estimate the vertical elastic modulus of each layer in the pavement, based on the shape and magnitude of the deflection bowl with the assumption the thickness and material properties of each layer are known.

Regardless of which deflection device is employed, the sensors used to measure deflection have finite accuracy and measurement errors will exist which contain both random and systematic components.

The most obvious way to measure the magnitude of this error is to take repeated measurements at the same location and load on the pavement. Repeat measurements may also reveal non-resilient behaviour in the pavement, if completed in reasonable succession. These repeat measurements are achieved most efficiently with a FWD which does not require the test vehicle to be moving.

How these measurement errors are transformed by the back-analysis process is of interest.

A4.1 Sensitivity analysis

The traditional linear elastic back analysis process iteratively applies a forward calculation (like that carried out by CIRCLY) to arrive at a solution. It is, therefore, possible to examine the deflection bowl produced by the forward calculation to determine the extent deflection measurement errors influence the resultant moduli. A sensitivity analysis of the back-analysis process is therefore required.

A4.1.1 Relative sensitivity

The relative-sensitivity of the modulus $\boldsymbol{E}$ to variations in the deflection $\boldsymbol{\delta}$ is expressed in Equation A4.1.1(a).

Equation A4.1.1(a) – Relative sensitivity

$$\boldsymbol{S}_{\delta}^{\boldsymbol{E}} = \frac{\%\,\boldsymbol{change\ in\ E}}{\%\boldsymbol{change\ in}\,\delta} = \frac{\Delta \boldsymbol{E}/\boldsymbol{E}}{\Delta\delta/\delta}$$

or

Equation A4.1.1(b) – Relative sensitivity

$$\boldsymbol{S}_{\delta}^{\boldsymbol{E}} = \frac{\boldsymbol{\partial E}}{\boldsymbol{\partial}\delta}\,\frac{\delta}{\boldsymbol{E}}$$

Consider the example of unbound granular pavement variants presented in Table A4.1.1(a).

Table A4.1.1(a) – Unbound granular pavement variants

Layer	Thickness (mm)	Modulus E (MPa)	Poisson's Ratio
Base	200	250, 275, 300, 325 and 350	0.35
Sub-base	150	130	0.35
Subgrade	Semi-infinite	66 (~CBR 7)	0.45

The bowl obtained from the forward calculation (CIRCLY) of the 300 MPa base modulus variant of Table A4.1.1(a) has been adjusted to provide a back analysis (*EfromD3*) which results in matching moduli. The forward calculation was carried out using a 300 diameter uniformly loaded plate (FWD) with a contact stress of 750 kPa and sublayering compatible with the back calculation. The resulting bowl is indicated in Table A4.1.1(b) and had a low back calculated fit error of 0.85%.

Table A4.1.1(b) – 300 MPa base modulus back analysis bowl

Offset (mm)	Deflection δ (mm)
0	1.542
200	0.970
300	0.763
450	0.547
600	0.422
900	0.279
1500	0.163

The back analysis of the pavement variants of Table A4.1.1(a) yield the results of Table A4.1.1(c). Only the maximum deflection has been altered, all other deflections (sensors or offsets) remain unaltered to ensure the results reflect the error associated with a single sensor.

Table A4.1.1(c) – Back analysed base modulus versus maximum deflection

Maximum deflection δ (mm)	Base modulus E (MPa)	Fit error (%)
1.654	250	1.2
1.594	275	0.94
1.542	300	0.85
1.498	325	0.94
1.460	350	1.11

In this case, the relationship between $\boldsymbol{E}$ and δ can be represented by the following approximation (R^2 = 0.999):

Equation A4.1.1(c) – Approximation of modulus by power of the deflection

$$E = a\ \delta^{b}$$

in which case, the relative sensitivity of Equation A4.1.1(b) becomes

Equation A4.1.1(d) – Relative sensitivity of modulus to change in deflection

$$S_{\delta}^{E} = a\ b\ \delta^{(b-1)}\ \frac{\delta}{a\ \delta^{b}} = b$$

If the relationship between the modulus and the deflection can be modelled by a power law, the power or index is equal to the relative sensitivity of the relationship.

In this example, relative sensitivity S_{δ}^{E} of the base modulus E with respect to the maximum deflection δ is -2.7

This indicates standard deviation of a back calculated modulus may be several times the deflection.

Traditionally, the 90th percentile of deflections have been used to design pavements. It is tempting to assume 10th percentile modulus is the equivalent of the 90th percentile deflection; however, as demonstrated, such an assumption may not be justified because of error amplification in the back-calculation process.

Some method for reducing the effect of measurement errors prior to back analysis would be helpful.

It is relevant to consider two back analysis methods employed in the past; the single bowl and mean bowl methods.

A4.2 Single bowl method

The single bowl method involves back analysing every bowl offered by the data collection equipment that passes quality assurance procedures.

The statistics of the resulting moduli are generated to identify a representative modulus for each section. The distribution of the moduli is rarely normally distributed and attempts to treat it as such occasionally results in a representative modulus that is negative. The assumption of a log-normal distribution prevents such results.

Sensor measurement errors occurring in the bowls are amplified (Section A4.1.1) in back-calculated modulus values, potentially resulting in over-expenditure on some pavements.

A4.3 Mean bowl method

Taking the mean of *n* 'similarly shaped' bowls, the standard error of the mean (SEM) deflections are a factor $\sqrt{n}$ lower than the error associated with the deflections of a single bowl; for the mean of two bowls, the error reduction is approximately 30% and, for 100 bowls, the error reduction is 90%.

The bowls used in this process must of similar shape; otherwise, the mean bowl shape will suffer a degree of distortion which will reduce the effective error reduction.

The traditional manual process involved recognising similar bowl shapes 'by eye' and eliminating isolated 'incongruent' bowl shapes from the analysis.

Typically, the number of bowl shapes identified in the manual process was limited and could not be repeated by other practitioners.

Statistics for this method are computed on the input (deflection side) of the back analysis rather than the output (modulus).

The analysis of deflection (input) statistics has advantages over modulus (output) statistics:

- it is aligned with traditional deflection measures
- a negative representative (90^{th} percentile) deflection is less likely
- it is less influenced by error amplification through the back-analysis process
- associated clustering can be used to identify outlier deflection bowls unlikely to yield reliable back-calculated results, and/or
- associated clustering significantly reduces the number of bowls to be back calculated, allowing the effects of different search ranges and seed values to be more thoroughly explored.

A5 Bowl comparison

To harness the advantages of the mean bowl method, an efficient and repeatable way of separating deflection bowls into groups with similar shape and magnitude is required.

A5.1 Dimensional reduction

The FWD is equipped typically with at least nine sensors, each of which contributes a measurement to the deflection bowl.

Let the number of deflection measurements per bowl be *n*.

To compare any two bowls, it is tempting to examine the differences between the deflections at each corresponding geophone. This would lead to an n-dimensional analysis; however, if the deflections of the two bowls were regressed against each other, assuming a simple linear model, a slope, intercept and $\boldsymbol{R^2}$ can be determined, resulting in a three-dimensional comparison.

Additionally, if a degradation in $\boldsymbol{R^2}$ can be accepted as an alternative to including an intercept, the comparison would become two-dimensional with parameters of just the slope and $\boldsymbol{R^2}$.

The dimensionality of the problem has thus been reduced from *n* to 2.

For any given location on a road pavement, deflection equipment measures the deflection of road surface at a series of longitudinal offsets from a vertically-applied load. This results in a series (which can be regarded as a vector) of deflection measurements, commonly referred to as a bowl, at each location.

Within the state-controlled road network, it is usually sufficient to define the location of each bowl in terms of road section ID, lane, wheel path and chainage; however, a site identifier should be introduced where structural differences can be identified or where a different rehabilitation treatment from adjacent areas of the pavement would be considered.

Additionally, within each site, a section identifier should be introduced where structural differences can be identified from a plot of deflection values or their derivatives versus the longitudinal reference (chainage).

In the case where a defined wheel path has not been used, a lateral offset from a centreline or some other road marking is acceptable.

To allow verification of the location, GNSS coordinates should also be supplied; this is especially useful for off-road measurements.

Early attempts at grouping of deflection data were limited to manual procedures which tended to be somewhat subjective.

The methodology described in this document provides a repeatable process which will produce the same results with the same input.

A5.2 Statistical cluster analysis

The earliest recorded use of cluster analysis to separate data into groups was in the field of anthropology in 1932, considerably before introduction of modern computers; however, with the development of mainframe computers in the 1950s, renewed interest was generated, resulting in a wider array of metrics and linkage methods.

A5.2.1 The metric

Each cluster is defined by the proximity of or distance between its objects or members. This measure of distance is usually referred to as the metric.

In three-dimensional Cartesian space, the distance coordinates x, y and z usually have equal weighting and the distance between two objects with coordinates (x_1, y_1, z_1) and (x_2, y_2, z_2) is simply the Euclidean distance $\sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2 + (z_2 - z_1)^2}$; however, FWD deflection bowls typically have a minimum of 9 coordinates (geophones) and may have up to 17 geophones, depending on the make.

The Euclidean distance between any two bowls will be weighted in favour of the deflection measured by the geophone closest to the load, because this is usually the largest deflection in the bowl. The Euclidean distance is not well suited to judging the similarity of deflection bowls if the entire bowl is to be used effectively.

In addition to the Euclidean distance, several other distance measures have been developed by others which weight the coordinates in different ways. These include:

- Binary, $r = \frac{\sum_{i=1}^{n}|XOR(f(x_i),g(y_i))|}{\sum_{i=1}^{n}|OR(f(x_i),g(y_i))|}$
- Canberra, $r = \sum_{i=1}^{n}\frac{|x_i - y_i|}{|x_i + y_i|}$
- Euclidean, $r = \sqrt{\sum_{i=1}^{n}(x_i + y_i)^2}$
- Manhattan, $r = \sum_{i=1}^{n}|x_i - y_i|$
- Maximum, $r = \max(|x_i - y_i|)$
- Minkowski, $r = \sqrt[p]{\sum_{i=1}^{n}|x_i - y_i|^p}$

Of these, the Canberra distance is promising as it provides near equal weighting to all deflection coordinates; however, it has the disadvantage that, while giving a measure relative shape, the relative magnitude of the two bowls is not well-represented.

For this reason, it was decided to adopt what shall be termed the simple linear regression metric.

A5.2.1.1 Simple linear regression metric

The most obvious method of comparing two bowls is to plot them against each other in a spreadsheet chart, deflection versus deflection, matching geophone pairs according to their offset from the load. In this form, it is almost instinctive to fit a regression line to the data and examine the slope, intercept and $\boldsymbol{R^2}$ of the relationship between the two bowls. If the two bowls are identical, the slope will be equal to 1, the intercept will be 0 and $\boldsymbol{R^2}$ will be 1.

The ordinary least squares regression is used in spreadsheets and the underlying assumption is the independent variable (x) contains negligible measurement error while the dependent variable (y) contains significant measurement error; however, when regressing deflection bowls drawn from the same population, the measurement error in both variables will be similar. In this case, orthogonal or Deming Regression, with a variance ratio of 1, should be used instead of ordinary least squares regression.

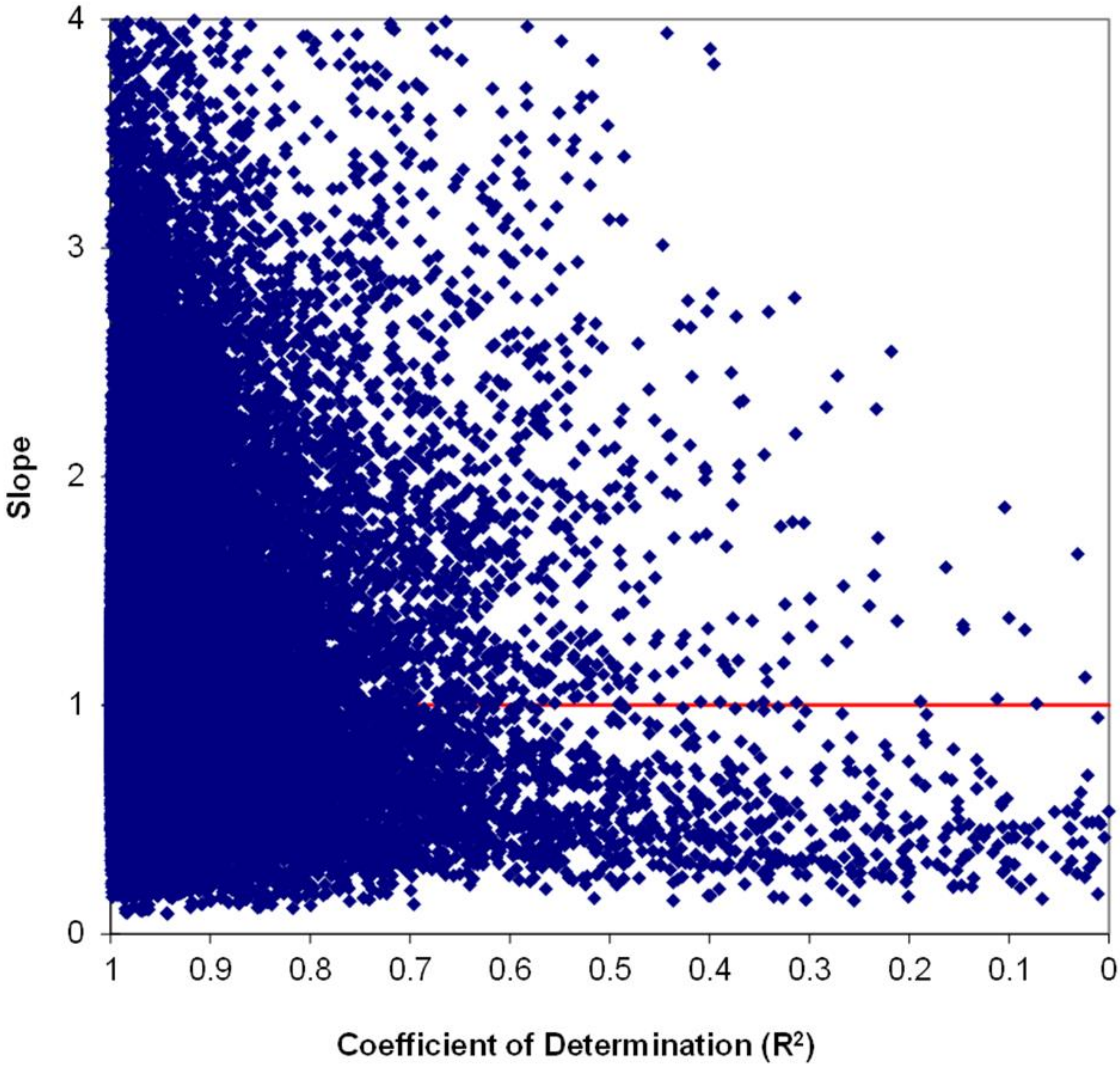
If the two bowls are near identical, the intercept will be negligible and can be suppressed (set to zero). If an intercept is required, the regression will compensate by reducing $\boldsymbol{R^2}$ which is consistent with a difference in shape; this is an acceptable simplification.

Simple linear regression (slope and $\boldsymbol{R^2}$ with no intercept) can be used to represent the relative magnitude and shape of the two deflection bowls.

If the slope and $\boldsymbol{R^2}$ are statistically independent, it would be desirable to use them as orthogonal components of a metric.

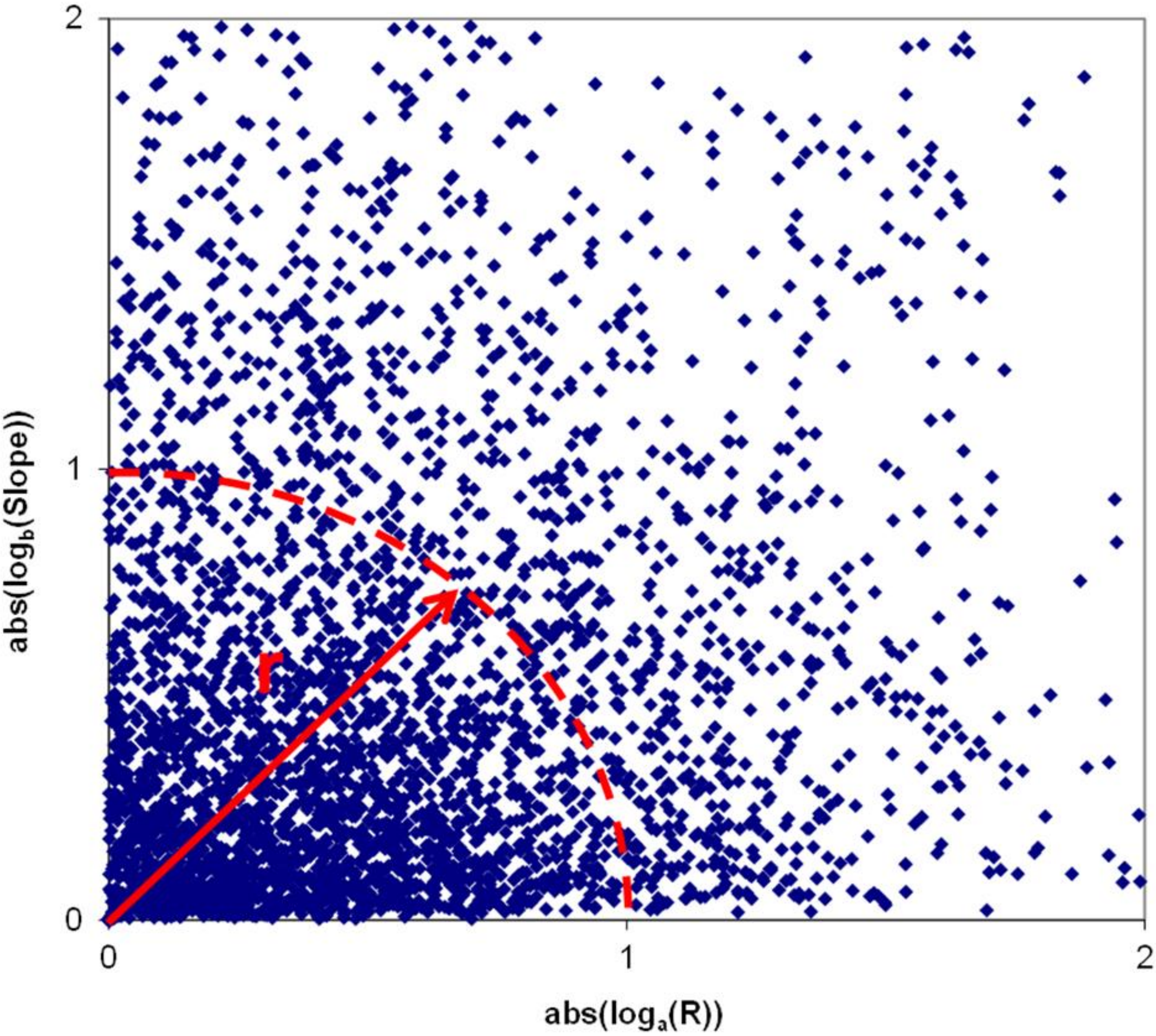
In Figure A5.2.1.1(a), the slope and $\boldsymbol{R^2}$ of a small local authority road network are plotted against each other and the unmodified regression parameters do not serve well as the orthogonal components of a metric. The primary reason for this is the value of the slope lies in the domain (0, ∞), while $\boldsymbol{R^2}$ lies in the domain (0, 1).

Figure A5.2.1.1(a) – Unmodified regression parameters



By allowing for the equivalence of the slope and its reciprocal and applying a logarithmic transform to both slope and $\boldsymbol{R^2}$, the basis for a modified Euclidean metric *r* is obtained as indicated in Figure A5.2.1.1(b).

Figure A5.2.1.1(b) – Transformed regression parameters



A5.2.2 Slope

The regression slope is a measure of the relative magnitude of one bowl with respect to the other and can be compared to the ratio of Maximum Deflections.

Since it is not important whether Bowl 1 or Bowl 2 provides the dependent or independent variable, the reciprocal of slope must be considered equal to the slope with respect to its contribution to the metric. To achieve this, the absolute value of the logarithm of the slope is used as a metric component in place of the slope. By scaling the Slope by its maximum tolerable value m_{max} and setting the cluster analysis cut-off threshold to 1, m_{max} becomes the base of the logarithm employed to transform the slope m to $m_{transform}$ as per Equation A5.2.2.

Equation A5.2.2 – Transformed slope

$$m_{transform} = abs\left(\frac{ln(m)}{ln(m_{max})}\right)$$

Equation A5.2.2 indicates a slope $\boldsymbol{m}$ equal to m_{max} will transform to a value of 1, thus ensuring only slope values less than m_{max} will be considered when constructing bowl groups. m_{max} is subsequently referred to as the slope discriminant. This discriminant controls the extent to which relative deflection bowl amplitude (depth) is used to determine membership of bowl groups.

The relationship between the unmodified regression slope values and the transformed values is indicated in Table A5.2.2.

Table A5.2.2 – Transformation of slope

$\boldsymbol{m}$	$\boldsymbol{m_{transform}}$
0	∞
$\frac{1}{m_{max}}$	1
1	0
m_{max}	1
∞	∞

The trial value of the slope discriminant for a section of the pavement can be determined by examining a plot of D_0 versus chainage and deciding what maximum ratio of deflection values can be accepted for two bowls to belong to the same group.

Where the initial sectioning based on maximum deflection is considered to represent bowl magnitude adequately, it is unnecessary to include the influence of slope in subsequent clustering. To remove the dependence of clustering on slope, its discriminant should be increased to a value of least 2.

A5.2.3 Coefficient of determination $\boldsymbol{R^2}$

The regression coefficient of determination or $\boldsymbol{R^2}$ is a measure of the similarity of the shape of the two bowls.

The absolute value of the logarithm of $\boldsymbol{R^2}$ is used as a metric component instead of $\boldsymbol{R^2}$; in the rare instances where $\boldsymbol{R^2}$ is negative, it is converted to a small positive value.

By scaling the $\boldsymbol{R^2}$ by its minimum tolerable value R^2_{min} and setting the cluster analysis cut-off threshold to 1, R^2_{min} becomes the base of the logarithm employed to transform R^2 to $R^2_{transform}$ as per Equation A5.2.3(a).

Equation A5.2.3(a) – Transformed $\boldsymbol{R^2}$

$$R^2_{transform} = abs\left(\frac{ln(R^2)}{ln(R^2_{min})}\right)$$

Equation A5.2.3(a) indicates an R^2 equal to R^2_{min} will transform to a value of 1, thus ensuring only R^2 values greater than the minimum tolerable R^2_{min} will be considered when constructing bowl groups. R^2_{min} is subsequently referred to as the R^2 discriminant. This discriminant controls the extent to which deflection bowl shape is used to determine membership of bowl groups.

The relationship between the unmodified regression slope values and the transformed values is indicated in Table A5.2.3.

Table A5.2.3 – R^2 transformation

R^2	$R^2_{transform}$
0	∞
R^2_{min}	1
1	0

The transformed slope and R^2 are then used to form a linear metric indicated in Equation A5.2.3(b) or a modified Euclidean metric r as indicated in Equation A5.2.3(c).

Equation A5.2.3(b) – Linear formula

$$r = x + y$$

Equation A5.2.3(c) – Euclidean formula

$$r = \sqrt{(x^2 + y^2)}$$

where

$x = m_{transform}$ = the transformed regression slope

$y = R^2_{transform}$ = the transformed regression R^2

The metric assumes the regression relationship between any two pavement structures, in terms of their bowl shapes, will be near-linear if they represent similar structures. The relationship between two bowls can be expressed as in Equation A5.2.3(d) where the deflection bowls are treated as data vectors.

Equation A5.2.3(d) – Vector formula

$$\tilde{y}_i = m\tilde{y}_j$$

where

$\tilde{y}_i$ = the deflection bowl (vector) at location i

$\tilde{y}_j$ = the deflection bowl (vector) at location j

m = the regression Slope

and

R^2 = the coefficient of determination

Where the two bowls are of the same shape, the value of R^2 will be equal to 1.

Where the two bowls are identical, the expected value of both R^2 and the slope m will be equal to 1.

The formulation of the model represented by Equation A5.2.3(d) provides
two independent (orthogonal) variables capable of being used as the components of a metric indicating the similarity of any two bowls; the regression slope and R^2.

By computing the regression slope and R^2 for every combination of two bowls, a matrix of metrics, usually referred to as a distance (or dissimilarity) matrix, can be constructed to support the use of the hierarchical clustering methodology referred to in this *Manual*.

Using the regression slope and R^2 requires a metric cut-off threshold to select the number of bowl groups sought in the solution. This threshold is set to 1, requiring metric components slope and R^2 to be scaled appropriately.

A5.2.4 Linkage methods

Statistical clustering requires a metric and a defined method for linking groups (clusters) of data.

At the commencement of an agglomerative process, each bowl is assigned to a group of its own. The groups are then sequentially combined into larger groups, until all bowls become members of the same group. The method employed to decide group membership is referred to as the linkage method.

Several linkage methods have been added to the agglomerative hierarchical clustering process over the decades since its inception.

A5.2.4.1 Single linkage

At each step of this method, the two clusters separated by the shortest distance are combined. The definition of 'shortest distance' is what differentiates the different agglomerative clustering methods. In single linkage clustering, a single element pair links two clusters, namely those two elements (one in each cluster) closest to each other. The shortest of these links remaining at any step causes the fusion of the two clusters whose elements are involved. The method is also known as nearest neighbour clustering.

A drawback of this method is chaining phenomenon: the gradual growth of a cluster as one element at a time is added. The clusters formed via single linkage clustering may be forced together due to single elements being close to each other, even though many of the elements in each cluster may be very distant to each other. This may lead to impractically heterogeneous clusters and difficulties in defining classes that could usefully subdivide the data.

A5.2.4.2 Complete linkage

In this method, the link between two clusters contains all element pairs, and the distance between clusters equals the distance between those two elements (one in each cluster) farthest away from each other. The shortest of these links remaining at any step causes the fusion of the two clusters whose elements are involved. This method is also known as farthest neighbour clustering.

Complete linkage clustering avoids a drawback of the chaining phenomenon associated with the single linkage method. Complete linkage tends to find compact clusters of approximately equal diameter.

A5.2.4.3 Mean linkage

Alternatively referred to as unweighted pair group method with arithmetic mean, at each step the distance between any two clusters A and B is taken to be the average of all distances between pairs of objects 'x' in A and 'y' in B – the mean distance between elements of each cluster. The algorithm of this method weights each element in the candidate cluster equally, regardless of its structural subdivision.

A5.2.4.4 McQuitty linkage

Alternatively referred to as weighted pair group method with arithmetic mean, the algorithm for this method differs from the unweighted pair group method with arithmetic mean algorithm by weighting the member admitted most recently to a cluster equal with all previous members.

A5.2.4.5 Centroid linkage

Alternatively referred to as unweighted pair group method with centroid, at each step, the distance between any two clusters A and B is taken to be the distance between the centroid of the objects in each cluster. This method's algorithm weights each element in the candidate cluster equally, regardless of its structural subdivision.

A5.2.4.6 Median linkage

Alternatively referred to as weighted pair group method with centroid, at each step, the distance between any two clusters A and B is taken to be the median of all distances between pairs of objects 'x' in A and 'y' in B – the median distance between elements of each cluster. This method's algorithm differs from the unweighted pair group method with arithmetic mean algorithm by weighting the member admitted most recently to a cluster equal with all previous members.

Note: The median is not a monotone distance measure; the number of groups may not increase by decreasing the metric cut-off value. This results in dendrograms with inversions or reversals difficult to interpret.

A5.2.4.7 Ward's minimum variance method

Ward's method attempts to form clusters with minimal variance and, like the complete linkage method, tends to result in compact spherical clusters.

A5.3 Constraints

The 'distance' matrix used by the clustering process has a finite capacity determined by the size of computer memory. Typically, a matrix consisting of more than a million elements (over 1000 bowls) can be accommodated; however, efforts should be made to keep the size of this matrix to a minimum, if only to reduce processing time. For this reason, the cluster analysis is only carried out within a pre-defined section: the appropriate definition of sites and sections can be used so the dissimilarity matrix does not exceed memory constraints.

The selection of R^2 and slope discriminant values is a manual procedure; for example, as the R^2 (or slope) discriminant is relaxed, the number of bowl groups produced by the cluster analysis will decrease, to the point where a single bowl group contains all bowls, potentially resulting in distorted bowl shape being supplied to the back analysis process. Conversely, if the discriminants are tightened, the number of bowl groups will increase to the point where each bowl group contains just one bowl, resulting in nil measurement error reduction. Neither of these extremes is very useful.

The complete linkage method produces acceptable results; corresponding discriminant values for back analysis follow.

Values for the R^2 discriminant in the vicinity of 0.98 are appropriate for most pavements. Undamaged bound pavements tend to require higher values for the R^2 discriminant than unbound pavements.

Values for the slope discriminant in the vicinity of 1.3 are appropriate for most pavements. Where increasing the R^2 discriminant causes a gap to open between bowls (two or more populations indicated) in a group, the slope discriminant should be reduced to split the group and remove the gap.

Where the bowl groups are to be used for project-wide visualisation or selection of subsurface sampling locations, an R^2 discriminant value around 0.9 and slope discriminant value around 3 may be required for visualisation purposes.

A6 Methodology

A6.1 Processing sequence

1. Capture FWD data to a format suitable for processing. Currently, FWD data are stored in a MS Access table called FWD. The structure of this table is given in Figure A6.1(a). FWD data are often supplied in a variety of spreadsheet formats, depending on the vendor. To harmonise these data, they are copied and pasted to the standard FWD table.

Figure A6.1(a) – Falling weight deflector data table structure

Field Name	Data Type	Description (Optional)
RejectYN	Yes/No	Reject record if True
ID	AutoNumber	Generated
Source	Short Text	Folder in which FWD files are stored
FileName	Short Text	FWD file name (Road_Site_Lane_Wheelpath_NominalLoad_Drop naming convention preferred)
TestDateTime	Date/Time	Date and time of test
Chainage_m	Number	Linear reference for the FWD test in metres
CentreLineOffset_m	Number	Offset from centre line in metres where tests have not been carried out by wheelpath
SubSite	Short Text	Optional
Stress_kPa	Number	Measured plate stress in kiloPascals
SurfaceTemp_C	Number	Surface surface temperature in degrees celsius
AirTemp_C	Number	Ait temperature in degrees celsius
D0_mm	Number	Deflection at the centre of the load
D200_mm	Number	Deflection at an offset of 200 millimetres from the centre of the load
D300_mm	Number	Deflection at an offset of 300 millimetres from the centre of the load
D450_mm	Number	Deflection at an offset of 450 millimetres from the centre of the load
D600_mm	Number	Deflection at an offset of 600 millimetres from the centre of the load
D750_mm	Number	Deflection at an offset of 750 millimetres from the centre of the load
D900_mm	Number	Deflection at an offset of 900 millimetres from the centre of the load
D1200_mm	Number	Deflection at an offset of 1200 millimetres from the centre of the load
D1500_mm	Number	Deflection at an offset of 1500 millimetres from the centre of the load
MaxDefl_mm	Number	Normalised deflection at the centre of the load in millimetres
Curvature_mm	Number	D0 minus D200 in millimetres
DeflectionRatio	Number	D250 divided by D0 where D250 is the arithmetic mean of D200 and D300
SubgradeCBR	Number	Estimated California Bearing Ratio (CBR) of the Subgrade (0.5911 * D900_mm^-1.4759)
Latitude_deg	Number	Test Latitude in degrees (optional)
Longitude_deg	Number	Test Longitude in degrees (optional)
Section	Short Text	Assigned by Sectioning procedure
Cluster	Number	Assigned by Statistical Clustering procedure

2. Separate bowls into sites where a structural difference is indicated or where an alternative rehabilitation treatment to that applied to adjacent pavement will be considered. Sites are identified based on a gap in chainage (longitudinal) but can also be defined based on transverse references such as carriageway, lane or even wheel path. The file annotation table of the database can be used to define or redefine sites by any combination of file name, lane or wheel path. Figure A6.1(b) presents the structure of the file annotation table.

Note:

- All fields should be filled out correctly, particularly those controlling the formatting of sectioning charts (MinimumScale_km, MaximumScale_km, MajorUnit_km and MinorUnit_km).
- The meaning of each deflection file can be assigned automatically if a file naming convention such as Road_Site_Lane_Wheelpath_NominalLoad_Drop is adopted.
- Transport and Main Roads uses odd-numbered lanes for its gazettal (prescribed) direction and even-numbered lanes for the anti-gazettal (counter) direction, eliminating the need to include direction code explicitly in the file name. The MoistureAdjustment_YN field is used to identify wheel paths which require a DMAF to be applied prior to back analysis.

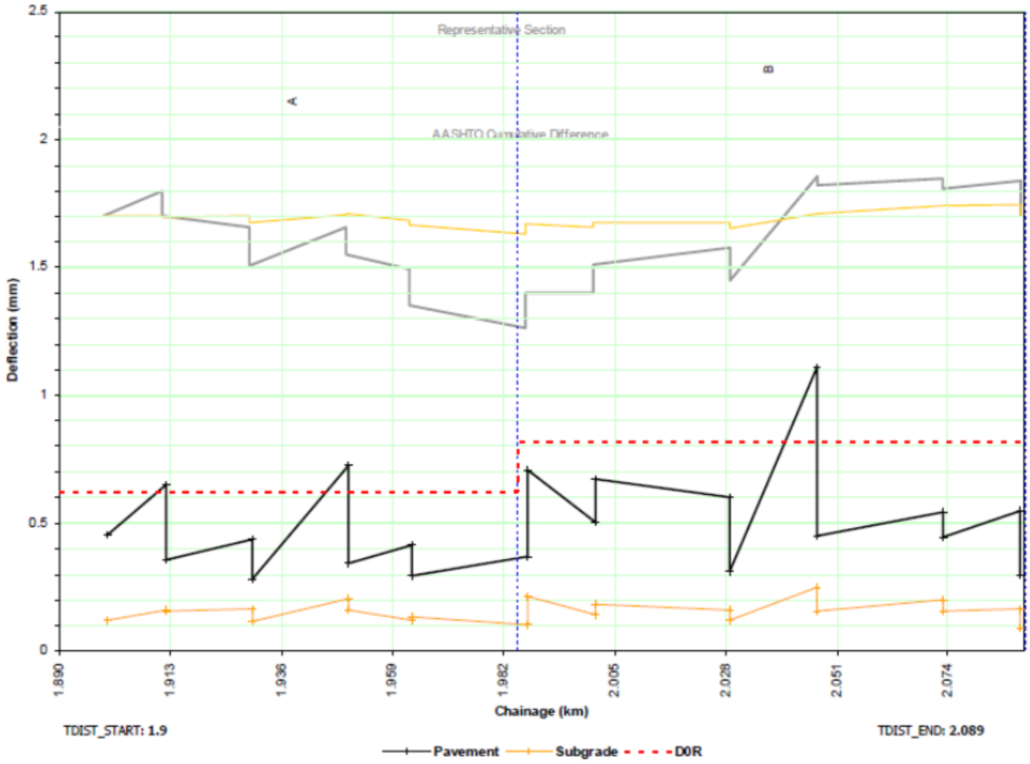
Figure A6.1(b) – Falling weight deflector file annotation table structure

Field Name	Data Type	Description (Optional)
Source	Short Text	Folder in which FWD files are stored
FileName	Short Text	File name (Road_Site_Lane_Wheelpath_NominalLoad_Drop naming convention preferred)
RoadID	Short Text	Road Identifier (4 characters max.)
RoadSectionID	Short Text	Road Section Identifier (4 characters max.)
Site	Short Text	Site (8 characters max.)
Lane	Short Text	Lane (1 character, e.g. 1, 2, A, Q, etc.)
Wheelpath	Short Text	Wheelpath (1 character, e.g. L, R, I, O)
WheelpathOffset	Number	Integer giving the ordinal position of the wheelpath on quasi geospatial charts (typically 1, 2, 3, or 4 for two lane roads)
NominalLoad_kN	Number	Nominal (targetted) Load in kiloNewtons
TestDate	Date/Time	Date that deflection testing was carried out
Start_km	Number	Linear reference of the first deflection test in kilometres
End_km	Number	Linear reference of the last deflection test in kilometres
MeanStress_kPA	Number	Mean deviator stess in kiloPascals
MeanLoad_kN	Number	Mean deviator load in kiloNewtons
RoadName	Short Text	Road name (255 characters max.)
RoadSectionName	Short Text	Road Section name (255 characters max.)
MinimumScale_km	Number	Minimum scale value for sectioning charts in kilometres
MaximumScale_km	Number	Maxmum scale value for sectioning charts in kilometres
MajorUnit_km	Number	Major scale unit for sectioning charts in kilometres
MinorUnit_km	Number	Minor scale unit for sectioning charts in kilometres
SampleInterval_m	Number	Mean deflection test interval in metres
NBowls	Number	Number of tests carried out
D0Min_mm	Number	Minimum deflection at the centre of the load in millimeters
D0Max_mm	Number	Maximum deflection at the centre of the load in millimeters
MoistureAdjustment_YN	Yes/No	Moisture Adjustment to be applied , True or False

3. Separate each site into longitudinal sections based on relative deflection levels, historical records, GPR results, field observations (for example, seal changes) or a project-wide cluster analysis with a low R^2 discriminant value. To assist with sectioning, deflection data may be processed further to produce a cumulative difference (AASHTO, 1993) plot as indicated in Figure A6.1(c). The cumulative difference is a modified numerical integration of the deflection data which has the effect of removing high-frequency information and converting deflection levels to slopes. In the example of Figure A6.1(c), two sections (A and B) have been adopted, based on subtle changes in cumulative difference slopes. It is acceptable to use several wheel paths in the sectioning process and Figure A6.1(c) includes two wheel paths which gives rise to two deflection results at many chainages and vertical segments in the plot. Disregarding the vertical segments, the overall trends are clear.

Note: A left mouse click on the MS Access sectioning chart will add an end-of-section marker, as indicated by the vertical blue dashed line of Figure A6.1(c). It is recommended section breaks be placed between, rather than at, bowl locations to ensure section membership is unambiguous.

Figure A6.1(c) – Sectioning chart



4. For each section, calculate the representative maximum deflection $D0_r$ and representative subgrade deflection $D900_r$ for each wheel path.

5. For each wheel path, estimate the subgrade CBR using $D900_r$ for a 40 kN load in Equation A6.1.

Equation A6.1 – California Bearing Ratio as a function of $D900_r$

$$CBR = \frac{0.5911}{D900_r^{1.4759}}$$

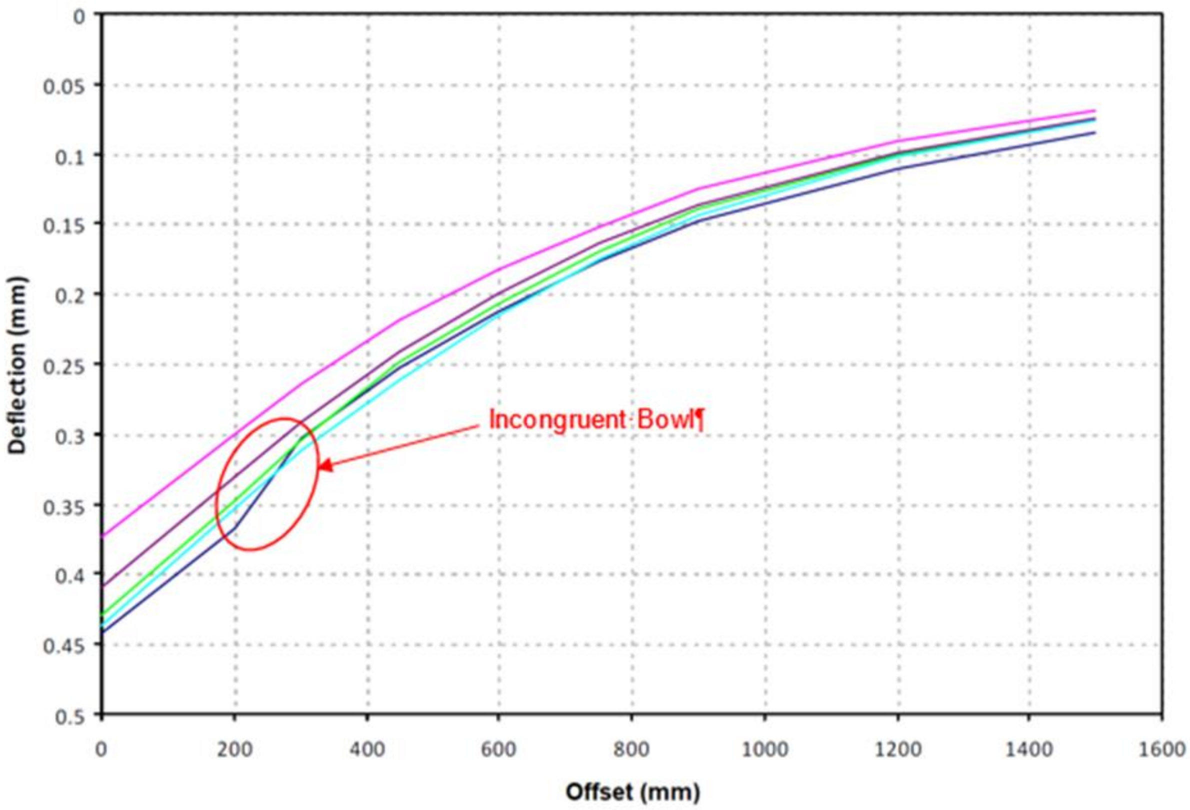
Note: Deflections for loads other than 40 kN can be 'normalised' to 40 kN by multiplying by 40 and dividing by the measured load.

6. Commence statistical clustering of bowl shapes in each section with the complete linkage method with an R^2 discriminant value in the vicinity of 0.9 or 0.98 and a slope discriminant value in the vicinity of 3 or 1.3, depending on whether the bowl groups are to be used for project-level visualisation or back-calculation respectively.

7. Increase the R^2 discriminant and decrease the slope discriminant until bowl groups contain bowls of predominantly the same shape. One or two incongruent bowls can be tolerated, as long as they do not extend significantly beyond the general envelope of the group, because the overall bowl shape of the group is provided by the mean bowl shape.

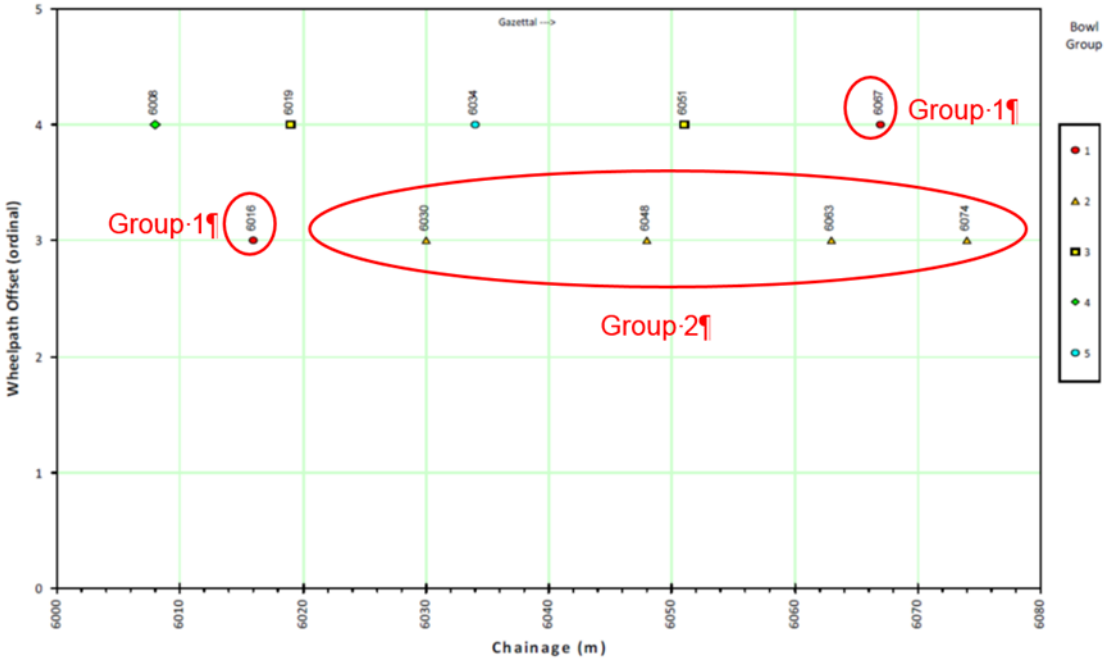
8. Review bowl charts and, if necessary, increase R^2 to reduce the occurrence of incongruent bowls (for example, Figure A6.1(d)).

Figure A6.1(d) – Incongruent bowl



9. Select larger bowl groups for back-calculation. Groups containing a single bowl may be the result of geophone measurement errors or indicative of isolated pavement defects / repairs; therefore, groups containing at least two bowls (preferably more) are recommended. No more than one or two bowl groups are selected for back analysis, although 'predominant' bowl groups with low deflection may also be selected if there is a possibility, they might influence the feasibility of the rehabilitation design. Both IWP and OWP should be sampled by the selected bowl groups to allow the effects of moisture to be estimated. Geospatial charts like Figure A6.1(e) can be used to examine the geospatial extent and cohesiveness of bowl groups to assist with selection of groups for back analysis. In the example of Figure A6.1(e), although Group 1 has the highest deflection, indicated by a low group label, Group 2 is a predominant group (contains a significant proportion of the bowls in the section) and, depending on the back analysis results, treatment might be to dig out isolated soft spots indicated by Group 1, followed by a wider area rehabilitation design based on the back analysis of Group 2.

Figure A6.1(e) – Geospatial chart



10. Compute the mean bowl for each group by calculating the arithmetic mean of deflections at each geophone. The mean bowl for each group will include the mean maximum deflection D_0g.

11. The moisture adjustment method described in Appendix B should be applied to wheel paths, except the method is confined to bowl groups rather than sections; therefore, the bowl group or groups selected for back analysis should include both IWP and OWP.

12. Submit the moisture adjusted bowl of each group to a back-calculation process.

A7 Implementation

The current implementation of SCA uses MS Access 2013 to generate the orthogonal regression parameters of slope and R^2 for every bowl combination within a section. MS Access then submits these combinations as a data file, together with a code file (processing instructions) to the R statistical computing language (R) (https://www.r-project.org/). MS Access then retrieves a comma separated value (.csv) file containing the cluster assignment of input bowls.

A7.1 Data file

The data file is a .csv file containing four fields:

- IDx, first bowl identifier (integer)
- IDy, second bowl identifier (integer)
- slope, regression slope (floating point), and
- R2, regression coefficient of determination (floating point).

The bowl identifier is matched with a record in a database table containing bowl details such as road, site, section, lane, wheel path, normalised load and chainage. Each data file represents a section and may span multiple lanes and wheel paths.

Figure A7.1 provides an example of a data file for a small section containing six bowls. Where a bowl is regressed against itself (IDx = IDy) both slope and R^2 are equal to 1, as would be expected for identical bowls.

Figure A7.1 – Data file example

IDx	IDy	Slope	R2
1	1	1	1
1	2	1.04402	0.915943
1	3	1.002677	0.99639
1	4	1.133642	0.993507
1	5	1.201951	0.997689
1	6	1.069658	0.995388
2	1	0.948228	0.871008
2	2	1	1
2	3	0.95064	0.874107
2	4	1.075785	0.899547
2	5	1.140721	0.900466
2	6	1.014746	0.869692
3	1	0.997027	0.996093
3	2	1.040778	0.911222
3	3	1	1
3	4	1.130656	0.99768
3	5	1.198549	0.997484
3	6	1.066188	0.983702
4	1	0.881595	0.992458
4	2	0.921118	0.923959
4	3	0.884257	0.997509
4	4	1	1
4	5	1.05996	0.998029
4	6	0.942663	0.977282
5	1	0.831817	0.997465
5	2	0.869194	0.92884
5	3	0.834163	0.997449
5	4	0.943273	0.998138
5	5	1	1
5	6	0.889638	0.988915
6	1	0.934566	0.995699
6	2	0.976154	0.920802
6	3	0.936813	0.985954
6	4	1.059077	0.98176
6	5	1.123149	0.990577
6	6	1	1

A7.2 Code file

The database contains a table, illustrated at Figure A7.2, which includes a draft version of the code file. This table is identical to the submitted code file, except it contains extensible markup language (.xml) format labels replaced by the database prior to submission.

.xml labels include the R workspace folder (a portion of the local hard drive for which the user has write permission), the name of the output file including the file extension and importantly, the slope and R^2 discriminant values.

As shown in Figure A7.2, the discriminants scale the slope and R^2 components of the metric. Additionally, because the logarithmic transformation of the metric components and cluster tree is always cut at a value of 1, slope and R^2 discriminants indicate the cut value on the respective axes.

Figure A7.2 – Draft R code file

```
setwd(“”<RWorkSpaceFolder>””)
R_Input = read.csv(“”R_ClusterInput.csv””, header = TRUE, sep = “”,””,
quote = “”””)
s = as.vector(ifelse((R_Input$Slope < 0 ), 10^2, R_Input$Slope))
s = abs(log(s))/log(<Slope>)  <——— Slope Discriminant
v = as.vector(ifelse((R_Input$R2 < 0), 10^-5, R_Input$R2))
v = abs(log(v))/log(<R2>)  <——— R2 Discriminant
n = sqrt(nrow(as.matrix(v)))
myLabel = as.vector(matrix(R_Input$Idy, nrow=1, ncol=n, byrow = FALSE))
## Slope
Slope = matrix(s, ncol=n, byrow = FALSE)
## Coefficient of Determination
R2 = matrix(v, ncol=n, byrow = FALSE)
myDist = Slope + R2
d1 = as.dist(myDist, diag = FALSE, upper = FALSE)
hc = hclust(d1, method = “”<method>””, members=NULL)
hc$height = sort(hc$height, decreasing = FALSE, na.last = NA)
myCluster = cutree(hc, k = NULL, h = 1)  <——— Dendrogram (tree) cut value
R_Output = cbind(myLabel, myCluster)
write.table(R_Output,file = “”<RworkSpaceFolder>\\<ROutputFile>””, quote
= FALSE, sep = “”,””)
q(save = “”no””, status = 0, runLast = TRUE)
```

A7.3 Output file

The output .csv format file contains two important fields:

- BowlID, bowl identifier (integer)
- ClusterID, statistical cluster identifier (integer).

Table A7.3 presents an example of the output file for the six-bowl example of Table A7.1. It indicates four clusters have been identified. Cluster 3 contains bowls 3, 4 and 5. All other clusters contain a single bowl.

Table A7.3 – Output file example

BowlID	ClusterID
1	1
2	2
3	3
4	3
5	3
6	4

Note: Once the output file has been read by the database, the statistics of each cluster are computed, and the cluster renumbered according to the mean maximum deflection of the cluster. The renumbered clusters are then called bowl groups, with the lowest numbered bowl group corresponding to the highest deflection. This allows bowl groups to be reported (deflection bowl charts and geospatial charts) with bowl shapes likely to be responsible for the more critical (higher) deflections to be reported at the top of the list within each section.

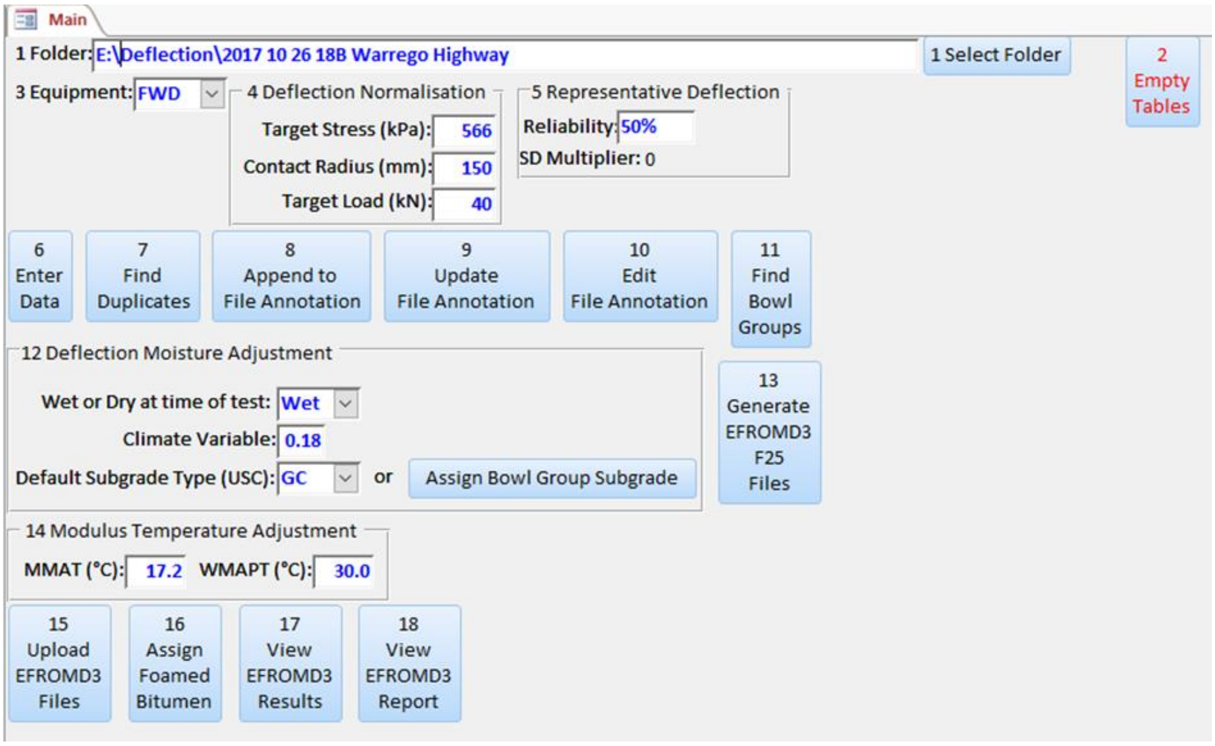
A7.4 Interface

The primary interface is provided currently by MS Access 2013 (32 bit). A MS Access 2016 (64 bit) is also available if required. The main form provides access to the table in which FWD data are stored.

A7.4.1 Main form

Figure A7.4.1 presents the main form of the interface. This form provides facilities for the entry of FWD data and information required to interpret these data. Also included is functionality for generating a back-analysis input file (currently *EfromD3* F25 format) for each section.

Figure A7.4.1 – Main form (MS Access)



A7.4.1.1 Instructions

The following steps correspond to the buttons indicated in Figure A7.4.1.

1. Assign project folder by entering the folder into the 'Folder' field or use the 'Select Folder' button which invokes the Windows folder picker dialogue.

2. Use the 'Empty Tables' button to remove data associated with previous projects. This is coloured red to indicate it should only be used to initialise the database for a new project and incorrect use can result in loss of data associated with the new project.

3. Select the type of equipment from the drop-down list of the 'Equipment' field. This entry is used to determine how to normalise deflections for load (refer Step 4) and appears in report headers.

4. Enter 'Deflection Normalisation' values.

   Note: Clicking on one of the entry fields will cause it to be estimated from the other two fields.

5. Enter the 'Representative Deflection' for bowl groups. This determines how the mean bowl of each group will be escalated. It is assumed the maximum deflection (D_0) of bowl groups is

normally distributed and a value of 50% leaves the mean bowl unadjusted ('SD Multiplier' = 0). A value of 90% will escalate the mean bowl to match the 90th percentile D_0 value of the group. The 50% value is recommended, since subsequent selection of bowl groups for back analysis is based on comparing the resulting group D_0 value as a quantile of section D_0 values (refer Step 11). Where the bowl group contains a single bowl, no escalation is possible and the unescalated bowl is used.

6. Click the 'Enter Data' button to open the data table. Paste deflection data (typically normalised data from a spreadsheet) to the table. If the stress of the data varies from the 'Target Stress' specified in Step 4, the data will be normalised.

   Note: Deflections at standard offsets of 0, 200, 300, 450, 600, 750, 900, 1200 and 1500mm from the load are expected. Where non-standard offsets are encountered, it is suggested deflections at these offsets be interpolated using a cubic spline method or similar. A variety of spline add-ins are available for use in spreadsheets and R provides a variety of packages suited to the task.

7. Click the 'Find Duplicates' button to list data records not identified uniquely in terms of file name and chainage. Return to Step 6 to adjust or remove duplicate records to ensure unique references are available to match clustering results with data. Duplicate records arise occasionally where the equipment operator suspects a recorded bowl may contain erroneous measurements and initiates additional confirmation measurements. Suspect records are not always removed prior to delivery and this step is required to confirm the record to be retained.
8. 'Append to File Annotation' to generate annotation (header) records for each file based on the data entered in Step 6.
9. 'Update File Annotation' to update file annotation records automatically with additional information based on the standard file naming convention (RoadSection_Site_Lane_Wheelpath_NominalLoad_Drop).
10. 'Edit File Annotation' to update file annotation records manually. This includes selecting IWP for moisture adjustment (MoistureAdjustment_YN field).
11. The 'Find Bowl Groups' button opens a similarly-named form, allowing all procedures indicated in Section A7.4.2 to be completed.
12. Complete the 'Deflection Moisture Adjustment' fields from information obtained using the methods described in Appendix B. Use the 'Assign Bowl Group Subgrade' button to assign a subgrade type to individual bowl groups and allow deflection moisture adjustment of wheel paths selected in Step 10.
13. Generate back analysis files in 'EfromD3F25Files' subfolder of the folder identified in Step 1. An F25 file will be generated for each section assigned in Step 11. Each file will contain the respective bowl groups selected in Step 11. *EfromD3* will require properties such as thickness, material type, Degree of Anisotropy and Poisson's Ratio to be entered manually for each layer in each F25 file.
14. Enter 'Modulus Temperature Adjustment' values. Mean monthly air temperature is used by the Modified Bell's Equation (AP T248) to estimate the mid-depth temperature of bituminous layers from measured surface temperature, while the remaining field is used to adjust the modulus of asphalt from the estimated temperature to the WMAPT for the location, using Equation 23 of AGPT02. The temperature T adjustment factor F used for the modulus of

foamed bitumen is $F = -0.02\,T + 1.5$ (<50mm thick) and $F = -0.01\,T + 1.25$ (≥50 and <100mm thick) based on the values in Table 4.9.8.5.2 and where F is not permitted to exceed 1 (T≤25°C).

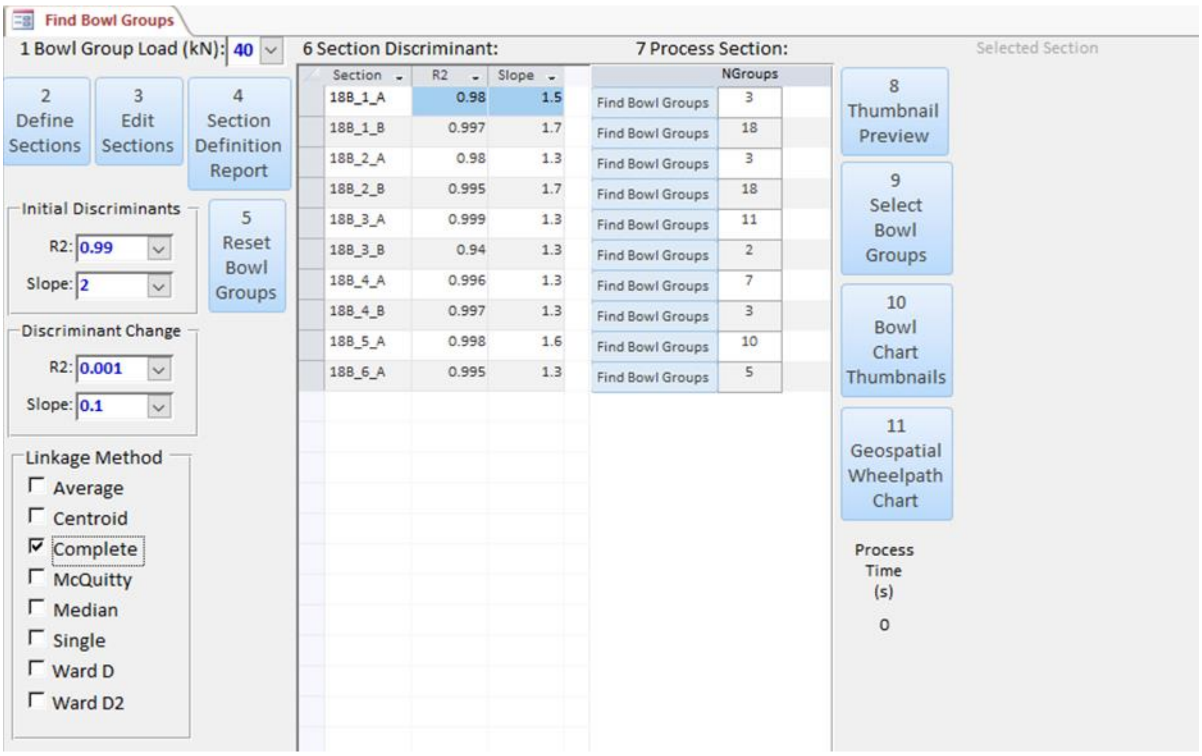
15. *EfromD3* produces a series of files with the extension .rxl. These are MS Excel .xls format files; to allow MS Access to read them, the file extension should be changed to .xls and saved in the EfromD3F25Files subfolder of Step 13. The 'Upload EfromD3 Files' button uses a Windows file picker dialogue to select .xls files to upload to a table.
16. The 'Assign Foamed Bitumen' button provides the opportunity to identify *EfromD3* 'AC' material that should have moduli adjusted for temperature based on Table 4.9.8.5.2 (Transport and Main Roads foamed bitumen formula) rather than Austroads default as indicated in Step 14.
17. Alternately, use the 'View EfromD3 Results' button to check the results of the back analysis before proceeding to Step 18.
18. 'View EfromD3 Report' for a formatted back analysis report that can be printed to a Portable Document Format (.pdf) file. The formatting of this report is complex and takes some time to complete.

A7.4.2 Find bowl groups

Figure A7.4.2 presents the 'Find Bowl Groups' form which interfaces with R performing the statistical clustering of the FWD data.

Note This form must be left open when generating back analysis files.

Figure A7.4.2 – Find Bowl Groups form (MS Access)



A7.4.2.1 Instructions

The following steps correspond to the portions of the Find Bowl Groups form indicated in Figure A7.4.2.

1. Select the 'Bowl Group Load (kN)' using the drop-down list derived from a summary of the FWD data entered in Step 6 of Section A7.4.1.
2. Define sections using deflection charts for each site (refer Figure A6.1(c)). Section end markers can be added by left clicking on the chart or editing the table to the right of the chart manually. Choose section breaks between, rather than at, bowl locations to avoid ambiguity regarding section membership.

   Note: Sections appearing in the charts are marked with an alphabetic character while the fully-qualified section label also includes the road section ID and site as a prefix.

3. Review and edit sections in tabular format if necessary.
4. Print the 'Section Definition Report' as a .pdf file.
5. Set initial discriminant values (0.9–0.98 for R^2 and 3–1.3 for slope) and click (twice, but do not double-click) the 'Reset Bowl Groups' button, to reset each section in the section discriminant sub-form to the right of the button. Caution should be exercised in using this button because it will delete the contents of the section discriminant form before regenerating revised sections; therefore, any discriminants to be reused for sections that have not been revised should be documented for re-entry before pushing this button. This button is intended for use where the section definitions in Step 2 have been modified.
6. Shift-click or Ctrl-click on the R^2 and slope fields to apply the increments and decrements entered in the discriminant change fields at the left of the form prior to Step 7. Avoid setting either discriminant to 1 as this corresponds to either identical bowl shapes or depth, which has the potential to return many bowl groups containing just a single bowl and may produce a metric singularity or divide overflow error. A section discriminant row can also be selected to indicate which section to use for the geospatial report of Step 11.
7. Left click the 'Find Bowl Groups' button next to each section in the 'Process Section' sub-form to submit data to R. The duration of this process is displayed as 'Process Time(s)' to the right of the sub-form after completion. This process returns bowl group information such as the number of bowl groups found which is returned in 'NGroups' field of the sub-form. Adjust the R^2discriminant as per Step 6 and left-click the 'Find Bowl Groups' button until the bowls in each group appear to have a similar shape (no incongruent bowls, refer Step 8). If multiple statistical populations are suggested by the appearance of gaps between the bowls of the thumbnail chart (refer Step 8) of a predominant group, or a group likely to be selected for back analysis, the slope discriminant should be reduced until the group splits into two groups, eliminating the gap. A slope discriminant less than 1.2 is seldom required.
8. The 'Thumbnail Preview' button can be used to display bowl chart thumbnails in a separate form (close before repeating Step 7) indicating the constituency of the bowl groups. These charts can be redisplayed by selecting the section row in the 'Section Discriminant' sub-form.
9. Select bowl groups for back analysis in the tick box next to the bowl group of interest. The predominant bowl group (largest number of bowls) is highlighted in yellow and should be selected for reference purposes.

10. The bowl group with the highest representative maximum deflection (D_0) should be selected (Bowl Group 1, the first thumbnail). To assist with bowl group selection, the title of each thumbnail chart contains the number of bowls and wheel paths included in the group. If this bowl group only contains a single wheel path, an additional bowl group should be selected to include an alternative wheel path. Moisture correction will be applied to wheel paths selected in Section A7.4.1.1 Step 10 in accordance with this *Manual,* except this adjustment is carried out on bowl groups rather than sections; therefore, selected bowl groups should include both IWP / OWP for estimation of moisture adjustment effects. As an example, Figure A7.4.2.2 indicates Bowl Groups 2, 5, 6, 7 and 13 have been selected for back-calculation. Selected groups contain both OWP (L) and IWP (R) for the purposes of moisture adjustment.
11. Geospatial charts are generated for the single section selected in Step 6 and will need to be printed individually via the 'Geospatial Wheelpath Chart' button, preferably as .pdf files and subsequently combined into a single file if required as a report. The 'Bowl Chart Thumbnails' Button should be used to print a report to a .pdf file. This report contains all sections and indicates bowl groups selected for back-calculation in Step 9.
12. Return to the 'Main' form by clicking on the 'Main' tab but do not close the 'Find Bowl Groups' form as the bowl group load defined in Step 1 is required for the generation of back-calculation input files. Complete Section A7.4.1.1 Step 7. Moisture adjustment of IWP is not permitted to influence the grouping of bowls and is not included in bowl chart thumbnails nor geospatial charts. Section A7.4.1.1 Step 7 includes this adjustment just prior to back analysis.

Note: The 'Find Bowl Groups' form contains an button to permit details of R to be entered.

A7.4.2.2 Select bowl groups

Figure A7.4.2.2 indicates how a subset of bowl groups may be selected for back calculation. An envelope of groups is selected based on the size (number of bowls) and quantile (percentile) of the group. The quantile of the maximum deflection (D_0) of the entire section which matches the mean maximum deflection of the group (D_{0g}) is reported as the quantile of the group.

Groups are selected, commencing at the predominant (largest) group automatically highlighted in yellow), followed by the next largest group with a higher quantile until a quantile near 90% (either above or below) is reached. It is recommended no more than six bowl groups be selected for back calculation as this corresponds to the limit of the back analysis report referenced in Section A7.4.1.1, Step 18.

Figure A7.4.2.2 – Bowl group envelope

Bowl Group Select

RoadSectionID 18B Site 1 Section B

BowlGroup	NBowls	Select	Quantile	D0Mean_mm
1	1	☐	100%	0.116
2	2	☑	93%	0.109
3	1	☐	89%	0.106
4	2	☐	88%	0.103
5	4	☑	85%	0.101
6	8	☑	69%	0.095
7	10	☑	67%	0.095
8	3	☐	56%	0.092
9	3	☐	52%	0.089
10	2	☐	47%	0.087
11	5	☐	46%	0.086
12	3	☐	29%	0.081
13	11	☑	23%	0.077
14	2	☐	23%	0.076
15	2	☐	23%	0.076
16	3	☐	18%	0.073
17	1	☐	3%	0.067

A7.4.3 Creating a bowl group Keyhole Markup Language Zipped file

The bowl group Keyhole Markup Language Zipped (.kmz) file assists with selecting test pit locations and excavation options associated with the calibration of the back-calculation process. Each test pit must return the thickness and material description of each layer of the pavement at the specified location.

The process for producing the .kmz file is as described in Section A7.4.1 and Section A7.4.2, except only a single site and section are defined, to represent all bowls in the project.

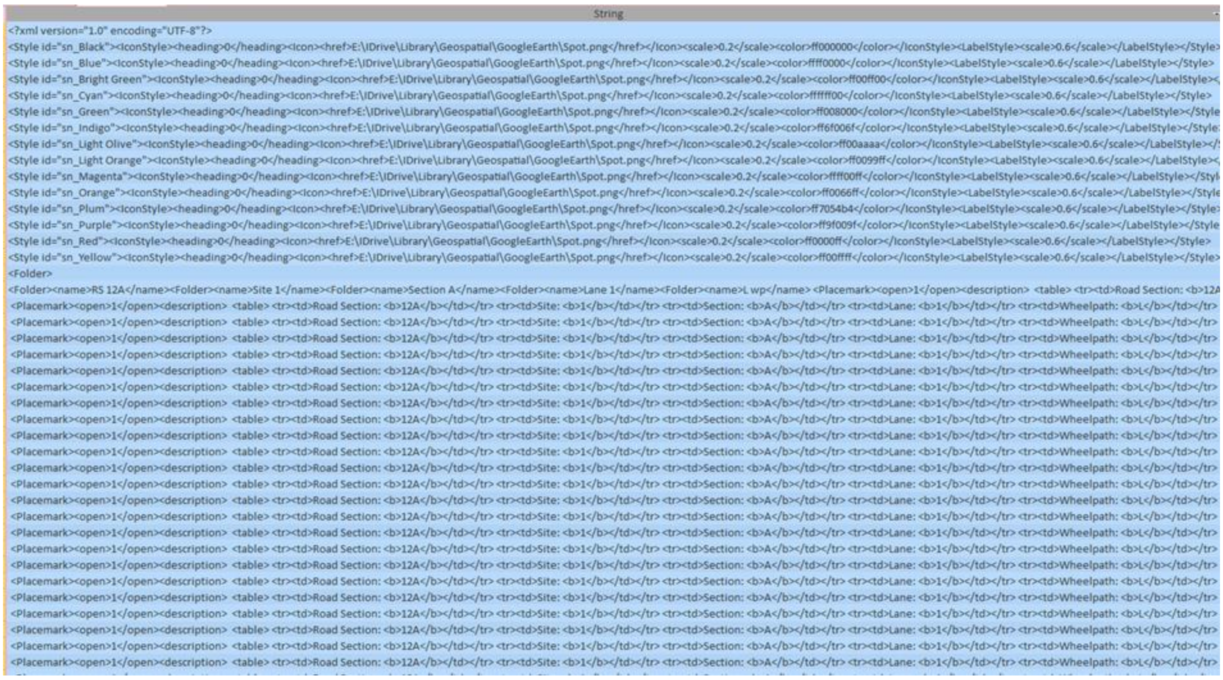
Figure A7.4.3 presents the Google Earth form which performs the necessary tasks to generate a Keyhole Markup Language (kml) table.

Figure A7.4.3 – Google Earth form (MS Access)

A7.4.3.1 Instructions

1. Click 'Generate Placemarks' button to assign deflection details and brief chart generating code to each bowl placemark.
2. Click 'Number Placemarks' button to assign a sequence number to be used to sort each bowl into a folder.
3. Click 'Generate Folders' button to create appropriate folder start and end labels.
4. Click 'Generate kml' button to assemble a kml table using folders, placemarks and common information.
5. Click 'View kml Table' button to view the kml table similar to Figure A7.4.3.1(a).

Figure A7.4.3.1(a) – Keyboard Markup Language table




```
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<Style id="sn_Yellow"><IconStyle><heading>0</heading><Icon><href>E:\IDrive\Library\Geospatial\GoogleEarth\Spot.png</href></Icon><scale>0.2</scale><color>ff00ffff</color></IconStyle><LabelStyle><scale>0.6</scale></LabelStyle></Style>
<Folder>
<Folder><name>RS 12A</name><Folder><name>Site 1</name><Folder><name>Section A</name><Folder><name>Lane 1</name><Folder><name>L wp</name> <Placemark><open>1</open><description> <table> <tr><td>Road Section: <b>12A
 <Placemark><open>1</open><description> <table> <tr><td>Road Section: <b>12A</b></td></tr> <tr><td>Site: <b>1</b></td></tr> <tr><td>Section: <b>A</b></td></tr> <tr><td>Lane: <b>1</b></td></tr> <tr><td>Wheelpath: <b>L</b></td></tr>
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```
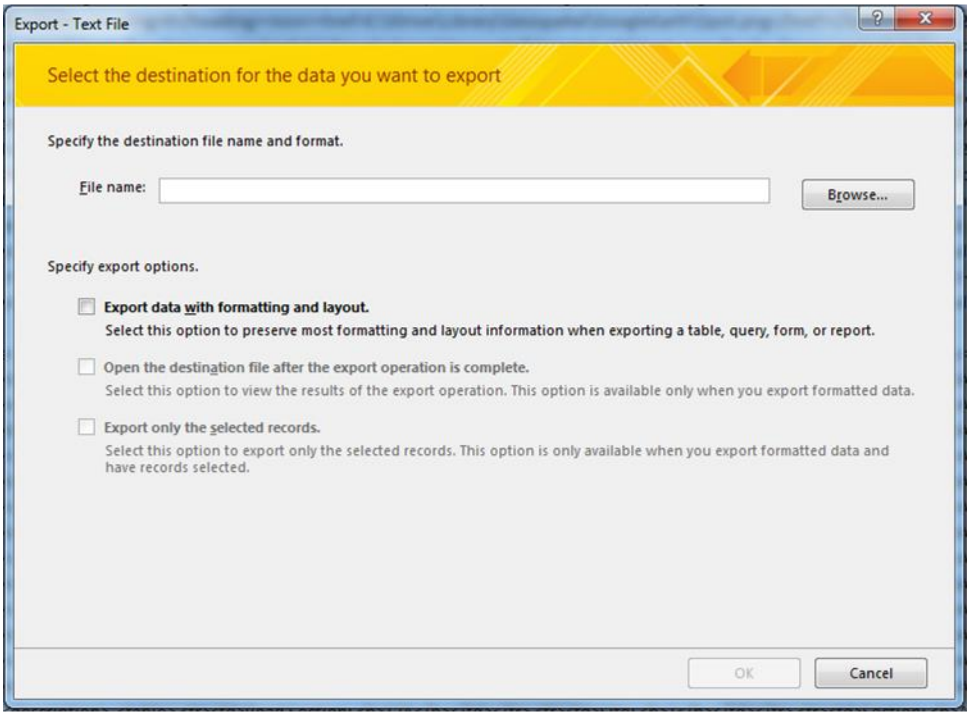


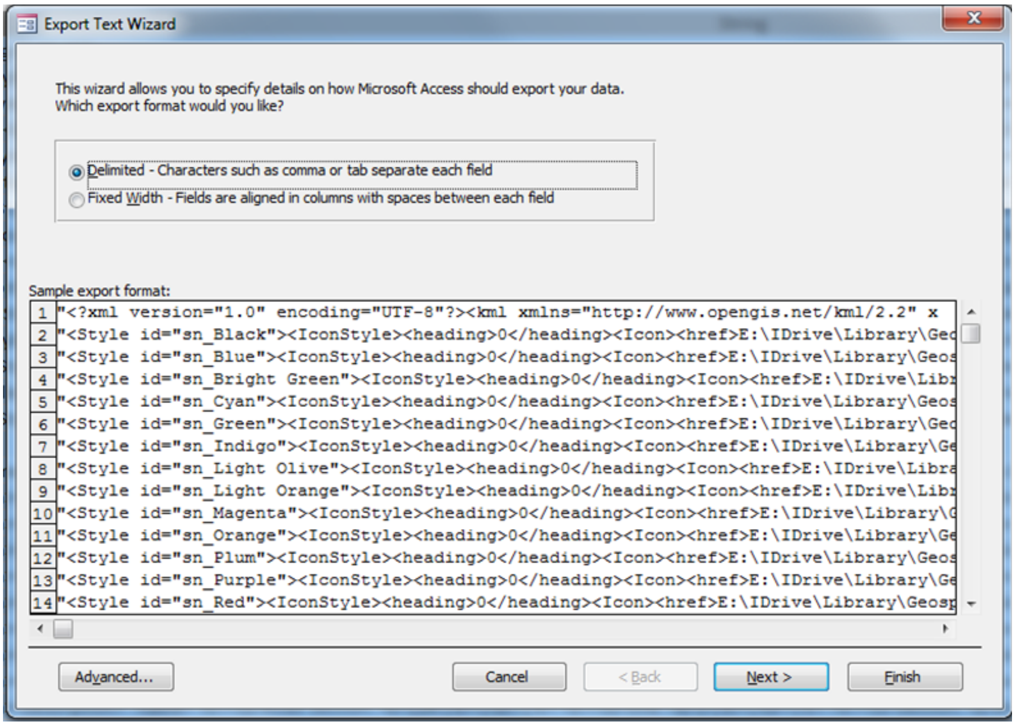
6. Export the table to a standard text document (.txt) file using Alt-xt keystrokes.
7. Select a destination (usually the project folder) for the .txt file using the 'Browse' button as per Figure A7.4.3.1(b) and click the 'OK' button.

Figure A7.4.3.1(b) – Select text file destination



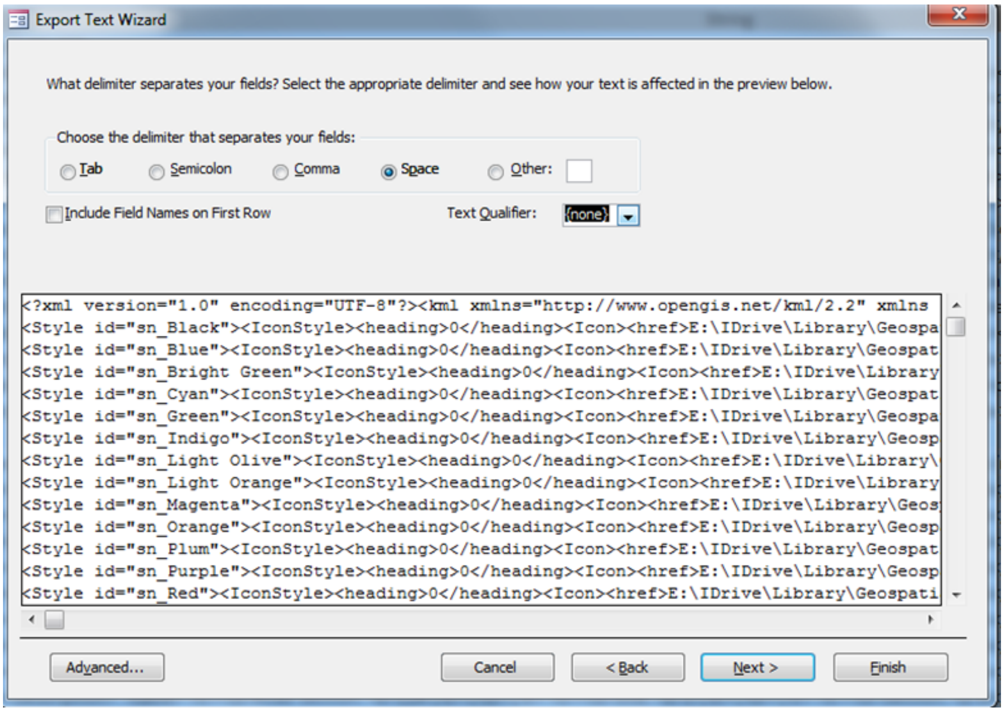
8. Select the 'Delimited file format' and 'Next>' buttons with keystrokes Alt-d and Alt-n, refer Figure A7.4.3.1(c).

Figure A7.4.3.1(c) – Select 'Delimited file format'



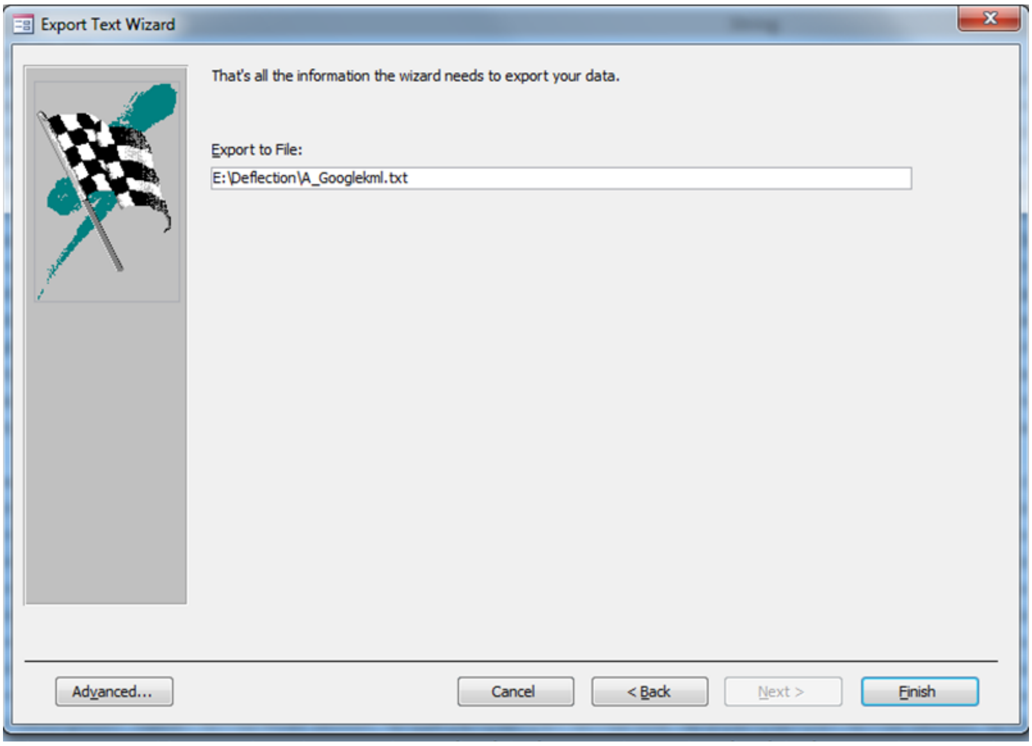
9. Select 'Space' as the delimiter (Alt-p keystroke), '{none}' as the Text Qualifier and the 'Next>' button (Alt-n keystroke), refer Figure A7.4.3.1(d).

Figure A7.4.3.1(d) – Select delimiter and text qualifier



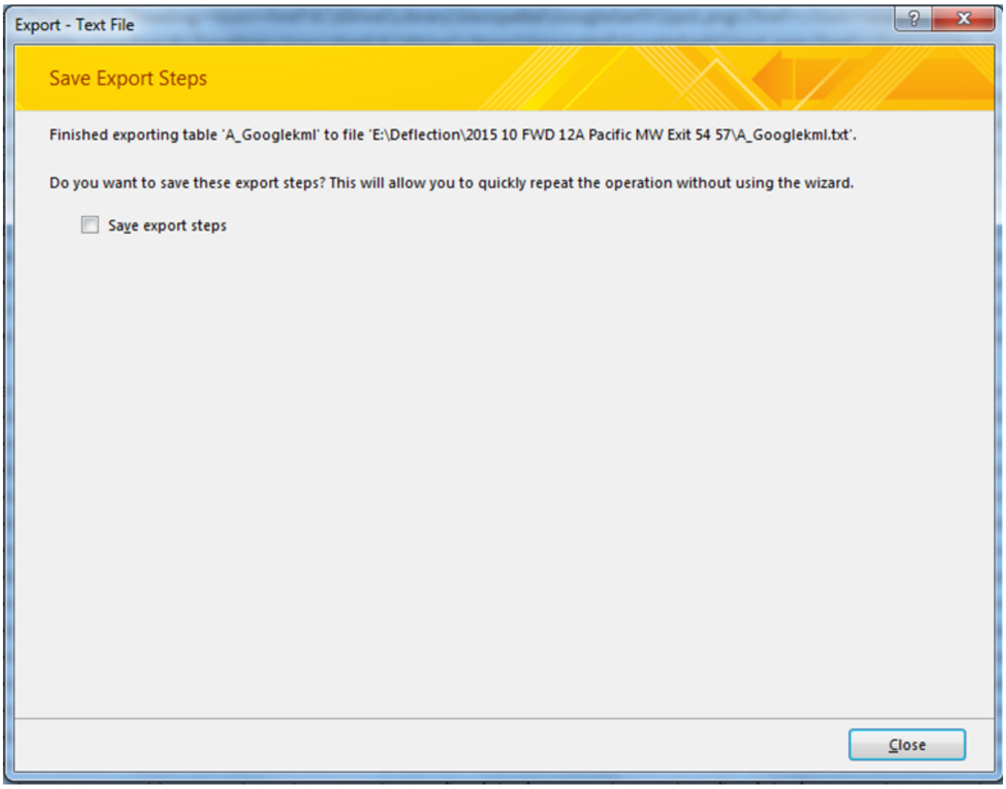
10. Confirm the save location and select the 'Finish' button (Alt-f keystroke), refer Figure A7.4.3.1(e).

Figure A7.4.3.1(e) – Save the file



11. Select 'Close' button (Alt-c keystroke), refer Figure A7.4.3.1(f).

Figure A7.4.3.1(f) – Select 'Close''

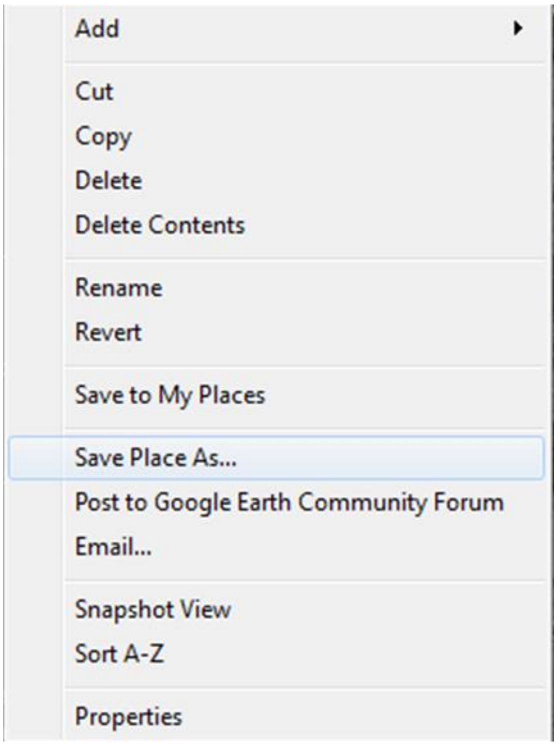


12. Go to the saved text file (A_Googlekml.txt) and change the file extension from .txt to .kml.
13. Double click on the .kml file or otherwise open it in Google Earth (Ctrl-o keystroke).
14. Save the loaded file to a kmz file by right-clicking on it under the 'Temporary Places' folder of Google Earth (left-hand side of Google Earth as per Figure A7.4.3.1(g)).

Figure A7.4.3.1(g) – Uploaded Keyboard Markup Language file

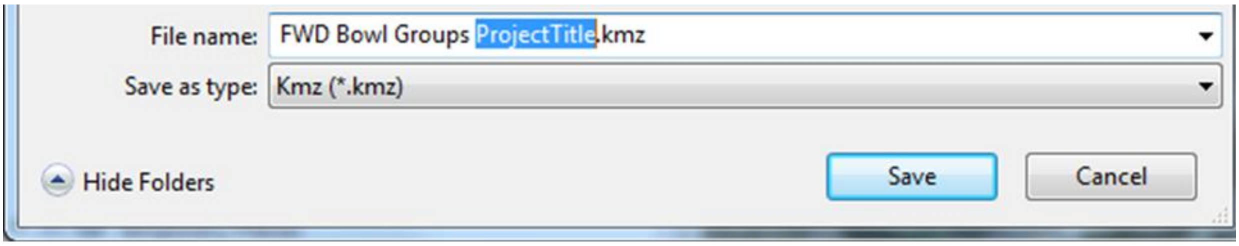
15. Select 'Save Place As'. Refer Figure A7.4.3.1(h).

Figure A7.4.3.1(h) – Select 'Save Place As'



16. Select the save location (usually the Project Folder), ensuring 'Save as type Kmz (*.kmz)' is selected. Add a 'Project Title' to the file name as appropriate, then click the 'Save' button. Refer Figure A7.4.3.1(i).

Figure A7.4.3.1(i) – Save as appropriately named Keyboard Markup Language Zipped file



Appendix B – Deflection moisture adjustment

B1 Background

The influence of rainfall on the moisture level and, therefore, the elastic modulus in pavement layers, depends on:

- the intensity and duration of rainfall events
- seasonal evaporation
- the type and AEP adopted for the drainage system design
- flow in the porous pavement media (*A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils*) (Genuchten, 1980)), (*Equations for the Soil Water Characteristic Curve*) (Fredlund, 1994)), and/or
- the sensitivity of the layer elastic modulus to the moisture content

B1.1 Traditional method

If possible, deflection testing should be undertaken when the subgrade is in the weakest condition, normally the wet season.

Ideally, as correction factors are influenced by things such as subgrade type, rainfall, location of water table and pavement types, they should be developed from studies conducted by each Transport and Main Roads Region or District.

Where this is not possible, guidance is provided in Table B1.1. The values in Table B1.1 are for use when providing designs based on the deflection reduction method. To correct for moisture, the D_r and representative D_{900} obtained from testing in a specific season are multiplied by the correction factor given in Table B1.1. The corrections are not applied to individual results.

The factors given in Table B1.1 are for guidance in situations where more reliable information is not available. They relate to pavements of full-width construction with fair drainage conditions. Caution should be exercised when applying them to other situations (for example, to existing 'boxed' pavements).

The OWP are susceptible to environmental forces while the moisture of the other wheel paths are relatively constant. For this reason, moisture correction factors are only applied to the deflections measured in wheel paths other than the OWP, to attempt to simulate the anticipated weakest condition of the pavement.

Table B1.1 – Seasonal moisture correction factors for a pavement with a thin asphalt surfacing or seal

[1]Pavement condition	[2]Correction factor for Transport and Main Roads Regions / Districts			
	For deflections measured during the end of wet season	For deflections measured during the end of dry season		
	All Regions / Districts	Rockhampton District Metropolitan, North Coast, South Coast and Wide Bay / Burnett Regions (includes Brisbane, Bundaberg, Gold Coast, Gympie, Ipswich and Logan, Moreton and Sunshine Coast Districts)	Emerald District Darling Downs, Central West, North West and South West Regions (includes Toowoomba and Warwick Districts)	Far North, Mackay / Whitsunday and Northern Regions
Weak pavements (D_r>1.5mm)	1 (1)[3]	1.2 (1.3)[3]	1.1 (1.2)[3]	1.2 (1.4)[3]
Intermediate pavements (1.5mm>D_r>0.9mm)	1 (1)[3]	1.2 (1.3)[3]	1.1 (1.2)[3]	1.2 (1.5)[3]
Strong pavements (D_r<0.9mm)	1 (1)[3]	1.2 (1.3)[3]	1.1 (1.2)[3]	1.2 (1.6)[3]

Notes:

[1] D_r values quoted in this table are for deflection results from Benkelman Beam, deflectograph or FWD testing with a 40 kN load.

[2] Applied only to wheel paths other than OWP. In situations where the water table is within 1m of the subgrade level throughout most of the year, no correction should be applied.

[3] Values in brackets apply for silty and clayey silt subgrades where greater variation in deflection level may be expected.

When comparing OWP deflections to moisture corrected deflections in other wheel paths:

- if the corrected deflection/s are higher than corresponding OWP deflection/s, then the corrected deflection/s should be used to determine D_r, and/or
- if the corrected deflections are lower than the corresponding OWP deflection/s, a check should be made to ensure other factors are not controlling the OWP deflection (for example, 'box type' construction trapping moisture or providing inadequate lateral restraint). If there are no obvious defects in the OWP to cause high deflections, then the OWP deflections should be used to determine D_r.

A detailed assessment of the pavement including the pavement structure, material properties (porosity) and drainage design is required to determine the initial rate at which moisture enters and leaves the pavement; traditionally, the adjustment of deflection for moisture has not included an explicit allowance for these factors.

Most moisture infiltration was a result of an extreme rainfall event beyond the capacity of the drainage design. According to the department's RDM, an AEP of 2% is typical for cross-drainage while an AEP of 10% is used for the design of side drains. The likelihood of extreme rainfall events should be included in the climatic characterisation of any site likely to be affected adversely by high moisture levels.

Two deficiencies have been addressed in the revision of the earlier method.

The season (wet or dry) is not well-defined in quantitative terms and it has been left to the practitioner to decide between the two. Given the erratic nature of rainfall in Queensland, the outcome is likely to be somewhat subjective. The revised method examines daily rainfall, evaporation and the long-term mean daily rainfall to derive an objective assessment as to whether a DMAF should be applied.

Climate was categorised based on administrative areas aggregated into three climatic zones. While these zones are reasonably aligned with climatic information available from the Bureau of Meteorology, the result is a broad quantisation of the climatic influence on pavement moisture levels. This quantisation results in difficulties where the climate of a site, slightly to one side of a boundary, is significantly different to another site, slightly to the other side of the boundary. Another difficulty arises in defining the zone boundaries and deciding on which side of a boundary a site lies. The revised approach adopts the Bureau of Meteorology intensity, frequency and duration tables as the basis for calculating a climate variable for a specific site. Since the intensity, frequency and duration table is determined from the latitude and longitude of the site, the climate variable is continuous geospatially, dispensing with the need to define boundaries or supply a reference map.

B1.2 Deflection moisture adjustment factor

Although the DMAF allows a continuous value to be evaluated for the seasonal factor, it remains quantised (0 = wet or 1 = dry) and the definition of what constitutes wet or dry conditions has not been quantified. To resolve this issue, the rate at which moisture is lost from the pavement must be considered, and this will determine the period, after the rainfall and before deflection testing, in which the effects of residual moisture should be considered.

B2 Season variable

The decision as to whether to apply a DMAF or not is influenced by the moisture level in the pavement relative to some reference level. The reference level must be related to the normal, typical or steady state moisture in the pavement and can only be determined from long-term observations, preferably over a period exceeding the design life of the pavement. It is unlikely the loss of moisture from a pavement following rainfall will progress in a linear fashion. It is far more likely to be related to the amount of residual moisture in the pavement and the simplest model for this is a percentage loss per unit of time.

B2.1 Rainfall

B2.1.1 American Association of State Highway and Transportation Officials climate model

Regarding the modelling of rainfall, the AASHTO enhanced integrated climate model (*Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* 2nd edition) (AASHTO, 2015)) includes hourly rainfall figures from at least six meteorological stations near the project site. To estimate the rainfall at the site, the latitude, longitude, elevation of each station is also used. Although the details of the method used to estimate site-specific rainfall are not published, machine learning methods can be used to interpolate multivariate data.

The Bureau of Meteorology intensity, frequency and duration tables for rainfall, developed in association with Engineers Australia and the University of New South Wales, use a similar method to generate site-specific tables from the latitude and longitude.

B2.1.2 Daily rainfall

Hourly rainfall data over an extended period (some years) are available via request to the Bureau of Meteorology and involves payment of a fee; daily rainfall data for a meteorological station, often including several decades of data, can be extracted by the user from the relevant Bureau of Meteorology web pages (under http://www.bom.gov.au/climate/data/stations/). These files are supplied as .csv files under the 'All years of data' button. The .csv file is accompanied by a .txt file which describes the contents of the .csv file and gives the latitude, longitude and altitude of the station above mean sea level. A file of station details for Queensland can also be downloaded as a fixed width text file from http://www.bom.gov.au/climate/cdo/about/sitedata.shtml. This file is more useful than other similar lists, in that it contains information indicating the period over which data have been collected for each station.

The latitude, longitude, altitude and date fields are used as the independent variables, with rainfall being the dependent variable with which the interpolation is calibrated or trained.

The latitude, longitude, altitude and date for the site when supplied to the interpolation function will return the interpolated rainfall for the site on that date.

Rainfall for the site is to be interpolated for the 365 days preceding the deflection testing.

The nnet procedure of R is used to interpolate daily rainfall based on the latitude, longitude and elevation of the project site.

MS Access database *TMRClimate* has been developed to download and interpolate rainfall data from the nearest six Bureau of Meteorology stations; contact ET_PMG_Director_Pavement_Rehabilitation@tmr.qld.gov.au.

B2.1.3 Daily evaporation

Daily evaporation data are not as readily available as rainfall data; however, evaporation is less erratic than rainfall and mean monthly figures are an acceptable alternative. These figures are represented as a series of online maps (http://www.bom.gov.au/jsp/ncc/climate_averages/evaporation/index.jsp) indicating mean monthly evaporation. A grid file (space delimited .txt file) accompanies these maps and can be downloaded for each month. Table B2.1.3(a) describes the six header records which indicate the structure of the data included in the file. As indicated, the cellsize is 0.25 degrees which corresponds to approximately 30 kilometres.

Table B2.1.3(a) – Bureau of Meteorology evaporation grid file header records

Record	Parameter	Value	Description
1	ncols	178	Number of columns (longitude)
2	nrows	142	Number of rows (latitude)
3	xllcorner	111.875	Lower left corner of x (longitude)
4	yllcorner	-44.625	Lower left corner of y (latitude)
5	cellsize	0.25	Cell size in degrees (~30 km)
6	NODATA_value	-9999	Value for no data (for example, ocean)

Unfortunately, the format of this file is not suited to processing by the interpolation function. For this reason, the *TMRClimate* application is used to combine all 12 monthly grid files into a single table. Because of the long-term nature of these data, this single table does not need to be updated very often (no more than once a year).

The rate of evaporation depends on factors such as cloudiness, air temperature and wind speed; however, mean monthly evaporation ϵ is distinctly cyclic and can be approximated by a sinusoidal function of the month t of the year as indicated in Equation B2.1.3(b).

Equation B2.1.3(b) – Evaporation model

$$\epsilon = a\,sin\big(b(t + c)\big) + d + \varepsilon$$

where seed values for the parameters are estimated as follows

a = the amplitude (half the range) of evaporation $\approx max\big(abs(\in_t - d)\big)$

b = the angular frequency $= \frac{2\pi}{Period\ (months)} = \frac{2\pi}{12}$

c = the seasonal delay with respect to the numbering of the months, varies with the site but typically lies between -10 and -9 and in this range, and can be estimated from the adjacent four months of the year using Equation B2.1.3(c).

Equation B2.1.3(c) – Seasonal delay

$$c \approx \frac{\sum_{t=8}^{11}\left(\frac{arcSin\left(\frac{(\in_t - d)}{a}\right)}{b} - t\right)}{4}$$

Equation B2.1.3(d) – Mean daily evaporation

d = mean daily evaporation $= \frac{\sum_{t=1}^{12}\in_t}{12}$

ε = error term representing a small variation from the sinusoid.

The annual evaporation cycle at a specific site can be characterised by four parameters plus a small error.

The *TMRClimate* application uses the nnet procedure of R to interpolate daily rainfall based on the latitude and longitude of the project site and month of the data.

The same application uses the interpolated evaporation data to fit a sinusoidal model using the 'nls' procedure of R Statistical Computing Language.

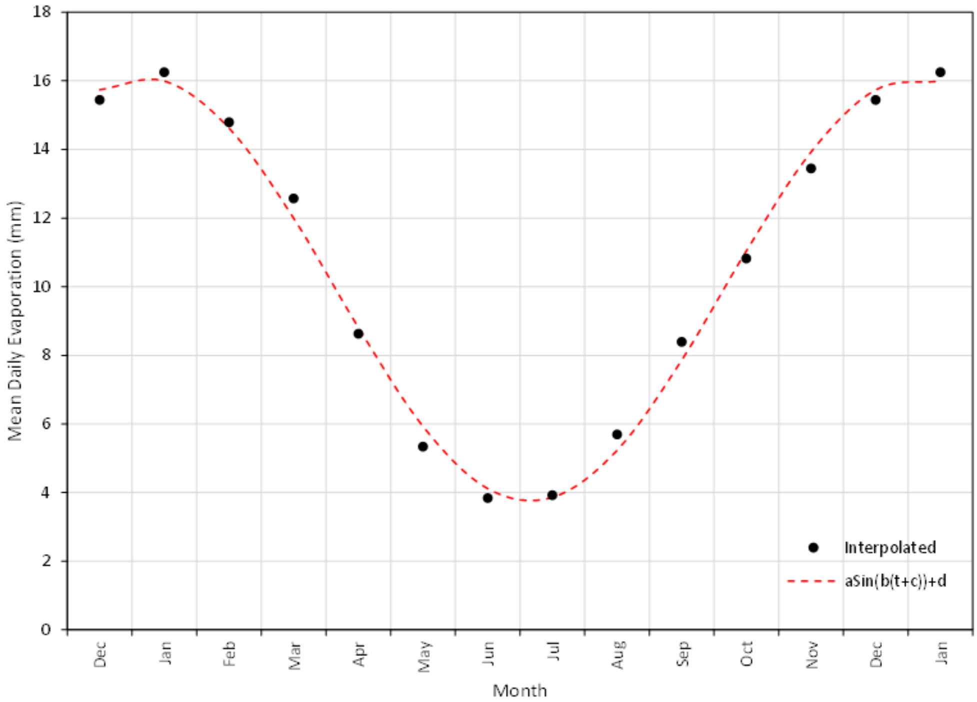
Table B2.1.3(b) presents the evaporation cycle parameters for several sites produced with the *TMRClimate* application.

Table B2.1.3(b) – Evaporation cycle parameters for selected sites

Location	Latitude (degrees)	Longitude (degrees)	Amplitude a (mm)	Phase c (months)	Mean daily evaporation d (mm)	R^2
Birdsville Aero	-25.8983	139.3536	6.17	-9.65	9.93	0.9924
Longreach Aero	-23.4397	144.2828	4.35	-9.59	6.17	0.9947
Brisbane Aero	-27.4178	153.1142	1.69	-9.11	3.95	0.9922
Cairns Aero	-16.8736	145.7458	2.70	-9.75	3.24	0.9756

Figure B2.1.3 compares the interpolated mean daily evaporation for Birdsville Airport with the modelled evaporation.

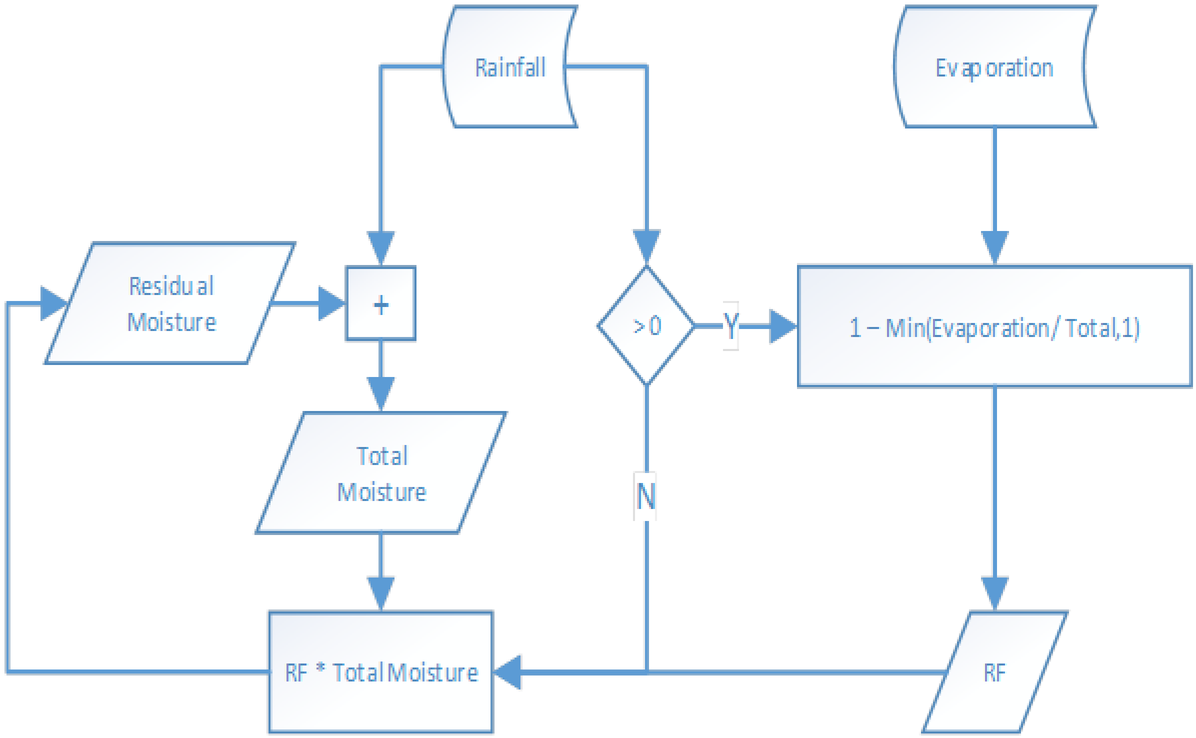
Figure B2.1.3 – Annual evaporation cycle Birdsville Aero



B2.1.4 Moisture Loss

Figure B2.1.4(a) describes the algorithm used to obtain daily moisture levels.

Figure B2.1.4(a) – Daily moisture loss algorithm



Adjustment of pavement deflection data is applied to restore deflections to the equilibrium moisture state of the pavement rather than a 'wet' state, which is an infrequent and variable occurrence (refer Section B3.1). A model of the pavement moisture loss following a series of rainfall events is required.

The reduction in pavement moisture levels is assumed proportional to the evaporation, although runoff will likely account for additional losses in the days immediately following the rainfall event.

Equation B2.1.3(b) is used to compute the mean daily evaporation for a specific month. To estimate the mean daily evaporation on the day d in the year, Equation B2.1.4 is used to convert to a month equivalent time t.

Equation B2.1.4 – Convert day to month

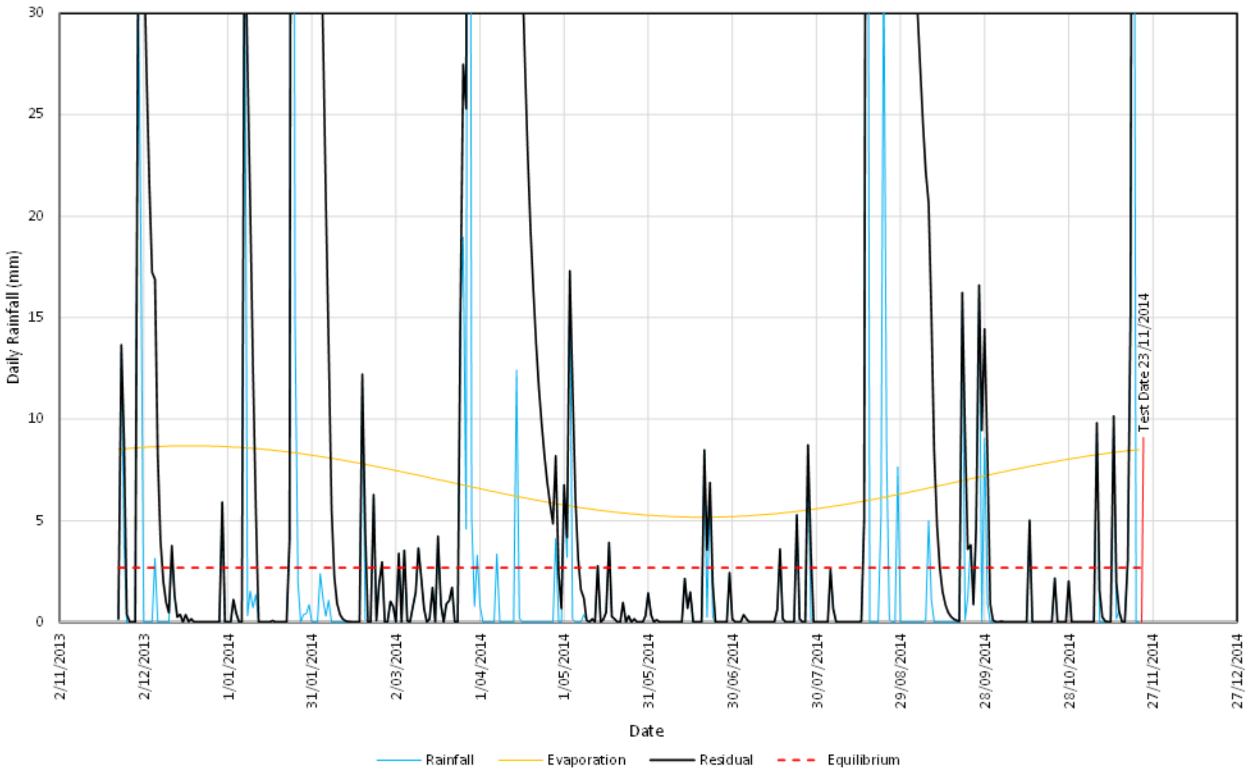
$$t \approx \frac{12d}{365} + 0.5$$

If the residual moisture level is above the long-term mean daily rainfall, the season factor of the DMAF is set to 0 (wet): otherwise, it is set to 1 (dry) and the DMAF will be greater than 1. Further calibration of this model could allow intermediate values to be adopted.

The *TMRClimate* application has been developed to implement this procedure and provides important site-specific dryback estimates.

Figure B2.1.4(b) plots simulated moisture levels from data interpolated from site latitude, longitude and elevation using the *TMRClimate* application.

Figure B2.1.4(b) – Simulated site-specific moisture levels



B3 Climate variable

B3.1 Extreme rainfall

The Bureau of Meteorology web site (http://www.bom.gov.au/water/designRainfalls/revised-ifd/?) provides site-specific estimates of extreme rainfall events in the form of intensity, frequency and duration tables based on the latitude and longitude of the site.

Note: The frequency (return period) has now been replaced by the AEP.

The AEP 20% (5-year return period) 15-minute duration rainfall was found to have the highest correlation (of all intensity, frequency and duration tables' entries) with the DMAF climate variable. The relationship between this rainfall, $\rho_{AEP20\%15}$, and the climate variable is defined in Equation B3.1.

Equation B3.1 – Climate variable

$$Climate\ Variable = a\left(\frac{\rho_{AEP20\%15}}{b}\right)^{c}$$

where

a = 59.81

b = 50 and

c = 8.75

Equation B3.1 is intended for interpolation purposes, and not for extrapolation; values of the climate variable should be limited to the range 0.08 (Semi-arid) to 1 (Tropical).

Defined in this way, the climate variable is a function of the latitude and longitude of the site. The DMAF is now related specifically to the likelihood of an extreme rainfall event, in common with other types of infrastructure whose design makes use of the intensity, frequency and duration tables.

Appendix C – Overlay design figures for deflection reduction method

C1 Assumptions

Several assumptions have been made to develop overlay design aids for unbound granular pavements with thin bituminous surfacing:

- The modulus of the granular material increases with thickness as described by Equation C1(a) (AGPT02).

Equation C1(a) – Layer support theory

$$E_{v\ granular} = E_{v\ subgrade}\ 2^{\left(\frac{total\ granular\ thickness}{125}\right)}$$

- The subgrade modulus and CBR are related (*Technical Basis for the Pavement Design Manual* (Angell, 1988)) as indicated in Equation C1(b).

Equation C1(b) – Subgrade modulus as a function of California Bearing Ratio

$$E_{subgrade} = 19\ CBR_{subgrade}^{0.68}$$

- Granular material is modelled as having a maximum achievable modulus of 350 MPa (CBR 80).
- All unbound granular layers are modelled as cross-anisotropic material (Degree of Anisotropy equal to 2).
- Granular layers are modelled with a Poisson's Ratio of 0.35.
- The subgrade is modelled with a Poisson's Ratio of 0.45.
- To obtain deflection values, a FWD is modelled as a 150mm-radius, uniformly-loaded plate with a contact stress of 566 kPa (40 kN load).
- To obtain the design traffic, a full standard axle model is used with a 92mm-radius contact area for each tyre with a contact stress of 750 kPa, tyres separated by 330mm and wheel gear separated by 1800mm.
- A traffic multiplier of 1.6 was adopted.

C1.1 Tolerable Deflection

Figure C1.1(a) – Tolerable deflection for normal design standard for falling weight deflectometer and Benkelman Beam with 40 kN loading

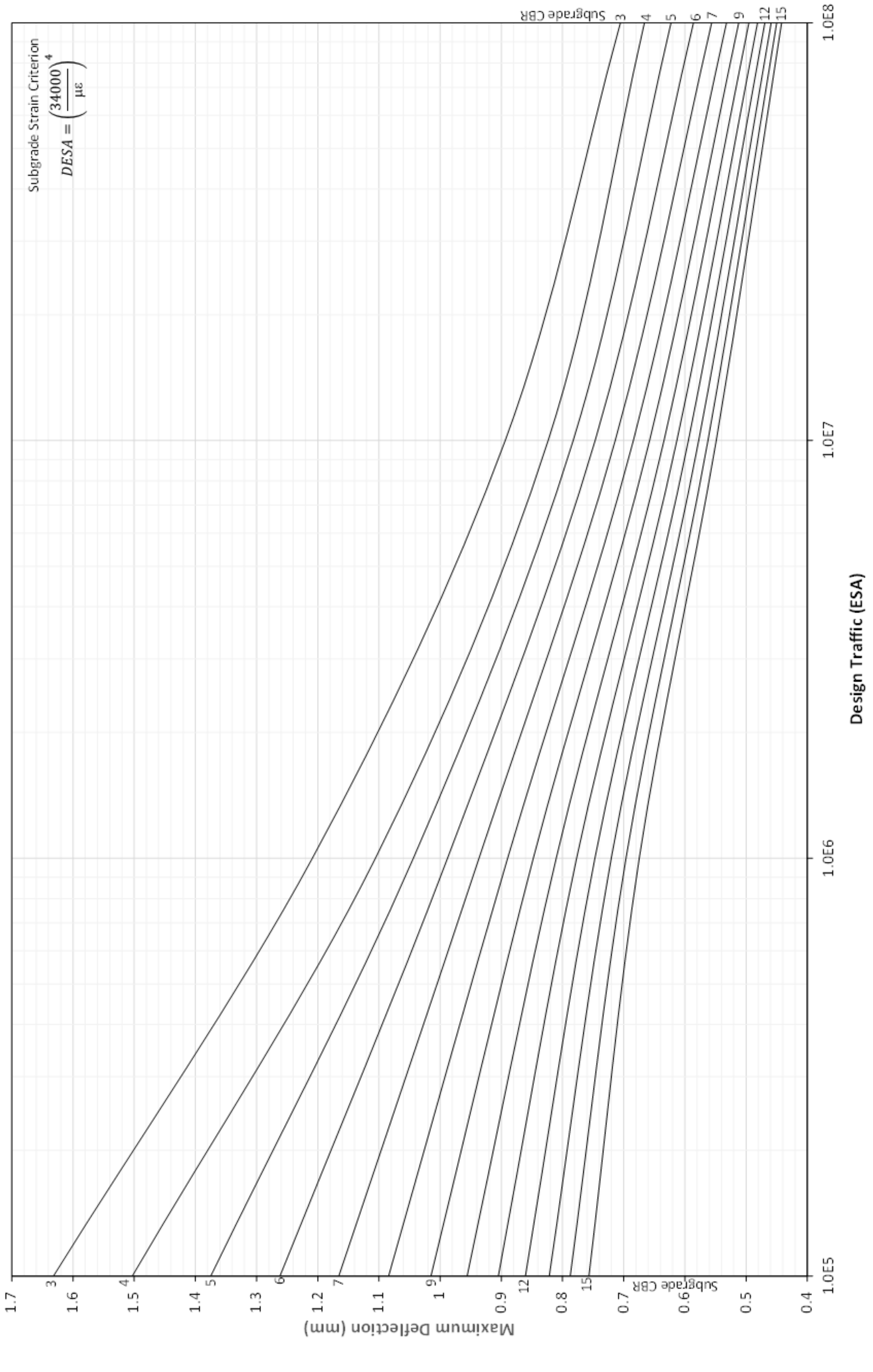


Figure C1.1(b) – Tolerable deflection for second design standard for falling weight deflector and Benkelman Beam with 40 kN loading

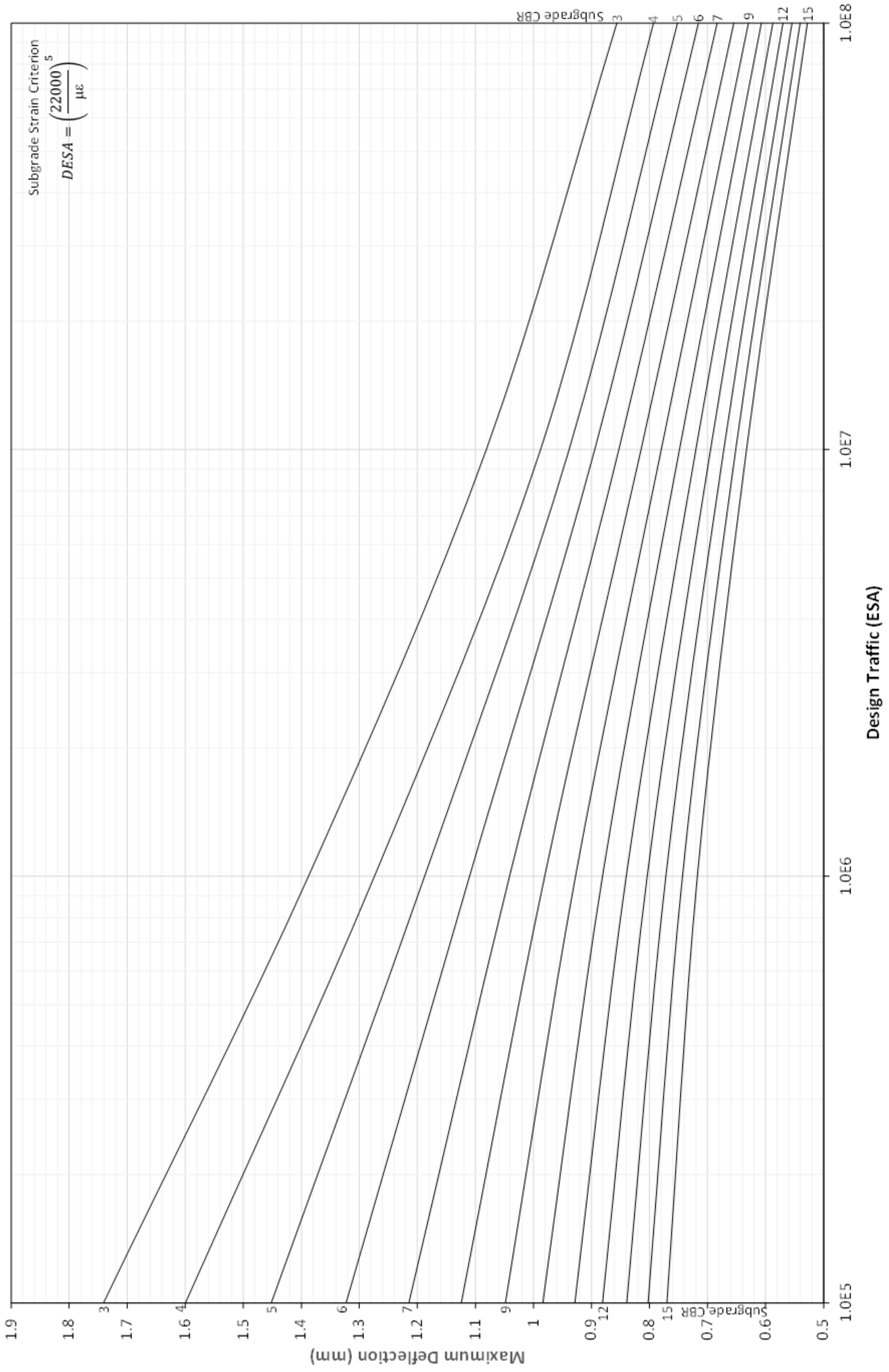


Figure C1.1(c) – Tolerable deflection for moderate to heavy traffic for deflectograph

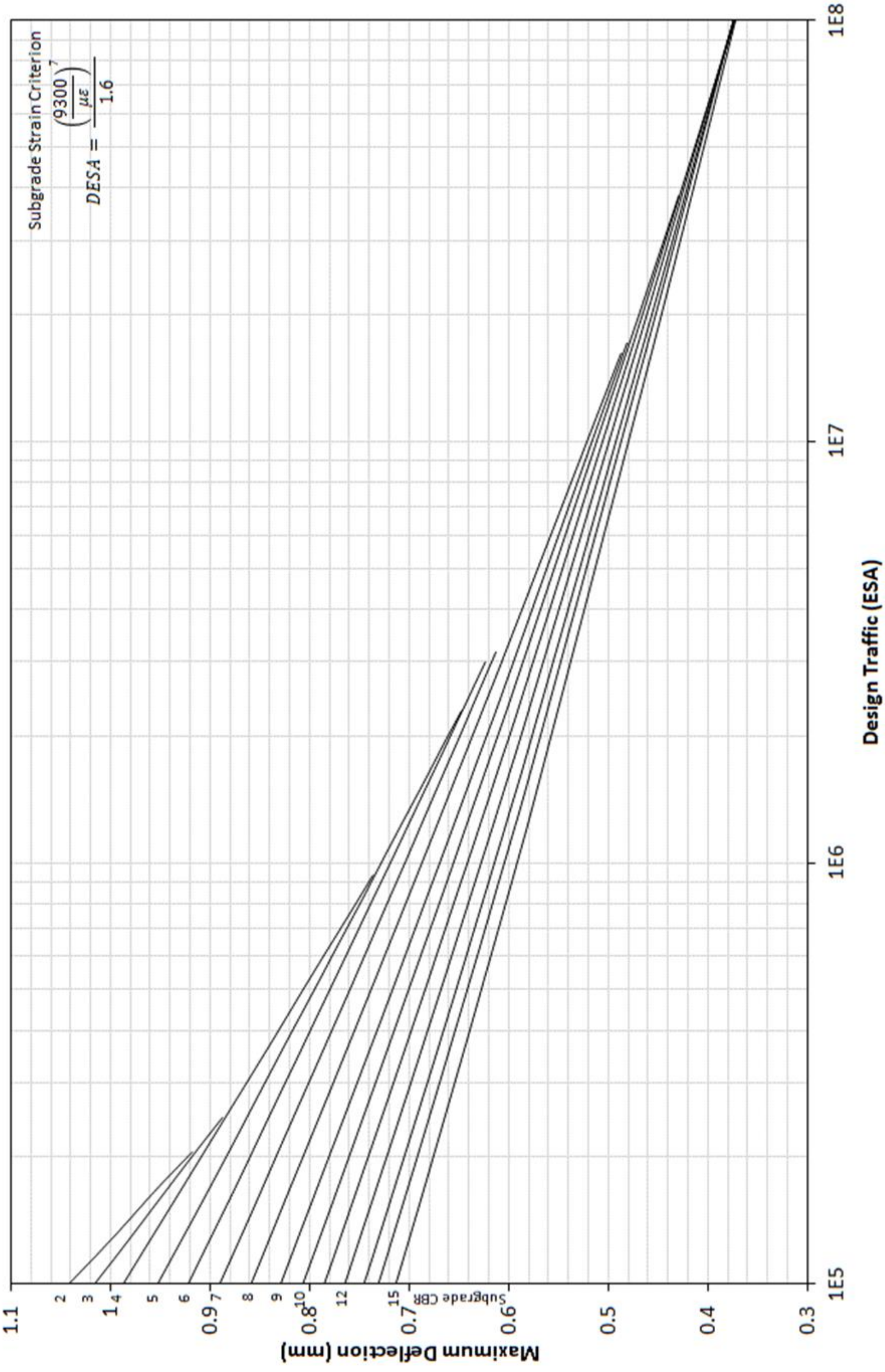
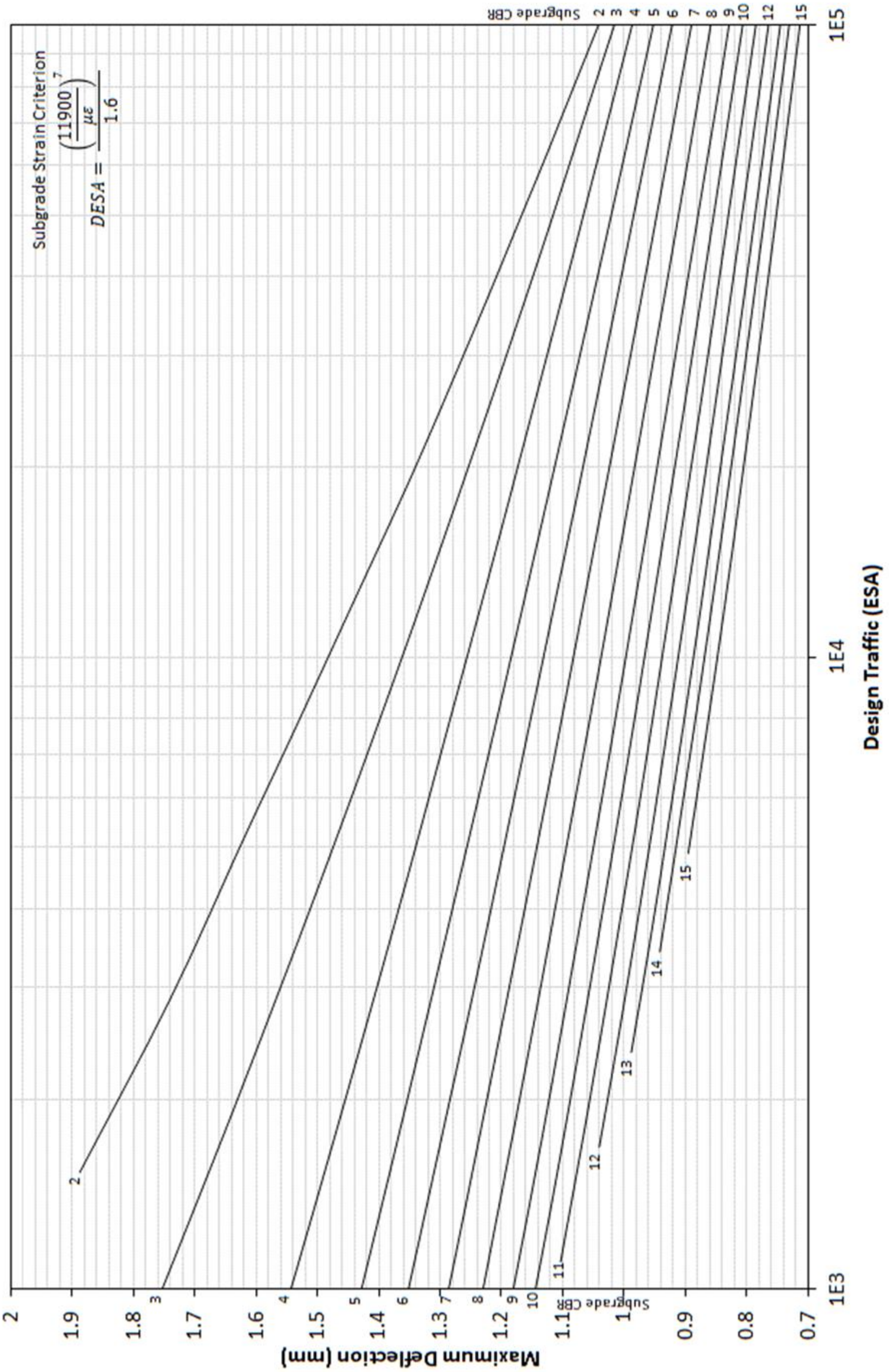


Figure C1.1(d) – Tolerable deflection for light traffic for deflectograph



C1.2 Granular overlay design

Figure C1.2(a) – Granular overlay design chart where all overlay material complies with MRS05 and MRTS05 and has a minimum California Bearing Ratio as indicated (subgrade CBR 3)

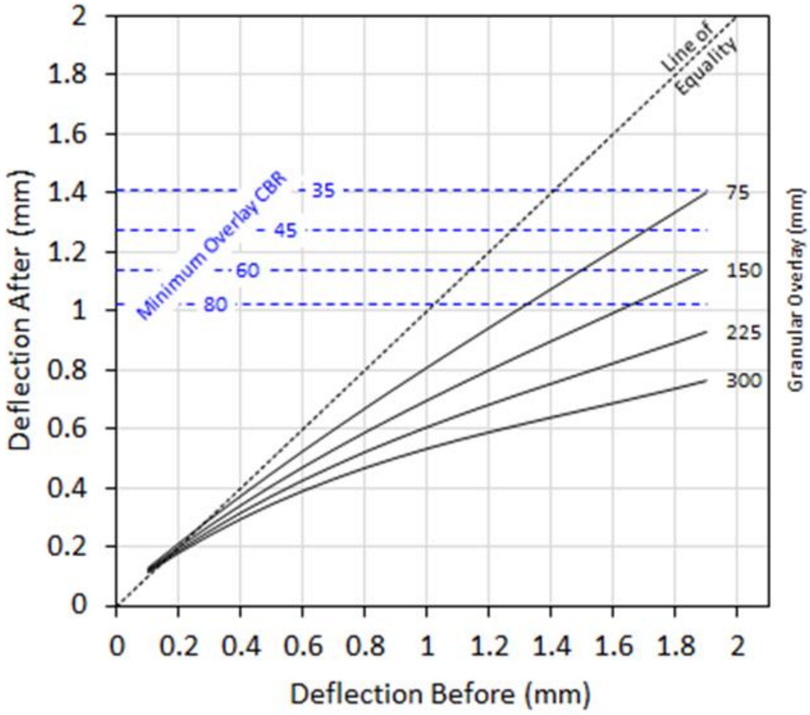
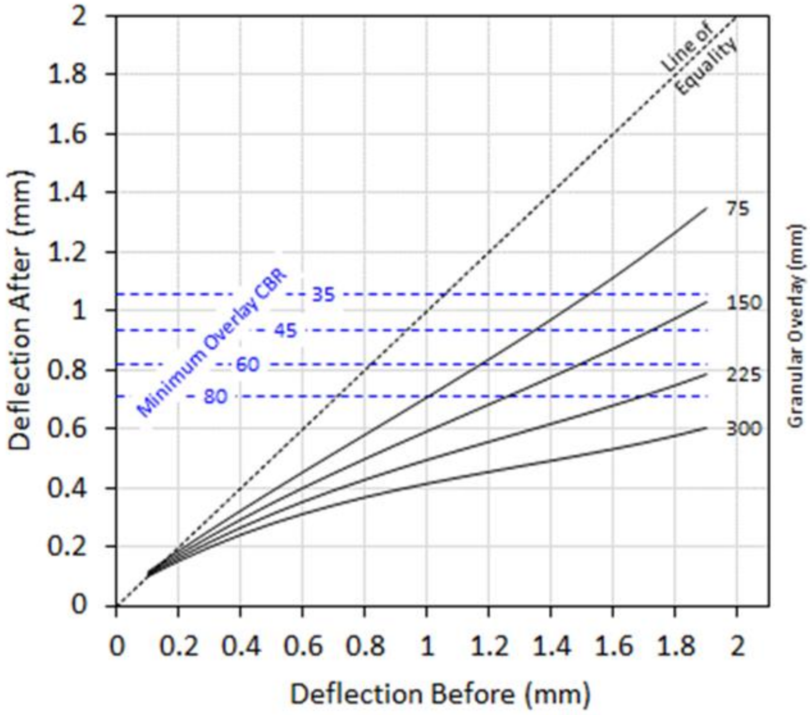


Figure C1.2(b) – Granular overlay design chart where all overlay material complies with MRS05 and MRTS05 and has a minimum California Bearing Ratio as indicated (subgrade CBR 15)



These charts have been produced using the Transport and Main Roads spreadsheet *UnBound Granular deflection reduction*. These charts should be used where subgrade CBR lies between 3–15.

C1.3 Asphalt overlay deflection reduction

Figure C1.3(a) – Asphalt overlay deflection reduction design chart for weighted mean annual average pavement temperature Zone 1 and a heavy vehicle operating speed of 50 km/h

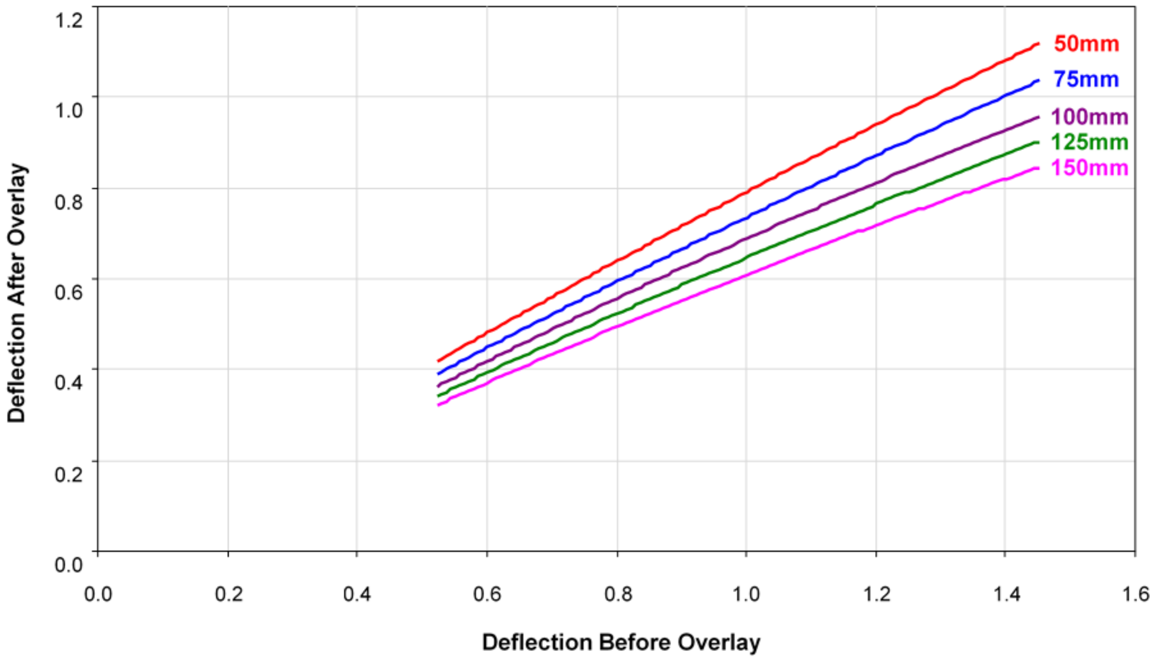


Figure C1.3(b) – Asphalt overlay deflection reduction design chart for weighted mean annual average pavement temperature Zone 1 and a heavy vehicle operating speed of 80 km/h

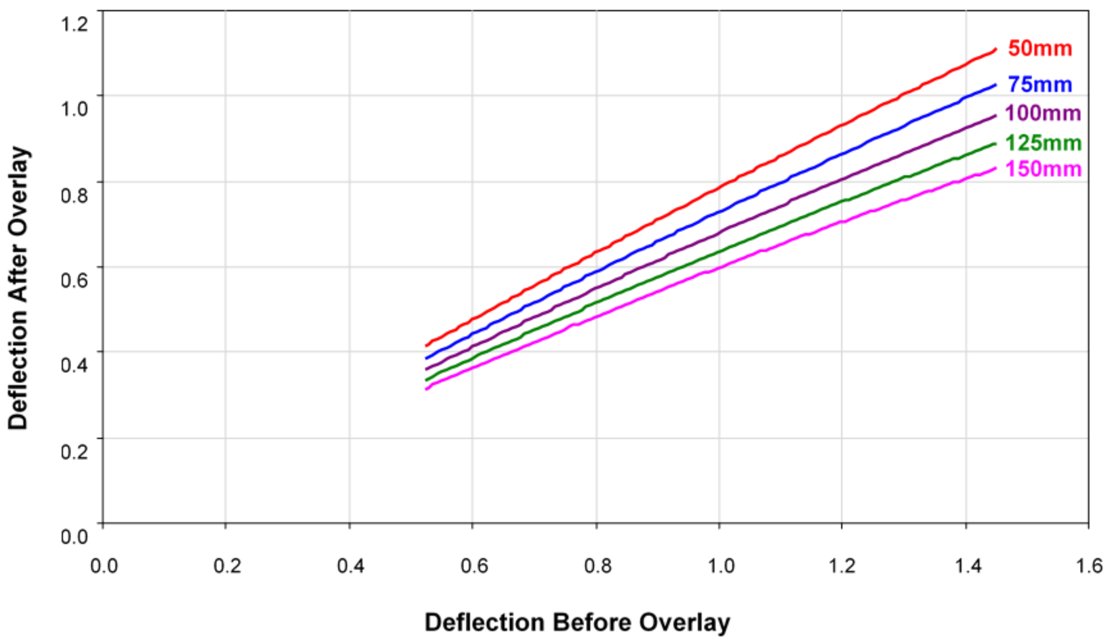


Figure C1.3(c) – Asphalt overlay deflection reduction design chart for weighted mean annual average pavement temperature Zone 2 and a heavy vehicle operating speed of 50 km/h

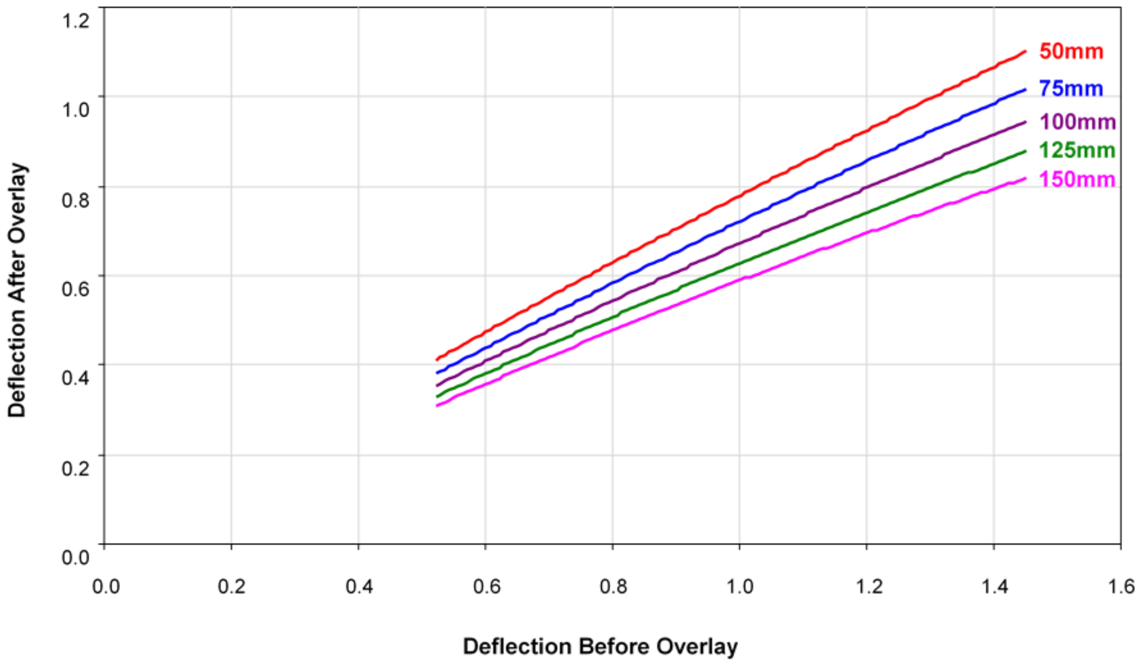


Figure C1.3(d) – Asphalt overlay deflection reduction design chart for weighted mean annual average pavement temperature Zone 2 and a heavy vehicle operating speed of 80 km/h

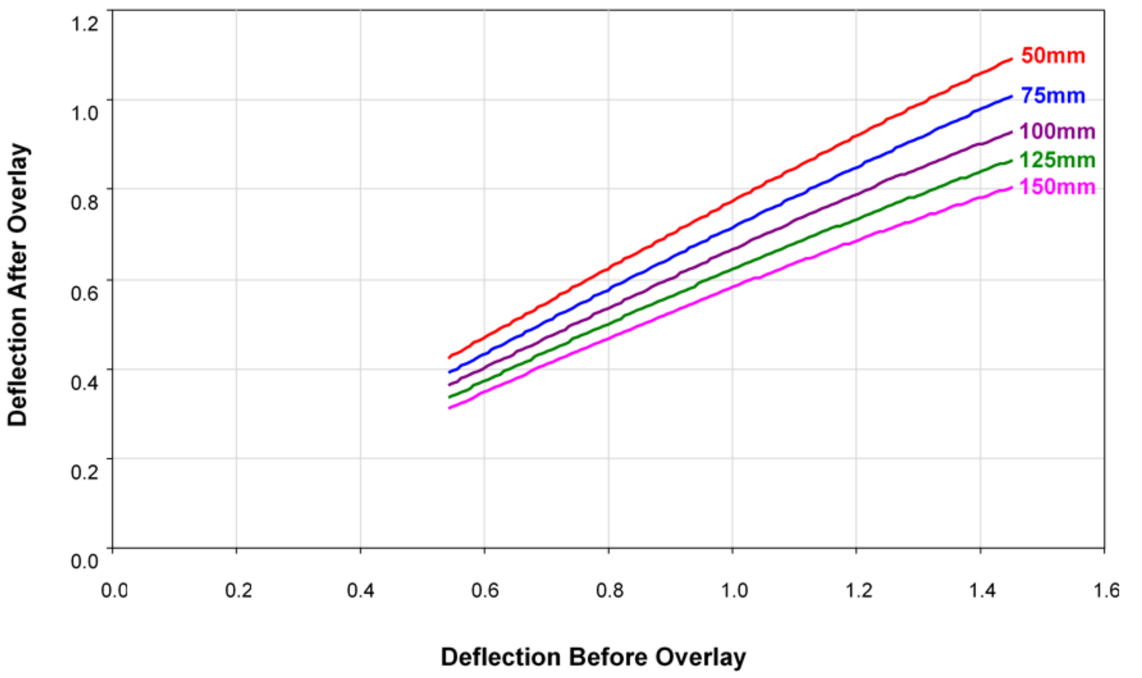


Figure C1.3(e) – Asphalt overlay deflection reduction design chart for weighted mean annual average pavement temperature Zone 3 and a heavy vehicle operating speed of 50 km/h

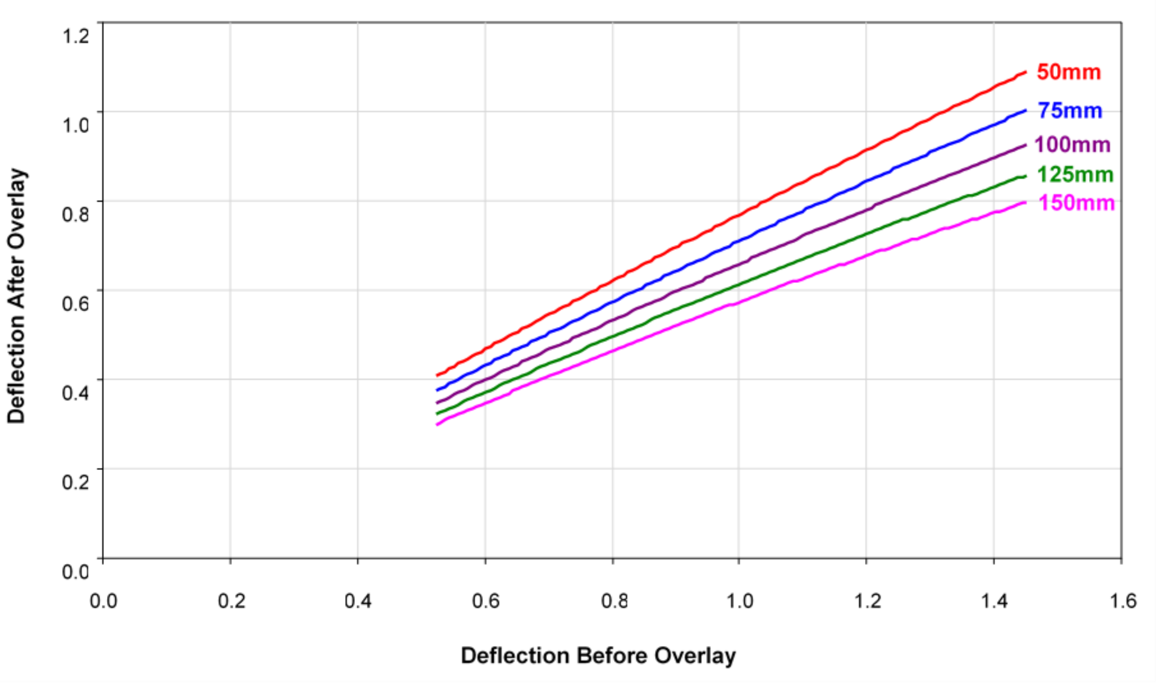


Figure C1.3(f) – Asphalt overlay deflection reduction design chart for weighted mean annual average pavement temperature Zone 3 and a heavy vehicle operating speed of 80 km/h

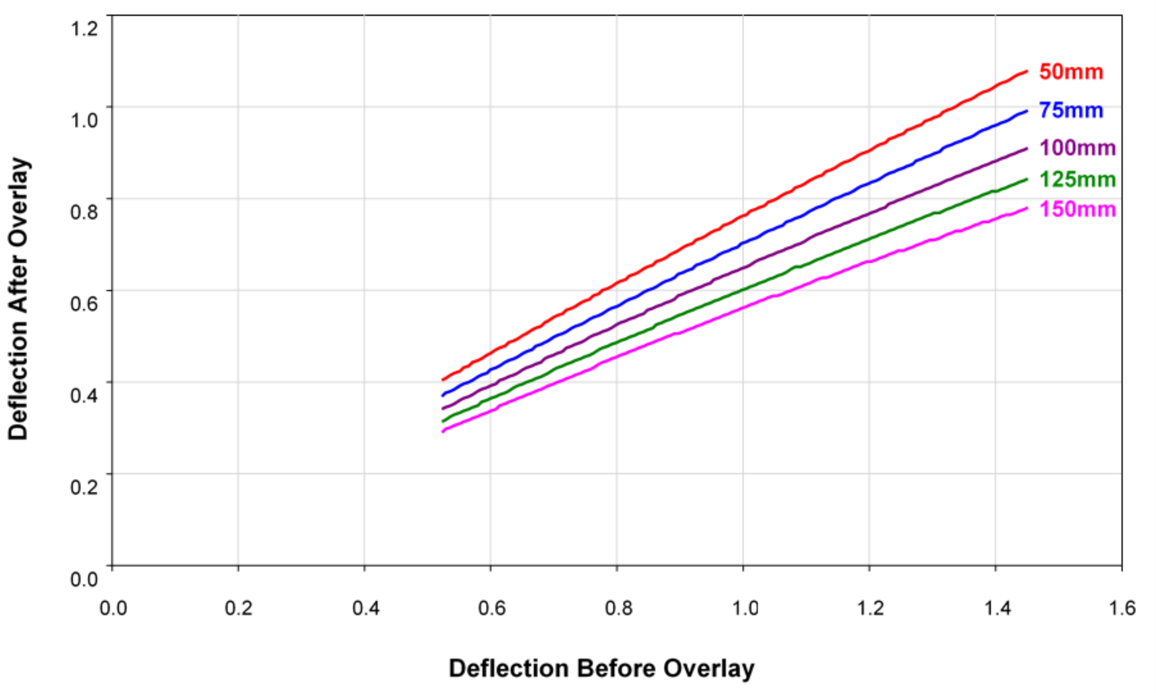


Figure C1.3(g) – Asphalt overlay deflection reduction design chart for weighted mean annual average pavement temperature Zone 4 and a heavy vehicle operating speed of 50 km/h

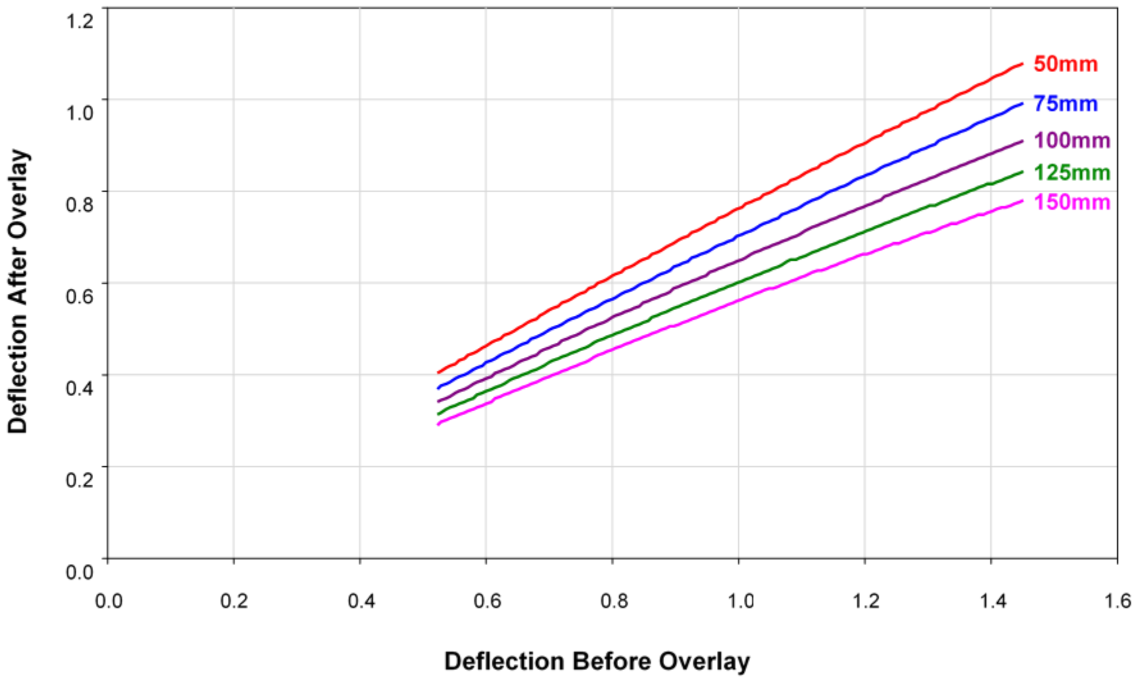
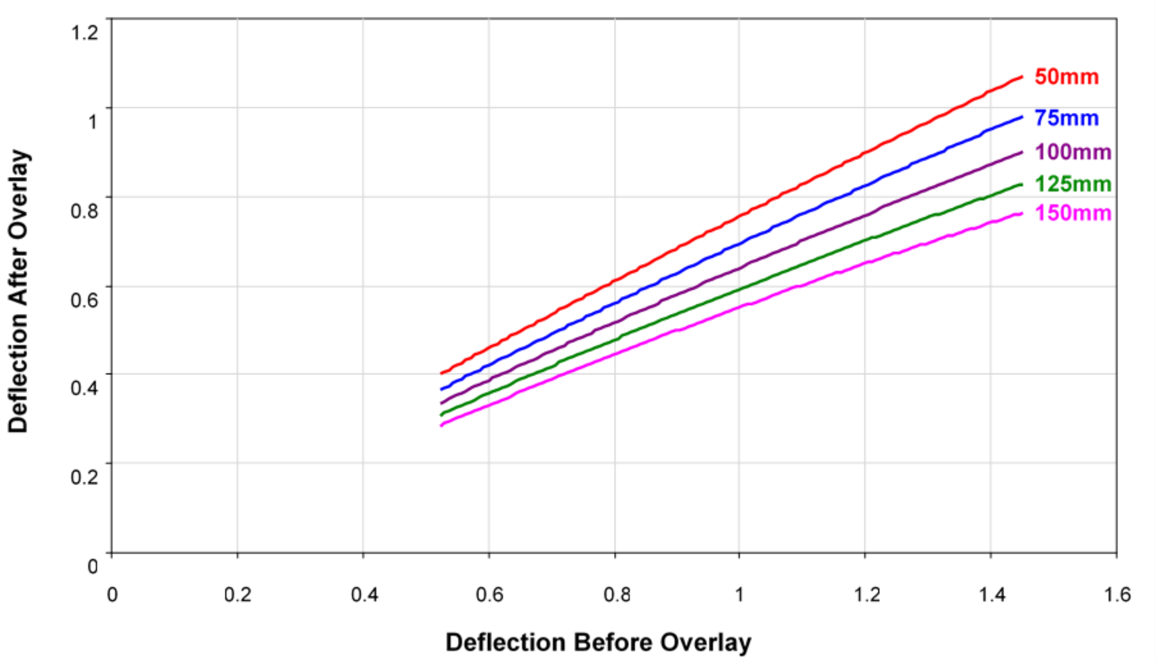


Figure C1.3(h) – Asphalt overlay deflection reduction design chart for weighted mean annual average pavement temperature Zone 4 and a heavy vehicle operating speed of 80 km/h



C1.4 Asphalt overlay curvature reduction

Figure C1.4(a) – Asphalt overlay curvature reduction design chart for weighted mean annual average pavement temperature Zone 1 and a heavy vehicle operating speed of 50 km/h

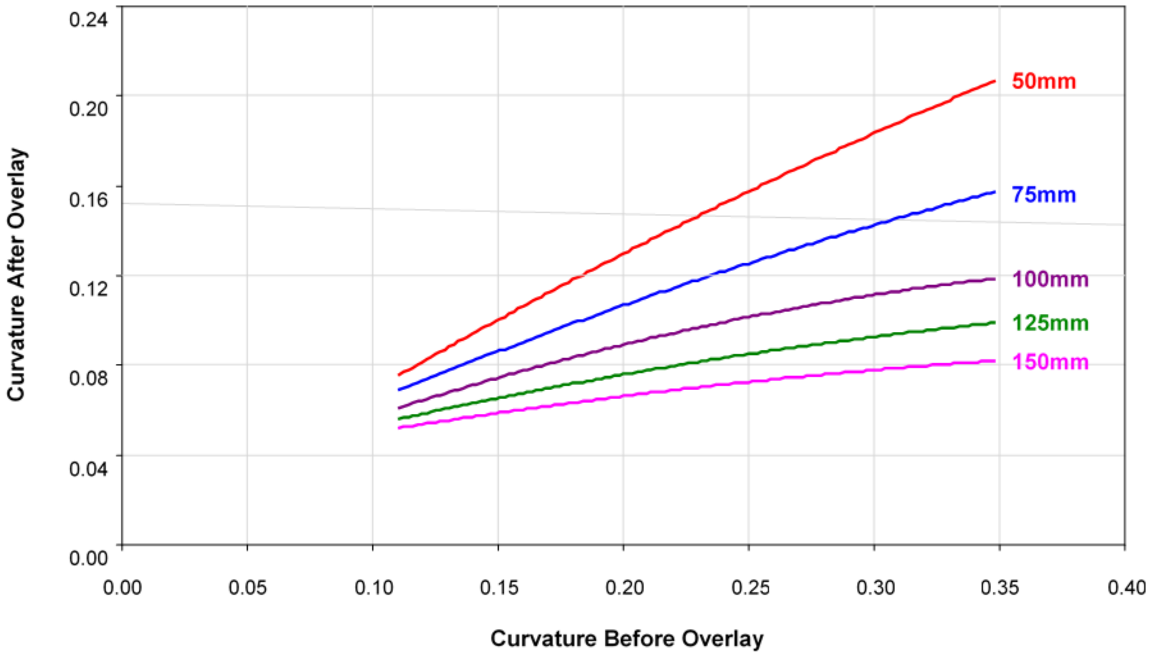


Figure C1.4(b) – Asphalt overlay curvature reduction design chart for weighted mean annual average pavement temperature Zone 1 and a heavy vehicle operating speed of 80 km/h

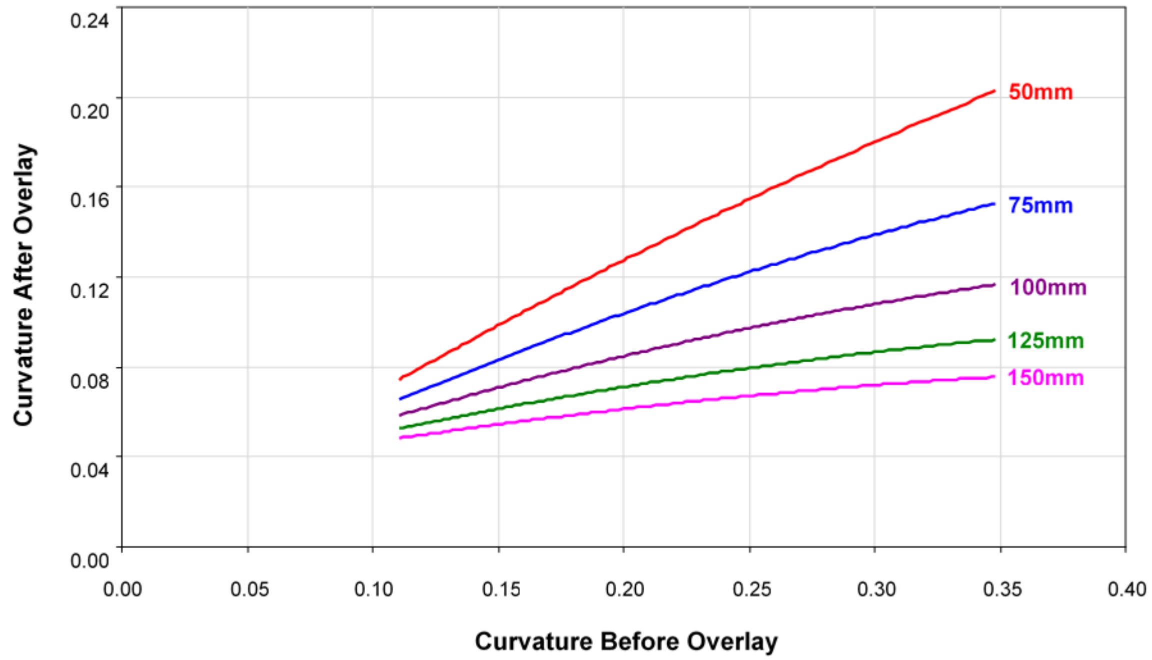


Figure C1.4(c) – Asphalt overlay curvature reduction design chart for weighted mean annual average pavement temperature Zone 2 and a heavy vehicle operating speed of 50 km/h

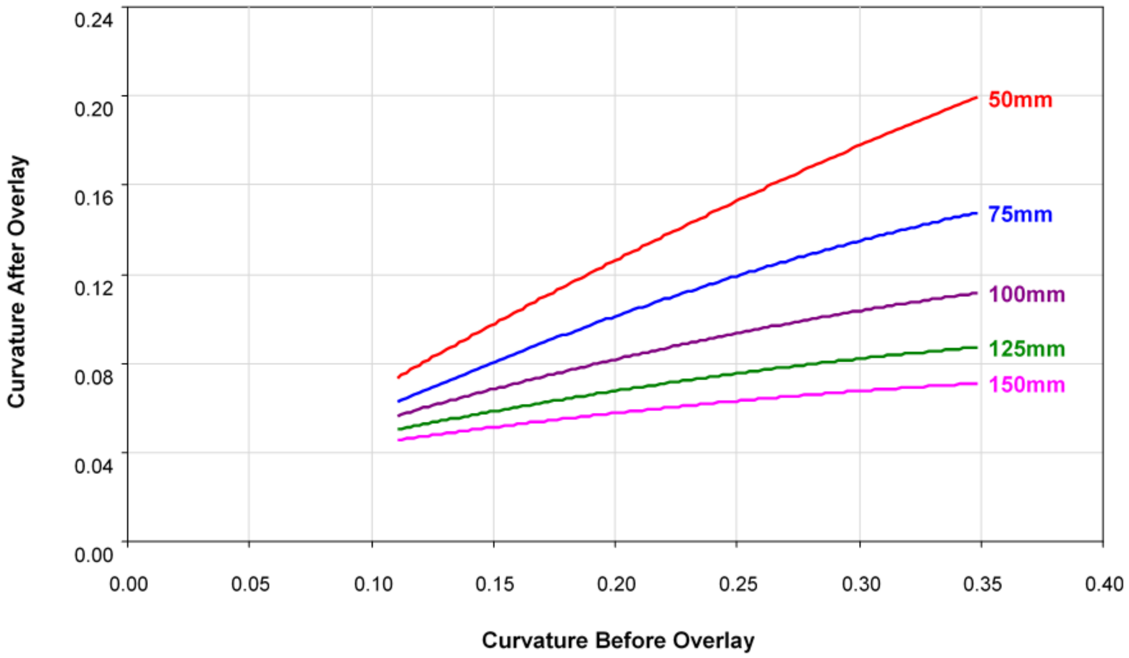


Figure C1.4(d) – Asphalt overlay curvature reduction design chart for weighted mean annual average pavement temperature Zone 2 and a heavy vehicle operating speed of 80 km/h

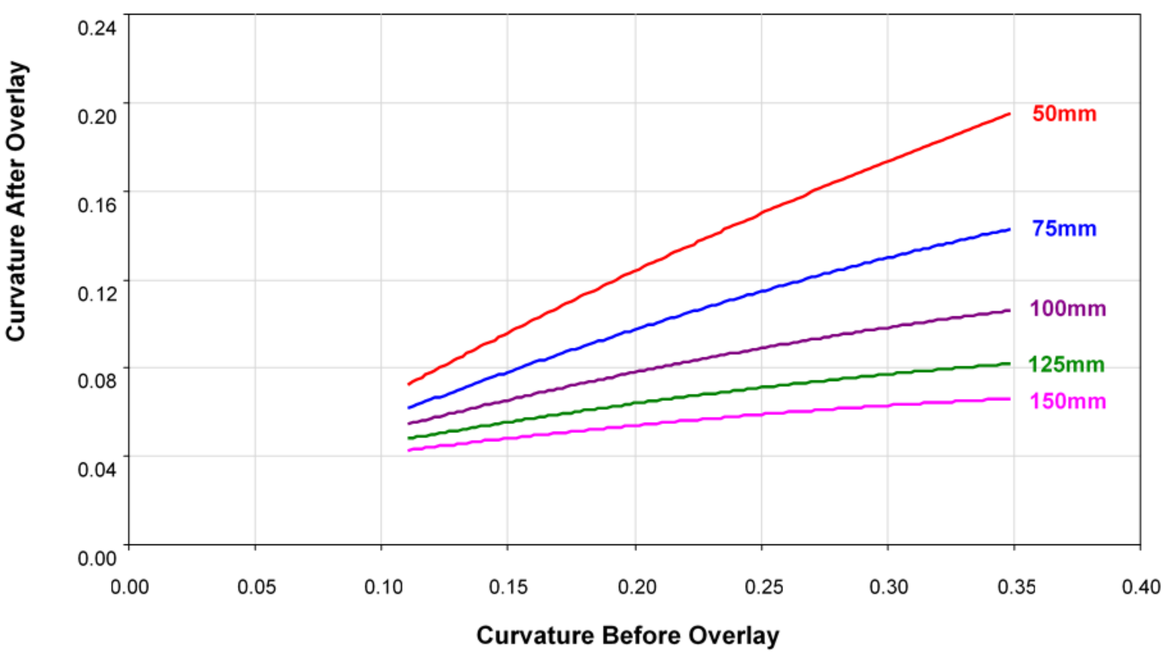


Figure C1.4(e) – Asphalt overlay curvature reduction design chart for weighted mean annual average pavement temperature Zone 3 and a heavy vehicle operating speed of 50 km/h

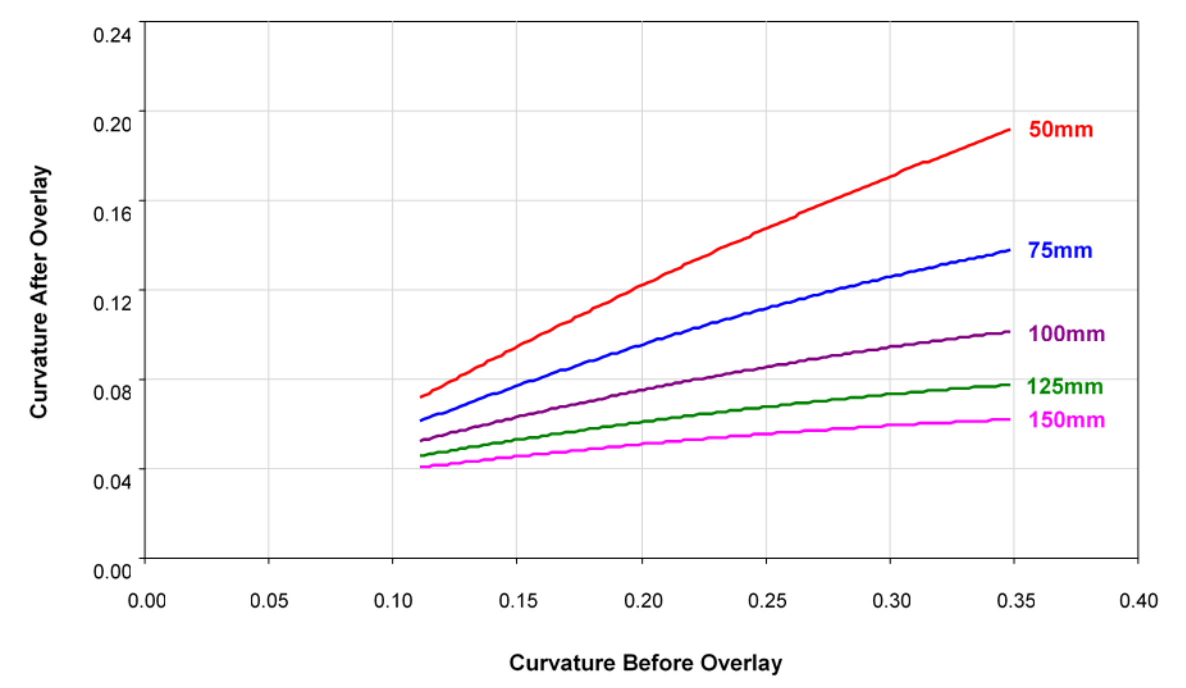


Figure C1.4(f) – Asphalt overlay curvature reduction design chart for weighted mean annual average pavement temperature Zone 3 and a heavy vehicle operating speed of 80 km/h

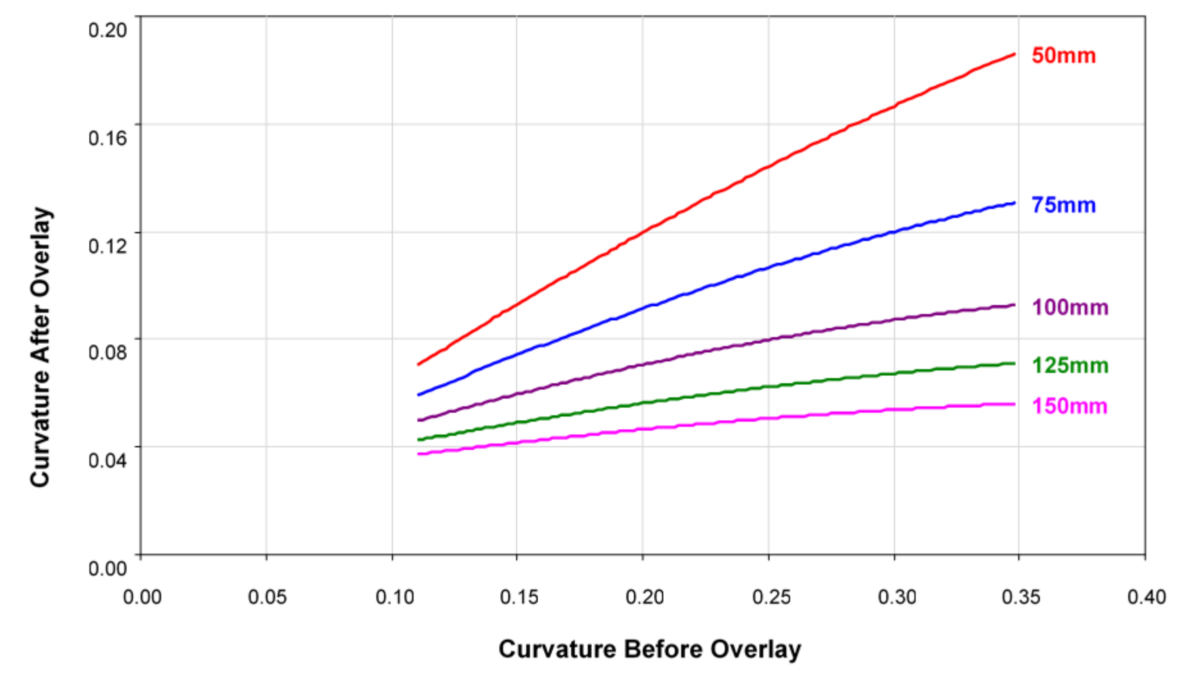


Figure C1.4(g) – Asphalt overlay curvature reduction design chart for weighted mean annual average pavement temperature Zone 4 and a heavy vehicle operating speed of 50 km/h

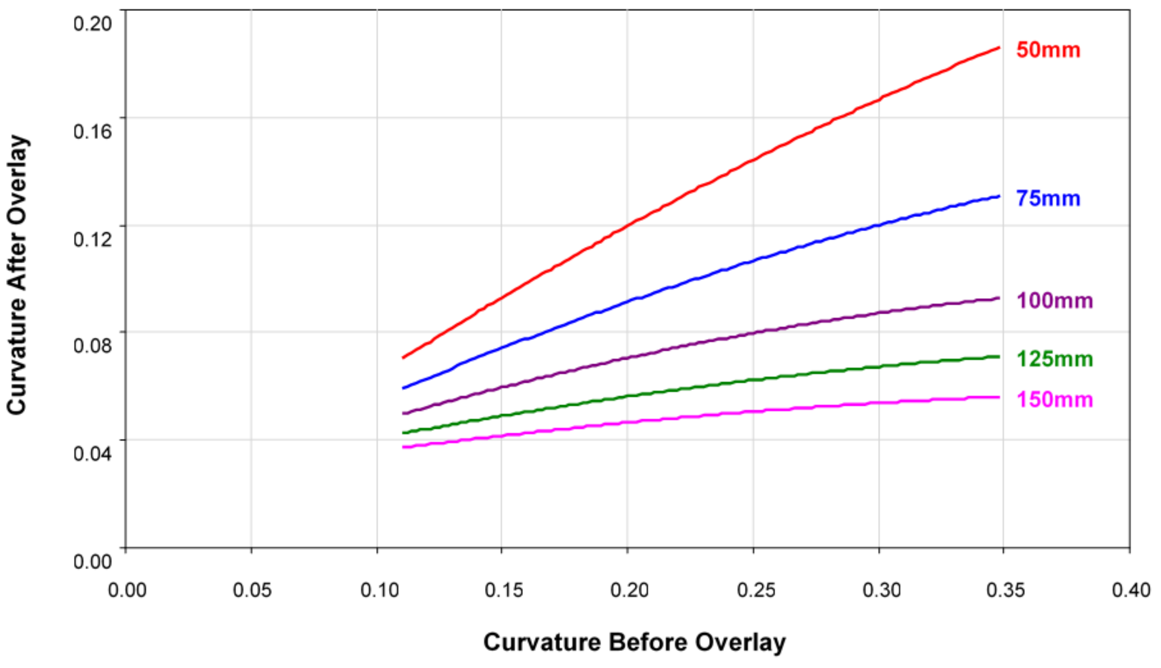
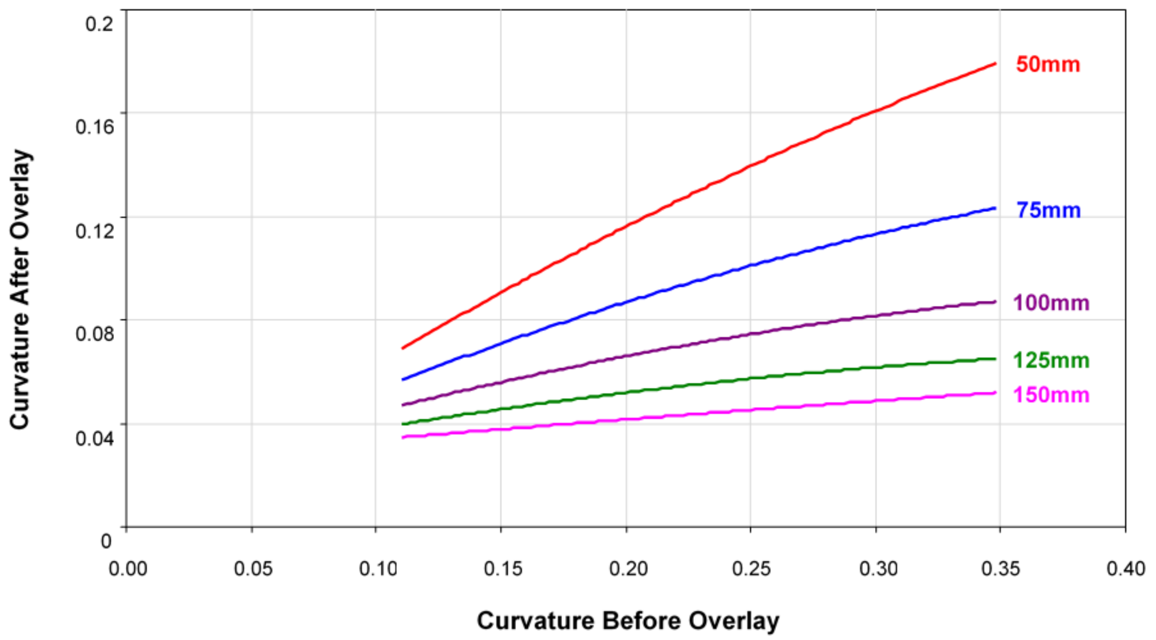


Figure C1.4(h) – Asphalt overlay curvature reduction design chart for weighted mean annual average pavement temperature Zone 4 and a heavy vehicle operating speed of 80 km/h



C1.5 Asphalt overlay curvature design traffic

Figure C1.5(a) – Asphalt overlay curvature versus design traffic loading design chart for overlay thicknesses from 75–150mm for weighted mean annual average pavement temperature Zone 1 and a heavy vehicle operating speed of 50 km/h

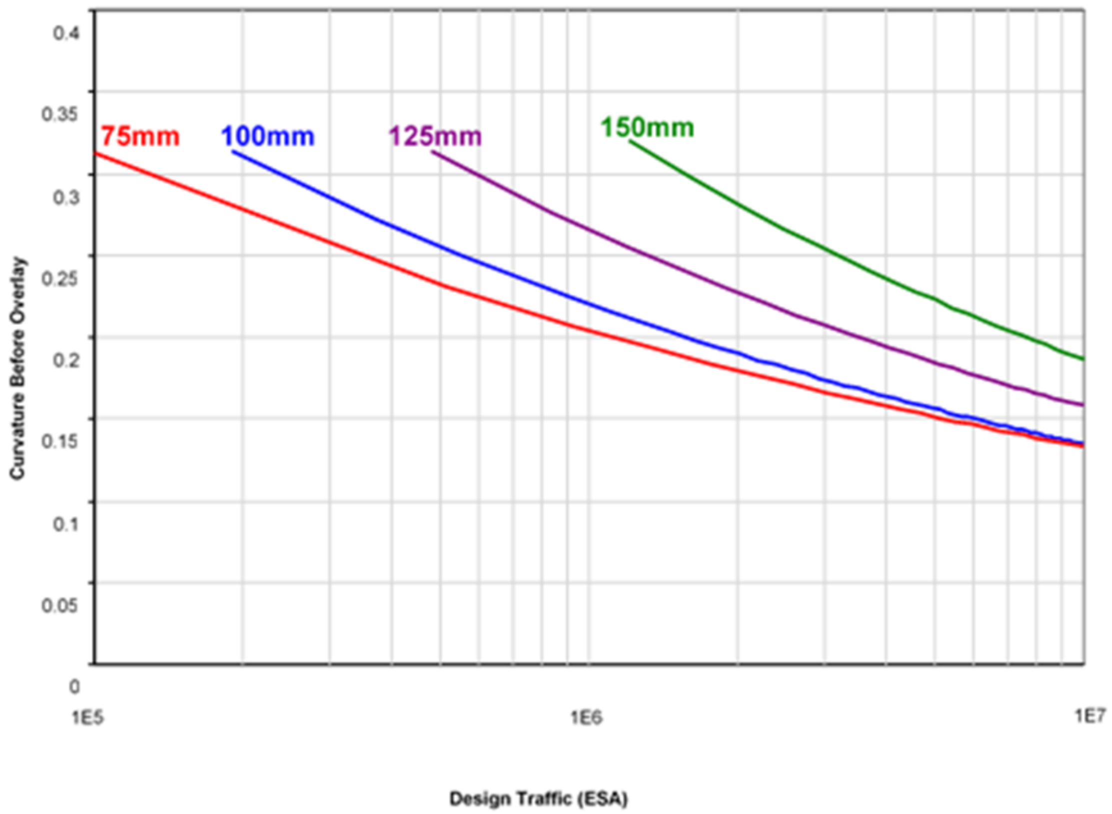


Figure C1.5(b) – Asphalt overlay curvature versus design traffic loading design chart for an overlay thickness of 50mm for weighted mean annual average pavement temperature Zone 1 and a heavy vehicle operating speed of 50km/h

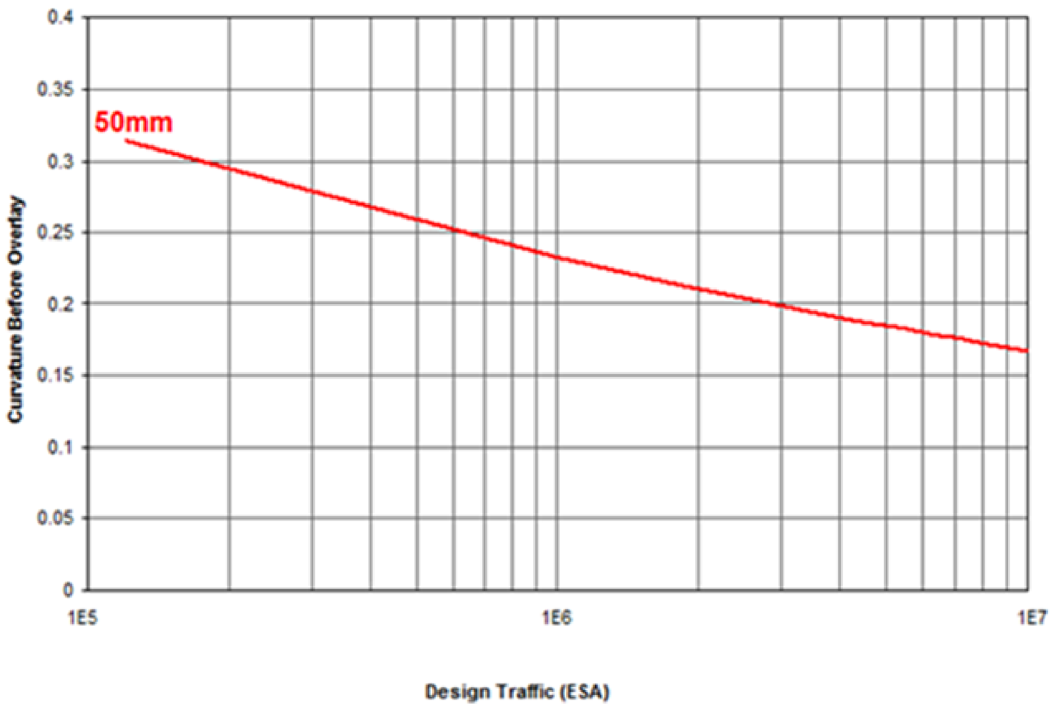


Figure C1.5(c) – Asphalt overlay curvature versus design traffic loading design chart for overlay thicknesses from 75–150mm for weighted mean annual average pavement temperature Zone 1 and a heavy vehicle operating speed of 80 km/h

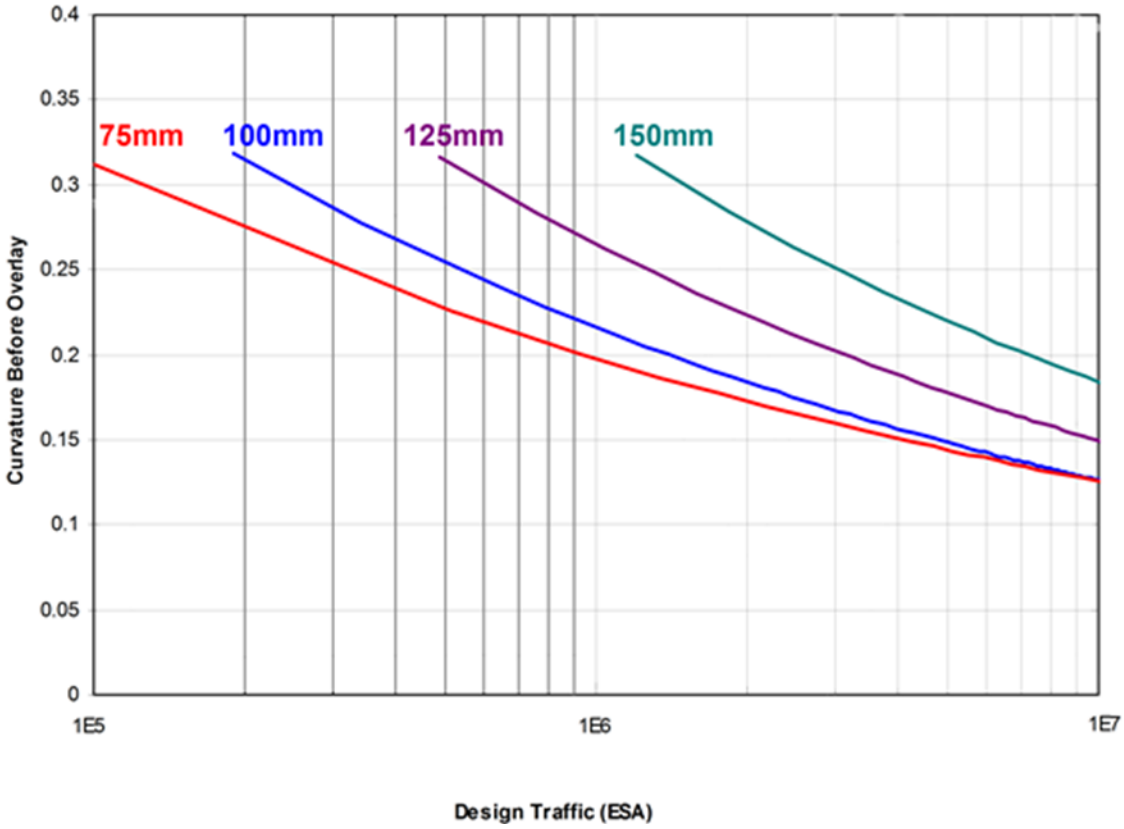


Figure C1.5(d) – Asphalt overlay curvature versus design traffic loading design chart for an overlay thickness of 50mm for weighted mean annual average pavement temperature Zone 1 and a heavy vehicle operating speed of 80 km/h

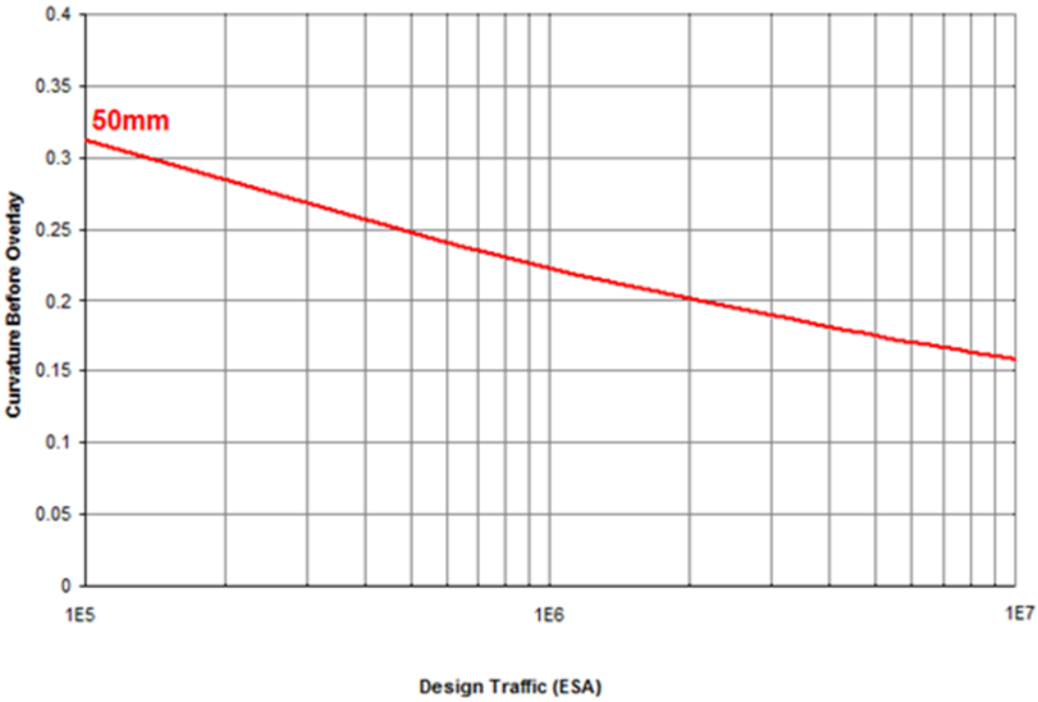


Figure C1.5(e) – Asphalt overlay curvature versus design traffic loading design chart for overlay thicknesses from 75–150mm for weighted mean annual average pavement temperature Zone 2 and a heavy vehicle operating speed of 50 km/h

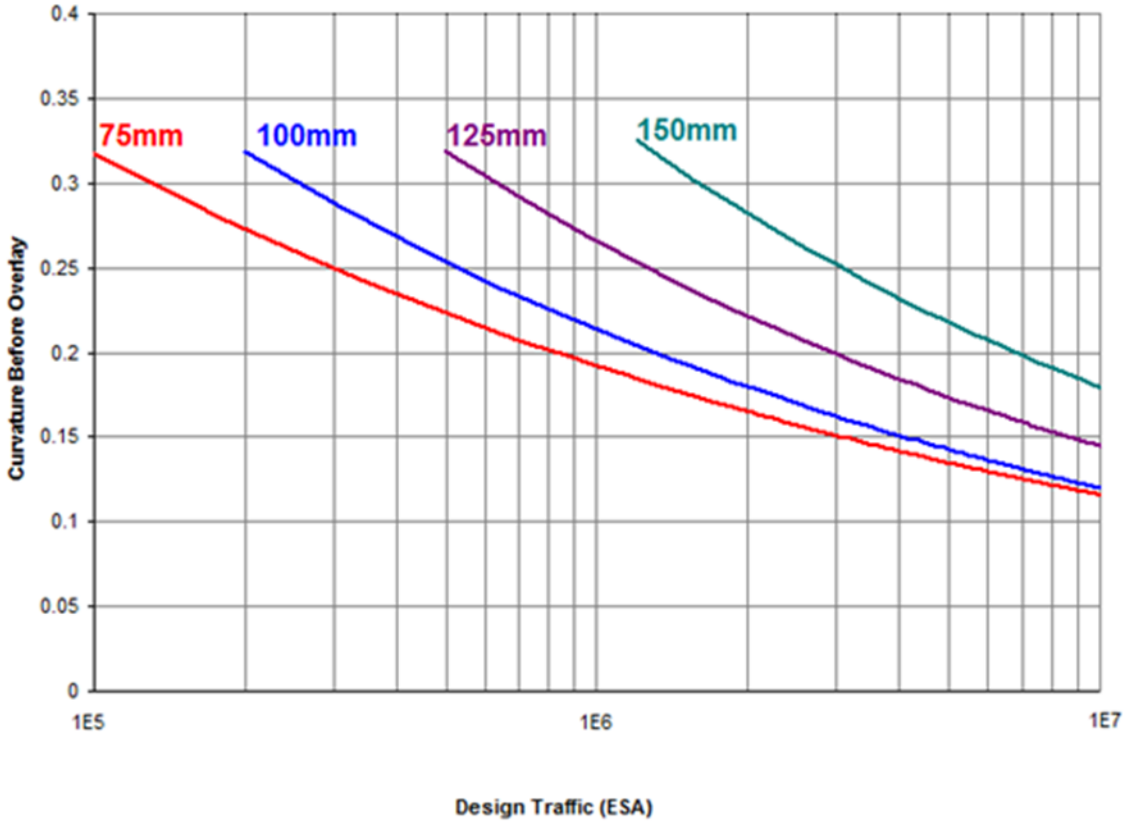


Figure C1.5(f) – Asphalt overlay curvature versus design traffic loading design chart for an overlay thickness of 50mm for weighted mean annual average pavement temperature Zone 2 and a heavy vehicle operating speed of 50 km/h

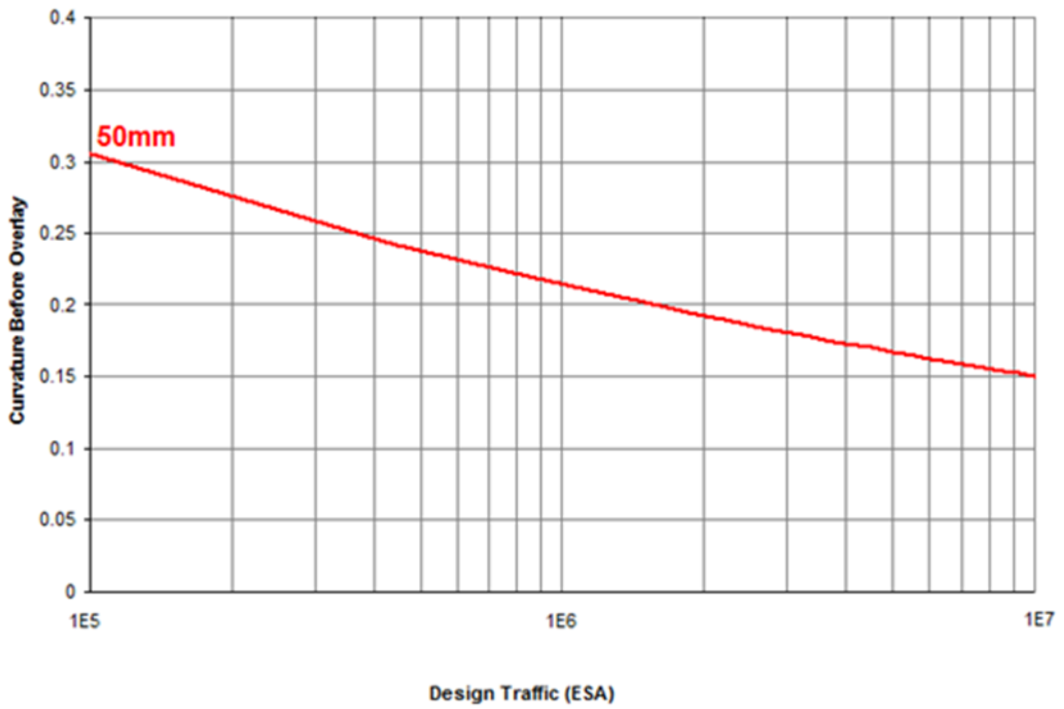


Figure C1.5(g) – Asphalt overlay curvature versus design traffic loading design chart for overlay thicknesses from 75–150mm for weighted mean annual average pavement temperature Zone 2 and a heavy vehicle operating speed of 80 km/h

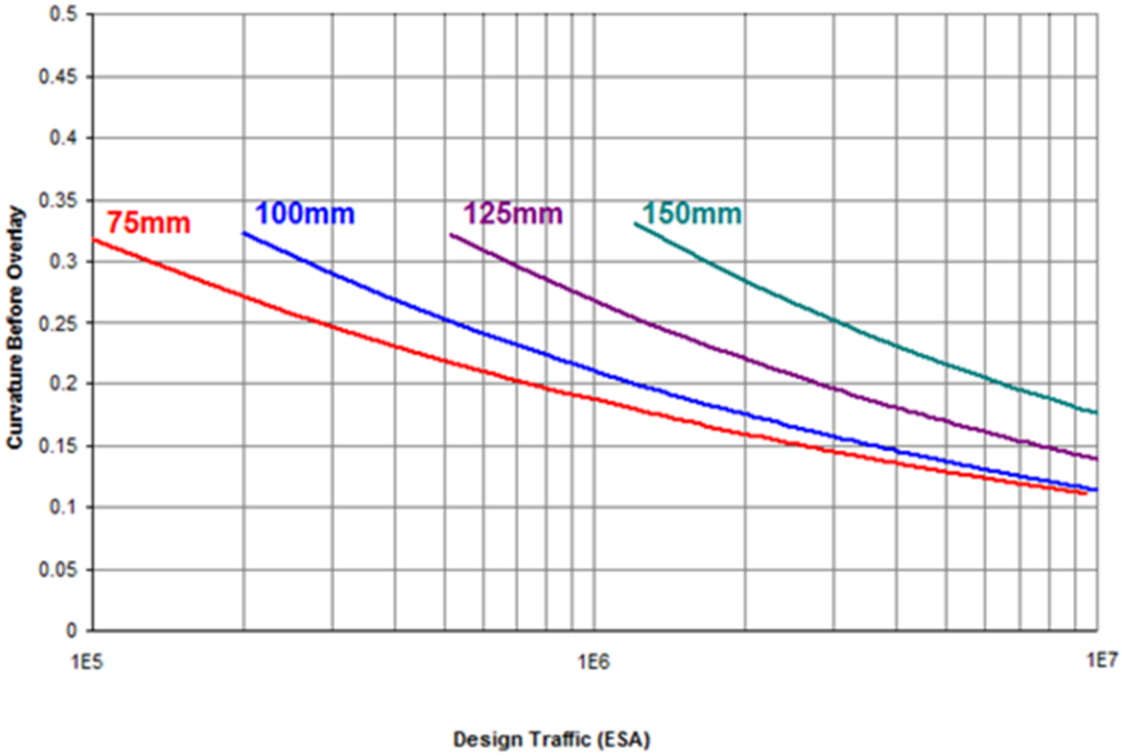


Figure C1.5(h) – Asphalt overlay curvature versus design traffic loading design chart for an overlay thickness of 50mm for weighted mean annual average pavement temperature Zone 2 and a heavy vehicle operating speed of 80 km/h

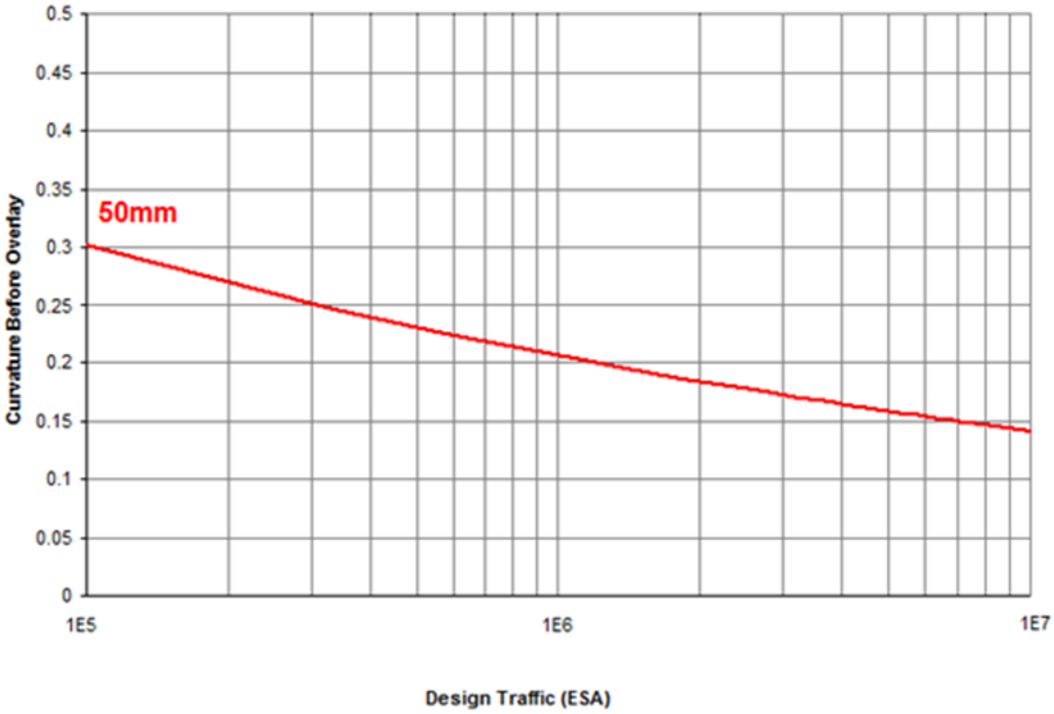


Figure C1.5(i) – Asphalt overlay curvature versus design traffic loading design chart for overlay thicknesses from 75–150mm for weighted mean annual average pavement temperature Zone 3 and a heavy vehicle operating speed of 50 km/h

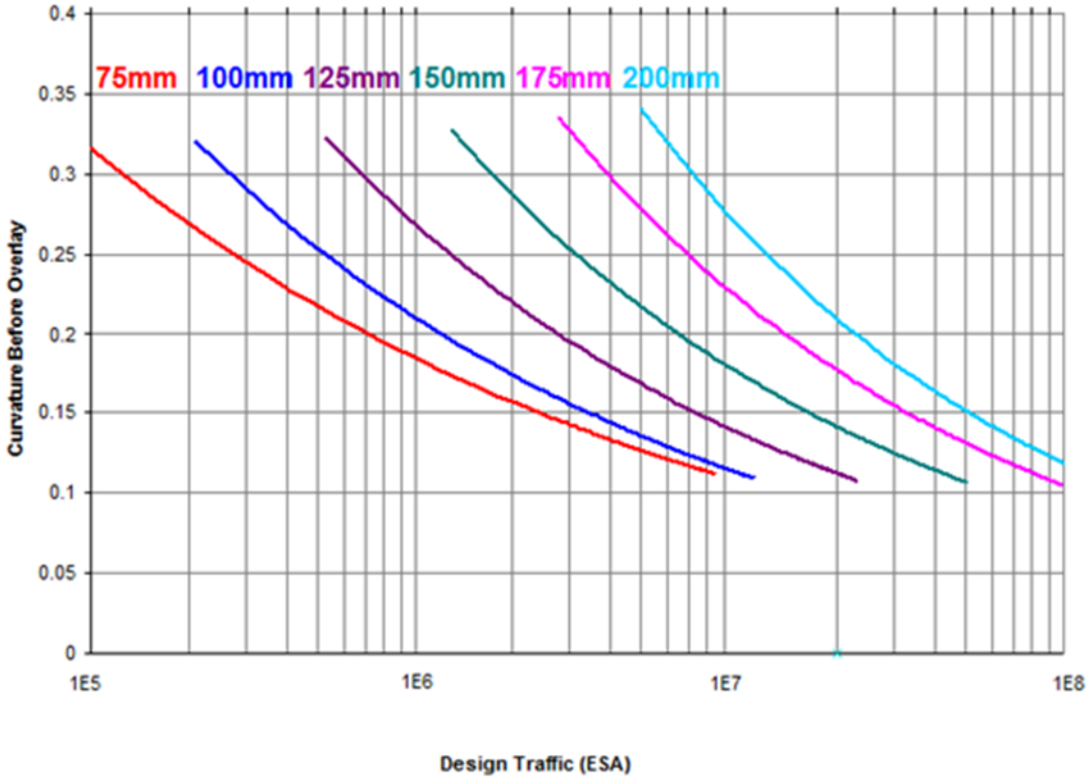


Figure C1.5(j) – Asphalt overlay curvature versus design traffic loading design chart for an overlay thickness of 50mm for weighted mean annual average pavement temperature Zone 3 and a heavy vehicle operating speed of 50 km/h

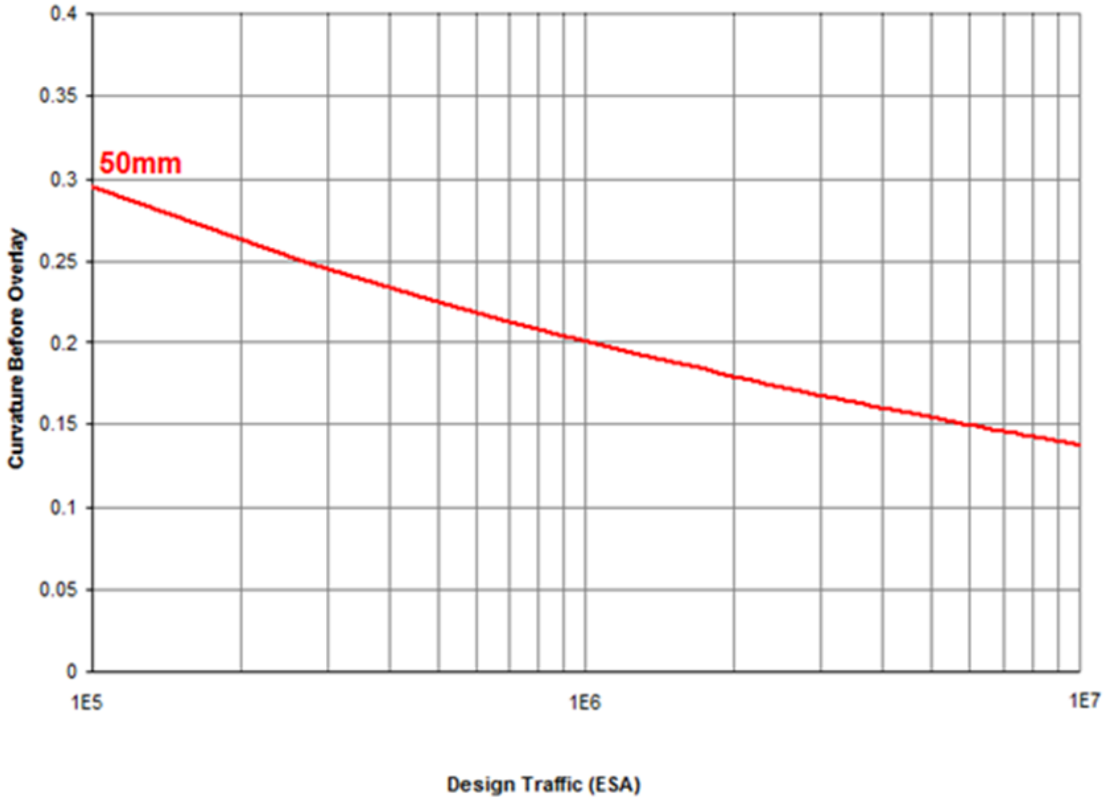


Figure C1.5(k) – Asphalt overlay curvature versus design traffic loading design chart for overlay thicknesses from 75–150mm for weighted mean annual average pavement temperature Zone 3 and a heavy vehicle operating speed of 80 km/h

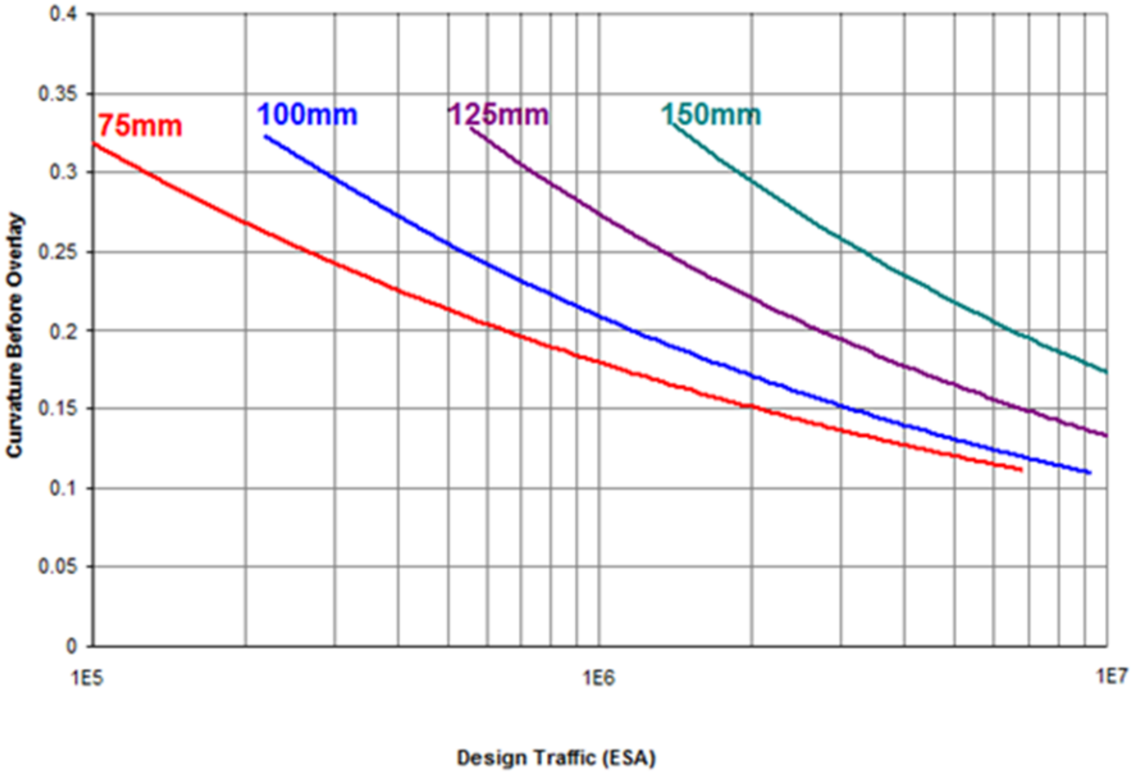


Figure C1.5(l) – Asphalt overlay curvature versus design traffic loading design chart for an overlay thickness of 50mm for weighted mean annual average pavement temperature Zone 3 and a heavy vehicle operating speed of 80 km/h

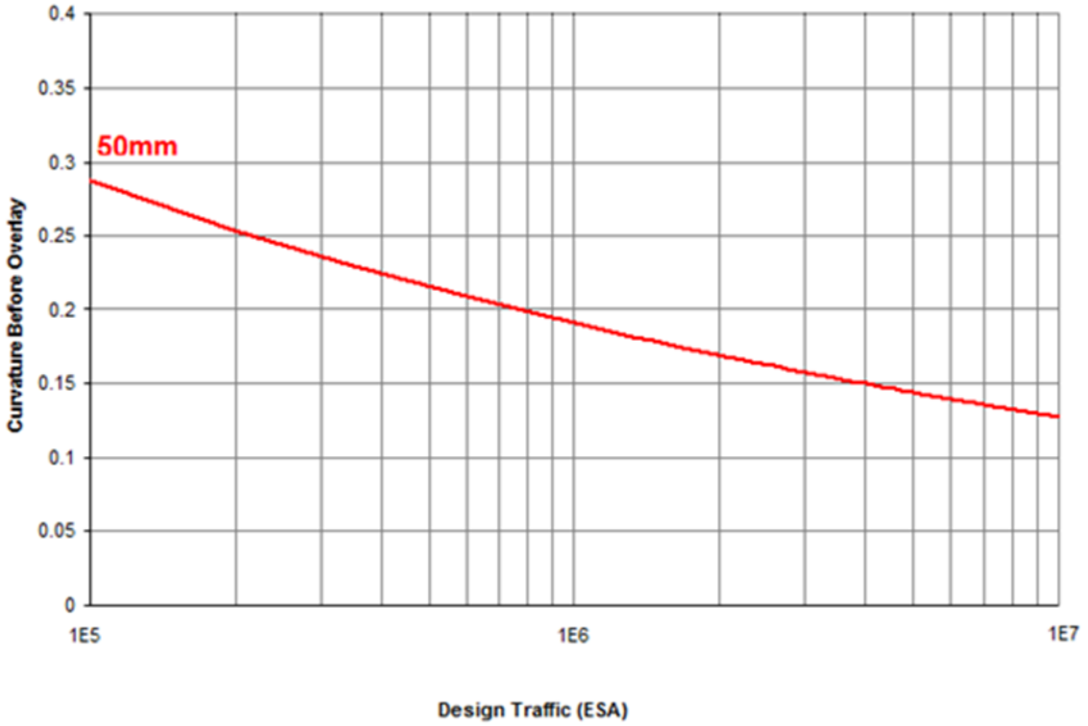


Figure C1.5(m) – Asphalt overlay curvature versus design traffic loading design chart for overlay thicknesses from 75–150mm for weighted mean annual average pavement temperature Zone 4 and a heavy vehicle operating speed of 50 km/h

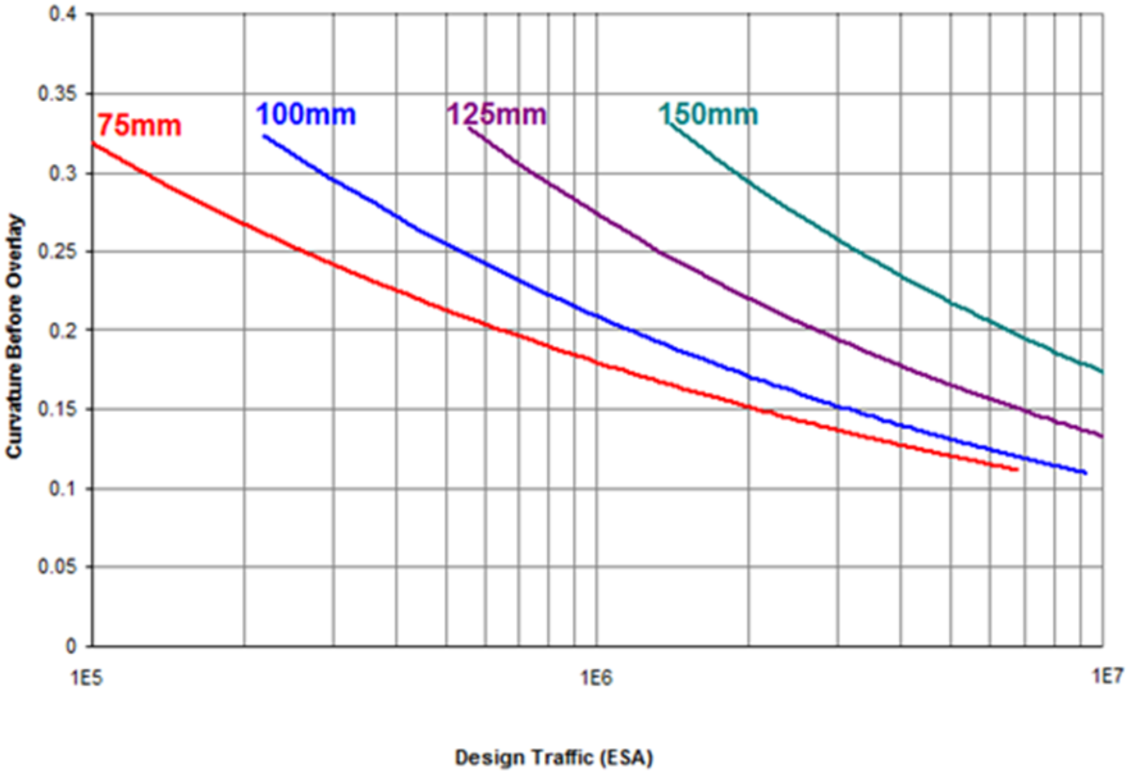


Figure C1.5(n) – Asphalt overlay curvature versus design traffic loading design chart for an overlay thickness of 50mm for weighted mean annual average pavement temperature Zone 4 and a heavy vehicle operating speed of 50 km/h

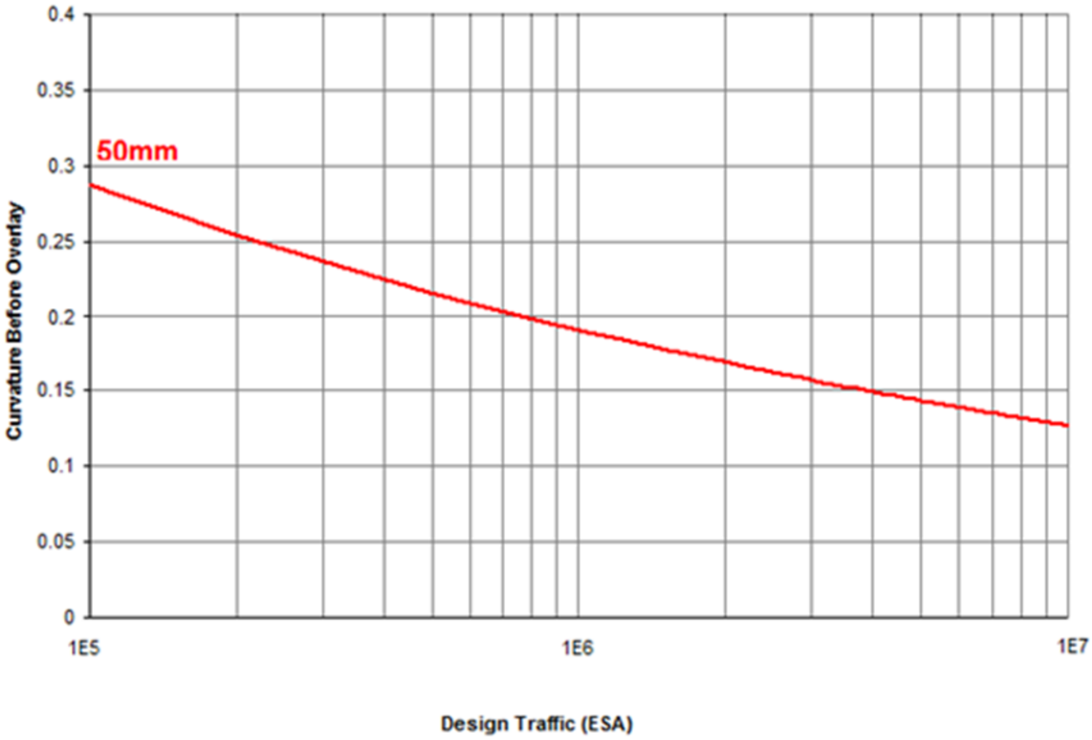


Figure C1.5(o) – Asphalt overlay curvature versus design traffic loading design chart for overlay thicknesses from 75–150mm for weighted mean annual average pavement temperature Zone 4 and a heavy vehicle operating speed of 80 km/h

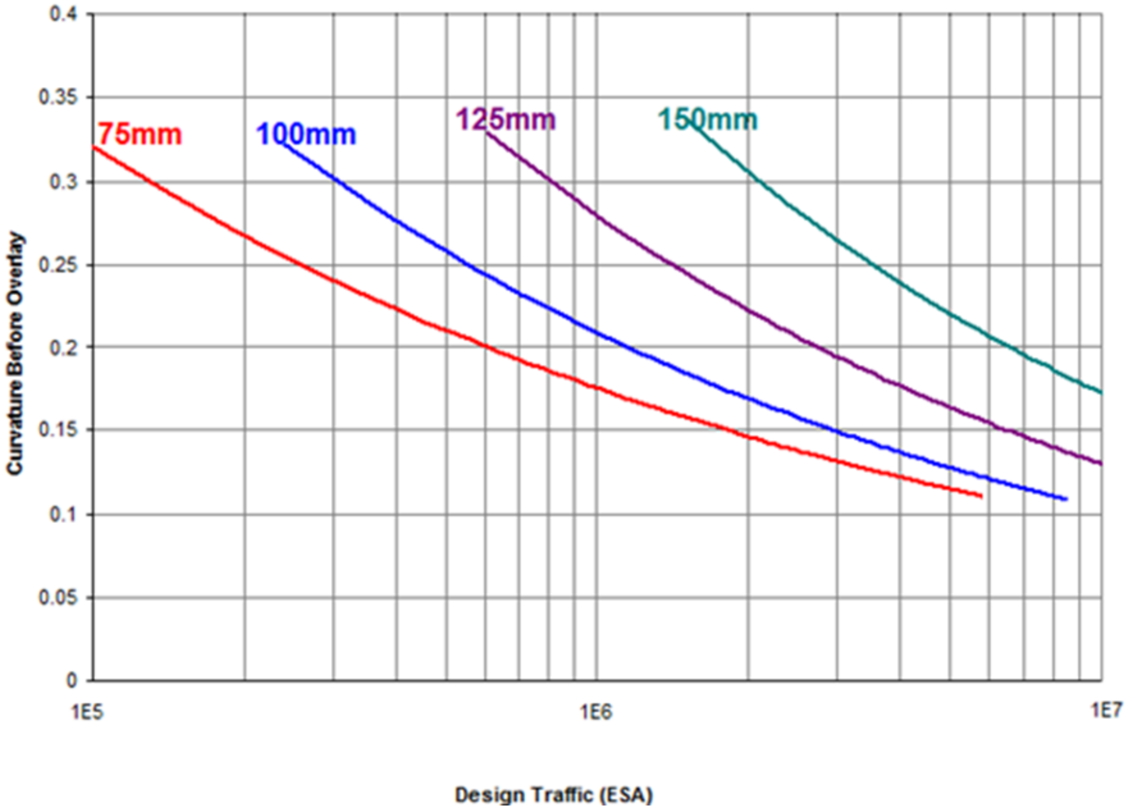
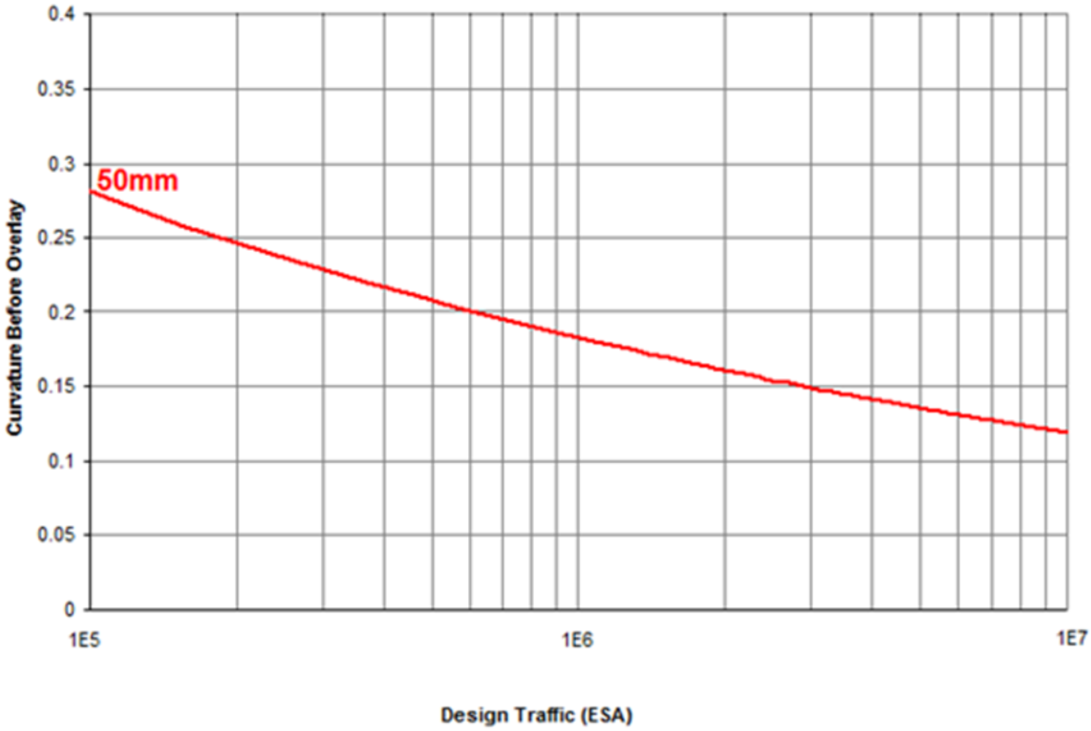


Figure C1.5(p) – Asphalt overlay curvature versus design traffic loading design chart for an overlay thickness of 50mm for weighted mean annual average pavement temperature Zone 4 and a heavy vehicle operating speed of 80 km/h



C2 Foamed bitumen structural design charts

Two separate set of charts have been developed, one for far north Queensland and another for south-east Queensland due to the effect of WMAPT on the asphalt modulus.

For north Queensland, WMAPT has been assumed as 36.9°C, and temperature correction has been made to the asphalt modulus accordingly.

For south-east Queensland, WMAPT has been assumed as 32.0°C; no temperature correction is required for the asphalt modulus.

It is assumed no temperature correction is required for foamed bitumen layers for all Queensland regions.

For these charts, a project reliability of 97.5% has been assumed, and a traffic multiplier of 1.1 has been assumed for both asphalt and foamed bitumen. For the subgrade, a traffic multiplier of 1.6 has been assumed.

C3 Foamed bitumen stabilisation (project reliability = 97.5%)

Where a project reliability other than 97.5% and/or traffic multiplier other than 1.1 is required, the allowable traffic loading of the chart shall be corrected as per Equation C3.

Equation C3 – Foamed bitumen allowable traffic loading correction factor

$$ATL_{Corrected} = \frac{RF}{0.67} \times \frac{1.1}{MF} \times ATL_{Chart}$$

where

MF = the alternative traffic multiplier for asphalt criteria (SARA/ESA)

ATL_{Chart} = the allowable traffic loading obtained from the chart (developed with project reliability = 97.5% and MF = 1.1)

$ATL_{Corrected}$ = the corrected allowable traffic loading for alternative project reliability and/or MF

RF = the reliability factor for asphalt criteria corresponding to an alternative project reliability – refer to Table C3 (derived from Table 6.16 of AGPT02)

Table C3 – Suggested reliability factors for asphalt fatigue

Project reliability	80%	85%	90%	95%	97.50%
Reliability factor	2.5	2	1.5	1	0.67

C4 Convert allowable traffic loading from chart to alternative project reliability and traffic multiplier values (example)

Step 1: Obtain the allowable traffic loading (ATL_{Chart}) from the chart for project reliability = 97.5%, traffic multiplier = 1.1.

Pavement profile: 50mm AC, 250mm foamed bitumen, and 300mm granular buffer

Foamed bitumen modulus =1400 MPa

Subgrade CBR = 5%

Region: Far North Queensland

Chart to be used: Figure C5(a)

Chart Reading: ATL_{Chart} = **2E6 ESAs**

Step 2: Calculate corrected allowable traffic loading ($ATL_{Corrected}$) for alternative project reliability = 95%, traffic multiplier = 1.4

For project reliability = 95%, corresponding RF value = 1 (from Table C3)

Use Equation C3:

$$ATL_{Corrected} = \frac{RF}{0.67} \times \frac{1.1}{MF} \times ATL_{Chart}$$

Substitute RF = 1, MF = 1.4, ATL_{Chart} = 1.1E6 ESAs:

$$2.34E6 = \frac{1}{0.67} \times \frac{1.1}{1.4} \times 2E6$$

Allowable traffic loading for project reliability = 95% and traffic loading = 1.4 = **2.34E6 ESAs**

C5 Spray seal surfacing

Figure C5(a) – Spray seal 250mm foamed bitumen treated material allowable traffic loading

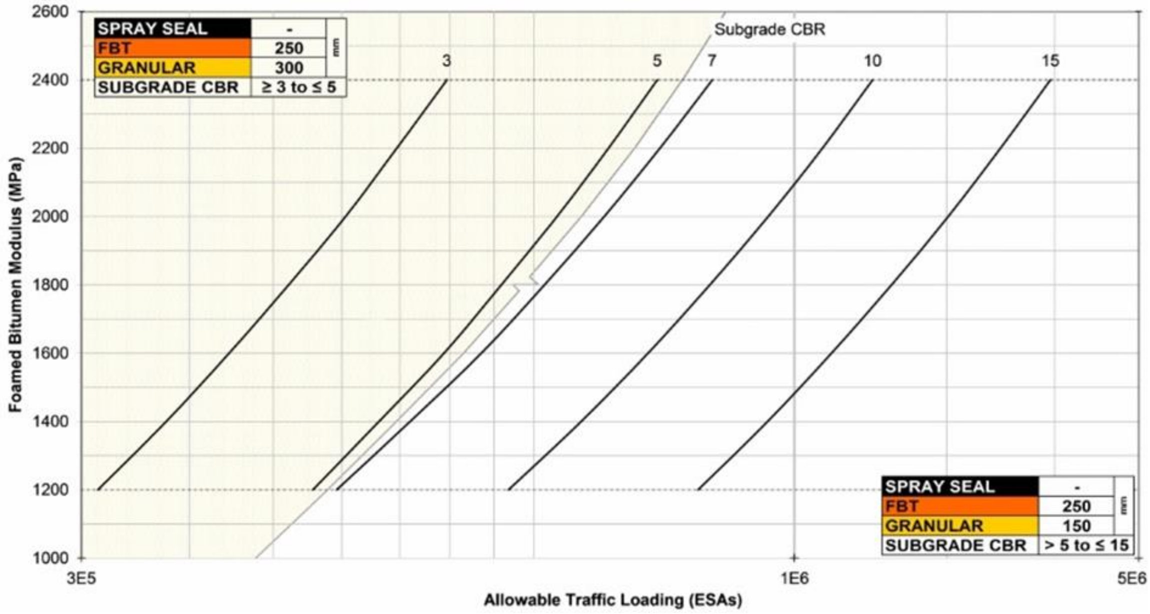


Figure C5(b) – Spray seal 300mm foamed bitumen treated material allowable traffic loading

Notes:

- Operating speed is 80 km/h.
- Construction tolerance not included.
- Maximum modulus adopted for granular layer is 150 MPa (refer AGPT02, Table 6.4)

Minimum thickness of unbound granular pavement material required below foamed bitumen treated material layer (mm)	Minimum CBR strength of unbound granular material required below foamed bitumen treated material layer (%)	Design subgrade CBR (%)
300		≥3–≤5
150	35	>5–≤15

C6 Far north Queensland 50mm asphaltic concrete overlay

Figure C6(a) – Far north Queensland 50mm asphaltic concrete 250mm foamed bitumen treated material allowable traffic loading

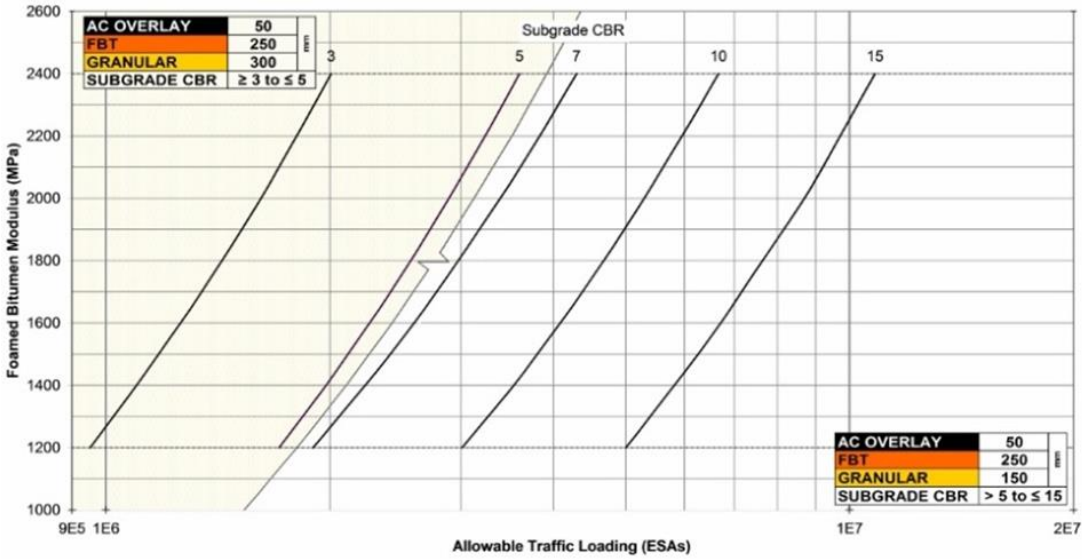
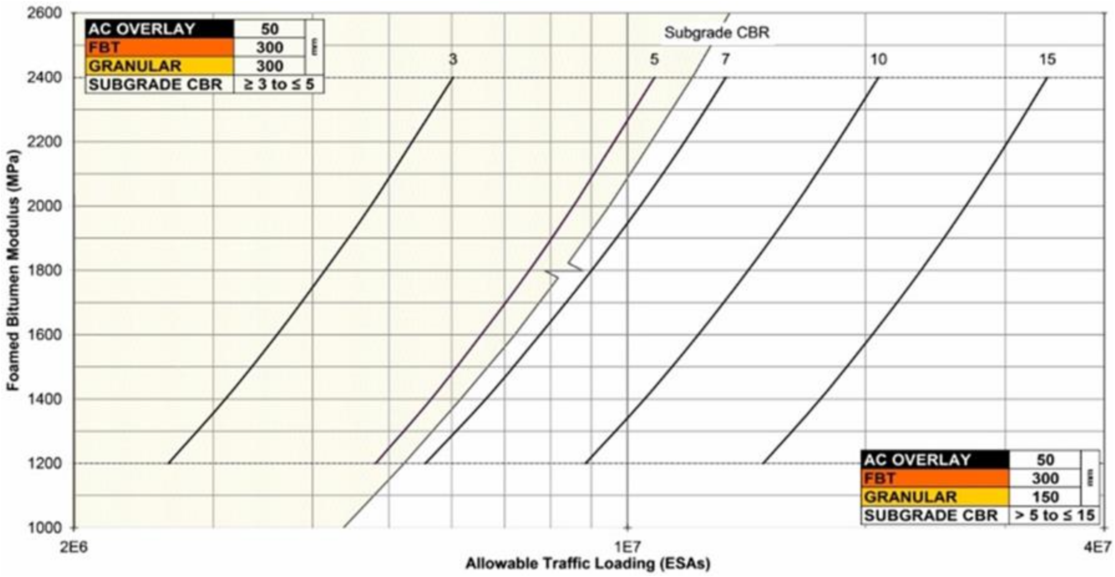


Figure C6(b) – Far north Queensland 50mm asphaltic concrete 300mm foamed bitumen treated material allowable traffic loading



Notes:

- Asphalt type considered in design: AC14 (A5S), AC20 (C600), WMAPT 36.9ºC.
- Operating speed is 80 km/h.
- Construction tolerance not included.
- Maximum modulus adopted for granular layer is 150 MPa (refer AGPT02, Table 6.4).

Minimum thickness of unbound granular pavement material required below foamed bitumen treated material layer (mm)	Minimum CBR strength of unbound granular material required below foamed bitumen treated layer (%)	Design subgrade CBR (%)
300		≥3–≤5
150	35	>5–≤15

C7 Far north Queensland 100mm asphaltic concrete overlay

Figure C7(a) – Far north Queensland 100mm asphaltic concrete 25 mm foamed bitumen treated material allowable traffic loading

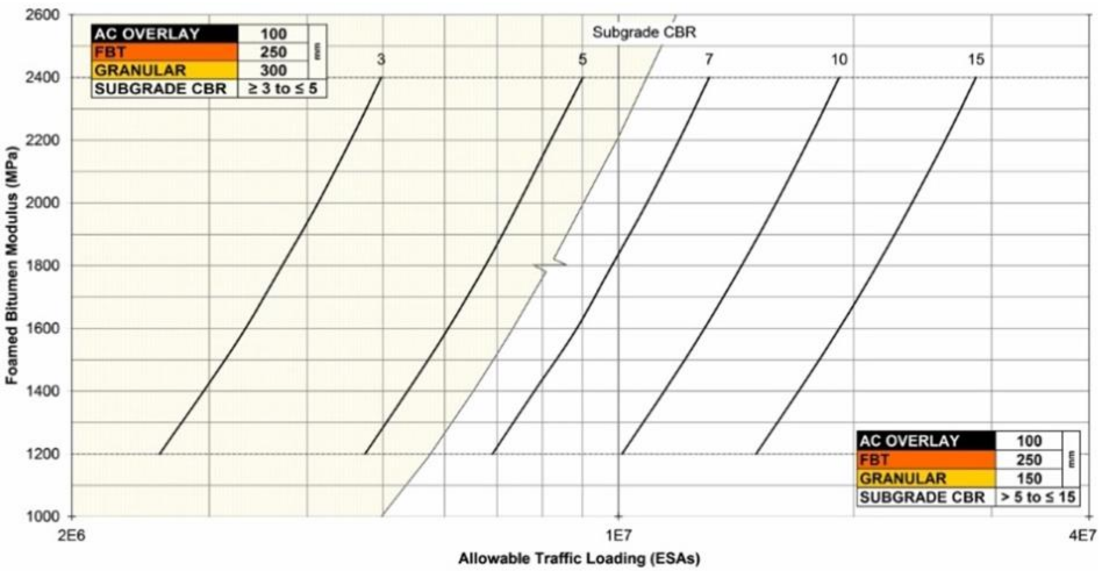
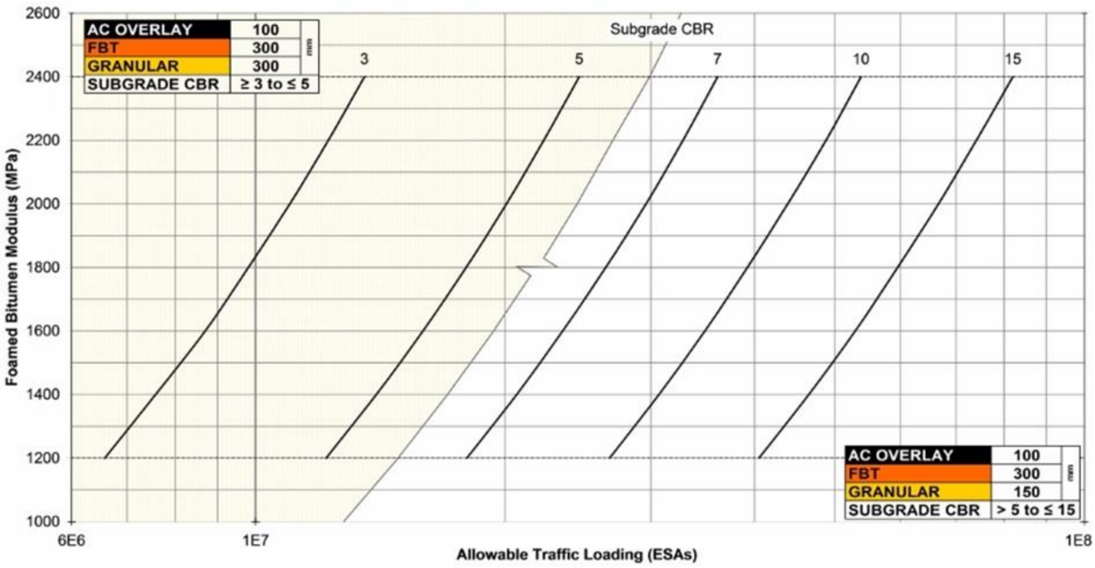


Figure C7(b) – Far north Queensland 100mm asphaltic concrete 300mm foamed bitumen treated material allowable traffic loading



Notes:

- Asphalt type considered in design: AC14 (A5S), AC20 (C600), WMAPT 36.9ºC.
- Operating speed is 80 km/h.
- Construction tolerance not included.
- Maximum modulus adopted for granular layer is 150 MPa (refer AGPT02, Table 6.4).

Minimum thickness of unbound granular pavement material required below foamed bitumen treated material layer (mm)	Minimum CBR strength of unbound granular material required below foamed bitumen treated material layer (%)	Design subgrade CBR (%)
300		≥3–≤5
150	35	>5–≤15

C8 Far north Queensland 150mm asphaltic concrete overlay

Figure C8(a) – Far north Queensland 150mm asphaltic concrete 250mm foamed bitumen treated material allowable traffic loading

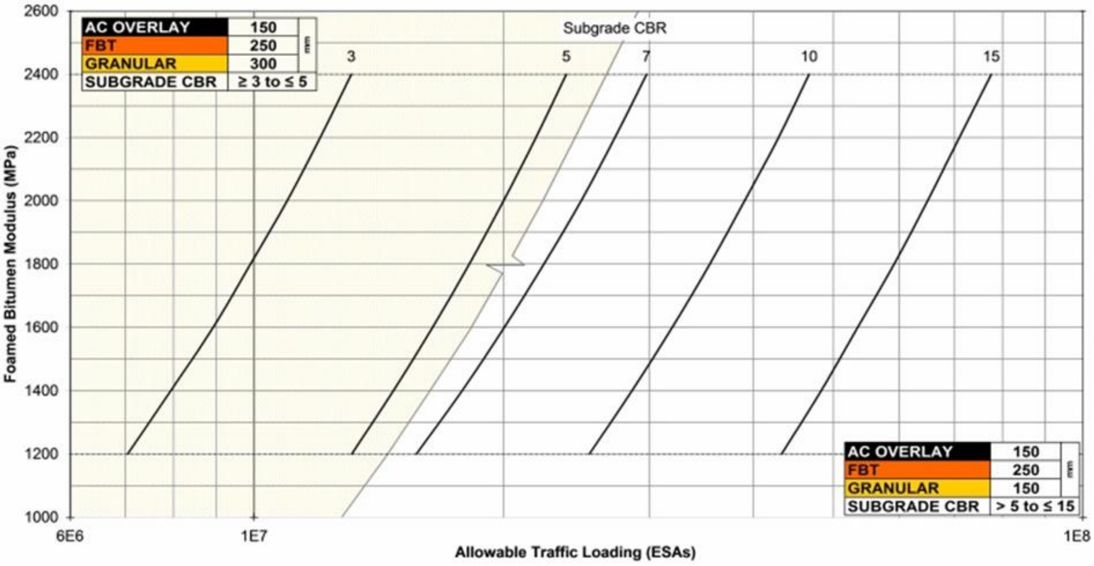


Figure C8(b) – Far north Queensland 150mm asphaltic concrete 300mm foamed bitumen treated material allowable traffic loading

Notes:

- Asphalt type considered in design: AC14 (A5S), AC20 (C600), WMAPT 36.9ºC.
- Operating speed is 80 km/h.
- Construction tolerance not included.
- Maximum modulus adopted for granular layer is 150 MPa (refer AGPT02, Table 6.4).

Minimum thickness of unbound granular pavement material required below foamed bitumen treated material layer (mm)	Minimum CBR strength of unbound granular material required below foamed bitumen treated material layer (%)	Design subgrade CBR (%)
300		≥3–≤5
150	35	>5–≤15

C9 South-east Queensland 50mm asphaltic concrete overlay

Figure C9(a) – South-east Queensland 50mm asphaltic concrete 250mm foamed bitumen treated material allowable traffic loading

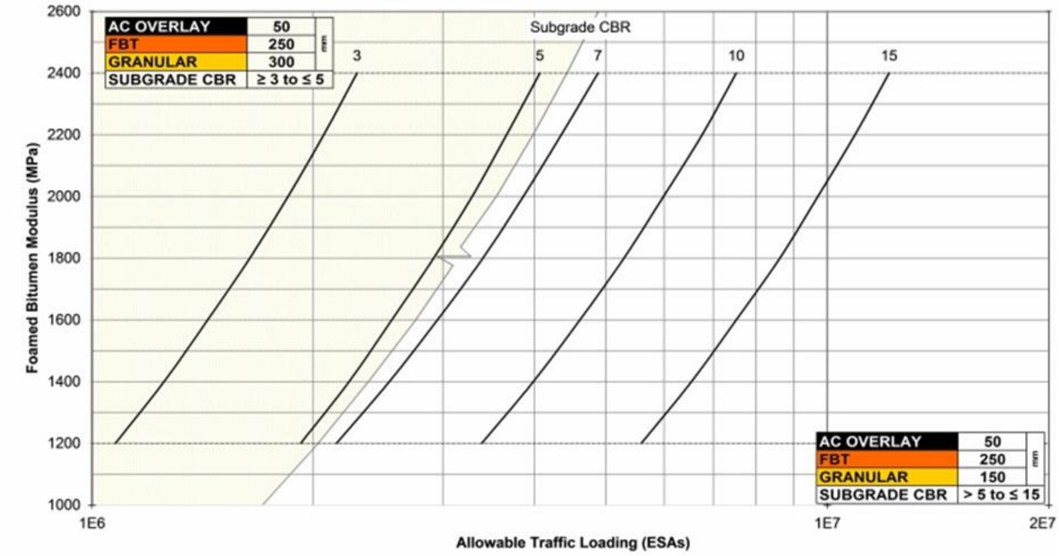
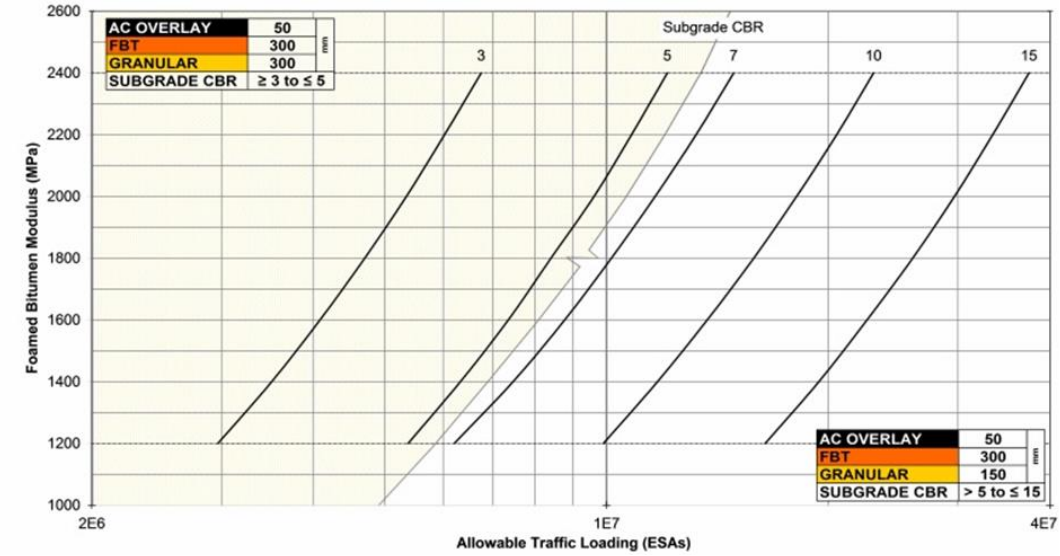


Figure C9(b) – South-east Queensland 50mm asphaltic concrete 300mm foamed bitumen treated material allowable traffic loading



Notes:

- Asphalt type considered in design: AC14 (A5S), AC20 (C600), WMAPT 32ºC.
- Operating speed is 80 km/h.
- Construction tolerance not included.
- Maximum modulus adopted for granular layer is 150 MPa (refer AGPT02, Table 6.4).

Minimum thickness of unbound granular pavement material required below foamed bitumen treated material layer (mm)	Minimum CBR strength of unbound granular material required below foamed bitumen treated material layer (%)	Design subgrade CBR (%)
300		≥3–≤5
150	35	>5–≤15

C10 South-east Queensland 100mm asphaltic concrete overlay

Figure C10(a) – South-east Queensland 100mm asphaltic concrete 250mm foamed bitumen treated material allowable traffic loading

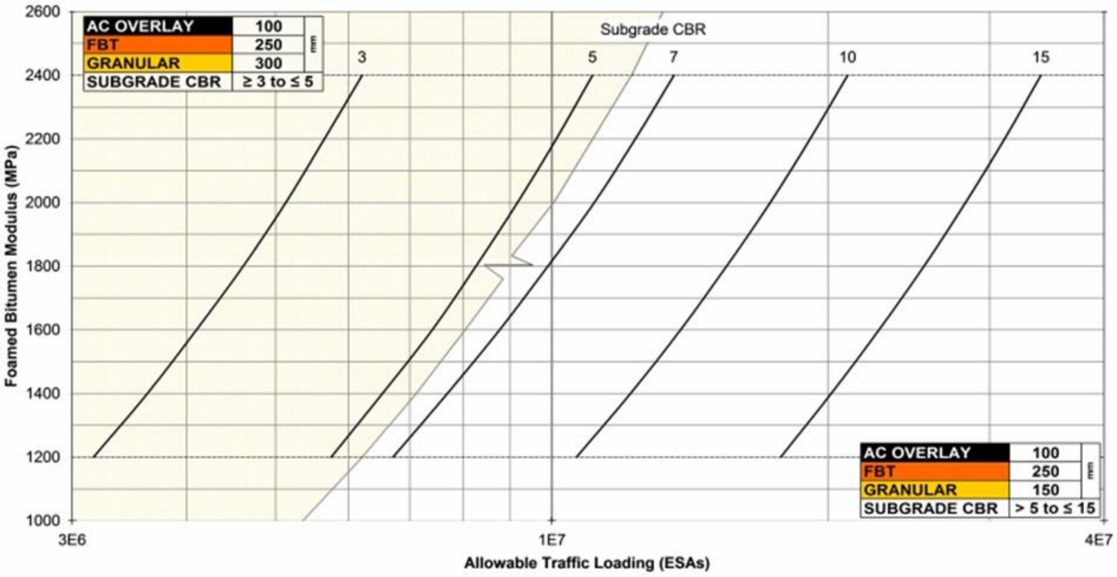
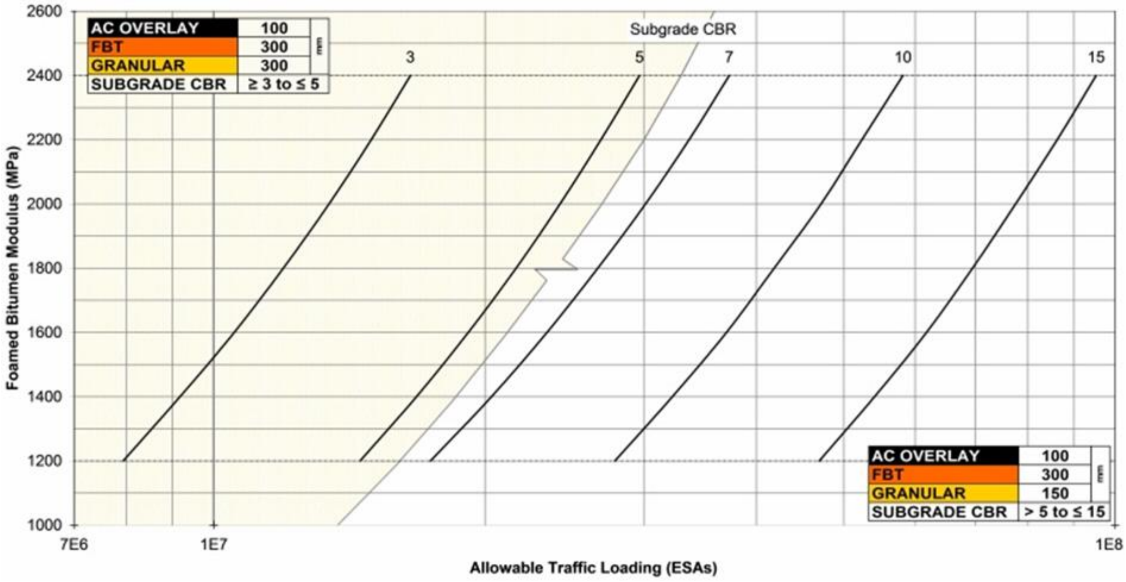


Figure C10(b) – South-east Queensland 100mm asphaltic concrete 300mm foamed bitumen treated material allowable traffic loading



Notes:

- Asphalt type considered in design: AC14 (A5S), AC20 (C600), WMAPT 32ºC.
- Operating speed is 80 km/h.
- Construction tolerance not included.
- Maximum modulus adopted for granular layer is 150 MPa (refer AGPT02, Table 6.4)

Minimum thickness of unbound granular pavement material required below foamed bitumen treated material layer (mm)	**Minimum CBR strength of unbound granular material required below foamed bitumen treated material layer (%)**	**Design subgrade CBR (%)**
300		≥3–≤5
150	35	>5–≤15

C11 South-east Queensland 150mm asphaltic concrete overlay

Figure C11(a) – South-east Queensland 150mm asphaltic concrete 250mm foamed bitumen treated material allowable traffic loading

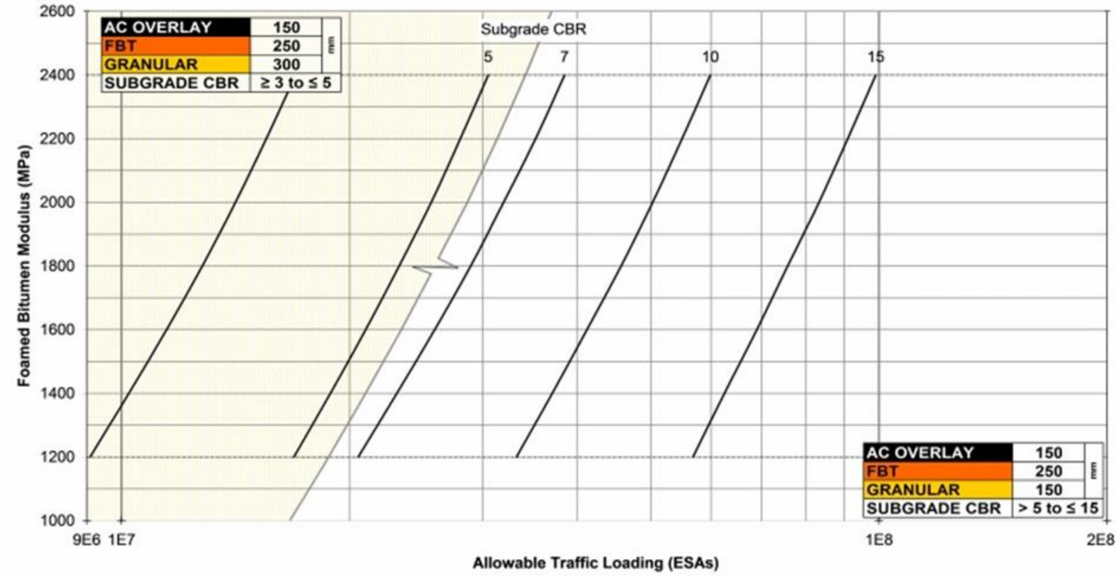
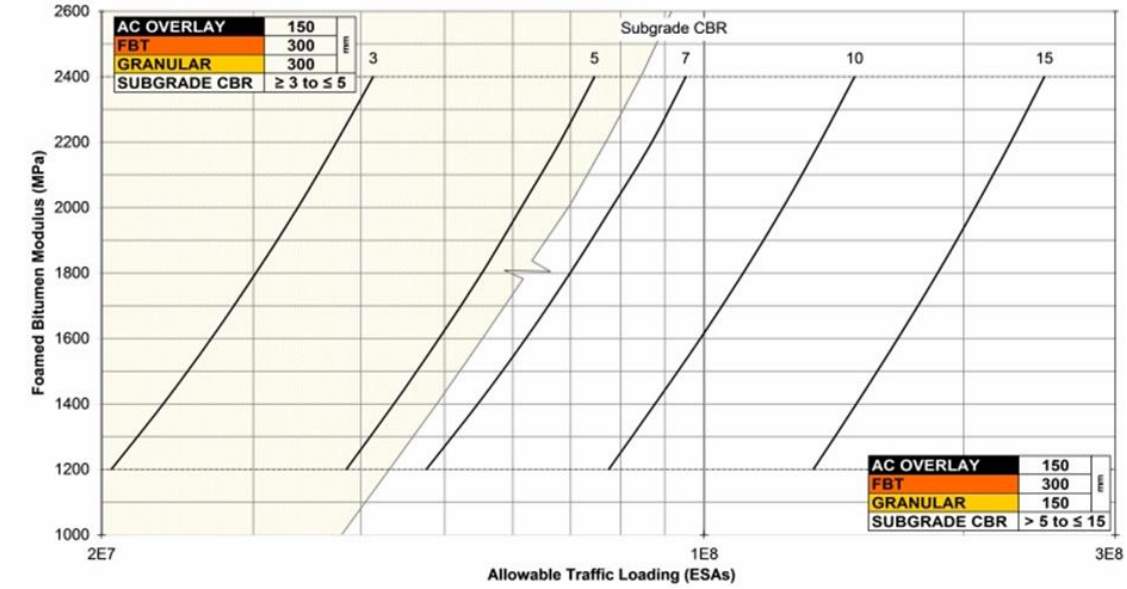


Figure C11(b) – South-east Queensland 150mm asphaltic concrete 300mm foamed bitumen treated material allowable traffic loading



Notes:

- Asphalt type considered in design: AC14 (A5S), AC20 (C600), WMAPT 32ºC.
- Operating speed is 80 km/h.
- Construction tolerance not included.
- Maximum modulus adopted for granular layer is 150 MPa (refer AGPT02, Table 6.4).

Minimum thickness of unbound granular pavement material required below foamed bitumen treated material layer (mm)	Minimum CBR strength of unbound granular material required below foamed bitumen treated material layer (%)	Design subgrade CBR (%)
300		≥3–≤5
150	35	>5–≤15

C12 Further reading

Relevant publications include AGPT05; AGPT02; MRS05; MRTS05; MRS30; MRTS30; PDS; RPDM speed properties; and Vuong, 1988.