Roads

Master Specification

RD-PV-D1 Pavement Design (Austroads Supplement)

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RD-PV-D1 Pavement Design (Austroads Supplement)

1 Introduction

Scope of the Guide and this Part

1.1 Chapter 12 of the Guide describes procedures for the design of lightly trafficked pavements with a design traffic loading of less than 1 x 105 ESA.

1.2 Whilst this Supplement may not in all areas specifically address such pavements, the majority of the documented recommendations and requirements are also applicable to lightly trafficked pavements. Engineering judgement and appropriate risk management practices should be used to determine where the design requirements for lightly trafficked pavements differ from those for moderately and heavily trafficked pavements.

1.3 Construction of pavement designs shall comply with the DPTI Master Specification for Roadworks. Conformance with this Specification is a fundamental prerequisite to achieving the design parameters provided in this Supplement, and the pavement life.

1.4 Where there is a difference between content in the Guide and this Supplement, the requirements of this Supplement shall govern.

1.5 The Austroads Guide Part 2 and this Supplement are intended for the design of new pavements founded on newly placed engineering fill or natural subgrade soils. Some projects may have areas of pavement that overlap existing pavements, for example, lane realignments, widenings and tie ins with existing roads. If there is an intention to utilise these existing pavements in the pavement designs for the project, then the designer should refer to the Austroads Guide to Pavement Technology Part 5 – Pavement Evaluation and Treatment Design (2011) and the DPTI Supplement to Part 5 for further guidance on how to assess and, if suitable, incorporate existing pavement layers into the design.

1.6 It is critical that the project scope clearly defines the expected pavement design period and other design criteria for all pavement works within the project extent.

2 Pavement Design Systems

General

2.1 For moderately and heavily trafficked roads, the following pavement configurations are constructed by DPTI:

Rural roads

2.2 Unbound granular pavement with sprayed seal surfacing - extensively used due to their low cost and available material sources. The quality required for the base layer is dependent on the traffic loadings. Where horizontal shear stresses are high due to turning traffic, an asphalt surfacing layer may be placed.

2.3 Full depth asphalt pavement may be used for widenings and strategically important road projects.

Urban roads

2.4 Thin asphalt surfaced unbound granular pavement – one or two layers of asphalt, over Class PM1 basecourse and Class PM2 subbase materials. The minimum thickness of asphalt required varies from 40 to 80 mm, depending on the design traffic loading.

2.5 Full Depth Asphalt Pavements – thick asphalt on a Class PM2 subbase layer. For moderately to heavily trafficked roads the asphalt thickness that ensures adequate fatigue life usually exceeds 200 mm and will consist of between 3 to 6 asphalt layers, on a 150 mm granular subbase.

2.6 Asphalt surfaced cemented subbase pavement - often referred to as an asphalt cemented composite pavement, comprising thick asphalt on two cemented subbase layers placed on the same day. For heavily trafficked roads, a minimum asphalt thickness of 175 mm is used to inhibit reflection cracking. For moderately trafficked roads lower asphalt thicknesses (refer Figure RD-PV-D1 3-1 may be used to reduce initial construction costs, but increased performance risks and higher maintenance costs need to be considered in the whole-of-life costing. The design life excludes the post cracking phase (refer 8.8).

2.7 Deep Strength Asphalt Pavements - minimum 175 mm thickness of asphalt on a single 150 - 200 mm cemented (fully bound) subbase. When the design traffic exceeds 10^7 ESA, the asphalt thickness is determined without allowing for a cemented post-cracking phase.

Heavy Duty Pavement Types

2.8 Some heavily trafficked roads (e.g. motorways and freeways) have high level performance requirements and need to be designed to minimise traffic delays due to road maintenance during their service lives. Such pavements commonly have a design traffic loading exceeding 10^7 ESA and are referred to as “heavy-duty” pavements in this Supplement.

2.9 The following pavement types are considered to be heavy-duty pavements:

Flexible Pavements:

a) Full depth asphalt

b) Deep strength asphalt pavement, no post-cracking

Rigid Pavements:

c) Jointed plain (unreinforced) concrete pavements (PCP)

d) Jointed reinforced concrete pavements (JRCP)

e) Continuously reinforced concrete pavements (CRCP)

2.10 National Highways in rural locations with traffic loadings exceeding 10^7 ESA may utilise a sprayed seal unbound granular pavement. This pavement type is not considered to be a low maintenance, heavy-duty pavement type. However, it may be utilised in this situation, due to project specific conditions and considerations. These include project location, existing pavement types, available materials, available construction plant, climate, acceptable performance standards and maintenance / whole of life cost considerations.

2.11 The preceding flexible pavement types are familiar to South Australian industry and asset owners, with significant design, material supply, construction and operational expertise available. Historically, DPTI has limited experience with rigid (concrete) pavements, although construction of its first major PCP motorway pavement will commence in 2018.

2.12 There are other flexible pavement types included in the Austroads Guides which are used by some States. Caution needs to be applied if considering their use in South Australia, given there is probably limited local engineering, materials and operational expertise.

Heavy Duty Pavement Type Selection

2.13 The factors affecting the selection of heavy-duty pavement type include:

a) Differential settlement predictions;

b) Scale of works;

c) Construction of works under traffic (greenfield or brownfield site);

d) Required surfacings type, including consideration of noise, drainage, texture, durability, etc.;

e) Maintenance requirements;

f) Acceptable pavement performance outcomes, including roughness, rutting, texture, skid resistance and cracking, which may be contractually specified; and

g) Economics, including whole-of-life costing.

2.14 These factors need to be assessed on a project specific basis. On major projects DPTI may undertake this assessment internally and specify the acceptable pavement types for each project element.

Heavy Duty Pavement Support

2.15 To increase the founding strength and uniformity of pavement support, heavy-duty pavements are usually founded on an engineered subgrade, typically comprising:

a) Engineered Fill (select fill) – constructed in accordance with RD-EW-C1 “Earthworks”, using compacted layers of Type A or Type B Class materials, or

b) In situ stabilised subgrades - typically lime stabilisation of clay subgrades, built in accordance with RD-PV-S2 “Plant Mixed Stabilised Pavement”.

2.16 The typical minimum thickness requirements for heavy duty pavement supporting layers are shown in Table RD-PV-D1 2-1. However, these should be reviewed based upon site specific geotechnical investigation and design considerations, as required to achieve the performance criteria outlined in DPTI Design Standard: EW100: Earthworks for Roads and RD-EW-C1 “Earthworks”.

2.17 For example, the Volumetric Change requirements of EW100 can lead to substantially greater depths of subgrade dig out and replacement (up to several metres) when expansive clay subgrades are present. Similarly, weak, wet, organic materials and other deficient subgrades will require additional geotechnical investigation, design and treatment.

2.18 The greater standard of this Supplement, and RD-EW- C1 “Earthworks” shall apply, unless otherwise stated in project specific documentation.

Table RD-PV-D1 2-1 Typical Minimum Support Requirements for Heavy-Duty Pavements

Subgrade Design CBR (%)	Support Treatment Options and Material Quality Treatments* (CBR)	Minimum Thickness (mm)#
> 10	PM2 or Characteristic Strength† ≥ 30	150
3 – 10	In situ Lime Stabilisation‡	250
	150 mm PM2 or Characteristic Strength† ≥ 30 over 150 mm Type A or B and Characteristic Strength† ≥ 15	300
< 3	150 mm PM2 or Characteristic Strength† ≥ 30 over 250 mm In situ Lime Stabilisation‡	400
	150 mm PM2 or Characteristic Strength† ≥ 30 over 350 mm Type A or B and Characteristic Strength† ≥ 15	500

* Shall comply with RD-EW-C1 “Earthworks” and RD-PV-S1 “Supply of Pavement Materials” of the DPTI Master Specification for Roadworks

† Characteristic Strength defined in Section 5: Selected Subgrade Materials (i.e. equal to 10th percentile of 4 day soaked CBR)

‡ Laboratory investigation of binder content to ensure long term Characteristic Strength ≥ 30

Geofabric or geogrids, subsoil drainage and other treatments may also be needed for weak or wet subgrades. Expansive soils typically require a minimum pavement support thickness of 600mm and/or other appropriate moisture control measures.

2.19 The heavy-duty flexible pavement types most likely to be constructed by DPTI are indicated in Table RD-PV-D1 2-2.

2.20 Major projects may include reference pavement designs in the tender documents to further inform pavement type selection.

Table RD-PV-D1 2-2 Typical Heavy-Duty Flexible Pavements

Pavement		Full depth asphalt	Asphalt on single layer cemented material
Depth below road surface (mm)	100	200 mm (min) asphalt(excludes Open Graded AC) Prime**	175 mm (min) asphalt (excludes Open Graded AC) Use SAMI* when asphalt < 200

	200 300	150 mm PM2 granular working platform or in situ stabilisation as per Table RD-PV-D1 2-1	150 – 200 mm plant mixed cemented subbase materials (single layer)
	400 500	Type A or B fill or in situ stabilisation as per Table RD-PV-D1 2-1	150 mm PM2 granular working platform or in situ stabilisation as per Table RD-PV-D1 2-1
	600 700	Subgrade	Type A or B fill or in situ stabilisation as per Table RD-PV-D1 2-1
			Subgrade
Notes		Asphalt thickness variable, usually 250 - 350 mm **Cutback bitumen (e.g. AMC0, AMC00) or emulsion prime.	Asphalt thickness variable, usually > 200 mm on 150 – 200 mm cemented subbase * Strain Alleviating Membrane Interlayer

2.21 The selection of pavement type requires some knowledge and experience of the local configuration details and materials that have previously proven successful. The design, construction and performance of pavement configuration types that are first time applications to a locality are likely to involve additional risk factors that require careful consideration.

2.22 The Supplement and the Guide do not contain provisions for settlement below the pavement layers. Where required, additional investigations and assessments shall be carried out to determine if settlement may occur and, if so how this affects the choice of pavement type.

2.23 The design procedures in the Guide have been developed over many years using mechanistic modelling and in-service field performance data, as well as from substantial, ongoing research programs run by Austroads. These pavements were generally designed and constructed to outlast 20 years or more of trafficking, with the loading spread more or less evenly over the design period. However, in some situations (e.g. temporary pavements) the pattern of loading differs markedly from that on which the procedures in the Guide were based. Section 7: Initial Daily Heavy Vehicles in the Design Lane provides direction on how the design traffic is calculated for such situations.

Overview of Pavement Design Systems

Input Variables (Design Traffic)

2.24 This Supplement refers to pavements within three traffic loading categories, comprising lightly, moderately and heavily trafficked roads. The following guidance is given on the load intensities of these categories:

a) Lightly trafficked roads have design traffic loadings less than 1 x 10^5 ESA;

b) Moderately trafficked roads have design traffic loadings greater than or equal to 1 x 10^5 ESA and less than 5 x 10^6 ESA; and

c) Heavily trafficked roads have design traffic loadings greater than or equal to 5 x 10^6 ESA.

2.25 Heavily trafficked roads with a design traffic loading above 1 x 10^7 ESA require a heavy-duty pavement type, as per the considerations outlined in Section 2.1 to 2.23.

2.26 DPTI endorses use of the upper limit on design traffic for asphalt fatigue (see Section 7: Design Traffic for Mechanistic – Empirical Design Procedure).

Input Variables (Project reliability)

2.27 Minimum project reliability levels to be used on DPTI roads are shown in Table RD-PV-D1 2-3.

Table RD-PV-D1 2-3 Minimum Project Reliability Levels

Road Class(1)	Project Reliability (%)
Motorways	95
Urban and Rural Arterial, Urban and Rural Connector	95
Access	90

(1) These Classes use the 2018 DPTI Road Classifications and correspond to the following historical descriptions:

a) Freeways & Expressways align with Motorways,

b) Highways and Main Roads align with Urban and Rural Arterial and Connector,

c) Other Roads aligns with Access.

3 Construction and Maintenance Considerations

General

3.1 The design procedures in this Supplement assume that appropriate DPTI standards of construction and maintenance are used. These standards are generally defined by the DPTI Master Specification for Roadworks (https://www.dpti.sa.gov.au/) with additional information also available from various Austroads publications, DPTI internal reports, and the proceedings of technical conferences.

3.2 Unless appropriate construction standards are met, modulus, thickness or other critical properties assumed in the design model may not be achieved and adverse pavement performance could be expected.

Extent and Type of Drainage

3.3 Due to the relatively low rainfall and high evaporation rates in South Australia, sub-surface drains are generally not provided for pavements except in areas where definite water seepage is identified. These situations include hillside cuttings and areas where the median or verge irrigation may lead to water entering the pavement. Motorway class projects require use of subsurface drains along the full length of the main alignment, to provide greater protection to the high value pavement asset.

3.4 Ground water site investigations should preferably be undertaken during the wetter months as these flows may be seasonal. Where the site investigation occurs outside the wettest period and seepage observations are inconclusive, sub-surface drains may need to be installed in some high risk areas as a precautionary measure. A critical review of the pavement and drainage design should occur where unexpected groundwater is identified during construction.

Use of Stabilisation

3.5 Where cemented material layers are placed close to the surface of the pavement, reflection of shrinkage cracking must be expected. In such situations, crack sealing and maintenance patching may be required. Crack sealing in dense graded asphalt surfacings is often more effective than crack sealing of sprayed seals.

3.6 The thickness of asphalt or granular material required above a cemented material layer to inhibit reflection cracking will depend on a number of factors, which include traffic loading, environment, quantity and type of binder used, curing practices, parent material properties, construction conditions and degree of subgrade support.

3.7 The minimum required cover of dense graded asphalt to inhibit reflection cracking is illustrated in Figure RD-PV-D1 3-1. The reduction in cover with lower traffic loading is in part related to the higher tolerable amounts of surface cracking for lower traffic volume roads. If a thickness less than 175 mm is used, higher crack sealing and patching maintenance costs need to be considered in the whole-of-life costing. DPTI field performance studies conclude that the use of a SAMI reduces the severity of reflective cracking and should be placed where the asphalt cover is less than 200mm.

3.8 Granular materials may be used as cover either solely or in conjunction with asphalt subject to the following criterion:

3.9 Equivalent thickness of dense graded asphalt = (0.75 $\times$ thickness of granular material cover) + (thickness of asphalt cover)

3.10 Figure RD-PV-D1 3-2 illustrates the combinations of asphalt and granular thicknesses that inhibit reflection cracking and are considered equivalent to 175 mm dense graded asphalt.

Figure RD-PV-D1 3-1 Minimum Cover to Inhibit Reflection Cracking

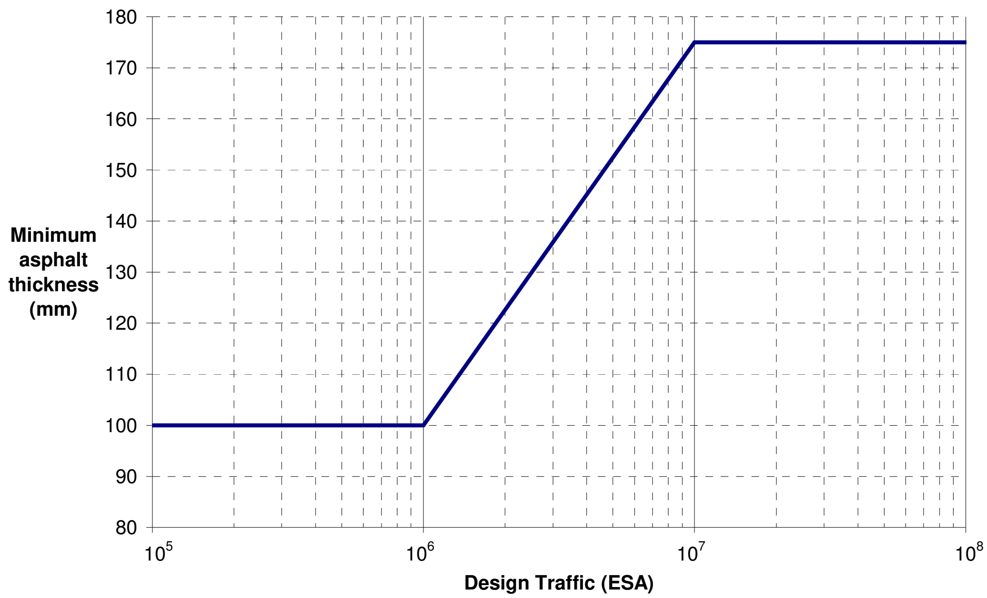
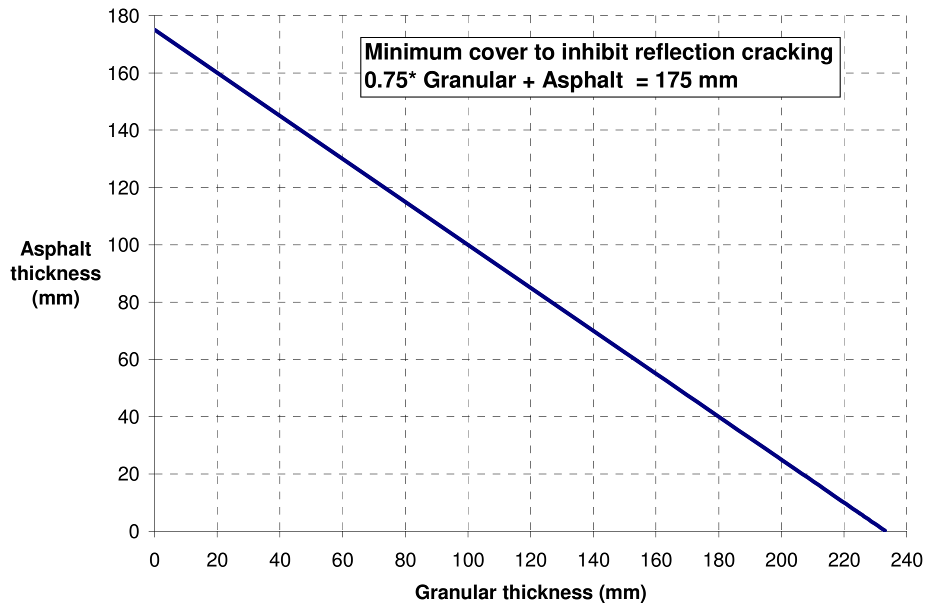


Figure RD-PV-D1 3-2 Asphalt and Granular Thicknesses to Inhibit Reflection Cracking.

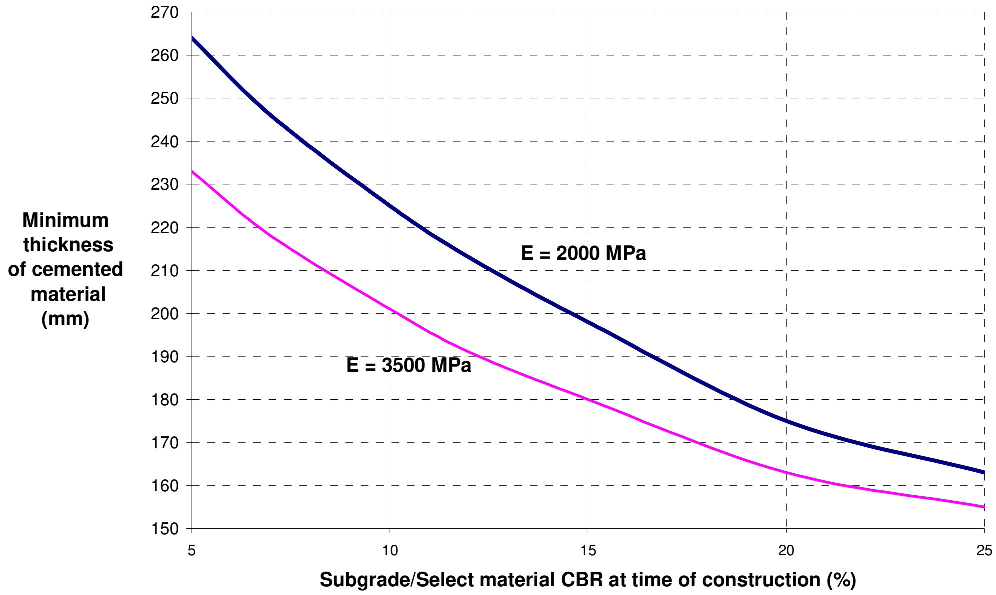


Single Layer Plant Mixed Cemented materials

3.11 The achievement of the specified compaction of cemented materials is essential for the development of the modulus and fatigue characteristics assumed in the design. The lower part of the cemented course is particularly critical as this is the zone in which maximum tensile strains occur. This requirement for adequate compaction limits the maximum layer thickness in these materials. For the construction of new pavements, the maximum single layer cemented thickness is 200 mm. The ability to obtain the required field density within the operating constraints of the construction equipment in urban areas (e.g. vibration) may reduce this maximum thickness for some projects.

3.12 Cemented materials may incur fatigue damage due to construction traffic unless they are sufficiently thick and adequately supported by the underlying materials. The minimum thicknesses of cemented material necessary to avoid fatigue damage during construction are given in Figure RD-PV-D1 3-3 and depend on the strength of the underlying material at the time of construction. For thicknesses less than those given in Figure RD-PV-D1 3-3, the fatigue damage during construction needs to be considered in assessing the fatigue life of the cemented material.

Figure RD-PV-D1 3-3 Minimum Thicknesses of Cemented Materials to Avoid

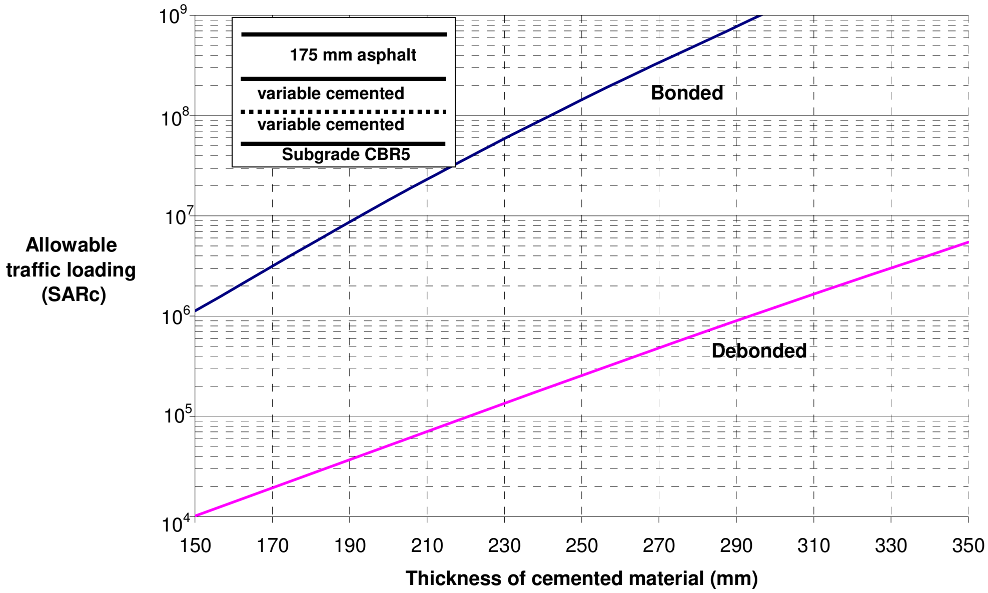


Multi-layer plant mixed cemented materials*

3.13 When the cemented material thickness of new pavements exceeds the maximum single layer thickness, multi-layer construction is necessary. For these pavements, the minimum and maximum thickness of each layer is 125 mm and 175 mm respectively. Hence the total thickness of cemented material for two-layer construction can vary between 250 and 350 mm. No more than two layers are considered fully bonded together in the mechanistic design calculation (refer Section 8. (CL8.1 to 8.4)).

3.14 In the construction of multi-layered cemented materials, layers designed to be fully bonded need to perform structurally as a single layer, otherwise they should be modelled as two unbonded layers. The effect on fatigue life of inadequate bonding between cemented material layers can be evaluated using mechanistic modelling. Figure RD-PV-D1 3-4 illustrates an example where the cemented material fatigue life decreases by a factor in excess of 100 when the interface between two layers is smooth rather than rough.

Figure RD-PV-D1 3-4 Effect of Lack of Bonding Between Cemented Material Layers



3.15 Shrinkage cracks which propagate to the pavement surface provide pathways for the infiltration of moisture, which can lead to debonding at layer interfaces within the pavement and/or weakening of the underlying granular layers and subgrade. Hence same day placement of multiple layers is essential, not only to achieve effective chemical bonding between the layers, but also to ensure shrinkage cracking is continuous through the multiple layers. If this does not occur and shrinkage cracking terminates at the interface between layers, surface moisture entering the pavement and accumulating at the interface is likely to initiate debonding and be exacerbated by the pumping effect of dynamic wheel loadings. The delay between commencing placement of each successive layer should not exceed 6 hours. RD-PV-S2 “Plant Mixed Stabilised Pavement” provides further construction requirements.

3.16 The moisture ingress to the subgrade through shrinkage cracks extending full pavement depth also requires consideration. It is difficult to accurately quantify this effect on subgrade strength and it will vary from site to site. In general, the adopted subgrade design CBR values should not exceed the soaked CBR test results.

3.17 Note that the long-term performance characteristics of multi-layer cemented material construction have yet to be fully established in South Australia. Many lane kilometres of moderately to heavily trafficked pavements have been constructed from 1989 onwards. Generally, these pavements consist of about 140 – 150 mm asphalt over two layers of cemented materials placed on the same day. With the exception of varying degrees of shrinkage or environmental cracking, these pavements tend to be indicating satisfactory structural performance. The exception is on some heavily trafficked DPTI freeways, where some issues linked to debonding of the CTSB layers have occurred. As a consequence, use of asphalt-two layer CTSB pavements as a heavy duty pavement has not been permitted on recent major motorway projects.

3.18 In selecting this type of construction, designers need to be aware that the risk of premature distress increases as the design traffic loading increases.

Pavement Layering Considerations

3.19 The number and thickness of pavement layers should be determined based on DPTI specification limits, construction practicalities and performance requirements.

3.20 Careful consideration must be given to the design thickness (also called nominal thickness, or the nominal compacted thickness on pavement work schedules) versus the actual as-constructed thickness that may be realised when layer level tolerances are considered. Section 11 of this Supplement provides further guidance on level tolerance specification.

3.21 The following pavement design layer thickness constraints and preferences apply:

a) All layer thicknesses shall conform to DPTI Specifications and requirements of Table RD-PV-D1 3-1, as well as the following items.

b) Thin-Asphalt on Granular Pavements – for pavements designed in accordance with AGPT-2 Empirical Method (Figure 8.4), the minimum AC design thicknesses in Table RD-PV-D1 3-2 shall apply.

c) A minimum total thickness equivalent to 100 – 175 mm asphalt is required over cemented subbase (refer CL 3.5 to 3.12)).

d) The following minimum asphalt wearing course design thicknesses shall be provided with full depth asphalt pavements and deep strength asphalt pavements:

i) Open Graded Asphalt – OG14 35 mm

ii) Dense Graded Asphalt – AC10 40 mm

iii) Stone Mastic Asphalt - SMA10 40 mm(1)

- Note: (1) This minimum thickness applies when a water proofing spray seal interlayer is placed below the SMA. Where no spray seal is provided, a minimum SMA thickness of 45 mm shall apply.

e) Specification RD-BP-S2 “Supply of Asphalt” includes Type AC14HB, which has lower air voids and additional binder to provide improved fatigue resistance. To inhibit rutting of these mixes, a minimum thickness of cover of 125 mm of dense-graded asphalt is required. SMA wearing courses may be included as part of this 125 mm, but Open graded wearing courses are not to be included.

f) High bitumen content asphalt fatigue layers at the bottom of full depth asphalt pavements should be 60 mm design thickness.

g) Consecutive asphalt layers should generally differ by not more than one mix size.

h) Use of AC20 (DGA28 by Austroads designations) on new works is not acceptable due to a higher risk of mix segregation. This mix is generally intended for rehabilitation works being conducted under live traffic, where construction issues dictate the need to maximise layer thickness.

i) Sprayed seal wearing courses are not given a nominal design thickness in work schedules, with finished surface levels for construction conformance measured on the top of the granular basecourse. However, where a sprayed seal is used as an interlayer, e.g. a 10 mm SAMI below an OGA wearing course, or a 7 mm spray seal below an SMA10 wearing course, then the nominal ALD of the aggregate is typically nominated as the design thickness in the pavement work schedules (5 mm usually). This interlayer thickness is not to be included in design calculations. Table RD-PV-D1 3-1provides an example of this for an OGA wearing course with a 10 mm SAMI.

Table RD-PV-D1 3-1 Pavement Layer Design Thicknesses

Material	Nominal Size (mm)	Typical DPTI Standard Materials	Design Thickness Range[1] (mm)
Asphalt	7	FineAC7	25 – 35
	10	FineAC10	35 – 50
	10	AC10	40 – 55
	14	AC14	50 – 80
Unbound Granular	20	PM1A, PM1, PM2, PM3	80 - 175
	30	PM1, PM2	90 - 175
	40	PM1, PM2, PM3	100 - 200
	55	PM3	130 -220
Plant Mixed Cement Stabilised – Single Layer Construction	All Sizes	SPM2C4	125 – 200*
Plant Mixed Cement Stabilised – Multiple Layer Construction	All Sizes	SPM2C4	125 – 175*

(1) Also referred to as “nominal compacted thickness” on pavement work schedules.

* For layer thicknesses less than 100 mm, the top of existing base is mixed with the new granular material to provide a minimum 100 mm thickness to compact.

Table RD-PV-D1 3-2 Minimum Asphalt Design Thicknesses

Lane DTL (ESA)	Minimum AC Design Thickness	Comments
Thin Asphalt on Granular Pavements		
$< 5 \times 10^5$	40 mm(1)	C320 binder acceptable.
5×10^5 to 1×10^6	80 mm(1)(2)	A polymer modified binder (PMB) must be used in both asphalt layers.
$> 1 \times 10^6$	Minimum thickness of 80 mm(1)(2)	Asphalt layers must satisfy fatigue criterion with GMP analysis (CIRCLY). Greater than 80 mm asphalt may be required, as determined from the GMP. Polymer modified binders should be specified as per Table RD-PV-D1 3-3.
Rural Intersections	40 mm	Single asphalt layers may be used over sprayed seals to withstand localised high horizontal shear stresses in rural locations. Polymer modified binder strongly preferred.(3)

(1) The design shall also incorporate a bituminous prime on the basecourse surface.
(2) Two asphalt layers have proven necessary to provide a structure with acceptably low risk of premature distress and reduce the likelihood of potholes in locations with higher traffic loadings.
(3) This treatment is unlikely to achieve the design life of the underlying pavement structure (e.g. 30 years), with asphalt cracking expected within this period. It is generally adopted in recognition that the AC overlay will provide a greater service life than a sprayed seal surfacing would in this application, and where provision of a heavy-duty pavement structure that would achieve the design life is not viable. The pavement designer should develop a heavy-duty pavement design alternative that will achieve the design period to allow for comparison of the two options, with the 40 mm asphalt overlay option adopted subject to the Principal's acceptance of the associated performance risk.

Use of Strain Alleviating Membrane Interlayers

3.22 To inhibit reflection cracking, a strain alleviating membrane interlayer (SAMI), generally size 10 mm S25E 1.8 – 2.0 l/m2, shall be applied on top of cemented material subbase layers when the dense mix asphalt cover is less than 200mm. The design thickness of a SAMI shall not be included in the pavement thickness calculations but should be accounted for in the pavement construction schedule levels, with 5 mm typically allowed for its thickness.

Improved Subgrades

3.23 Lime stabilisation of soft subgrades or excavation and replacement are treatments that are commonly used as construction expedients in DPTI works. Adoption of an improved design subgrade modulus due to these treatments should only occur if the long-term properties have been validated by thorough field and laboratory testing. These treatments are mechanistically modelled in the same manner as selected subgrade material.

Surfacing Type

3.24 Table RD-PV-D1 3-3, Table RD-PV-D1 3-4 and Table RD-PV-D1 3-5 provide guidance for the selection of sprayed seal and asphalt surfacing types.

Table RD-PV-D1 3-3 Asphalt Surfacings

Item	Considerations and Requirements
Wearing Course Selection – Texture and Noise	OGA can provide good skid resistance, high texture, good noise reduction and minimise water spray for improved visibility and drainage. It has high in-situ voids, usually in excess of 20%, resulting in a lower service life than DGA & SMA. SMA has a lower texture, less noise reduction and spray reduction compared to OGA, but is better than DGA with these characteristics. It has low in-situ air voids of approximately 5.0% and a high polymer modified binder content, providing a longer service life than OGA and DGA. SMA is typically more expensive than DGA, which is in turn more expensive than OGA. However, the cost of the water proofing SAMI / spray seal also need to be considered with SMA and OGA, during both initial construction and in future resurfacing works (where the seal must be reinstated also, if removed). SMA is generally the preferred wearing course mix for higher speed roads (typically 80 km/hr or higher), in providing acceptable texture and the best durability. However, OGA may be be required for drainage purposes, particularly on high speed sites with substantial gradients (usually assessed from water film thickness analyses by drainage engineers), or, for noise mitigation needs, or to match existing assets and other reasons. A lower texture is generally acceptable on lower speed roads, which also typically have no significant noise mitigation or drainage issues. DGA is therefore the typical wearing course in lower speed urban environments, with SMA or OGA reserved for more heavily trafficked routes, or where there are project specific issues warranting their increased cost.
Spray Seal / SAMI requirements below Asphalt Wearing Courses.	A sprayed seal or SAMI should be provided as a waterproofing layer below a Stone Mastic Asphalt (SMA) or Open Graded Asphalt (OGA) wearing course, respectively. For SMA, the waterproofing seal is not required if the nominal thickness is equal or greater than 45mm and the in-situ specification RD-BP-C3 voids target of 5.0% is achieved. No water proofing seal is required below DGA.

Table RD-PV-D1 3-4 Guide To The Selection Of Asphalt Types[3]

Course[4]		General Mix Designations[5]	Binder Class	Target Mix Design Air Voids %)	Applications/Comments
Wearing	Heavy Duty Wearing Course	AC10M	A5E	4.0	Motorway, significant grades (Table PV-D1-6), approaches to heavily trafficked signalised Intersections (50-150 m, but consider level of saturation), Roundabouts, Bus Lanes, Bus Stops.
	Medium Duty Wearing Course	AC10M	A15E	4.0	Other signalised and non-signalised Intersections, mid-block zones.
		OG14	A15E	20	Non-signalised Intersections, mid-block zones, no high horizontal shear locations.
		SMA10	A5E[1]	3.5	Non-signalised Intersections, mid-block zones.
Structural	Modified Intermediate Course	AC10M, AC14M	A5E[1]	4.0	Motorway, significant grades, approaches to heavily trafficked signalised Intersections, Roundabouts, Bus Lanes, Bus Stops. AC10M required where wearing course delayed.
	Standard Intermediate Course	AC10M, AC14M	C320	4.0	Other Intersections, mid-block zones. AC10M required where wearing course delayed.
	Base Course	AC14M	C320	4.0	Normal Works.
		AC14HB	C320	2.5	High binder fatigue resistant bottom asphalt layer for full depth asphalt configurations.
Special Surfacings	Medium Duty Thin Flexible	SMA7	A15E	3.5	Texture, Noise Reduction, no high horizontal shear locations.
		OG10	A15E	20	Texture, Noise & Spray Reduction, no high horizontal shear locations
	Footpaths, Bikeways, Crossovers and Carparks and Sport Recreation Courts	FineAC7L	C170[2]	4.0	Light maintenance vehicle trafficking only.
	Carparks and Local Roads	FineAC10L	C170[2]	4.0	Medium to heavy maintenance vehicles (mainly rigids), minimal large articulated heavy vehicles.

(1) A15E may also be approved.
(2) C320 may also be approved.
(3) This table reflects asphalt mixes and binders currently available in the SA market. There are other alternatives available in the Austroads Guide series which are also acceptable, following validation for SA use.
(4) Table RD-PV-D1 3-5 provides guidance on Course descriptions and position within the pavement structure.
(5) DPTI asphalt mix size designations are based on ensuring there is 10% of this nominal size material within the mix. (e.g. AC10 has a minimum 10% material retained on 9.5 mm sieve and AC14 has minimum 10% retained on 14 mm sieve etc.) These may be coarser than other typical interstate mixes.

Table RD-PV-D1 3-5 Thick Asphalt Pavement Course Descriptions

Layer Number	DPTI Course Name	Comments
1	Wearing Course	Surface layer.
2	Intermediate Course	Directly below wearing course. Sometimes referred to as levelling course.
3	Base Course	All asphalt layers below intermediate course are referred to as base courses. There may be more than one base course for thicker asphalt pavements, with each base course numbered sequentially (i.e. base course 1, basecourse 2, basecourse 3, etc.). The bottom base course layer may be a high binder AC14HB layer in some configurations.
4	Subbase	Granular Subbase (e.g. PM2/20) or Cement Treated Subbase (e.g. SPM2C4).

Sprayed Seal Surfacings

3.25 The detailed design of sprayed seal surfacings is beyond the scope of AGPT-2 and this supplement. DPTI requires seal design to be undertaken in accordance with the following documents:

3.26 DPTI Master Specification:

a) RD-BP-C5 “Application of Sprayed Bituminous Surfacing” and its commentary (current edition).

b) RD-BP-D2 “Design of Sprayed Bituminous Surfacing”.

3.27 However, for the purpose of pavement design Table RD-PV-D1 3-6 and Figure RD-PV-D1 14-1 in Appendix 2: Nominal Sprayed Seal Design may be used as a guide for completing pavement schedules, as per the examples in Table RD-PV-D1 11-2, Table RD-PV-D1 11-3 and Table RD-PV-D1 11-4.

Table RD-PV-D1 3-6 Guide to the Selection of Sprayed Bituminous Surfacings

<table>
<tr><th>Period When Treatment Applied</th><th>Open to Traffic Immediately</th><th>Initial Trafficking Between April to September</th></tr>
<tr><td>October to March*</td><td>Prime† pavement with very light prime (AMC 00) or medium prime (AMC 0) followed in not less than three days by 14/7 or 16/7 double seal.</td><td rowspan="2">Apply prime† and 7 or 10 mm seal followed the next summer by 14/7 or 16/7 double seal after several weeks of summer trafficking has occurred.</td></tr>
<tr><td>April</td><td>Very light prime followed by 14/7 or a 16/7 seal.
Where a geotextile is required‡ adopt a 7 mm or 10 mm initial seal. Ensure this seal receives warm weather trafficking for 2 weeks, and then apply 14/7 or 16/7 geotextile double seal.</td></tr>
<tr><td>May to September</td><td colspan="2">7 mm or 10 mm primer seal#. Traffic in warm weather for 2 weeks (emulsion) or 3 months (cutback) prior to applying final seal in summer. Preferably postpone surfacing treatment until October and apply prime and double seal.</td></tr>
</table>

* In areas north of Port Augusta, October to April may be appropriate
† Selection of prime type depends on type of basecourse material and porosity of surface. Embedment allowances over primed basecourse are determined in accordance with Austroads Seal design methods. Maximum permissible ball penetration values will depend on traffic volume and composition.
‡ Application of geotextile seals should be limited to the period between November to March inclusive.
# It is advisable to leave cutback primer seals exposed for 6 – 12 months. Emulsion primer seals may have a final seal applied after several weeks of trafficking in hot weather.
Notes:

a) The presence of salt in the basecourse can result in damage to new seals. Where salinity may be an issue, specialist advice should be sought.

b) Normally C170 bitumen used for new construction. In marginal weather conditions polybutadiene (PBD) based binder should be used in the bottom coat of double seals.

c) Geotextile Seals should not be used in areas of high shear forces. E.g. tight bends, intersection approaches, steep inclines, etc, where they may debond.

Bridge Deck Surfacings:

3.28 Bridge decks typically have an asphalt wearing course as the running surface, to allow periodic maintenance, provide a uniform wearing course along the road and accommodate minor shape correction during construction.

3.29 The following minimum detailing requirements apply to DPTI bridge deck asphalt surfacings:

a) The bridge deck wearing course mix will typically match the wearing course on the pavements approaching the bridge, and may therefore be either dense graded asphalt, SMA or OGA. Where OG is adopted, consideration should be given to surface levels and lateral drainage requirements from the OGA.

b) A polymer modified binder is required in all asphalt to provide maximum durability.

c) A second asphalt layer is required below the wearing course to provide additional cover over the deck structure for future maintenance works.

d) A very light prime is to be applied to the concrete surface (typically 0.2 – 0.3 l/m2 depending on the surface texture and the finish, ideally confirmed by trial during construction). The prime may be a specialised proprietary product in which case it must be applied strictly as per manufactures instructions. Otherwise a cutback bituminous prime shall be allowed to cure for 3 days prior to the application of any subsequent surfacing treatment. It should be noted that some concrete curing compounds might react adversely with bitumen. If this is the case, the curing compound needs to be removed prior to the application of the prime.

e) Application of a 10mm S25E SAMI (1.6 – 1.8 l/m2) to the primed concrete bridge deck surface is required, to assist in bonding and for water proofing.

f) The surfacing treatment thickness dead load must be considered in the structural design of the bridge.

3.30 The resulting required minimum bridge deck configurations then are as per Table RD-PV-D1 3-7.

Table RD-PV-D1 3-7 Guide to the Selection of Bridge Deck Surfacings

Layer	OGA	SMA	DGA	Design Thickness (mm)
Wearing Course	OG14(1)	SMA10(1)	AC10(1)	40
Levelling Course(2)	AC10	AC10	AC10	40
Spray Seal	10 mm SAMI	10 mm SAMI	10 mm SAMI	5
Prime	Very Light Prime	Very Light Prime	Very Light Prime	-
Concrete Bridge Deck				Total 85

(1) Wearing course selection as per Table RD-PV-D1 3-3 and Table RD-PV-D1 3-4.
(2) More than one layer, possibly with P&R of initial new AC layers, may be necessary to achieve levels. Bridge designer must be informed in relation to applied pavement dead loads.

Pavement Joint Considerations & Treatment Extents

3.31 The design of joints for rigid (concrete) pavements is an integral consideration in their design and should conform to the requirements of Section 9 of the Supplement.

3.32 For flexible pavements, joints can occur at various locations within the pavement, including:

a) At the interface of new works with existing pavements, which may be transverse, diagonal, longitudinal or other shaped joints, depending on the alignment of the existing and new pavements;

b) At the edges and ends of a construction run, e.g. at the end of a production shift within the same pavement configuration;

c) At changes in pavement configuration;

d) At the interface with other road elements like kerb and gutter, subsurface drainage, etc.

3.33 The structural competency of flexible pavements at construction joints is generally not as sound as in other areas. The joint is a discontinuity in the pavement, forming a plane of weakness, and the adjacent pavement materials tend to be weaker and more permeable. In addition, stiffness differences between different pavement types can contribute to differential deflections and performance issues. This creates a significant risk of deformation, cracking and other distress at joints, which is substantially increased where the joint will be trafficked.

3.34 The location of joints and the design of joint details to mitigate performance risks is therefore an important part of pavement design.

3.35 Joints should preferably not be located in wheel paths, or where unavoidable, detailed appropriately. Such details can include stepping layer terminations and / or using reinforcing geofabrics and geogrids, as appropriate to the pavement configurations, support conditions, traffic loads, project scope and other factors.

3.36 Joint details are required for all pavement joints in a project, including:

a) Each combination of unique abutting pavement types, which includes joints between new and existing pavements at the extent of works.

b) Edge and end joints at the end of construction runs within the same configuration.

c) Joints between pavements and other road elements, like kerb and gutter, central medians and subsurface drainage.

3.37 These joint details must be shown on the project drawings.

3.38 In addition, the following minimum requirements apply to pavement treatment extents from consideration of jointing and other issues, unless prior approval from DPTI has been obtained:

a) Pavement treatments should extend to the edge of lanes to avoid joints in the wheel paths;

b) The pavement treatment should extend to the limit of geometric changes, so that the new and existing surface levels match, and to realise these geometric changes;

c) The extent of pavement treatments must match the extent of pavement marking (traffic control layout) changes, to ensure that all existing pavement marking is removed;

d) The extent of pavement treatment on intersecting Local Government roads must extend, as a minimum, to the extents of the Commissioner of Highways maintenance responsibility, as defined in Section 3.1 of the DPTI Operation Instruction 20.1 Care, Control & Management of Roads (Highways) by the Commissioner of Highways (Section 26 of the Highways Act) (DPTI, 2008). (https://www.dpti.sa.gov.au/standards/tass)

Shoulders

3.39 Shoulders provide structural support to the pavement edge, as well as providing additional trafficable width for road users. They are typically required where no kerb and gutter is present on urban fringe and rural roads, and on Motorway class roads, where the shoulder forms an emergency break down lane.

3.40 Shoulders in rural and urban fringe environments may be sealed or unsealed (or part sealed and unsealed). The pavement and shoulders are typically either a spray sealed or thin asphalt surfaced, unbound granular pavement. Motorway class pavements are typically “heavy duty” pavement configurations with asphalt surfaced pavements and shoulders.

3.41 If not designed and constructed using the same adjacent trafficked lane pavement composition, then the functional purpose of the shoulder needs to be carefully considered so that appropriate materials and thicknesses are used. This is particularly important where the shoulder is sealed and may be used as a heavy vehicle climbing lane, for parking of vehicles or may be frequently trafficked on the inside edge of curves, with general traffic wander across lanes or at property entrances.

3.42 The pavement structure used in the shoulder will also be affected by the sealed and unsealed shoulder widths, including bike lanes, if present. Minimum seal widths are generally derived from Austroads Road Design requirements rather than pavement structural design considerations.

3.43 Subject to these considerations, the following minimum pavement structure requirements are applicable:

Urban fringe / rural environment – sprayed seal or thin asphalt on granular pavements:

3.44 Sealed part of shoulder:

a) Edge line to 0.5 m outside edge line - same pavement structure as adjacent trafficked lane;

b) 0.5 m outside edge line – either same pavement structure as adjacent trafficked lane, or reduced structure designed in accordance with Section 8 (CL 8.20 to 8.21). Matching shoulder thickness to the adjacent trafficked lanes layer thicknesses should be considered for constructability, where possible.

3.45 Unsealed part of shoulder:

a) Minimum total thickness of granular materials is 150 mm for major Rural Arterials and 200 mm for National Highways. The uppermost 100 mm or more should be a Class 2 pavement material or similar with reasonably high plasticity index (6 to 8) to provide low permeability and good surface integrity (resistance to ravelling). A greater thickness may be required from structural considerations, as per Section 8 (CL 8.20 to 8.21).

Motorways – Asphalt Surfaced Pavements:

3.46 Sealed Shoulder:

a) Edge line to 0.5 m outside edge line - same pavement structure as adjacent trafficked lane;

b) 0.5 m outside edge line - either same pavement structure as adjacent trafficked lane, or reduced structure designed in accordance with Section 8 (CL 8.20 to 8.21). Matching shoulder thickness to the adjacent trafficked lanes layer thicknesses should be considered for constructability, where possible.

c) Unsealed shoulders are generally not acceptable on a Motorway class asset.

Rigid Pavements:

3.47 Section 9 (CL 9.18 to 9.20) of the Guide describes how shoulders are considered as an integral part of the base thickness design process for rigid pavements.

Settlement

3.48 The Supplement and the Guide do not contain provisions for settlement below the pavement layers. Where required, additional investigations and assessments should be carried out to determine if settlement may occur and, if so whether pre-treatment (such as drainage and surcharge, or expansive clay subgrade treatments) of the formation is required to reduce the amount of settlement after the pavement is constructed.

3.49 DPTI Design Standard - EW100: Earthworks for Roads provides further guidance on the requirements for design of earthworks for roads, including material selection, construction methodology, geometric design, water / moisture control measures and surface treatment.

Frequency of Maintenance Treatments

3.50 The whole-of-life costing (Chapter 10) requires the maintenance costs of pavement alternatives to be estimated over the analysis period.

3.51 The Austroads Guide to Pavement Technology Part 3: Pavement Surfacings (2009). provides guidance on the range of expected service lives of surfacings. There are some situations (e.g. expansive clay subgrades, subgrades subject to settlement) where higher than normal frequency and extent of maintenance are required to seal the pavement and restore shape.

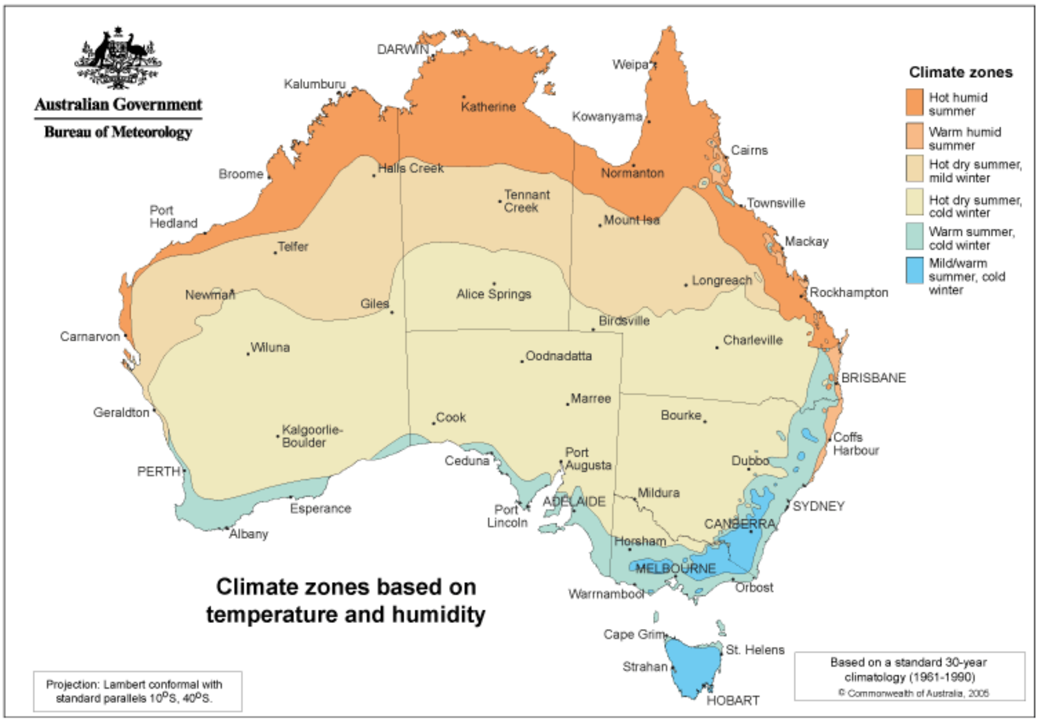
4 Environment

General

4.1 Australian climatic zones based on temperature and humidity are indicated in Figure RD-PV-D1 4-1. Most of the coastal areas of South Australia experience warm summers and cool winters, while the more scarcely populated regions are generally within the Hot, Dry Summer, Cold Winter zone.

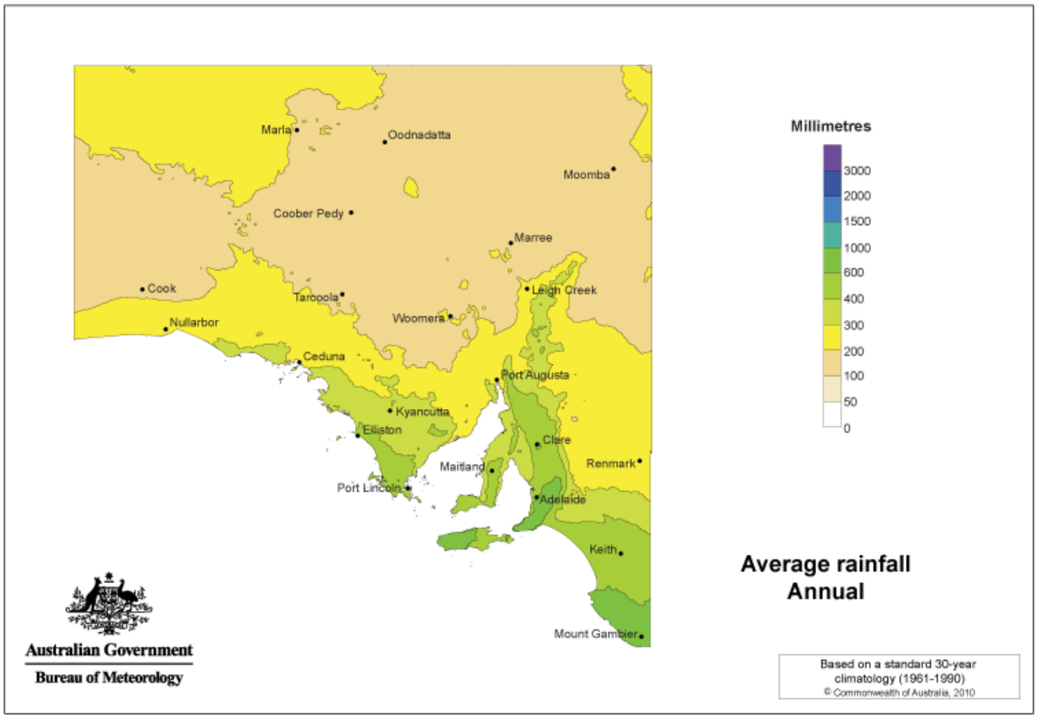
4.2 South Australia is relatively dry compared to other Australian states and typically has low rainfall and high evaporation rates as shown in Figure RD-PV-D1 4-2 and Figure RD-PV-D1 4-3. Except for the south east corner of South Australia, the remainder is wholly arid or semi-arid. The south east corner, including Adelaide, has dry summers with median annual rainfall of 400 – 800 mm, mostly in the winter months.

Figure RD-PV-D1 4-1 Australian Climatic Zones



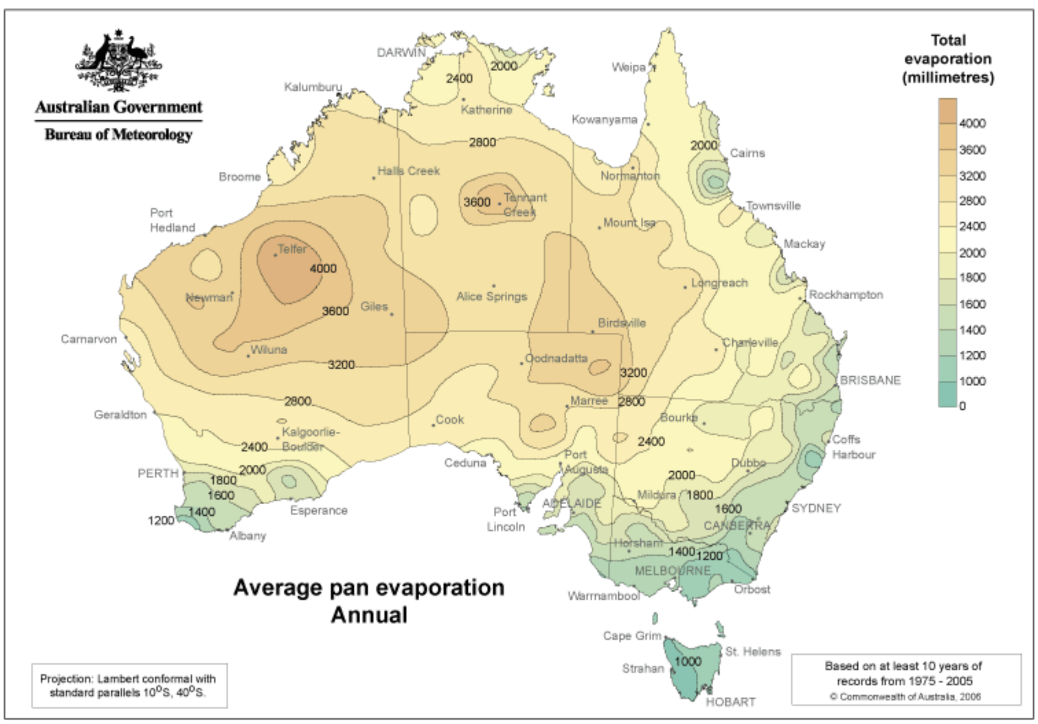
(www.bom.gov.au/climate/averages)

Figure RD-PV-D1 4-2 Average Annual Rainfalls for South Australia



(www.bom.gov.au/climate/averages)

Figure RD-PV-D1 4-3 Average Evaporation Map - Annual



(www.bom.gov.au/climate/averages)

5 Subgrade Evaluation

Factors to be considered in Estimating Subgrade Support

Moisture Changes during Service Life (Expansive Soils)

5.1 Soils change in volume as their moisture content changes. The magnitude of the volume change depends on:

a) The reactivity (shrink/swell potential) of each material under the pavement;

b) Extent (width and depth) of each material; and

c) Magnitude of changes in moisture content.

5.2 Hence strategies to inhibit loss of shape due to Expansive soils include:

a) Minimising the changes in moisture content from the time of construction;

b) Removing the expansive soil (expansive soils) in the zone of seasonal moisture change and replacement with a non-expansive material; or

c) Treating the material to reduce its reactivity, e.g. lime stabilisation.

5.3 The Guide lists a number of construction approaches.

5.4 South Australia has substantial deposits of expansive clay soils, including soils in the Extremely Reactive category (by AS2870 definitions). In the event that expansive clay soils have been identified onsite, a geotechnical assessment is required to assess the risks of future pavement shape loss and the most appropriate treatment strategy.

5.5 In wetter areas of Australia where expansive soils expand in volume after construction, some road agencies place a non-expansive fill to surcharge the expansive soil to minimise the expansion. This may not always be appropriate for South Australia as often the expansive soils reduce in volume rather than expand.

5.6 Moisture changes in expansive soils may be reduced by covering them with a low permeability select fill layer with maximum Weighted Plasticity Index (WPI) of 1200.

5.7 Historically, DPTI have generally adopted two approaches for expansive clay soils:

a) For Minor projects, a minimal disturbance approach to preparing subgrades is generally used. This approach recognises the potentially higher undisturbed soil strengths of overconsolidated expansive clay subgrades, and the equilibrium moisture conditions that may be encountered. Reworking these subgrades or exposing them during wet periods can lead to building in undesirable moisture contents that are above equilibrium, with the risk of shrinkage deformation and cracking of the pavement surfacing as the subgrade dries back to an equilibrium condition. Moisture management during construction is a critical requirement with this approach.

b) On Major projects, RD-EW-D1 “Design of Earthworks for Roads” is used. This provides a design method and performance criteria for calculating expansive soil volumetric changes. Application of this method can result in the need to dig out and replace up to 2 m, or more, of subgrade soils on more expansive sites, or sites with desiccated or wet subgrades.

5.8 This is information is provided as a general guide only and a project specific assessment must be undertaken to assess each project’s conditions to develop a suitable design approach that will achieve performance requirements. Critical factors in this assessment include geotechnical conditions at the site and the reactivity of the soil profile, as well as potential moisture changes. Required performance standards and acceptable performance risk are also key issues.

Laboratory Determination of Subgrade CBR and Elastic Parameters

5.9 Laboratory preparation and testing conditions to be adopted to determine the subgrade design CBR are as follows:

a) Density Ratio of 98% using Standard compactive effort and optimum moisture content; and

b) 4.5 kg surcharge and 4-day soaking.

5.10 This test condition aligns with Table 5.2 of the Guide, allowing classification of the expansive nature of the soils and provides a lower bound strength value.

5.11 It is recommended that prior to the 4-day soaking, specimens of cohesive materials also be tested unsoaked to assess their sensitivity to moisture variations that may occur during construction and in service.

5.12 The vertical design modulus of a subgrade is determined from its design CBR and Equation 2 of the Guide. A maximum vertical modulus of 100 MPa shall apply to normal soil subgrades and where sound rock formations exist, a maximum value of 150 MPa is applicable.

5.13 If in situ stabilisation of the subgrade is undertaken after detailed field and laboratory investigations have verified the long-term performance properties (Little, 1995), these stabilised layers are characterised as selected subgrade materials (refer Section 5 (CL 5.17 to 5.23)).

Adoption of Presumptive CBR Values

5.14 The presumptive subgrade CBR value for each material type shall not exceed the lowest of the range of values given in Table 5.4 of the Guide.

5.15 Generally, the use of presumptive CBR values is not acceptable on DPTI projects, except for minor works, and only subject to DPTI approval. A site-specific pavement and geotechnical investigation should be undertaken. This is discussed further in Section 5 (CL 5.24 to 5.26).

Selected Subgrade Materials

5.16 Select fill or selected subgrade materials may be provided above the in situ subgrade to:

a) Provide a working platform on which to compact pavement layers, particularly over soft subgrades;

b) Increase the strength and uniformity of the supporting pavement substrate;

c) Increase the height of pavements on embankments; and

d) Reduce moisture changes of highly expansive subgrades (refer Section 5.8).

5.17 In some situations (refer Section 3.23) it may be necessary to excavate soft or expansive subgrades prior to placement of the fill.

5.18 Fill materials shall meet the requirements of Classification A or B of RD-EW-C1 “Earthworks” and the following:

a) The Characteristic Strength of a fill material is defined as the tenth percentile value (i.e. mean – 1.3 x standard deviation) of the laboratory 4-day soaked CBR results from at least six samples. The samples shall be taken in accordance with the test procedure TSA-MAT-TP226 with at least one sample from each of six Lots. The design CBR of any particular fill material shall not be greater than two-thirds of the Characteristic Strength.

b) The top 150 mm of fill shall have a minimum Characteristic Strength of CBR 15% and a maximum vertical design modulus of 100 MPa.

c) Fill materials with a WPI greater than 1200 shall not be used directly below pavements.

d) Swell values determined with 4.5kg surcharge and 98% Standard compaction should be < 1% wherever possible.

5.19 DPTI Design Standard: Earthworks for Roads (EW100) provides presumptive design parameters for conforming Type A, B & C fill for use in mechanistic modelling.

5.20 Lime stabilisation of clay subgrades (3% - 5% hydrated lime) has proven a successful and cost-effective construction expedient for DPTI on a number of projects. Where minimal or no field and laboratory testing is undertaken for these applications, such treatments are not considered in the pavement design calculations.

5.21 However, if appropriate site investigations and laboratory testing for lime demand and Unconfined Compressive Strength (UCS) are used to verify the long-term properties of the lime stabilised subgrade (Little, 1995), the structural contribution of the layer may be considered in the same manner as a selected subgrade material. The Characteristic Strength is determined and the design CBR calculated as two thirds of this value, with an upper limit of CBR 15. The design vertical modulus of the top sublayer (refer Section 8.2.2) is 10 times the design CBR of the lime stabilised material up to a maximum value of 150 MPa.

5.22 The designer must ensure that the laboratory curing regime aligns with anticipated field conditions and that a suitable construction verification scheme of parameters adopted in the design is implemented.

Pavement & Subgrade Investigation

5.23 Most DPTI projects require a site-specific pavement and subgrade investigation. This includes projects being undertaken by others that impact and interface with DPTI road assets, which generally must conform with DPTI standards within the DPTI owned area. An example of this is a new development, where the proposed access roads will connect to an existing DPTI road.

5.24 A properly scoped site investigation will provide valuable data to the designer. Benefits include:

a) Identification of the subsurface conditions, including soil types, presence of any fill, moisture condition and in situ strength.

b) Sampling of subgrade soils for use in laboratory CBR testing (and other testing as appropriate), informing the selection of the design CBR value and assessment of subgrade soil reactivity;

c) Identification of existing pavement configurations (where they are affected by the new works), allowing design of appropriate joint details between new and existing pavements, as well as consideration of the existing pavement structure, performance and remnant life;

d) Adoption of less conservative design parameters, from the better understanding of the subsurface conditions and increased designer confidence, leading to reduced pavement thicknesses and costs. Or, adoption of lower design parameters where conditions are worse than may have been assumed, which would otherwise have delivered a deficient design if presumptive values had been used.

5.25 The results of the site investigation can also be included in the construction contract tender as background information. This can lead to reduced tender prices, since the contractor has a greater understanding of site conditions, and reduced uncertainty and risk. A design optimised for actual site conditions can also avoid increased costs and delays during construction from needing to undertake redesign or additional works related to subsurface issues when they are only identified for the first time during construction.

6 Pavement Materials

General

6.1 The Guide and the Supplement provide procedures for pavement design using materials that comply with DPTI Master Specification for Roadworks, which provides specifications for:

a) Unbound granular materials (recycled and quarry based), including pavement materials, sands, ballast, sealing and asphalt aggregates;

b) Asphalt, Sprayed seals, Slurry Surfacings, and bituminous binders;

c) Controlled Low Strength Material (CLSM);

d) Stabilised materials, including cement, lime and foamed bitumen;

e) Geotextiles.

6.2 For non-standard materials, similar performance relationships and presumptive modulus values may not be applicable.

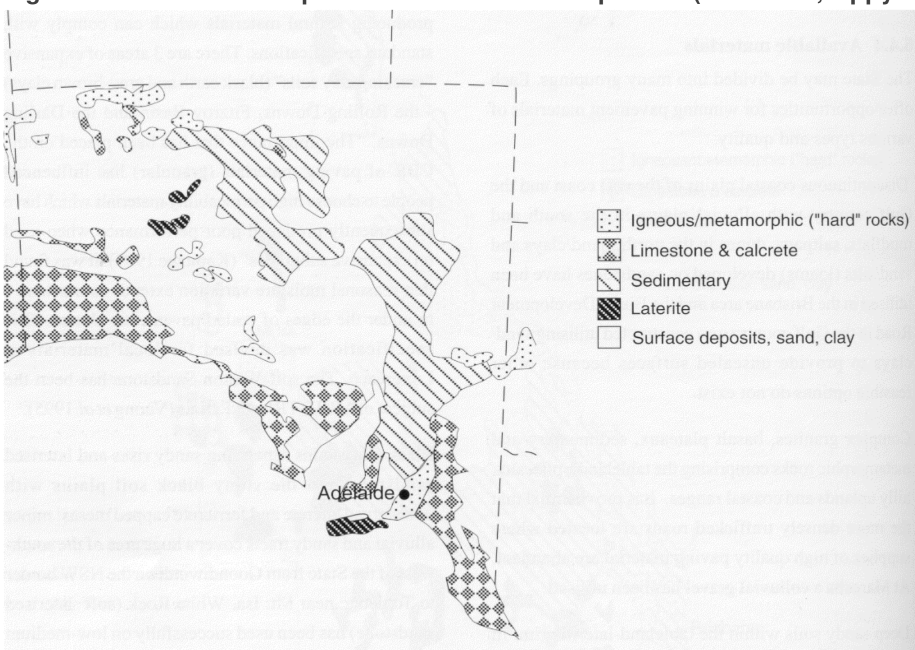
Unbound Granular Materials

Availability of materials

6.3 A simplified surface rock map of South Australia is illustrated in Figure RD-PV-D1 6-1. It is apparent that hard rock sources are very limited in extent. Igneous rocks occur in some small outcrops in the central and far north-west of the state. Basalt is quarried near Mt Gambier. Metamorphic hard rocks are to be found on the Eyre Peninsula and in the Mt Lofty/Flinders Ranges. Otherwise sedimentary and superficial surface materials cover the state. Calcretes, with or without processing, are extensively used and perform satisfactorily on light to moderately trafficked roads. In outback areas a wide range of local materials are utilised including high gypsum content rubbles, shales, tableland stone, iron pan, river gravel and clays.

6.4 Further details on material availability can be found in Robinson, Oppy and Giummarra (1999).

Figure RD-PV-D1 6-1 Simplified Surface Rocks Map of SA (Robinson, Oppy And Giummarra, 1999)



Standard granular materials

6.5 The quality and strength characteristics required of granular materials depends upon the following factors and their interactions:

a) Traffic (volume, axle group types and loads)

b) Climate

c) Pavement configuration and drainage

d) Subgrade

6.6 Lightly traffic roads in dry environments can more successfully use lower quality granular materials than roads with higher traffic loadings in wet environments.

6.7 RD-PV-S1 “Supply of Pavement Materials” details requirements for standard granular materials comprising crushed quarry products, natural gravel, sand and recycled pavement materials. These specifications include the range of products that meet traditional grading based manufacturing tolerances, as well as those that use performance based mix design limits to deliver the required stiffness, shear strength and deformation resistance properties.

6.8 Specifications for higher standard Quarried Pavement Materials were developed for the construction of very heavily trafficked unbound granular pavements with thin surfacings, and are designated PM1A or PM1B heavy duty materials. They provide improved stability and workability and are compacted to 100% Modified Maximum Dry Density and dried back to no greater than 60% Optimum Moisture Content, and are placed in layers not exceeding 125mm thickness. However, as their availability is limited, project specific assessment is required.

6.9 The general DPTI pavement material types are summarised in Table RD-PV-D1 6-1. Size 40 mm materials are unsuitable as base layers as they tend to segregate during placement and do not provide adequate surface tightness and finish.

Table RD-PV-D1 6-1 Standard Granular Material Types

Material Type/Class	Source	Size (mm)	Primary Usage
PM1A*, PM1B*, PM1† (Class 1)	Quarried	20	Base
		30	
PM1† (Class 1)	Recycled	20	
		30	
PM2† (Class 2)	Quarried	20	Upper subbase, lower subbase, working platforms for heavily and moderately trafficked roads, base for lightly trafficked roads
		30	
		40	
	Recycled	20	
		30	
		40	
PM3† (Class 3)	Quarried	20	Working platforms for moderately trafficked roads and lower subbase layers for lightly trafficked roads
		40	
		55	
		75	
	Recycled	20	
		40	

* Heavy duty pavement materials use only a grading based specification
† Grading based or mix design specifications
Note: Recycled, heavy duty and mix design products require project-specific consideration and DPTI approval.

Non-standard granular materials

6.10 Marginal or non-standard granular materials should only be used after consideration of:

a) The documented performance history of the proposed material
b) Costs relative to standard materials
c) The predicted traffic loading
d) The climate at the site
e) The moisture sensitivity of the subgrade
f) The quality and uniformity of the materials as shown by laboratory testing
g) Consequences of poor performance
h) Suitability and cost-effectiveness of mechanical or chemical stabilisation

6.11 These materials commonly have lower moduli than standard granular materials, so greater thickness is required to reduce the stresses/strains on the subgrade. However, it is not always possible to obtain equivalent performance by using thicker layers of non-standard materials.

6.12 Thicker pavement layers for lower moduli materials may result in the same subgrade strain as for thinner layers of standard materials; thus the extent of rutting of the subgrade is similar. However, the use of the non-standard materials may result in inferior performance due to deformation within the pavement layers under traffic loading leading to rutting and early pavement distress. The use of repeated load triaxial testing to provide the values for modulus and permanent deformation will assist in predicting the performance of non-standard materials compared to standard materials. Specialist advice should be sought in undertaking such evaluations.

6.13 The existing DPTI road network includes pavements with layers of coarse gravel to cobble sized particles, which are often referred to as “macadam”. These materials can typically be placed with a graded profile from coarser to finer materials, and can provide a strong, robust, relatively moisture insensitive layer that provides good pavement performance, with moduli greater than conventional crushed rocks.

6.14 Macadam is generally not used in modern pavement designs for a number of reasons, including the inability to conduct nuclear density testing on such coarse materials for QA purposes during construction, as well as the cost of placement and materials. Pavement designs that consider utilising existing remnant macadam layers should refer to AGPT-5 and the DPTI Part 5 Supplement for further guidance for this.

Determination of Modulus of Unbound Granular Materials

6.15 The modulus of granular materials can be determined using repeated load triaxial equipment. As the modulus depends on density, moisture content and stress state, it is essential that the material be tested under conditions which approximate those occurring in service.

6.16 DPTI adopts the following conditions for testing of granular materials characterisation:

a) 80% modified optimum moisture content

b) 98% modified maximum dry density

c) Stress conditions as defined by Austroads (2007) and documented in test procedure TSA-MAT-TP 183.

6.17 The maximum allowable design modulus from direct measurement shall be 350 MPa.

Presumptive values

6.18 In determining the top vertical moduli of DPTI Class 1 base materials, the typical values in Tables 6.3 and 6.4 of the Guide for normal standard crushed rock shall be used.

6.19 For base materials that do not conform to RD-PV-S1 "Supply of Pavement Materials" but have proven performance in the field, the maximum modulus shall be 300 MPa under thin bituminous surfacings.

Cemented Materials

Introduction

6.20 RD-PV-S1 "Supply of Pavement Materials" details various types of plant-mixed stabilised materials produced by the addition of cement, fly ash, lime, ground granulated blast furnace slag (GGBFS), bitumen or combinations of binders to granular material. As cemented materials need to include cementitious binding agents in sufficient amounts to produce a bound layer with significant tensile strength, not all RD-PV-S1 "Supply of Pavement Materials" stabilised materials meet this definition. The RD-PV-S1 "Supply of Pavement Materials" materials listed in Table RD-PV-D1 6-2 are those DPTI consider to be cemented materials. The source material may be natural quarried material or, where approved, recycled materials complying with RD-PV-S1 "Supply of Pavement Materials". In addition, stabilised material may be specified by either binder content or strength.

Table RD-PV-D1 6-2 Cemented Material Types(1)

Specification Type	Binder	Min 28 Day UCS	20 mm Class 2(2) (PM 2/20)	30 mm Class 2(2) (PM 2/30)	40 mm Class 2(2) (PM 2/40)
Binder Control	Target binder 4% Type GB cement	-	SPM2/20C4	SPM2/30C4	SPM2/40C4
	Target binder 5% Type GB cement	-	SPM2/20C5	SPM2/30C5	SPM2/40C5
Strength Control	Cement, fly ash GGBFS, and/or lime	4MPa	SPM2/20C4MPa	SPM2/30C4MPa	SPM2/40C4MPa
		5MPa	SPM2/20C5MPa	SPM2/30C5MPa	SPM2/40C5MPa

(1) Materials with a 28 day UCS less than 4 MPa (AS 1141.51) are not used in cemented designs because of durability concerns.
(2) Class 1 materials may be substituted for Class 2
GGBFS = Ground Granulated Blast Furnace Slag; GB = General Blended.

6.21 Finely graded gravels, clayey gravels, silty sands (>50% passing 0.425 mm sieve) and other materials which do not achieve significant particle interlock are not included in the definition of cemented materials as their fatigue performance would be variable and unpredictable.

Determination of Design Modulus

6.22 The moduli of cemented material are dependent on a number of factors, including:

a) Source material quality, grading etc

b) Binder type and quantity

c) Compaction and moisture

d) Curing regime

6.23 The design modulus for the cemented materials detailed in Table RD-PV-D1 6-2 shall be no greater than 3500 MPa. This maximum modulus assumes seven days initial curing with negligible traffic, as per DPTI construction specification RD-PV-S2 “Plant Mixed Stabilised Pavement”. Following initial curing, a primer seal may be placed to provide a stable moisture regime to promote longer term curing and to assist with bonding to any subsequently placed asphalt layers.

6.24 For cemented materials not complying with RD-PV-S1 "Supply of Pavement Materials” but which have proven field performance, the design moduli may be determined from UCS test results as follows:

$$E_{FLEX} = 1000\ UCS \qquad (6.11)$$

where E_{FLEX} = flexural modulus (MPa) of field beams at 28 days moist curing, and
UCS = Unconfined Compressive Strength (MPa) of laboratory specimens at 28 days.

6.25 The maximum design modulus for these non-standard materials shall not exceed 3500 MPa.

Factors Affecting the Fatigue Life of Cemented Materials

6.26 Cracking of cemented materials can occur:

a) Where environmentally induced stress exceeds the tensile strength of the bound material, e.g. due to shrinkage; and

b) At the end of the fatigue life for a bound pavement layer as a result of applied load applications exceeding the fatigue limit.

6.27 Environmentally induced stress can result from circumstances such as:

a) Volume change in pavement layers from moisture and/or temperature changes;

b) Curling (temperature differentials) and warping (moisture differentials); and

c) Substrate movement (settlement and/or volume change).

6.28 If cracking reflects to the surface of the pavement, this may lead to:

a) Detrimental materials such as water and incompressible materials entering the pavement and subgrade, causing damage and failure;

b) Underlying pavement layers abrading or eroding, leading to the formation of depressions; and

c) The cracks wearing and widening, leading to further deterioration of pavement functionality and structural integrity.

6.29 Surface cracking on heavily trafficked roads is minimised by providing a minimum cover equivalent to 175 mm thickness of dense graded asphalt (see Section 3 (CL 3.5 to 3.18)).

6.30 For lightly to moderately trafficked roads the tolerable amount of surfacing cracking is greater than for heavily trafficked roads. In such cases a lower minimum cover may be appropriate, but is required to be at least equivalent to 100 mm of dense graded asphalt (see Section 3 (CL 3.5 to 3.18)).

Determining In-service Fatigue Characteristics from Laboratory Fatigue Measurements

6.31 This content in the Guide is new to the 2017 edition and has not been applied to a DPTI project at the time of this Supplement's publication. The first-time application to a project is likely to involve additional risk factors that require careful consideration and mitigation.

Determining In-service Fatigue Characteristics from Laboratory Measured Flexural Strength and Modulus

6.32 This content in the Guide is new to the 2017 edition and has not been applied to a DPTI project at the time of this Supplement's publication. The first-time application to a project is likely to involve additional risk factors that require careful consideration and mitigation.

Asphalt

Introduction

6.33 The requirements for asphalt supply and construction are given in RD-BP-S2 "Supply of Asphalt" and RD-BP-C3 "Construction of Asphalt Pavement".

6.34 A guide to the selection of asphalt for particular applications is provided in Chapter 3 of this Supplement.

6.35 There has been substantial effort put into developing innovative asphalt mixes in recent years, including technology like EME2, A5E and perpetual pavement concepts. These technologies are at various stages of adoption in South Australia, with further advice available from DPTI. The design parameters provided for A5E mix are provided for information only, to allow comparative designs versus other mixes to be undertaken. Acceptance of use of a higher design modulus (greater than 1.0) for A5E binder must have prior approval from DPTI.

Determination of Design Modulus from Direct Measurement of Flexural Modulus

6.36 This approach is considered the best method of determining the flexural modulus of asphalt. However, the required volume and cost of laboratory testing required to develop the master curves for each individual mix, by each different asphalt supplier is relatively onerous and has not been undertaken, so far, in South Australia. The first time use of this method on a project is likely to involve additional risk factors that require careful consideration and mitigation, including development and validation testing and analysis.

6.37 The maximum asphalt design moduli obtained from this method shall not exceed the values in Table RD-PV-D1 6-3 for the given mix type, temperature and design speed, unless approved by DPTI.

Determination of Design Modulus from Measurement of ITT modulus

6.38 This test data is readily available, however it has not been used to determine a design modulus on a DPTI project to date. The first-time application on a project is likely to involve additional risk factors that require careful consideration and mitigation, including development and validation testing and analysis.

6.39 As noted in the Guide, the results of this test have significant scatter and "designers are advised not to assign a high level of accuracy or precision to a design modulus determined from the mean of a single set of triplicate specimens". An appropriate, project or volume based statistical relationship is not provided in the Guide. Use of this method will therefore require development of laboratory to field shift factors, with an acceptable supporting technical basis.

6.40 The maximum asphalt design moduli shall not exceed the values in Table RD-PV-D1 6-3 for the given mix type, temperature and design speed.

Means of Determining Asphalt Fatigue Characteristics

6.41 The use of laboratory based fatigue data in place of the Shell Relationship (Guide Equation 25) has not occurred or been proposed on a DPTI project at this time (or any other Australian State Road Authority project also, to the best of DPTI's knowledge.).

6.42 This approach is considered viable, subject to provision of appropriate levels of test data and development of an appropriate correlation between laboratory and field performance. The first-time application of this approach on a project is likely to involve additional risk factors that require careful consideration and mitigation, along with substantial testing and analysis.

DPTI Asphalt Design Moduli*

6.43 The asphalt design moduli for DPTI Registered Mixes shall be not less than 480 MPa for open graded, 830 MPa for stone mastic, and 1000 MPa for dense mix asphalt, and be determined from Table RD-PV-D1 6-3 and the following methodology.

6.44 The moduli provided in Table RD-PV-D1 6-3 are for the Weighted Mean Annual Pavement Temperature (WMAPT) of 27°C for Adelaide. For other pavement temperatures, the asphalt design modulus shall be the greater of the minimum indicated above, and that calculated from the values in Table RD-PV-D1 6-3 using the following equations (Jameson 2005b):

$$S_{WMAPT} = S_{27^{\circ}C} * e^{(A(WMAPT-27))} \quad (6.12)$$

$$A = (1 + 0.0307*(Vb - 11))*(0.014\ln(V) - 0.1579) \quad (6.13)$$

where S_{WMAPT} = asphalt modulus at the WMAPT (MPa)
$S_{27^{\circ}C}$ = asphalt modulus at 27°C (Table RD-PV-D1 6-3) (MPa)
Vb = volume of binder in mix (%)
V = heavy vehicle design speed (Table RD-PV-D1 6-4)
A = mix type and vehicle speed variable

6.45 WMAPT for various South Australian sites are given in Appendix B of the Guide.

Table RD-PV-D1 6-3 Asphalt Design Moduli at WMAPT Of 27°C

Mix Designation	Mix Size (mm)	Binder Class	Effective Binder Volume (%)	Design Modulus MPa at 27°C			
				10km/h	30km/h	50km/h	80km/h
AC10M	10	320	12.4	1220	2120	2630	3190
AC14M	14	320	11.7	1360	2330	2880	3480
AC20M	20	320	11.0	1520	2570	3160	3800
AC14HB	14	320	13.1	1330	2330	2910	3540
SMA10	10	320	15.1	830	1490	1880	2320
OG14	14	320	9.9	480	790	960	1150

6.46 As asphalt moduli vary with heavy vehicle speed, a design speed needs to be determined for each project. In the absence of site-specific heavy vehicle travel speed data, the presumptive design speeds given in Table RD-PV-D1 6-4 shall be used.

Table RD-PV-D1 6-4 Presumptive Heavy Vehicle Design Speeds

Project Location	Design Speed V (km/h)	
	Flat and up to 5% Grade	Over 5% Grade
Posted speed limit ≥ 80 km/h	80	50
50 km/h ≤ posted speed limit < 80 km/h	50	30
Posted speed limit < 50 km/h	30	10
Low radius roundabouts, bus stops, etc.	10	10

6.47 The modulus of an asphalt wearing course incorporating polymer modified binders may be estimated by applying the modulus adjustment factors in Table RD-PV-D1 6-5 below to the values for C320 binder mixes given in Table RD-PV-D1 6-3. These factors are the same as provided in Table 6.13 in the Guide, with additional local mixes added.

Table RD-PV-D1 6-5 Factors to Estimate the Modulus of Polymer Modified Binder Asphalts for DPTI Modulus at WMAPT 27 C (Table RD-PV-D1 6-3)

Austroads Binder Grade	Description of Binder	Softening Point – Approximately (C)	Modulus Adjustment Factor	Binder Availability
A10E	Styrene-Butadiene-Styrene (SBS) 6%	95	0.70	Not readily available in SA
A15E	Styrene-Butadiene-Styrene (SBS) 5%	90	0.75	Available.
DPTI Binder Grades				
A5E	Styrene-Vinyl- Styrene (SVS) Min 7.5%	95	1.6	Available. DPTI approval required for use.

6.48 The adjustment factor to estimate the modulus of High Flexural-Modulus Asphalt using the A5E binder shall be 1.6 times the values for C320 binder given in Table RD-PV-D1 6-3. Modulus adjustment factors exceeding unity for other PMBs shall not be adopted unless confirmed by laboratory resilient modulus testing of the project mix.

6.49 Design of Heavy Duty Wearing Courses and Modified Intermediate Courses utilising A5E shall be designed to 80 gyratory cycles.

Recycled Asphalt Pavement (RAP)

6.50 DPTI supports increasing the use of Recycled Asphaltic Pavement (RAP) within its asphalt mixes, as per RD-BP-S2 and RD-BP-C3.

6.51 Asphalt mixes with RAP are assumed to have the same design modulus and fatigue relationship as the equivalent mix with virgin materials.

7 Design Traffic

Procedure for Determining Total Heavy Vehicle Axle Groups

Selection of Design Period

7.1 The design periods given in Table RD-PV-D1 7-1 are required for new DPTI pavements.

Table RD-PV-D1 7-1 Pavement Design Periods for New Pavements

Road Classification	Design Period – Flexible Pavements (Years)	Design Period – Rigid Pavements (Years)
Motorway – Main Alignment and Ramps	40	40
Urban & Rural Arterial, Urban & Rural Connector	30	40
Access	20	Project Specific – Assessment Required

Initial Daily Heavy Vehicles in the Design Lane

7.2 DPTI has 8 Weigh-In-Motion (WIM) sites in operation in South Australia. These are located on major freeways and interstate freight routes and measure the axle group counts and mass in both directions of travel. Detailed Traffic load distributions are developed from this data and include NHVAG, ESA/HVAG, ESA/HV and Percent Heavy Vehicles for each site, as presented in Appendix 1: Traffic Load Distributions.

7.3 The following additional DPTI or project specific traffic data may also be available:

a) Urban and Rural Traffic Estimate Maps (publicly available at https://dpti.sa.gov.au/traffic_volumes). These provide the two way AADT and heavy vehicle estimates for the entire DPTI road network, with both urban and rural map sets available.

b) Intersection turning count surveys. These provide detailed turning count movements at intersections, with AADT and heavy vehicle counts provided for through and all turning movements at the intersection. These may be requested for relevant DPTI projects.

c) Mid-block Classification Counts. These are available for many DPTI managed roads, particularly rural roads. They are standard site classification counts, providing counts of each Austroads vehicle class and a total AADT and percent heavy vehicles. These may be requested where available for related DPTI projects.

d) Traffic Modelling and Forecasts. These traffic volumes are typically developed as part of network planning studies and for major road projects, with projected AADT and heavy vehicle counts developed at different times after opening of new or upgraded major roads. These can often give markedly different results depending on different scenario parameters and should be used with due consideration of their assumptions and limitations. The Austroads Guide to Traffic Management series provides further information on this.

7.4 Ensuring that the design traffic loading is developed from sufficient traffic data is a key pavement design activity, given the major influence traffic has on the required pavement type and configuration.

Short-term Heavy Loadings

7.5 The Guide design procedures have been developed over many years using mechanistic modelling and in-service field performance data. These pavements were generally designed and constructed to outlast 20 years or more trafficking, with the loading spread relatively evenly over the design period.

7.6 However, in some situations the pattern of loading differs markedly for that on which the procedures in the Guide were based. For instance:

a) Temporary pavements may be required to carry high daily traffic loadings but because of their limited operational life (e.g. 0-2 years) may have a relatively low design traffic loading if a similar 1 to 2 year design period is adopted.

b) In some areas of the State the haulage of grain results in large seasonal variations in traffic loadings.

c) Some bus ways in Adelaide have high loadings on weekends during the winter football season but low loadings at other times.

7.7 In such situations, the design traffic needs to be adjusted to allow for the greater impacts of these special loadings. For roads with intermittent or seasonal loadings, rather than the design traffic being calculated from the annual average daily heavy vehicle volume, the maximum daily heavy vehicle traffic per annum is used.

7.8 For temporary pavements with an operational period of less than 5 years, the design traffic is typically calculated using a 5 to 10 year design period with zero traffic growth rate using the maximum daily heavy vehicle volume. The actual design period adopted must consider the following risk factors and acceptable performance outcomes, as well as any other relevant factors:

a) The consequences of failure during the temporary pavements use, which may include safety risks, reputational damage and contractual penalties if maintenance works need to be undertaken to provide a serviceable pavement;

b) The intensity of traffic loading immediately upon opening to traffic;

c) The temporary pavement operating speed;

d) The inspection regime when in service.

Cumulative Number of Heavy Vehicles when Below Capacity

7.9 The following minimum heavy vehicle growth rates shall be adopted in consideration of predictions of road freight growth rates from 2008 to 2030 (BITRE 2010):

a) Motorway and Interstate Freight Routes: 3.9 %

b) Arterial and Connector: 3.0 %

7.10 Different values to these presumptive values may be appropriate, based upon a review of project specific data, and subject to approval by DPTI.

Cumulative Heavy Vehicle Axle Groups

7.11 Whenever possible, project specific weigh-in-motion (WIM) data should be used to determine NHVAG values rather than presumptive values.

7.12 Appendix E of the Guide provides the 2014 NHVAG values from all WIM sites throughout Australia. Additional 2016 DPTI WIM data that became available since the Guide was drafted is presented in this Supplement, and should be used in place of the 2014 data. This data is summarised in Table RD-PV-D1 7-4, with detailed data available for download on the DPTI website.

Estimation of Traffic Load Distribution (TLD)

7.13 South Australian WIM data has been filtered to remove axle group loadings that exceed the limits shown in Table RD-PV-D1 7-2. These values were determined from discussions with DPTI transport safety compliance officers and consideration of special heavy vehicle types and movements in SA. The processed WIM data is expected to provide the most reliable traffic load distributions for general pavement design purposes.

Table RD-PV-D1 7-2 Maximum South Australian WIM Axle Group Loadings

SAST	TAST	SADT	TADT	TRDT	QADT
9t	16t	18t	33t	40t	48t

Design Traffic for Flexible Pavements

Pavement Damage in Terms of Equivalent Standard Axle Repetitions

7.14 Whenever possible, project specific weigh-in-motion data should be used to determine relevant heavy vehicle parameters rather than presumptive values.

7.15 Appendix E of the Guide provides the 2014 heavy vehicle characteristics from weigh-in-motion sites throughout Australia, including the 8 sites in South Australia. More recent 2016 DPTI data has become available since the Guide was drafted, as per Table RD-PV-D1 7-4. The corresponding traffic load distributions are available for download on the DPTI website: https://www.dpti.sa.gov.au/standards.

7.16 For road projects where there are no suitable WIM data available, presumptive values for pavement design purposes are provided in Table RD-PV-D1 7-3 which relate to DPTI Road Categories and the largest Austroad Vehicle Class that uses the road. These must be used with careful project specific consideration of their suitability and sensitivity testing of pavement designs outputs with adopted TLD.

7.17 The presumptive Urban General Access TLD is applicable for moderately trafficked urban arterial and urban connector roads meeting these criteria:

a) Maximum Austroads Vehicle Class 9;

b) The percentage of Long Articulated Vehicles (Class 6 to 9) vehicles is less than or equal to 35% of the total heavy vehicle count.

Table RD-PV-D1 7-3 Presumptive Heavy Vehicle Characteristics for DPTI Road

	Urban Roads		Rural Roads			
Largest Austroads Vehicle Class allowed on Road(1)	Class 9	Class 11	Class 9	Class 10	Class 11	Class 12
Design TLD	Presumptive UGA	Wingfield - WI1	Bordertown - BTW	Bordertown - BTW	Oodla Wirra - OWS	Pimba - PIN

(1) Stage Government Regulations set the largest allowable vehicle on a particular route, as shown on the RAVnet online map system, refer https://www.dpti.sa.gov.au/ravnet and http://maps.sa.gov.au/ravnet/index.html.

Table RD-PV-D1 7-4 Traffic Characteristics of South Australian WIM Sites (2016)*

Road	ID	Location	Direction	Lane	NHVAG	ESA/HVAG	ESA/HV	Percent HVs
Barrier Highway	OWN	Oodla Wirra	N	OL	3.31	1.25	4.16	27.6
Barrier Highway	OWS	Oodla Wirra	S	OL	3.50	1.39	4.88	27.5
Dukes Highway	BTE	Bordertown	E	OL	3.47	1.13	3.53	27.0
Dukes Highway	BTW	Bordertown	W	OL	3.19	1.40	4.46	28.8
Eyre Highway	IKE	Iron Knob	E	OL	3.28	1.29	4.23	27.3
Eyre Highway	IKW	Iron Knob	W	OL	3.93	1.18	3.89	29.7
South East Highway	MOE	Monarto	E	OL	2.71	1.18	3.19	15.8
South East Highway	MOW	Monarto	W	OL	3.18	1.17	3.71	12.8
Stuart Highway	PIN	Pimba	N	OL	4.01	1.88	7.55	20.1
Stuart Highway	PIS	Pimba	S	OL	3.96	1.22	4.84	21.2
Sturt Highway	DUE	Dutton	E	OL	3.20	1.28	4.10	23.9
Sturt Highway	DUW	Dutton	W	OL	3.23	1.22	3.93	23.1
Riddoch Highway*	NAN	Naracoorte	N	OL	3.17	1.40	4.44	23.1
Riddoch Highway*	NAS	Naracoorte	S	OL	3.21	1.09	3.51	23.1
Port River Expressway	WI1	Wingfield	E	OL	2.42	1.14	2.75	14.0
Port River Expressway	WI2	Wingfield	E	IL	2.05	0.95	1.96	2.5
Port River Expressway	WI3	Wingfield	W	IL	2.46	1.03	2.52	4.8
Port River Expressway	WI4	Wingfield	W	OL	2.26	1.03	2.34	12.5

*2016 data was not available for the Riddoch Highway site, 2014 data has been used.

<u>Design Traffic for Mechanistic-empirical Design Procedure</u>

7.18 DPTI accepts use of the upper limits on design traffic for asphalt fatigue, which is a design traffic loading of 2 x 10^8 ESA for the Adelaide urban region (WMAPT of 27C).

7.19 Consideration should be given to the WOLC benefits of constructing the thickness corresponding to 2 x 10^8 ESA, given it is unlikely to experience bottom up fatigue cracking requiring full depth structural rehabilitation treatments.

8 Design of New Flexible Pavements

Mechanistic-empirical Procedure

8.1 The calculated layer thicknesses shall be rounded up to the nearest 5 mm. To allow for variations in the constructed layer thicknesses within the construction tolerances, 10 mm shall be added to the pavement layer which governs the overall allowable loading.

8.2 It is assumed that sprayed seals, geotextiles and SAMIs are non-structural. Geosynthetics that reinforce pavement layers or have load spreading properties are also excluded from the mechanistic modelling procedures.

8.3 Generally, the mechanistic modelling is undertaken assuming full bonding between layers, characterised as a “rough” interface in the CIRCLY program. However, when modelling pavements with more than two cemented material layers, only one of the interfaces between cemented material layers shall be modelled as fully bonded, and any other interfaces between cemented layers shall be modelled as a smooth interface.

8.4 Similarly, if the construction and maintenance procedures are likely to result in some degree of debonding during the design period, the interfaces between cemented material layers shall be modelled as smooth.

<u>Selection of Trial Pavement</u>

8.5 Refer to discussion in Section 2.

Procedure for Elastic Characterisation of Selected Subgrade and Lime-stabilised Materials

8.6 The maximum vertical modulus assigned to top sublayers shall not exceed 100 MPa, except:

a) For fully investigated and designed lime stabilised subgrades; and

b) Sound rock formations which have a maximum modulus of 150 MPa (refer Section 5 (CL 5.17 to 5.23)).

Procedure for Elastic Characterisation of Granular Material

8.7 Class 1, 2 and 3 pavement materials meeting the requirements of RD-PV-S1 “Supply of Pavement Materials” shall be considered as unbound granular materials and characterised in accordance with the Guide.

Consideration of post-cracking phase in Cemented Material and Lean Mix Concrete

8.8 For heavy-duty pavements incorporating cemented materials no allowance shall be made for the post-cracking phase of design life. For other road pavements where the design traffic is less than 10^7 ESA, designs may include the post-cracking phase of cemented materials if agreed by the project manager.

Design of Granular Pavements with Thin Bituminous Surfacings

8.9 For unbound granular pavements with asphalt surfacings less than or equal to 80 mm thick, the base and subbase requirements discussed in Section 8 (CL 8.13 to 8.17) below shall apply.

8.10 Where the total thickness of asphalt is 85 mm or more, the following minimum requirements apply:

8.11 Total asphalt thicknesses 85 to 150 mm:

a) An unbound granular base is required consisting of a minimum 125 mm of Class 1 Quarried Pavement Material or Class 1 Recycled Pavement Material;

b) An upper subbase consisting of a minimum 125 mm thickness of Class 2 Quarried Pavement Material or Class 2 Recycled Pavement Material shall be provided; and

8.12 Total asphalt thicknesses exceeding 150 mm:

a) An upper subbase consisting of a minimum 125 mm thickness of Class 2 Quarried Pavement Material or Class 2 Recycled Pavement Material shall be provided.

Empirical Design of Granular Pavements with Thin Bituminous Surfacings

Pavement Composition

8.13 As the traffic loading increases, the qualities and thicknesses of the base and subbase materials also needs to increase. Better quality materials are used in the upper levels of the pavement while a wider range of test property limits can be permitted for subbase.

8.14 Material properties and layer thicknesses for base and subbase layers are discussed below:

8.15 Base:

a) Class 1 Quarried Pavement Material or Class 1 Recycled Pavement Material, as specified in RD-PV-S1 "Supply of Pavement Materials” shall be used as base, with minimum thicknesses as follows:

b) For design traffic of up to 10^6 ESA, a minimum thickness of 150 mm;

c) For design traffic of 10^6 to 10^7 ESA, a minimum thickness of 250 mm; and

d) For design traffic exceeding 10^7 ESA, a minimum thickness of 300 mm.

8.16 Subbase:

a) A minimum 125 mm thickness of Class 2 Quarried Pavement Material or Class 2 Recycled Pavement Material shall be provided. In accordance with Table RD-PV-D1 6-1, lower subbase layers shall also comprise Class 2 Quarried Pavement Material, Class 2 Recycled Pavement

Material, Class 3 Quarried Pavement Material, or Class 3 Recycled Pavement Material, or other approved granular materials of similar strength.

8.17 DPTI accepts reduction of the subbase layer thickness by the thickness of a thin asphalt surfacing designed with Figure 8.4, as stated in the Guide (2nd para, pg 123).

Geogrid reinforcement

8.18 Where appropriate, geogrids are provided at the interface of the subbase and subgrade. Reinforcement can be achieved from any one, or combination of the following mechanisms (Perkins et. al., 1998):

a) Resistance to lateral spreading of the subbase aggregate as vertical loads are applied at the pavement surface.

b) Increased confinement afforded to the subbase causing an increase in the state of stress in that layer and correspondingly an increase in the modulus of the subbase (and base) layers.

c) Improved distribution of stress to the subgrade which generally results in the subgrade achieving a higher modulus.

d) Reduced shear stresses being transferred to the subgrade resulting in lower vertical strains being mobilised in the subgrade.

8.19 To date DPTI has primarily used geogrids on soft subgrades as a construction expedient without any reduction in pavement thickness or strength requirements. This low risk approach reflects the lack of local performance studies, although there is some overseas experimental evidence that supports the use of a thickness reduction methodology for geogrids. As with other developing treatments, the DPTI usual practice would be to incorporate trial sections of limited extent within new construction works (subject to project and asset manager approval). These would assist the assessment of placement issues, economic viability, and field performance prior to widespread adoption of a new design approach.

Shoulders

8.20 For design of sealed shoulders adjoining new pavement, a design traffic value for the shoulders of 2-5% of the adjacent trafficked lane pavement should be adopted. Where the sealed shoulder is likely to be frequently trafficked, 100% of the adjacent lane design traffic should be adopted.

8.21 The designer must also consider the long-term plans for the road in selecting the design traffic loading, particularly where full lane width sealed shoulders may become fully trafficked lanes in future arrangements.

Sprayed seal considerations

8.22 Table RD-PV-D1 3-2 should be used as a guide for the selection of initial spray seal treatments on granular pavements. Unless otherwise specified sprayed seal treatments shall be designed in accordance with Austroads Seal Design Procedures and the following exceptions and additions:

8.23 Sealing aggregate shall comply with RD-PV-S1 “Supply of Pavement Materials”.

8.24 The size of aggregate selected will vary according to the expected volume and composition of traffic. In general, a 14/7 double seal is appropriate as an initial surfacing treatment on a granular pavement. However, where traffic volumes exceed 2000 vehicles/lane/day or the percentage of heavy vehicles exceeds 15%, a 16/7 double seal should be considered.

8.25 Primed surfaces on granular pavements shall be cured for a minimum of 3 days prior to applying the final surfacing. Cement treated bases shall be primed with a very light prime at a rate of 0.4 - 0.6 l/m2 depending on the surface finish of the pavement.

8.26 For seal treatments on granular pavements, aggregate embedment can have a significant effect on binder application rates, and hence needs to be measured prior to final seal design. Table RD-PV-D1 8-1 provides general maximum limits for average ball penetration values at varying traffic volumes.

Table RD-PV-D1 8-1 Suggested Maximum Average Ball Penetration Values

Traffic Volumes (AADT/lane/day)	Ball Penetration (mm)
AADT/lane/day ≤ 1500	3.5
1500 < AADT/lane/day ≤ 2500	3.0
2500 < AADT/lane/day ≤ 3000	2.5
AADT/lane/day >3000	2.0

Note: The penetration values are a guide only, determination of an appropriate hardness value will depend on several factors including traffic composition, gradient and curve radii.

8.27 Where high traffic volumes require the use of low binder application rates, geotextile reinforced seals may be utilised to minimise the degree of aggregate embedment. However, this treatment should not be applied to areas subject to high shear forces such as intersections, tight corners and steep climbing lanes.

8.28 It is advisable to leave cutback primer seals exposed for 6 – 12 months. It is also recommended that cutback bitumen primer seals be trafficked for 3 months between October and March before placing a sprayed seal or asphalt surfacing less than 100 mm thick (refer Table RD-PV-D1 3-2).

8.29 Where it is not feasible to comply with the above time constraints an emulsion primer binder may be used, in which case the final surfacing can be applied after the emulsion primer seal has been subjected to several weeks of trafficking in hot weather.

8.30 Priming prior to spray sealing is mandatory.

Documentation of Pavement Design Calculations

8.31 Flexible pavement designs shall be supported by documented design calculations and methodology, including:

a) The subgrade and if relevant, selected subgrade material test results and procedures used to obtain design CBR values and hence layer moduli;

b) The processes used to estimate the moduli for each pavement material;

c) The design traffic calculations;

d) The DPTI CIRWIN Summary Report or Mincad CIRCLY Job Summary File, with layer thicknesses, elastic properties and CIRCLY critical strains;

e) The performance relationships used to estimate allowable loadings;

f) The allowable loadings for each distress mode; and

g) Adjustment of the governing layer thickness.

8.32 This is considered the minimum level of documentation required on a DPTI project, including works by others that impact DPTI road assets. Further information may be necessary on larger, more complicated designs.

Independent Design Review

8.33 Independent review of calculations and design must be undertaken and presented in the pavement design report.

9 Design of New Rigid Pavements

9.1 The update of Section 9 in this 2018 edition of the Supplement has been limited to updating the reference documents. The detailed technical content of this Section will be revised in a future update of the Supplement.

General

9.2 The Roads and Maritime Services (RMS) NSW is the pre-eminent Australian road authority in rigid pavement technology, having designed, constructed and maintained the majority of Australian concrete pavements over the last 40 years. Given the extremely limited use of concrete roads in South Australia, DPTI relies heavily on RMS's recommended practices for the design of concrete pavements. Consequently, the following additional advice to the Guide is mainly based on the RMS Supplement to the Austroads Guide to the Structural Design of Road Pavements (2015). This advice may be extended or superseded by RMS Supplement updates.

9.3 The Austroads design procedure assumes that the base and subbase will separate under traffic and environmental loading. Debonding procedures vary markedly according to both the type of subbase and the type of base (refer Table 9.10 of Guide) and are detailed in RMS specifications. Local climatic conditions should also be taken into account. Appropriate debonding can be critical to the performance of the pavement.

9.4 Further information on materials, design and construction aspects are available in the RMS QA Roadworks Specifications (for concrete pavements), RMS Guides to QA Specifications, and the RMS Standard Drawings - Pavement.

9.5 The Guide does not offer any guidance to the impact on settlement on pavement thickness and the designer should refer to RMS Guide for design of concrete pavements in areas of settlement.

Pavement Types

9.6 Refer to discussion in Section 2.2.

Roundabout Pavements

9.7 The design of roundabout pavements is a special case because the radii and vehicle speeds are substantially different to those normally encountered on the road network. This combination of radius and speed includes high centripetal forces and results in high outer wheel loads from commercial vehicles.

9.8 Special rules therefore apply to the design of roundabout pavements, whether designed in steel-fibre, mesh reinforced or plain concrete.

9.9 The following advice applies principally to low speed urban roundabouts but the concepts should also be considered in designing rural roundabouts, and possibly even low radius curves.

9.10 Because of the typically radial pattern of jointing in a roundabout (and the resulting odd-shaped slabs) it is difficult to correctly align dowels and they are therefore usually avoided in this application.

9.11 The thickness design for steel fibre concrete pavements (SFCP), being an undowelled pavement, would normally be controlled by erosion and the thickness usually would therefore be the same as that for a plain undowelled pavement. However, the geometry of roundabouts usually results in relatively low traffic speeds. Joint distress is unlikely to be controlling factor in the pavement life. For heavy vehicle speeds below about 30 km/h, the thickness design is therefore carried out considering fatigue only.

9.12 The higher flexural strength of SFCP results in a significantly reduced stress ratio factor and lower percentage fatigue for a given pavement thickness.

9.13 Concrete Roundabout Pavements – A Guide to their Design and Construction (RTA 2004) provides additional design advice.

Wearing Surface

9.14 Where an open graded asphalt wearing surface is required over continuously reinforced concrete pavements (CRCP), the minimum asphalt thickness is 60 mm, consisting of 30 mm of dense graded asphalt and a minimum 30 mm of open graded asphalt to allow future milling and resurfacing.

9.15 Thin asphalt wearing surfaces over a plain concrete base should be discouraged as the reflective cracking in these thin layers is difficult to maintain, even with a pre-treatment over the transverse contraction joint.

Factors used in Thickness Determination

Base Concrete Strength

9.16 For design traffic less than 10^6 HVAG a minimum 28-day characteristic compressive strength of 28 MPa is required to ensure adequate pavement surface durability. This equates to about 4 MPa concrete design flexural strength.

9.17 Steel fibre reinforced concrete pavement roundabouts are typically designed to a minimum 28-day flexural strength of 5.5 MPa.

Concrete Shoulders

9.18 The Guide recognises the structural contribution of an integrally cast channel gutter or kerb and gutter. DPTI recommends that concrete shoulders always be incorporated in the design of rigid pavements.

9.19 A separately placed channel comprised of structural grade concrete may also provide edge support to the pavement, but to a lesser extent, and hence does not warrant a reduction in pavement thickness. Wherever heavy vehicles are likely to travel along this edge a "no shoulder" design condition should be adopted.

9.20 If kerbing cannot be constructed integrally, special effort is warranted to maximise the contribution of a tied kerb and gutter by:

a) Specifying a slipform kerb (in contrast to an extruded one); and

b) Providing tie bars with adequate pull-out embedment to ensure the maintenance of load transfer by aggregate interlock.

Base Thickness Design

General

9.21 To allow for variations in the constructed layer thicknesses within the construction tolerances, 10 mm shall be added to the design base thickness and another 10 mm to allow for future low-noise diamond grinding.

Example Design Charts

9.22 These charts may be used in establishing a trial thickness for a given subgrade CBR and design traffic to commence the base thickness design. They are based on selected design factors and their relevance to a particular project may be limited.

Reinforcement Design Procedures

Reinforcement in Plain Concrete Pavements

9.23 Jointed reinforced concrete slabs are usually 8 to 15 metres long, but lengths in the range 10 – 12 m are recommended on the basis of economy and pavement performance. In addition, slabs longer than about 12 m are likely to provide noticeably lower ride quality because of the necessarily wider transverse joints.

9.24 In steel fibre concrete pavements, slab lengths should be limited to 6 m in the case of undowelled joints and 10 m for dowelled joints.

Reinforcement in Continuously Reinforced Concrete Pavements

9.25 The proportion of longitudinal reinforcing steel (p) in a cross section, or steel ratio, is initially determined using Equation 31 in the Guide. For example, in a typical Continuously Reinforced Concrete pavement using Y16 bars, assuming a crack width of 0.3 mm and a total shrinkage and temperature strain of 500 microstrain, the steel ratio is 0.67%. This is the minimum value of p.

9.26 The critical value of the proportion of reinforcing steel pcrit is determined by Equation 33. It is in inverse proportion to the yield strength of the steel and directly proportional to flexural strength of the concrete.

9.27 When flexural strengths are available from trial mixing of concrete to be supplied to the works, the proportion of reinforcing steel specified in the design should be checked using Equation 33. If the strength of the concrete is higher than anticipated the proportion of steel reinforcement will need to be increased or the base thickness reduced, provided the thickness design criteria are met.

9.28 Equation 34 of the Guide has been provided to determine the expected theoretical spacings of cracks. It is provided as a guide and is not to be used for design purposes under normal Australian climatic conditions to determine or adjust the required reinforcement.

Joint Types and Design

Joint Design

9.29 The design and layout of all pavement joints shall be carried out in accordance with RMS Standard Drawings - Pavement.

Documentation of Pavement Design Calculations*

9.30 Rigid pavement designs shall be supported by documented design calculations and methodology, including:

a) Concrete pavement type, subbase type and whether or not concrete shoulders are to be provided;

b) Subgrade and if relevant, selected subgrade material test results and procedures used to obtain design CBR values and equivalent subgrade strength using Equation 25 of the Guide;

c) Design traffic calculations, including expected load repetitions for each axle group load of each axle group type, in a format similar to Table L.1 of Appendix L the Guide;

d) Calculations of allowable load repetitions and percentage damage for each axle group of each axle group type in a format similar to Table L.2 of Appendix L of the Guide;

e) Adjustment of the calculated base layer thickness for construction tolerance; and

f) The specified base thickness taking into account the minimum thicknesses in Table 9.7 of the Guide.

9.31 For reinforced pavements, reinforcement design calculations in accordance with Section 9 (CL 9.23 to 9.28) of the Guide shall also be provided.

9.32 The following project-specific drawings shall be provided:

a) Joint layout plans for the project;

b) Cross-sections including traffic lane widths, shoulder widths, the location of longitudinal joints in base and subbase, sub-surface drainage;

c) Detailed drawings of each joint type (e.g. transverse contraction, construction, longitudinal) and special features (e.g. slab anchors, terminals at bridge structures; reinforcement, kerb and channel joints, junctions of concrete and flexible pavements).

10 Comparison of Designs

10.1 The validity of the economic comparisons invariably depends on the accuracy of the numerous assumptions and performance predictions that need to be made within each pavement whole-of-life costing model. For real pavements, the field performance can vary significantly between projects and differ from the typical or base expectations. Hence, it will often be necessary to consider the economic comparisons for the scenarios where rehabilitation and maintenance requirements are consistently either more or less than the average case. A comprehensive analysis would include economic comparisons of pessimistic, base and optimistic performance predictions and their associated maintenance costs over the analysis period.

Method for Economic Comparison

10.2 The Present Worth of Costs (PWOC) method shall be used to calculate the Whole-of-Life costs.

Real Discount Rate

10.3 A discount rate of 7% shall be used with sensitivity testing at 4% and 10%.

Road User Costs

10.4 Consideration must be given to the safety and service of road users and others who may be affected by the presence of the asset. Some issues to consider are:

a) Disruption caused by frequency of maintenance activity.

b) Roughness impacts on the cost of operating vehicles.

c) For strategic routes, implications of damage/disruption due to (perhaps low probability) catastrophic events (e.g. floods, earthquakes), subsidence, expansive clay subgrade etc.

d) Traffic noise from particular surfaces.

e) Environmental issues

f) During construction and maintenance

g) e.g. potential for dust, material disintegration or ravelling, fumes or contamination to the environment from certain road materials

h) Aesthetic or visual intrusion effects.

i) Traffic and pedestrian safety which may be affected by:

j) Surfacings (texture, colour/visibility, etc.)

k) Susceptibility of the pavement type to damage (e.g. rutting, cracking, ravelling)

l) Practically of adopting a different pavement type on a road length which is dominantly of another pavement type.

10.5 Although cost is a prime consideration in the selection of options, if any of the above non-measurable factors are considered important for the project under consideration, judgement will have to be used and the most economical solution may not be the most appropriate implementation of Design and Collection of Feedback

11 Implementation of Design and Collection of Feedback

Implementation of Design

Pavement Work Schedule

11.1 The DPTI Master Specification for Roadworks requires the Contractor to undertake pavement works in accordance with the Pavement Work Schedule for the project. Table RD-PV-D1 11-2, Table RD-PV-D1 11-3 and Table RD-PV-D1 11-4 are examples of these Schedules.

11.2 The following guidance is included in this Supplement to assist pavement designers in preparing these Schedules.

Design Levels, Layer Design Thickness and Critical Layer

11.3 To allow for variations in the constructed layer thicknesses within the construction level tolerances, 10 mm shall be added to the pavement layer which governs the overall allowable loading (designated the critical layer).

11.4 Layer thickness shall conform to Section 3 (CL 3.19 to 3.21) and Table RD-PV-D1 3-2 of this Supplement.

Level Tolerances

11.5 Table RD-PV-D1 11-1 provides general guidance on level tolerances that are applicable in most situations. The designer must carefully consider the overall pavement structure and which layer is the critical layer, as well as constructability, quality management systems and other factors, for each specific configuration to ensure these are suitable.

11.6 Table RD-PV-D1 11-2 to Table RD-PV-D1 11-4 provide some example pavement schedules with these tolerances have been applied. A pavement work schedule should be provided for each unique pavement type, and match the pavement types shown on the drawings.

Table RD-PV-D1 11-1 Pavement Layer Level Tolerances

Material	Layer	Top of Layer Level Tolerance (mm)	Bottom of Layer Level Tolerance (mm)
Asphalt	Wearing Course – AC10 or SMA(1)	+5, -0 adjacent kerb & gutter +/-5 elsewhere	+5, -10
	Wearing Course – OGA(2)	+/- 5	+5, -0(2)
	Intermediate & base course layers, except Bottom Asphalt layer – AC10, AC14	+/- 10	+/- 10
	Bottom Asphalt Layer(3)	+/- 10	+0, -10 if critical layer(3) +/- 10 if not critical layer
Stabilised Materials (fully bound)	CSTSB, FBS	+/- 15	+0, -15 if critical layer(3) +/- 15 if not critical layer
Unbound Granular Materials	Top basecourse layer (PM1)	+/- 10(4)	+0, -40 if critical layer(3) +/- 15 if not critical layer
	Other basecourse layers and subbase layers	+/- 15	+0, -40(5) if bottom layer(3) +/- 15 if not bottom layer (i.e. for intermediate layers)

(1) Dense mix and Stone Mastic asphalt wearing course surface levels are generally specified to match or sit slightly proud of the adjacent kerb water table level, to allow surface water to run onto the water table. Where there is no kerb and gutter present, or other adjacent surface level to consider, a more relaxed surface tolerance can be adopted. See Table RD-PV-D1 11-2 for an example.
(2) Open Graded asphalt typically sits proud of the water table / shoulder surface level to allow water to drain laterally out of this porous mix. The surface level of the underlying AC levelling course is then specified to match the adjacent water table, shoulder, etc., surface level. See Table RD-PV-D1 11-4 for an example.
(3) The bottom of the critical layer governing the pavement design life is given a level tolerance of +0 mm to ensure that the as-constructed thickness achieves the thickness used in design calculations. This tolerance typically applies to the bottom asphalt layer with full depth asphalt pavements, the bottom CTSB layer in asphalt – CTSB composite pavements, and either the bottom asphalt layer or bottom CTSB layer in deep strength asphalt pavements.
(4) This tolerance would apply where a sprayed seal surfacing is to be applied.
(5) A tighter tolerance of -20 mm (instead of -40 mm) is typically adopted on motorway class pavements with low target roughness counts to reduce the potential for longitudinal subgrade level variations that can negatively affect the finished pavement ride quality.

Nominal Compacted Thicknesses

11.7 Refer to Section 3 (CL 3.19 to 3.21) of this Supplement for guidance on layer thicknesses.

11.8 Asphalt layers shall comply with compaction criteria specified in RD-BP-C3 “Construction of Asphalt Pavement” of the DPTI Master Specification for Roadworks.

11.9 Plant mixed cement stabilised layers shall comply with compaction criteria specified in RD-PV-S2 “Plant Mixed Stabilised Pavement” of the DPTI Master Specification for Roadworks. For heavy-duty designs, a minimum 98% Modified Compaction shall be specified.

11.10 Unbound granular base (PM1) layers shall be compacted uniformly to the full depth and over the full width to an Ls value of not less than 98%. (RD-PV-C1 “Earthworks”).

11.11 Unbound granular subbase (PM2) layers shall be compacted uniformly to the full depth and over the full width to Ls of not less than 96%.

11.12 Fill materials shall comply with the compaction criteria specified in RD-EW-C1 “Earthworks” of the DPTI Master Specification for Roadworks.

11.13 Further reference should be made to the DPTI Master Specification for Roadworks, for additional information and detail.

Moisture Content of Granular Materials

11.14 Unbound granular materials shall comply with the following moisture criteria prior to sealing or placement of an overlaying material:

a) PM1 materials shall not have a moisture content exceeding 60% of optimum moisture content; and

b) PM2 and PM3 materials shall not have a moisture content exceeding 70% of optimum moisture content.

Pavement Drawings*

11.15 Pavement treatment plans are required to show the extent of the different pavement types. For new works, these are typically a part of the overall project drawing set, which may also include the general construction, road geometry, traffic control layout, ITS, drainage, and other drawings.

11.16 Pavement treatment plans shall be prepared in accordance with DPTI Drawing Presentation Standard DP 009 Pavement Treatment, which is available on the Department’s website.

11.17 In addition, other drawings showing any other details required for appropriate construction of the pavements must be provided. This would include pavement joint details, typical cross-sections and other general construction notes.

Table RD-PV-D1 11-2 Example of Pavement Schedule for an Asphalt Cemented Composite Pavement

Design Level of Upper Surface of Courses in Relation to Finished Design Levels (mm)	Level Tolerance (mm)	Nominal Compacted Thickness	Layer	Material[5]	Application Rates and Additional Requirements to Master Specification – Division Roadworks
00	+5, -0 (k&g) ±5 elsewhere				
		40 mm	Wearing Course[1]	AC10M[3] A15E	
-40	+5, -10				
		60 mm	Levelling Course[1]	AC14M C320	
-100	±10				
		75 mm	Base Course[1]	AC14M C320	
-175	±15				
		5 mm	SAMI	10mm S25E Spray Seal[2]	Aggregate: SA10-7, Spread rate 120 m^2/m^3, Precoat: IDF++ @ 4 L/m^3 Binder: S25E @ 1.9 L/m^2, Adhesion add: 1 part.
			Prime	AMC00[2]	0.8 L/m^2
-180	±15				
		315 mm	Subbase	SPM2/20QGC4[4] (20 mm PM2/20 with 4% Cement)	96% Modified Compaction Placed in two layers & same day
-495	+0, -20				
			Subgrade		

KEY TO ABBREVIATIONS

(k&g) Kerb and gutter locations only.
L/m2 Litres per square metre.
IDF++ 100parts IDF, 30 parts C170, 1.5 parts approved adhesion additive

(1) Tack Coat (CRS60 @ 0.2 L/m2 residual) to be applied in accordance with RD-BP-C3 “Construction of Asphalt Pavement” Clause 6.4 or as directed by the Superintendent.

(2) Spray rates are nominal values only and may vary due to stone ALD, surface texture, weather conditions, etc. Rates are to be verified by the Superintendent prior to application.

(3) Heavy duty wearing course to be used for 50m approach to intersections (Refer Pavement Plans for locations).

(4) Recycled crushed concrete with 4.5% cement, i.e. SPM1/20RG, C4.5 may be used as an alternative.

(5) Asphalt mixes with standard binder types reflects the design viscosity recommended. However, for mixes including aged binder from RAP, softer binder may be required to achieve the target viscosity.

PAVEMENT TYPE A

Table RD-PV-D1 11-3 Example Pavement Schedule for a Spray Sealed Unbound Granular Pavement

Design Level of Upper Surface of Courses in Relation to Finished Design Levels (mm)	Level Tolerance (mm)	Nominal Compacted Thickness	Layer	Material	Application Rates and Additional Requirements to Master Specification – Division Roadworks
			Wearing Course	16/7 Double Sprayed Seal(1)	Aggregate:-16 mm to SA16-10, Spread rate 95 m^2/m^3, Precoat: IDF++ @ 4 L/m^3, Binder:- S20E @ 1.0 L/m^2 Aggregate:-7mm to SA7-5, Spread rate 180 m^2/m^3, Precoat: IDF++ @ 4 L/m^3, Binder:- S20E @ 0.8 L/m^2
00	±10		Prime	AMC0(1)	Application Rate @ 1.0 L/m^2
-150	±15	150 mm	Base Course 1	PM1/20 QG	98% Modified Compaction 60% OMC Dryback
-300	±15	150 mm	Base Course 2	PM1/20 QG	98% Modified Compaction
-450	+0, -40	150 mm	Subbase	PM2/20 QG or RG	96% Modified Compaction
			Subgrade		

KEY TO ABBREVIATIONS

L/m2 Litres per square metre.

IDF++ 100parts IDF, 30 parts C170, 1.5 parts approved adhesion additive

(1) Spray rates are nominal values only and may vary due to stone ALD, surface texture, weather conditions, etc. Rates are to be verified by the Superintendent prior to application.

PAVEMENT TYPE B

Table RD-PV-D1 11-4 Example of Pavement Schedule for a Full Depth Asphalt Pavement

Design Level of Upper Surface of Courses in Relation to Finished Design Levels (mm)	Level Tolerance (mm)	Nominal Compacted Thickness	Layer	Material[4]	Application Rates and Additional Requirements to Master Specification – Division Roadworks
45	+5				
		40 mm	Wearing Course[1]	OG14 A15E[3]	
00	+5, -0 (k&g)	5 mm	SAMI	10mm S25E Spray Seal[2][3]	Aggregate: SA10-7, Spread rate 120 m^2/m^3, Precoat: IDF++ @ 4 L/m^3 Binder: S25E @ 1.9 L/m^2, Adhesion add: 1 part.
-40	+5, -10 elsewhere ±10	40 mm	Levelling Course[1]	AC10M C320	
-105	±10	65 mm	Intermediate Course[1]	AC14M C320	
-170	±10	65 mm	Base Course[1]	AC14M C320	
		60 mm	Lower Base Course[1]	AC14HB C320	
-230	+0, -10		Prime	AMC0[2]	1.0 L/m2
-380	+0, -20	150 mm	Subbase	PM2/20	96% Modified Compaction
			Subgrade		

KEY TO ABBREVIATIONS

(k&g) Kerb and gutter locations only.
L/m2 Litres per square metre.
IDF++ 100parts IDF, 30 parts C170, 1.5 parts approved adhesion additive

(1) Tack Coat (CRS60 @ 0.2 L/m2 residual) to be applied in accordance with RD-BP-C3 “Construction of Asphalt Pavement” Clause 6.4 or as directed by the Superintendent.

(2) Spray rates are nominal values only and may vary due to stone ALD, surface texture, weather conditions, etc. Rates are to be verified by the Superintendent prior to application.

(3) Heavy duty wearing course mix to be used and SAMI to be omitted within intersections and for 50m approach / departure to intersections (Refer Pavement Treatment Plans for locations).

(4) Asphalt mixes with standard binder types reflects the design viscosity recommended. However, for mixes including aged binder from RAP, softer binder may be required to achieve the target viscosity.

PAVEMENT TYPE B

Field assessment of materials during construction*

11.18 As specified in RD-EW-C1 “Earthworks”, proof rolling is used to identify any unsuitable material (i.e. soft spots). When areas of weak materials occur, the dynamic cone penetrometer (DCP) may be used to determine their depth and lateral extent.

11.19 The Clegg Impact Soil Hammer (Clegg, 1980) commonly known as the Clegg Hammer is a field instrument for indirectly determining the in situ strength or stiffness of compacted materials. Queensland Transport Technical Note 5 Application for Clegg Impact Soil Tester provides target impact values that may be a useful guide in assessing the field strength and integrity of compacted unbound granular layers.

11.20 Measurement of the surface deflections on completed granular layers using the Falling Weight Deflectometer (FWD) may also provide a guide to the consistency and uniformity of the construction work at these stages. In addition, indications of the competency of the pavement during construction can be gained by considering the magnitude of the measured maximum deflections.

11.21 Ideally, the additional granular thickness required to reduce the measured maximum deflection to the design deflection should not exceed the total thickness of granular layers yet to be placed. Where this condition is not met, appropriate remedial action or design modifications can be implemented to ensure the required structural adequacy of the completed pavement.

12 References

a) Austroads (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Austroads, Sydney.

b) Austroads (2009). Guide to Pavement Technology Part 3: Pavement Surfacings. Austroads, Sydney.

c) Austroads (2007). Determination of permanent deformation and resilient modulus characteristics of unbound granular materials under drained conditions. AG:PT/T053, Austroads, Sydney.

d) Clegg (1980). An Impact Soil Tester as an Alternative to California Bearing Ratio. Proc. 3rd ANZ Geomechanics Conference, 1, pp225-230.

e) Bureau of Infrastructure, Transport and Regional Economics (BITRE), 2010, Road Freight Estimates and Forecasts in Australia: interstate, capital cities and rest of state, Report 121, Canberra ACT.

f) Jameson, G.W (2005a). Utilisation of Post-cracking Phase of Cemented Materials Life. ARRB Group Report VE71073 for Transport SA.

g) Jameson, G.W (2005b). Asphalt Modulus Variation with Temperature. ARRB Group Report VE71073-5 for Transport Services Division, DTEI.

h) Little, D.N (1995). Handbook for Stabilisation of Pavement Subgrades and Base Courses with Lime. (Kendall/Hunt).

i) Perkins, S.W., Ismeik, M., Fogelsong, M (1998). Mechanical Response of a Geosynethic Reinforced Pavement System to Cyclic Loading. Proc. 5th International Conference on the Bearing Capacity of Roads and Airfields, Trondheim, Norway, Vol. 3, pp 1503-1512.

j) Queensland Transport (1988). Applications for the Clegg Impact Soil Tester. Technical Note 5 (QDMR: Brisbane).

k) Roads and Maritime Services, NSW. Guides to QA Specifications (Sydney).

l) Roads and Traffic Authority, NSW (2004). Concrete Roundabout Pavements – A Guide to their Design and Construction (Sydney).

m) Roads and Maritime Services, NSW (2015). Supplement to the Austroads Guide to the Structural Design of Road Pavements.

n) Roads and Maritime Services, NSW . RMS Standard Drawings - Pavement (Sydney).

o) Robinson, P., Oppy, T., Giummarra, G. (1999). Pavement Materials in Road Building: Guidelines for Making Better use of Local Materials. ARRB Transport Research.

p) Standards Australia (1997). AS2008. Residual Bitumen for Pavements.

13 Appendix 1: Traffic Load Distributions

13.1 Traffic Load Distributions for 2016 are available for download on the DPTI website, at https://www.dpti.sa.gov.au/standards except for the Naracoorte Site, where only 2014 data was available.

13.2 It is noted that the Sturt Highway WIM moved to Dutton from its previous location at Truro.

13.3 In addition, a presumptive TLD has been provided for Urban General Access Roads with lower Class vehicles, as discussed in Supplement Section 7.6.

Table RD-PV-D1 13-1 Traffic Load Distributions

Road	ID	Location	Direction	Lane
Barrier Highway	OWN	Oodla Wirra	N	OL
Barrier Highway	OWS	Oodla Wirra	S	OL
Dukes Highway	BTE	Bordertown	E	OL
Dukes Highway	BTW	Bordertown	W	OL
Eyre Highway	IKE	Iron Knob	E	OL
Eyre Highway	IKW	Iron Knob	W	OL
South East Highway	MOE	Monarto	E	OL
South East Highway	MOW	Monarto	W	OL
Stuart Highway	PIN	Pimba	N	OL
Stuart Highway	PIS	Pimba	S	OL
Sturt Highway	DUE	Dutton	E	OL
Sturt Highway	DUW	Duttoin	W	OL
Riddoch Highway*	NAN	Naracoorte	N	OL
Riddoch Highway*	NAS	Naracoorte	S	OL
Port River Expressway	WI1	Wingfield	E	OL
Port River Expressway	WI2	Wingfield	E	IL
Port River Expressway	WI4	Wingfield	W	OL

* 2014 Data only available for Naracoorte

14 Appendix 2: Nominal Sprayed Seal Design

14.1 The nominal residual binder application rate and aggregate spread rates may be assessed from the following figure. These are provided for provision of nominal application rates on pavement schedules for tendering and preliminary cost purposes. Final detailed seal designs should be undertaken in accordance with RD-BP-C5 “Application of Sprayed Bituminous Surfacing” and RD-BP-D2 “Design of Sprayed Bituminous Surfacing”.

Figure RD-PV-D1 14-1 Nominal Residual Application Rates