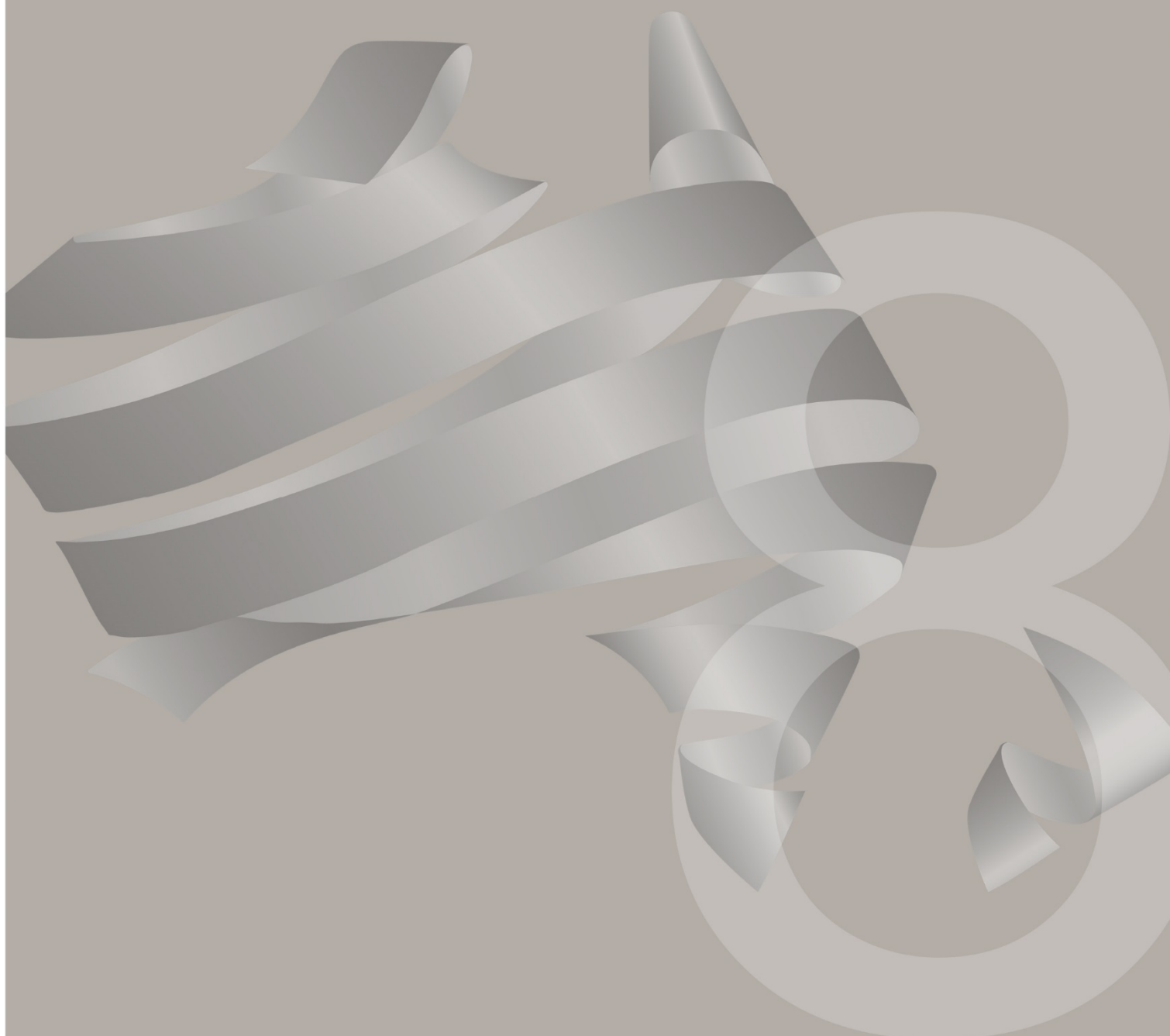


**Guide to Pavement Technology Part 8**  
Pavement Construction



# **Guide to Pavement Technology Part 8: Pavement Construction**



***Austroads***

Sydney 2009

## Guide to Pavement Technology Part 8: Pavement Construction

**First edition project manager:** Allan Jones

**First edition prepared by:** Ryan de Carteret, Lee Comport, John Metcalf and John Rebbechi

### Abstract

Part 8: Pavement Construction provides advice on the general requirements for the management of quality assurance, construction planning, earthworks, subsurface drainage, unbound pavements, stabilised pavements, sprayed bituminous surfacings, asphalt pavements and surfacings and concrete pavements. The advice has been generally developed from the approaches followed by Austroads member authorities. However, as it encompasses the wide range of materials and conditions found in Australia and New Zealand, some parts are broadly based.

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

### Keywords

pavement, construction, quality assurance, earthworks, subsurface drainage, unbound, stabilisation, sprayed bituminous surfacings, asphalt, concrete

### Edition 1.1 published August 2018

- Format updated.

Edition 1 published October 2009

**ISBN** 978-1-921551-58-1

Austroads Project No. TP1565

Austroads Publication No. AGPT08-09

Pages 187

© Austroads Ltd 2009

This work is copyright. Apart from any use as permitted under the *Copyright Act 1968*, no part may be reproduced by any process without the prior written permission of Austroads.

### Publisher

Austroads Ltd.  
Level 9, 287 Elizabeth Street  
Sydney NSW 2000 Australia

Phone: +61 2 8265 3300

austroads@austroads.com.au  
www.austroads.com.au



**Austroads**

### About Austroads

Austroads is the peak organisation of Australasian road transport and traffic agencies.

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

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

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

- Roads and Maritime Services New South Wales
- Roads Corporation Victoria
- Queensland Department of Transport and Main Roads
- Main Roads Western Australia
- Department of Planning, Transport and Infrastructure South Australia
- Department of State Growth Tasmania
- Department of Infrastructure, Planning and Logistics Northern Territory
- Transport Canberra and City Services Directorate, Australian Capital Territory
- Australian Government Department of Infrastructure and Regional Development
- Australian Local Government Association
- New Zealand Transport Agency.

This Guide is produced by Austroads as a general guide. Its application is discretionary. Road authorities may vary their practice according to local circumstances and policies. Austroads believes this publication to be correct at the time of printing and does not accept responsibility for any consequences arising from the use of information herein. Readers should rely on their own skill and judgement to apply information to particular issues.

# Contents

<b>1.</b>	<b>Introduction.....</b>	<b>1</b>
1.1	Scope of Part 8 of the Guide to Pavement Technology.....	1
1.2	Guide to Pavement Technology.....	1
<b>2.</b>	<b>Quality Assurance.....</b>	<b>3</b>
2.1	Quality Assurance and Quality Control.....	3
2.1.1	Quality Assurance.....	3
2.1.2	Quality Control.....	3
2.1.3	Responsibilities.....	4
2.1.4	Advantages Claimed.....	5
2.2	Conformance and Specification.....	6
2.2.1	Concept of Lots.....	6
2.2.2	Statistics and Quality Assurance.....	7
2.2.3	Selection of Samples.....	7
2.2.4	Limits in Standard Specifications.....	8
2.3	Systems.....	10
2.3.1	Quality Management System.....	10
2.3.2	Quality Assurance Manual (Quality Manual).....	11
2.3.3	Project Quality Plan (Quality Plan).....	12
2.3.4	Inspection and Test Plans (ITPs).....	12
2.3.5	Checklists.....	13
2.3.6	Hold Points.....	13
2.3.7	Non-conformity and Non-conformance Reports (NCRs).....	14
2.3.8	Corrective Action and Corrective Action Reports (CARs).....	15
2.3.9	Auditing.....	15
2.4	Surveillance of Construction Works.....	15
<b>3.</b>	<b>Construction Planning.....</b>	<b>17</b>
<b>4.</b>	<b>Earthworks.....</b>	<b>19</b>
4.1	Introduction.....	19
4.2	Earthworks Materials.....	19
4.2.1	Types of Earthworks Materials.....	19
4.2.2	Winning Earthworks Materials.....	20
4.2.3	Processing Earthworks Materials.....	22
4.2.4	Delivery of Earthworks Materials.....	22
4.3	Setting Out for Earthworks Construction.....	24
4.3.1	Manual Control.....	24
4.3.2	Automatic Control.....	25
4.4	Preparation for Earthworks Construction.....	25
4.4.1	Treatment of Unsuitable Material.....	26
4.4.2	Shear Key Requirements for Fills in Longitudinal Cut to Fill Zones.....	29
4.4.3	Cut Batter Stability.....	29
4.4.4	Treatment of Cuttings at Cut Floor Level.....	30
4.4.5	Treatment of Cut to Fill Zones.....	30
4.5	Embankment Construction.....	31
4.5.1	In Situ Density and Bulking of Materials on Excavation.....	31
4.5.2	Earthfill Embankments.....	33
4.5.3	Slopes.....	34
4.5.4	Fill at Structures.....	35
4.5.5	Completion of Formation (Surface of Earthworks).....	36
4.6	Geotextiles and Geogrids.....	36
4.6.1	Types, Specification and Supply.....	36
4.6.2	Handling and Storage.....	36

4.6.3	Robustness Requirements .....	37
4.6.4	Joining .....	37
4.6.5	Placement Requirements .....	39
4.6.6	Anchorage .....	42
4.7	Levelling and Trimming Operations.....	43
4.8	Compaction .....	44
4.8.1	Moisture Content .....	45
4.8.2	Layer Thickness, Type and Uniformity of Material being Compacted.....	47
4.8.3	Compaction Equipment .....	49
4.8.4	Environmental Influences .....	54
4.9	Conformance and Quality Testing.....	54
4.9.1	Material Supply .....	54
4.9.2	Test (or Proof) Rolling .....	54
4.9.3	Level Tolerances .....	55
4.9.4	Compaction.....	55
4.9.5	Frequency of Testing.....	57
<b>5.</b>	<b>Subsurface Drainage .....</b>	<b>58</b>
5.1	Trenching.....	58
5.2	Pipes.....	59
5.2.1	Prefabricated Structural (Fin) Drainage.....	59
5.3	Granular Filters.....	59
5.4	Geotextile Filters.....	62
5.5	No Fines Concrete Filters.....	64
5.6	Flushing.....	65
5.7	Finishing Details .....	65
<b>6.</b>	<b>Unbound Granular Pavements.....</b>	<b>67</b>
6.1	Introduction.....	67
6.2	Preparation of the Underlying Materials .....	67
6.3	Supply of Granular Materials.....	67
6.4	Quarrying of Granular Materials .....	68
6.5	Preparation of Granular Materials Prior to Delivery .....	69
6.6	Delivery of Granular Materials to Site.....	69
6.7	Placement of Granular Materials .....	69
6.7.1	Grader-placed Layers.....	69
6.7.2	Mechanical Spreaders.....	72
6.7.3	Jointing .....	74
6.8	Compaction of Granular Materials.....	74
6.9	Preparation Prior to Surfacing.....	75
6.10	Conformance and Quality Testing.....	77
6.10.1	Material Supply .....	77
6.10.2	Trial Pavement.....	77
6.10.3	Thickness and Level Tolerances .....	77
6.10.4	Compaction.....	78
6.10.5	Ride Quality .....	78
6.10.6	Frequency of Testing.....	79
<b>7.</b>	<b>Stabilised Pavements .....</b>	<b>82</b>
7.1	General.....	82
7.2	Specification and Supply of Materials to be Stabilised.....	82
7.3	In Situ Stabilisation.....	82
7.3.1	Production Levels .....	83
7.3.2	Initial Site Preparation .....	83
7.3.3	Application and Mixing of Powder Binders .....	83
7.3.4	Application and Mixing of Liquid Binders.....	88
7.3.5	Adding Water During Stabilisation.....	89
7.3.6	Mixing .....	89

7.3.7	Joints .....	91
7.3.8	Multi-layered Stabilised Pavements .....	92
7.3.9	Compaction.....	93
7.3.10	Curing .....	95
7.3.11	Levelling and Trimming .....	96
7.4	Plant Mix Stabilisation .....	96
7.4.1	Manufacture.....	96
7.4.2	Placement, Compaction and Curing.....	97
7.5	Granular (Mechanical) Stabilisation .....	98
7.6	Surface Sealing .....	98
7.7	Conformance and Quality Testing.....	98
7.7.1	Material Supply.....	98
7.7.2	Thickness and Level Tolerances.....	99
7.7.3	Compaction.....	99
7.7.4	Ride Quality .....	99
7.7.5	Binder Content.....	100
7.7.6	Unconfined Compressive Strength.....	100
<b>8.</b>	<b>Sprayed Bituminous Surfacing.....</b>	<b>101</b>
8.1	Preparation for Sprayed Bituminous Surfacing .....	101
8.1.1	Preparation for Initial Treatments of Granular Pavements.....	101
8.1.2	Preparation for Sprayed Seal Re-treatments .....	102
8.1.3	Preparation of Concrete Surfaces .....	104
8.1.4	Preparation of Timber Surfaces.....	104
8.2	Supply of Materials.....	104
8.2.1	Stockpiling Aggregates.....	104
8.2.2	Supply and Storage of Bituminous Materials .....	107
8.2.3	Field Preparation of Bitumen Binders.....	109
8.3	Placement of Sprayed Bituminous Surfacing .....	112
8.3.1	Preliminary Activities .....	112
8.3.2	Priming.....	115
8.3.3	Primersealing.....	117
8.3.4	Sealing.....	118
8.3.5	Other Seal and Reseal Treatments .....	122
8.3.6	Handwork.....	123
8.3.7	Cleaning Up.....	123
8.4	Conformance and Quality Testing.....	123
8.4.1	Material Supply .....	123
8.4.2	Bitumen Application Rate .....	123
8.4.3	Assessment of Loose Aggregate .....	124
8.4.4	Riding Quality .....	124
8.4.5	Treatment of Non-conformities.....	124
8.4.6	Audit and Surveillance of Sprayed Bituminous Surfacing Work.....	125
<b>9.</b>	<b>Asphalt Pavements and Surfacing .....</b>	<b>126</b>
9.1	Preparation for Asphalt Construction .....	126
9.1.1	Site Planning.....	126
9.1.2	Consideration of Climatic Conditions .....	126
9.1.3	Surface Preparation.....	127
9.1.4	Cold Milling .....	128
9.1.5	Pre-treatment of Granular Pavements .....	129
9.1.6	Pre-treatment of Asphalt Surfaces .....	130
9.1.7	Pre-treatment of Concrete Surfaces.....	130
9.1.8	Protection of Public Utilities .....	131
9.2	Transport of Asphalt .....	131
9.3	Transfer of Asphalt from Trucks to Paver .....	131
9.4	Placement of Asphalt .....	132
9.4.1	General.....	132

9.4.2	Spreading by Paver .....	133
9.4.3	Joints .....	136
9.4.4	Layer Thickness .....	138
9.5	Compaction .....	139
9.5.1	General .....	139
9.5.2	Compaction Equipment .....	139
9.5.3	Roller Numbers and Speed .....	141
9.5.4	Rolling Procedures .....	142
9.5.5	Mix Temperatures for Placing.....	143
9.6	Conformance and Quality Testing.....	144
9.6.1	Material Supply .....	144
9.6.2	Trial Pavement.....	144
9.6.3	Thickness and Level Tolerance.....	144
9.6.4	Compaction.....	145
9.6.5	Riding Quality .....	147
9.6.6	Audit and Surveillance of Asphalt Paving Contract Work .....	147
<b>10.</b>	<b>Concrete Pavements .....</b>	<b>148</b>
10.1	General.....	148
10.1.1	Sub-base Types .....	148
10.1.2	Base Types.....	148
10.1.3	Wearing Surface .....	149
10.1.4	Construction Process .....	149
10.2	Surface Preparation.....	150
10.3	Concrete Production and Delivery.....	150
10.3.1	Concrete Batch Plants.....	150
10.3.2	Delivery of Concrete .....	152
10.3.3	Retempering .....	153
10.4	Steel Reinforcement.....	154
10.5	Placement of Concrete .....	156
10.5.1	Placement by Slipform Paver .....	156
10.5.2	Consideration of Climatic Conditions .....	157
10.6	Compaction of Concrete.....	162
10.6.1	Compaction by Hand .....	163
10.6.2	Compaction by Slipform Paver .....	163
10.7	Finishing and Curing.....	164
10.7.1	Finishing .....	165
10.7.2	Curing .....	167
10.8	Joints .....	169
10.8.1	Sawn Joints .....	170
10.8.2	Crack Inducers.....	173
10.8.3	Dowel and Tie Bars .....	174
10.8.4	Sealants.....	175
10.9	Conformance and Quality Testing.....	177
10.9.1	General.....	177
10.9.2	Material Supply .....	178
10.9.3	Trial Pavement.....	178
10.9.4	Location of Steel Reinforcement .....	178
10.9.5	Thickness and Level Tolerances .....	178
10.9.6	Statistical Methods for Concrete Quality .....	179
10.9.7	Lot Definition for Concrete Paving.....	179
10.9.8	Concrete Slump.....	180
10.9.9	Compaction.....	180
10.9.10	Compressive Strength .....	180
10.9.11	Cracking.....	181
10.9.12	Ride Quality .....	181
<b>References</b>	<b>.....</b>	<b>182</b>

## Tables

Table 4.1:	Indicative densities and volume change factors .....	32
Table 4.2:	Indicative earthworks loss factors .....	32
Table 4.3:	Compaction of fill material at structures.....	36
Table 4.4:	Minimum overlap requirements.....	38
Table 4.5:	Types of compaction equipment for various materials .....	52
Table 4.6:	Visible deflection under test rolling .....	55
Table 4.7:	Example of compaction requirements for earthworks.....	57
Table 6.1:	Example of acceptance criterion for compaction .....	78
Table 7.1:	Advantages and disadvantages of methods for adding water during stabilisation .....	89
Table 8.1:	Preparation for resealing.....	103
Table 8.2:	Typical precoating rates.....	106
Table 8.3:	Binder overlap .....	113
Table 8.4:	Minimum pavement temperature for priming.....	115
Table 8.5:	Typical primer temperatures for spraying .....	116
Table 8.6:	Typical drying time of cutback primers.....	116
Table 8.7:	Typical life expectancy of a primed pavement.....	117
Table 8.8:	Primerbinder temperatures for spraying .....	117
Table 8.9:	Area effectively rolled per hour by a self-propelled multi-tyre roller.....	121
Table 8.10:	Indicative acceptable quantities of loose aggregate particles .....	124
Table 9.1:	Guide to selection of profiler .....	129
Table 9.2:	Typical asphalt layer thickness .....	139
Table 9.3:	Number of rollers (dense graded asphalt) .....	141
Table 9.4:	Typical rolling sequence .....	142
Table 9.5:	Typical permissible tolerances in shape .....	145
Table 9.6:	Typical in situ air voids (dense graded asphalt).....	146
Table 9.7:	Typical relative compaction (bulk density) .....	146
Table 10.1:	Typical construction procedure for concrete pavements .....	149
Table 10.2:	Typical texture depth requirements.....	166
Table 10.3:	Longitudinal texture drag characteristics .....	166
Table 10.4:	Typical factors that shorten the sawing window .....	171
Table 10.5:	Joint dimensions of field moulded sealants .....	176

## Figures

Figure 2.1:	Principal and contractor responsibilities .....	4
Figure 2.2:	Typical concrete paving run lots .....	6
Figure 2.3:	Example method for calculation of characteristic values.....	10
Figure 2.4:	Quality management system general requirements .....	11
Figure 2.5:	Example hold point .....	13
Figure 3.1:	Aerial view of a typical rural construction project.....	18
Figure 4.1:	Zoned embankment.....	20
Figure 4.2:	Dump truck delivering fill material .....	23
Figure 4.3:	Clearing of earthworks alignment .....	26
Figure 4.4:	Excavation of unsuitable material .....	27
Figure 4.5:	Geotextile as separation layer above and below drainage layer.....	28
Figure 4.6:	Example cross-section for benching of earth embankments.....	29
Figure 4.7:	Treatment of cut to fill zones.....	31
Figure 4.8:	Embankment construction .....	33
Figure 4.9:	Dumping of material in heaps to be spread .....	34
Figure 4.10:	Progressive revegetation of an embankment with grass and straw mulch .....	35
Figure 4.11:	Hand compaction adjacent to reinforced soil wall.....	35
Figure 4.12:	Example construction sequence for geotextile reinforced embankments .....	41
Figure 4.13:	Illustration of mud wave formation in a geotextile reinforced embankment.....	41
Figure 4.14:	Procedure for placement of fill over geotextile for moderate foundation conditions .....	42
Figure 4.15:	Grader spreading earthworks material .....	43



Figure 4.16:	Compaction of an earthworks layer .....	44
Figure 4.17:	Variation in dry density with moisture content at compaction .....	45
Figure 4.18:	Standard and modified compaction curves .....	46
Figure 4.19:	Modes of compaction .....	49
Figure 4.20:	Pneumatic multi-tyre roller compacting fill material .....	50
Figure 4.21:	Padfoot rollers compacting earthworks fill material .....	52
Figure 5.1:	Trenching for subsurface drainage .....	58
Figure 5.2:	Horizontal discharge unit .....	59
Figure 5.3:	Placement of granular material .....	60
Figure 5.4:	Placing two-stage filters .....	61
Figure 5.5:	Placing a first-stage geotextile filter .....	63
Figure 5.6:	Discharge pipe in trench, ready for second-stage filter .....	63
Figure 5.7:	Drain pipe with pre-fitted sock being positioned .....	64
Figure 5.8:	Pit with debris .....	65
Figure 5.9:	Pre-cast subsurface drain outlet .....	66
Figure 5.10:	Subsurface drain outlet in culvert headwall .....	66
Figure 6.1:	Typical pugmill used in a quarry for processing granular materials .....	68
Figure 6.2:	Grader trimming granular material .....	70
Figure 6.3:	Paver laying granular material .....	73
Figure 6.4:	Smooth drum steel-wheeled roller compacting granular material .....	75
Figure 6.5:	Water cart ensuring surface of granular material does not dry out prior to surfacing .....	76
Figure 6.6:	Survey vehicle featuring laser profilometer to measure ride quality .....	79
Figure 6.7:	Example of sampling and testing frequency requirements (1) .....	80
Figure 6.8:	Example of sampling and testing frequency requirements (2) .....	81
Figure 7.1:	A mechanised spreader with rear drop chute .....	84
Figure 7.2:	A reclaimer with an integrated powder binder hopper and load cells .....	84
Figure 7.3:	Spotting cement bags and hand raking binder into position .....	84
Figure 7.4:	Use of trays to verify the binder application rate .....	85
Figure 7.5:	Construction traffic over spread binder may alter distribution .....	86
Figure 7.6:	Desirable features of a mechanical spreader .....	87
Figure 7.7:	Lime slurry spreading .....	88
Figure 7.8:	Liquid binder (foamed bitumen) added directly through recycler .....	88
Figure 7.9:	Two pass mixing (1) .....	90
Figure 7.10:	Two pass mixing (2) .....	90
Figure 7.11:	A lens of no or little binder may occur when the first pass mixing operation is deeper than the second pass .....	91
Figure 7.12:	Typical spreading and mixing arrangements for half-width (A) and full-width (B) .....	92
Figure 7.13:	Typical two layered in situ stabilisation of heavily trafficked highways .....	93
Figure 7.14:	Longitudinal shrinkage cracks .....	94
Figure 7.15:	Static padfoot (left) vibrating padfoot (right) compactors .....	94
Figure 7.16:	Smooth drum and rubber tyred roller in operation .....	95
Figure 7.17:	Delamination of stabilised material .....	96
Figure 7.18:	Stationary mixing plant and pugmill chamber .....	97
Figure 7.19:	Paver laying stabilised material .....	97
Figure 8.1:	Typical rotary broom .....	102
Figure 8.2:	Typical vacuum broom .....	102
Figure 8.3:	Loading precoated aggregate with a front-end loader .....	106
Figure 8.4:	Road ranker (at rear) delivering binder to site .....	108
Figure 8.5:	Bitumen sprayer .....	115
Figure 8.6:	Truck mounted box spreader .....	119
Figure 8.7:	Aggregate spreader .....	119
Figure 8.8:	Pneumatic multi-tyred roller .....	120
Figure 9.1:	Rotary road broom .....	127
Figure 9.2:	Shape correction .....	128
Figure 9.3:	Tack coat sprayer in operation .....	130
Figure 9.4:	Materials transfer device .....	132
Figure 9.5:	Asphalt paver .....	133

Figure 9.6:	Typical paver components .....	134
Figure 9.7:	Paver with hydraulic screed .....	134
Figure 9.8:	Joint matching shoe .....	135
Figure 9.9:	Levelling beam .....	136
Figure 9.10:	Cutting disc attached to steel roller .....	137
Figure 9.11:	Paving in echelon (hot joints) .....	137
Figure 9.12:	Vibratory steel-wheeled roller .....	140
Figure 9.13:	Pneumatic multi-tyred roller .....	140
Figure 9.14:	Nuclear density testing.....	146
Figure 10.1	Typical concrete road.....	150
Figure 10.2:	Concrete batch plant.....	151
Figure 10.3:	Tipper trucks being used on a typical PCP project .....	152
Figure 10.4:	Limit of retempering .....	154
Figure 10.5:	CRCP reinforcement.....	154
Figure 10.6:	Erection of CRCP reinforcement.....	155
Figure 10.7:	Components of a slipform paver .....	156
Figure 10.8:	Slipform paver .....	157
Figure 10.9:	Effect of temperature on concrete compressive strength development with age .....	157
Figure 10.10:	Effect of temperature on concrete setting time .....	158
Figure 10.11:	Example of plastic shrinkage cracks.....	160
Figure 10.12:	Rate of evaporation of surface moisture from concrete.....	161
Figure 10.13:	Compressive strength versus density or air voids content .....	162
Figure 10.14:	Damp hessian being dragged behind a slipform paver .....	166
Figure 10.15:	Tyned concrete surface .....	167
Figure 10.16:	Sawing Window.....	170
Figure 10.17:	Failure of keyed joint.....	173
Figure 10.18:	Tied transverse construction joint .....	173
Figure 10.19:	Tie bars protruding from concrete slab at longitudinal joint .....	174
Figure 10.20:	Dowel bars located in specially prepared cages.....	174
Figure 10.21:	Joint sealant with backing rod.....	175
Figure 10.22:	Joint sealant loads .....	176
Figure 10.23:	Sealant extending down the side of the base concrete .....	177
Figure 10.24:	Normal distribution of concrete strengths .....	179

# 1. Introduction

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

## 1.1 Scope of Part 8 of the Guide to Pavement Technology

This part of the Austroads Guide to Pavement Technology is intended to give the practitioner an overview of the issues involved in the management of pavement construction.

The guide contains brief descriptions of the topics listed below:

- quality assurance
- earthworks
- subsurface drainage
- unbound granular pavements
- stabilised pavements
- sprayed bituminous surfacings
- asphalt pavements and surfacings
- concrete pavements.

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

## 1.2 Guide to Pavement Technology

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

- Part 1: Introduction to Pavement Technology
- Part 2: Pavement Structural Design
- Part 3: Pavement Surfacings
- Part 4: Pavement Materials:
  - Part 4A: Granular Base and Subbase Materials
  - Part 4B: Asphalt
  - Part 4C: Materials for Concrete Road Pavements
  - Part 4D: Stabilised Materials
  - Part 4E: Recycled Materials
  - Part 4F: Bituminous Binders
  - Part 4G: Geotextiles and Geogrids
  - Part 4H: Test Methods
  - Part 4I: Earthworks Materials

- Part 4J: Aggregate and Source Rock
- Part 4K: Seals
- Part 4L: Stabilising Binders
- Part 5: Pavement Evaluation and Treatment Design
- Part 6: Unsealed Pavements
- Part 7: Pavement Maintenance
- Part 8: Pavement Construction
- Part 9: Pavement Work Practices
- Part 10: Subsurface Drainage.

It is emphasised that this document should be used only as a guide and not as a limiting or standard specification. Some road agencies have published manuals or supplements that translate the guidance provided by Austroads into practice reflecting local materials, environments, loadings and pavement performance.

## 2. Quality Assurance

One of the greatest challenges in road construction is to consistently achieve work that conforms to the specified requirements (Austroads 1995).

Up until the mid-1980s the quality of Australian pavement construction was specified in terms of the end result, which was to be achieved by the contractor. The construction processes were assumed to be the business of the contractor and the main check on quality came at the completion of a piece of work when the properties of the completed work were measured and the work was either accepted or rejected by the principal. The quality assurance process was introduced in Australia in an attempt to improve product quality while ensuring that the quality of the product is the responsibility of the contractor.

AS/NZ ISO 9000 describes fundamentals of quality management systems and expresses the rationale for quality management systems as 'the quality management system approach encourages organisations to analyse customer requirements, define the processes that contribute to the achievement of a product which is acceptable to the customer, and keep these processes under control. A quality management system can provide the framework for continual improvement to increase the probability of enhancing customer satisfaction and the satisfaction of other interested parties. It provides confidence to the organisation in that it is able to provide products that consistently fulfil requirements'.

This section gives a brief overview of quality assurance as it relates to road construction. For further information on quality assurance refer to AS/NZS ISO 9000.

It is acknowledged that Centre for Pavement Engineering Education course notes (CPEE 2003, 2004 & 2006) were useful sources of information in preparation of this section.

### 2.1 Quality Assurance and Quality Control

#### 2.1.1 Quality Assurance

Quality assurance is defined as systematic action necessary to give confidence of satisfactory quality (Austroads 2008c) and includes production of documentary evidence that the defined quality has been achieved.

A quality assurance program is intended to assure the principal that the works have been constructed to the standards required of the specification. Under the program the contractor is required to undertake specified quality checks and tests so as to provide written confirmation of the adequacy of the works.

#### 2.1.2 Quality Control

Quality control is defined as those tests necessary to determine and control the quality of the product being produced (Austroads 2008c).

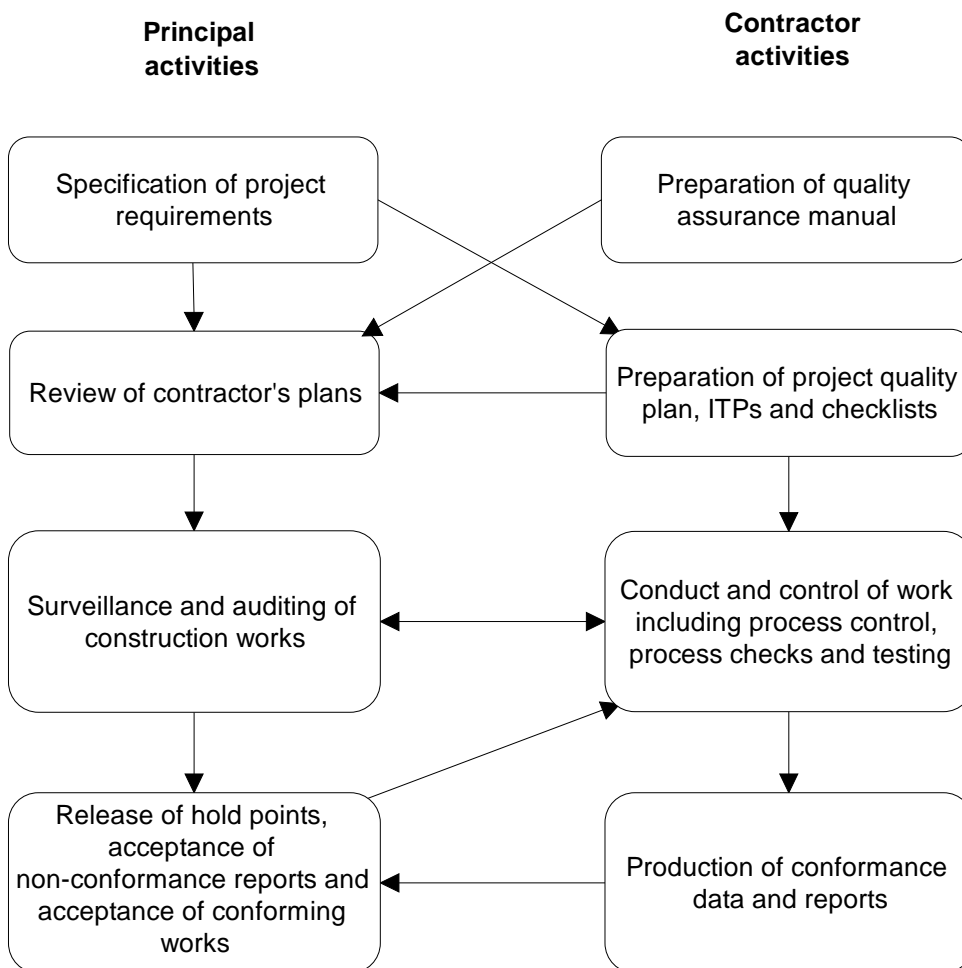
The common activities of quality control are inspection, sampling, testing, and statistical analysis, although the methods adopted may not be identical. Under most road construction contracts quality control is the responsibility of the contractor within an overall quality assurance context.

### 2.1.3 Responsibilities

There are a number of different contract methods used in Australia and New Zealand for pavement construction including direct control, conventional construct only, design and construct, design, construct and maintain, build own operate and alliance contracts. The type of contract used for any given project is typically a function of many issues including authority practice, project size, project risk, resource availability, delivery timeframe and financing. The form of contract significantly affects the division of responsibilities and risk allocation between the road authority and the constructor.

Figure 2.1 provides an overview of the activities to be undertaken by the principal and the contractor under a conventional, construct-only contract, which is the most common form of contract used in Australia and New Zealand for pavement construction. It should be noted that in some jurisdictions a superintendent/contract administrator is used to administer construction contracts on behalf of the principal. In such cases, some of the responsibilities of the principal are assigned to the superintendent/contract administrator. However, the division of responsibilities between the principal and superintendent/contract administrator is not consistent across road authorities and may even vary within jurisdictions depending on factors such as project risk and availability of resources.

**Figure 2.1: Principal and contractor responsibilities**



#### **Responsibilities of the principal**

Responsibilities of the principal include:

- specifying the work to a particular level including tolerances
- forming a partnership with the contractor to promote mutual trust

- carrying out sufficient surveillance to ensure that the contractor's procedures are being implemented and are effective in achieving the specified outcomes including quality, time, performance, environmental and safety
- ensuring that the contractor complies with the quality verification requirements and that the contractor's procedures reflect the requirements of the specifications
- auditing the contractor's records to assist in identifying deficiencies in the contractor's quality assurance system, identifying opportunities for improvements to the quality assurance system and identifying trends in non-conformance issues
- approving/accepting works, which demonstrate compliance with the specified requirements.

### ***Responsibilities of the contractor***

The contractor is responsible for establishing, undertaking and maintaining the necessary quality control and process checks, including testing, to ensure that specified criteria are met. The responsibility for achieving quality lies with the contractor because the contractor has control over the process. Hence, the contractor administers quality control, sometimes referred to as process control, to reach the specified quality set out in the contract documents (Austroads 1995).

For the principal to gain the desired assurance that the final product will meet the specified criteria, contractors are required to provide a specific quality plan for the contract. The plan must be based on the contractor's own quality system, detailing the processes, procedures, and inspection and test programs to be used in delivering the requirements of the specification (Austroads 1995). Contractors must also provide conformance data and reports for completed work that demonstrate that at least the specified criteria have been met.

### **2.1.4 Advantages Claimed**

Various advantages are claimed for the quality assurance programs. Whether these advantages are achieved depends on how conscientiously the system is applied.

#### ***Advantages for the contractor***

The system allows the contractor to integrate the testing of the works with the construction program. The contractor is not subjected to delays while the resources of the principal are mobilised and results obtained, apart from hold points. Generally, the contractor makes the decision to proceed to the next operation and the flow of the works is improved. The contractor is also obliged to assess the quality of their product and is provided with objective results, which allows them to make the necessary adjustments to achieve the specified product.

#### ***Advantages for the principal***

Overall, quality of the product is improved if more attention is paid to the processes and testing of the product as it is being constructed, as opposed to only once completed. Complete documentation of the quality of the product, lot by lot, is compiled allowing future performance of the project to be more easily related to the quality of the materials and construction processes employed. The potential for disputes and litigation is also reduced through the incremental approval or acceptance of the works and better definition of risk allocation between the two parties.

## 2.2 Conformance and Specification

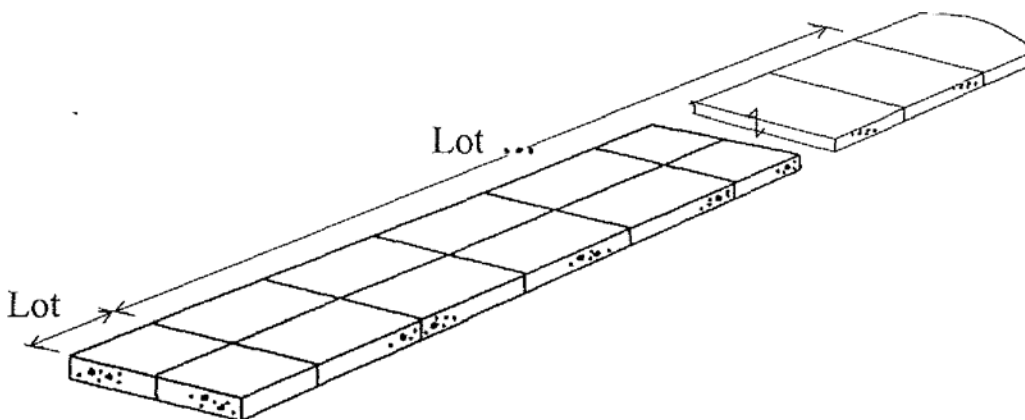
### 2.2.1 Concept of Lots

A key concept in quality assurance as applied to road construction is that of a lot.

A lot is defined as 'a continuous portion of homogeneous and/or representative material, or end product produced under essentially constant conditions usually within one shift' (Austroads 2008c). Work on site is carried out in lots (Figure 2.2), testing is by lots, verification to the specification is by lots and the contractor is paid by lots.

A typical example of a lot is a length of a single layer of road base that has been placed and compacted within a single shift.

**Figure 2.2:** Typical concrete paving run lots



Source: CPEE (2006)

In selecting the extent of a lot, homogeneity must be considered such that the variability within a lot is not caused by assignable factors such as changes in material, layer thickness, compaction equipment, rolling pattern, etc.

Where the material, method of placement or method of compaction varies separate lots should be designated. A typical example of this situation is concrete pavement construction where hand paved, or compacted, sections at the start and end of a concrete paving run should be designated as separate lots to the main slipformed paving run.

Accurate assignment and recording of lots provides the ability to trace the history, application or location of any item or activity, or similar items or activities, by means of record identification. This is vitally important when determining quality trends, determining the cause of and rectifying deficiencies.

It is common for construction specifications to require traceability from manufacture all the way through to placement. For example, traceability for concrete batches may be required to start at the batch plant and finish at the location where the material is incorporated into the works. In such cases, lot conformance records should include off-site details such as batch quantities and batching times in addition to on-site details such as testing details and location of placement.

Since a lot is a single homogeneous production unit, samples taken from it are deemed to be representative of its quality as a whole. It follows that the decisions concerning the acceptability of that quality apply to the whole lot and not just any particular part. Thus, if one sample taken from a lot fails to meet the specified requirements the whole lot is deemed to not comply. In addition, any rectification work to address non-conformity of a lot should be applied to the whole lot not just the location(s) of non-conforming samples.



### 2.2.2 Statistics and Quality Assurance

Any quality assurance system is likely to include a statistically based assessment scheme. The design of a statistically based assessment scheme requires knowledge of and judgement concerning:

- the variability of products and test procedures
- the consequences (risk) in accepting a non-complying product (unintentionally)
- weighing up the cost of reducing that risk by increased testing. Sample size affects probability of acceptance
- cost implications of transferring risk onto the contractor/supplier
- reliability and consistency of the upstream production and quality control procedures.

As with most disciplines, there are numerous statistical terms used in road construction. The following definitions are commonly used by pavement practitioners:

<b>Mean</b>	A number that represents the centre of a series of test results.
<b>Standard deviation</b>	The standard error of the mean is the standard deviation of the test results.
<b>Distribution</b>	Test results will always vary about some mean and the pattern of this variation is called the distribution.
<b>Histogram</b>	A bar chart of the number of test results recorded for each measurement.
<b>Normal distribution</b>	One of several probability distributions. It resembles a bell shape curve with the top of the bell representing the mean of the test results. The distribution is symmetrical and has simple mathematical properties relating to the standard deviation.
<b>Rolling mean</b>	Calculated mean using groups of consecutive results with progression in single increments.
<b>Tolerance</b>	The permissible deviation from a specified value.

The three key parameters of any statistical relationship are the:

- size of the population, including the definition of a lot of homogeneous quality
- size of the sample(s)
- method of choosing the random sample(s).

For additional guidance on basic statistical principles and methodologies refer to Grant (1964) and Wadsworth (1989).

### 2.2.3 Selection of Samples

Certainty in obtaining the true value of a lot property could be assured by measuring each and all of the smallest subdivisions of the lot. However, as this is impractical, the value of the property is estimated from measurements on a sample, which is defined as those portions of the lot taken to represent the whole.

As a single material can yield various values when different portions are tested, an adequate description of quality requires both the average quality and the variation in quality about the average. Appropriate sampling theory must be used whether selecting different test portions used to estimate the mean and variation in lot quality, or choosing the different increments to be combined into a representative test portion to obtain the average quality.

In order to make an unbiased decision, samples from within a lot must be drawn in an unbiased manner. Test sites or portions should not be chosen on the basis of appearance, good or bad, by expectations of high or low results or by any other form of personal bias. The deliberate selection of sites or portions, which appear to be average, is also invalid. An unbiased selection is achieved if testing portions are taken without regard to their condition or position so that all parts of a lot have the same probability of being sampled. All lot increments must have an equal and non-zero chance of selection. Classical unbiased sampling methods recommend random selection of test portions.

Elimination of bias has the further requirement that all lot increments be independent, i.e. that the test result from any site cannot be predicted from the result from another site. Where normal construction processes operate on paving materials, two very adjacent sites can be expected to have almost the same value for properties such as density, moisture content and subgrade support. Three sites across a pavement at each of three locations thus could not be construed as nine independent sites. Even for purely random selection it is necessary to define units of sufficient size that random differences could be expected to exist and that no more than one test is conducted in any unit.

The choice of sampling method should be balanced against speed, simplicity and cost. Statistical sampling plans, though theoretically sound, may be unsuitable if the cost of implementation is prohibitive.

Sampling selection should not be delegated to inexperienced and/or uninstructed personnel, as improper sampling can undermine all efforts to improve the precision of testing procedures.

Re-tests, after initial rejection and treatment of a non-conforming lot must be based on a full and separate re-sampling of the whole lot, not just locations where previous tests had failed.

NAASRA (1989) provides further details on lots, selection of lots, selection of samples and general statistical principles applying to pavement materials.

#### **2.2.4 Limits in Standard Specifications**

There are three primary methods used for setting limits in standard roadworks specifications:

- maximum and/or minimum limits
- specified values and tolerances
- characteristic value.

##### ***Maximum and/or minimum limits***

Any specified maximum or minimum limit should be regarded as an absolute limit and not used as a target value. The relevant limit should not be exceeded as the result of any variation in the construction and/or measurement process.

An example of a maximum or minimum limit is the shape requirements for the surface of an unbound pavement in RTA (2007a), which is that 'the surface must not deviate from the bottom of a 3 metre straight edge laid in any direction by more than 5 mm'.

##### ***Specified values and tolerances***

A specified value is regarded as a target value and any specified tolerances should be regarded as providing the absolute limits for any departure from the specified value. The construction of the works should be controlled so that the relevant tolerance is not exceeded as the result of any variation in the construction and/or measurement process.

An example of a specified value and tolerances is the pavement width requirements for an unbound pavement in RTA specification R71: Unbound and Modified Pavement Course which is that 'the width of the base and subbase layers must be as specified...and constructed with a tolerance of zero to +100 mm' (RTA 2007a).

### ***Characteristic value***

Where a characteristic value is specified, a statistical procedure in accordance with the relevant specification should be used to determine the value that represents the properties of a lot. The statistical procedure makes allowance for variations in the construction and measurement processes.

The characteristic value method of specifying limits is commonly used for determining conformance of a lot where a number of samples are taken and variability between samples is likely, such as compaction conformance of earthworks layers.

An example of a characteristic value is the compaction requirements for an unbound pavement in RTA (2007a), which is that 'conformity of a lot for compaction is achieved if the Characteristic Value of Relative Compaction of the lot, determined in accordance with RTA Q and reported to one decimal place, is not less than 102.0%'.

### ***Calculation of characteristic value***

Construction contracts specifying characteristic values will also typically specify the statistical procedure to be used in determining characteristic values and it should be noted that the method used varies significantly between road authorities.

Figure 2.3 is an example method used where the compliance of a lot is to be measured by the calculation of a characteristic value, determined by the analysis of several individual samples or measurements using a statistical procedure.

Figure 2.3: Example method for calculation of characteristic values

**L3.2 Method for Statistical Calculation for Conformity of Lots**  
 When acceptance criteria specify a maximum and/or minimum characteristic value of attribute (Q),  $Q_U$  and/or  $Q_L$  must be used to determine Q.  
 The calculation of the characteristic value of attribute (Q) for the lot must be as follows:

(a) Sample Size = 1  
 $Q_U = Q_L = \text{Test result}$

(b) Sample Size = 2  
 $Q_U = \text{highest test result}$   
 $Q_L = \text{lowest test result}$

(c) Sample Size > 2  
 $Q_U = \bar{x} + ks$   
 $Q_L = \bar{x} - ks$

where  $\bar{x}$  = arithmetic mean of attribute test results for all sub-lots  
 $s$  = standard deviation of sub-lot attribute test results

$$s = \sqrt{\frac{\sum_{i=1}^n (x_i - \bar{x})^2}{n - 1}}$$

$k$  = acceptance constant from Table Q/L.2 (based on 10% producer's risk)

**Table Q/L.2 – Acceptance Constant  $k$**

Sample Size	3	4	5	6	7	8	9	10 - 14	15 - 19	20 +
$k$	0.52	0.62	0.67	0.72	0.75	0.78	0.81	0.83	0.90	0.95

A lot achieves conformity if:  
 $Q_U \leq$  the specified upper limit for characteristic value of the attribute; and  
 $Q_L \geq$  the specified lower limit for characteristic value of the attribute.  
 If:  
 $Q_U$  is more than the specified upper limit for characteristic value; or  
 $Q_L$  is less than the specified upper/lower limit for characteristic value,  
 and reworking is subsequently undertaken, the complete lot must be resampled and retested to verify conformity.

Source: RTA (2007a)

## 2.3 Systems

### 2.3.1 Quality Management System

A quality management system is that aspect of the overall management function that determines and implements policy. A quality system incorporates the organisational structure, responsibilities, procedures, processes, and resources for implementing quality management. Although quality assurance is used extensively for work undertaken by contract, the system is also being progressively introduced into work undertaken by day labour.

Establishing and implementing a quality management system involves:

- identifying the areas and assessing the associated level (likelihood and impact) of risk of products and services not conforming with the specified requirements
- developing processes, generally documented as plans and procedures, to manage the risks

- identifying and providing resources and allocating responsibilities to suit the plans and procedures
- implementing the plans and procedures
- monitoring, auditing and improving the implementation of plans and procedures
- regularly reviewing and improving the quality management system.

Quality systems used both in Australia and internationally have been standardised and the appropriate standards used in Australia are known as the AS/NZ ISO 9000 series.

It is typical for contracts to specify the minimum quality assurance level and practices required for the works. An example of requirements for a quality management system is shown as Figure 2.4.

**Figure 2.4: Quality management system general requirements**

**QUALITY MANAGEMENT SYSTEM**

**General Requirements**

Develop a corporate Quality Management System that complies with all the requirements ISO 9001 and RTA Q. Implement and maintain the Quality Management System in accordance with ISO 9001 and RTA Q.

Apply the following quality assurance practices to the Work Under the Contract:

- (a) ensure that purchased items conform to specification before incorporating them in the Works;
- (b) plan and control work processes;
- (c) plan and carry out inspection and testing (including identification and traceability) to verify that the work processes are effective and that all finished work complies with the Contract;
- (d) careful selection of subcontractors and confirmation that their work complies with the Contract;
- (e) where the Specifications require plans, procedures, methods and forms to be documented, use these documents in implementing the Quality Management System for the Contract;
- (f) acknowledge and rectify any nonconforming work and improve work processes to prevent recurrence of nonconformities;
- (g) keep orderly records to demonstrate that the Works comply with the Contract; and
- (h) improve procedures and work practices when opportunities are identified to minimise errors, waste and product

Source: RTA (2008a)

### 2.3.2 Quality Assurance Manual (Quality Manual)

The quality assurance manual, also known as quality manual, is the primary document for a company's quality management system and sets out the general quality policies, procedures and practices of an organisation. The majority of road construction contracts require the contractor to provide a quality manual prior to the commencement of the work. This manual should contain the following:

- a general statement regarding the contractor's management attitude towards quality assurance
- details of the contractor's organisation and allocation of responsibilities in regards to implementation and continuance of the quality assurance system

- procedural checklists and verification checklists for all of the contractor's activities required to construct the works.

Commonly, the quality manual is a corporate document that may be integrated with other management systems such as those for environmental and occupational health and safety management.

### **2.3.3 Project Quality Plan (Quality Plan)**

A project quality plan is a document that sets out the specific quality practices, resources and sequence of activities relevant to a particular project. The quality plan is generally prepared using a format similar to the quality manual, but is related to the works undertaken on a specific project.

Project quality plans generally contain a number of sections including the management and organisational chart for the work, a works program, construction procedures, hold points, checklists, inspection and test plans and a range of administrative procedures such as measuring and testing equipment verification, authorisation of variations, audit test records, site instructions and construction methods.

Preparation of the project quality plan should begin with a pre-construction review of the required construction procedures. This works to facilitate quality assurance by defining the many separate tasks involved in delivering the project, permitting progressive certification of quality. This task breakdown should be linked to the inspection and test plans, so that all parties are fully aware of the inspection practices to be observed, the witness, check and hold points and, the measurement, material sampling and testing requirements for each task. The personnel involved in the certification process must be designated and provided with the required documentation for recording the certification steps.

Much of the quality assurance function is best conducted by independent verification, systematic, well planned, quantified and responsive to the level of confidence in the contractor's quality assurance system. An appropriate mix of observation, to verify planned work practices are being followed, review and documentation of hold points and witness points with provision for an audit procedure, and sampling and testing protocols, defining frequency and analysis, is required in the plan.

In practice, most contractors will have a generic form of project quality plan that is used as a shell document for adaptation to specific contract works as required. Generic project quality plans may also be acceptable for use on minor works that do not justify preparation of a separate project quality plan.

Additional guidance on the preparation of quality plans is provided in AS/NZS ISO 9004:2000.

### **2.3.4 Inspection and Test Plans (ITPs)**

Inspection and test plans are documents that specify the type and extent of the inspection and testing to be performed and acceptance requirements on a given activity or item (AS 3905.2).

Inspection and test plans should include (RTA 2008a):

- who performs the receiving, in-process and final inspections and/or testing and at what stage of the work
- how the inspection or test is to be carried out and recorded
- the acceptance criteria and frequency of inspection and testing
- who reviews inspection/test results, evaluates whether work conforms, determines what to do next if work does not pass a required inspection or test and closes out work lots
- when statistical analysis of test results is required
- when nonconformity control is addressed including closing out work lots
- who performs final review of all inspection/test results to confirm that all inspections and test have been carried out to completely verify conformity for each lot

- the time limits for testing, time constraints for submission, and hold and witness points that are nominated in the specifications
- the requirements for identification and traceability and the sampling methods.

### 2.3.5 Checklists

There are generally considered to be two types of checklists used in road construction works: procedural checklists and verification checklists.

Procedural checklists are used to identify each activity that must take place during the construction process. On the completion of each activity the person in charge of the work, typically the team leader, ganger, supervisor or foreman, completes and signs the checklist, signifying that correct construction procedures have been followed. This makes this person responsible and accountable for the completed work.

Verification checklists are used to verify that the work has been completed in accordance with the specified requirements. Verification checklists are typically completed by a responsible person one level removed from the person performing the work; usually the contractor's quality assurance engineer, who verifies that the work has been completed in accordance with the specification requirements. These checklists are often accompanied by the results of any tests or measurements undertaken to verify that the work complies with the specified requirements.

### 2.3.6 Hold Points

A hold point is a point in the construction process beyond which the work may not proceed without authorisation (AS/NZ ISO 9001). In road construction contracts hold points generally require written authorisation by the principal prior to commencing the next activity.

Hold points are introduced into a quality assurance system prior to the start of critical works where the consequences of failure are expensive. Common examples of hold points include submission of conformance records, for the underlying layer as well as the materials or mix to be used, prior to placing a concrete pavement, asphalt pavement or bitumen surfacing. Figure 2.5 is an example of a hold point, in this case prior to priming or primer sealing of a new pavement surface.

**Figure 2.5: Example hold point**

<b>Hold Point</b>	
Process Held:	Priming or primer sealing of new pavement surface.
Submission Details:	Conformance reports verifying that all level, thickness, compaction and trimming requirements of the Specification have been met for each lot.
Release of Hold Point:	The Principal will consider the submitted documents, prior to authorising the release of the Hold Point

Source: RTA (2007a)

Modern road construction contracts often include a multitude of hold points allowing both the principal and contractor to effectively minimise construction risk through incremental review and approval or acceptance of the works.

### 2.3.7 Non-conformity and Non-conformance Reports (NCRs)

A non-conformity is defined in AS/NZS ISO 9000 as non-fulfilment of a requirement. In relation to road construction, a non-conformity is commonly described as a non-conformance and is a deficiency in characteristics, documentation or process implementation which renders the quality of an item indeterminate or outside that required by the relevant specification, contract or regulation.

Non-conformance reports are raised and completed by the contractor when a non-conformity/non-conformance is identified. Non-conformance reports generally contain a section identifying the location, details and intended action to rectify the non-conformance and a separate section for the principal's response and acceptance or approval.

Generally the action taken to address a non-conformance takes one of the following forms:

- rejection of the lot – requiring rectification work and re-testing of the lot
- acceptance of the lot with conditions – conditions that may be applied include: additional rectification treatments (which were not specified but provide a fit-for-purpose product), extended warranty for that portion of the works and payment deductions
- acceptance of the lot without conditions – generally confined to isolated, very low risk non-conformances.

The principal may agree to accept the contractor's proposed action or specify a different action and will generally require evidence that the rectification work, if required, has been performed.

The primary determinant in what action the principal is willing to accept in response to a non-conformity should be based on the risk associated with the non-conformity. For example, while an isolated low compaction result in a lower earthworks layer may be accepted by the principal, conditional upon improvements to the compaction process for subsequent layers due to its perceived low risk, it is unlikely a low compaction result would be accepted in a pavement base course lot.

If a lot is rejected the whole lot should be reworked, which in the case of sprayed seals, asphalt or concrete may involve complete removal and replacement of the lot. It is not acceptable to rework only those areas where a non-conforming sample or samples are located.

Rectification treatments, as opposed to rejection of the lot, generally involve application of a further treatment that is designed to mitigate the potential detrimental effects of the non-conformity. Examples of rectification treatments include grinding of a concrete pavement surface to obtain the specified surface texture, application of a crumbed rubber reseal in place of a standard bitumen reseal over a non-conforming primerseal, and increasing the thickness of an asphalt base over a granular sub-base that did not meet the specified thickness. Typically, rectification treatments are only accepted by the principal where there is no additional cost to the principal, and the final product is equivalent to the specified product.

Extended warranties may be appropriate where the impact of a non-conformity is not known but is likely to become apparent within the extended warranty period. A typical example of an extended warranty response to a non-conformity is a sprayed bituminous seal in which the binder application rate exceeded the specified rate by more than an acceptable tolerance. In this case the risk of the seal bleeding is likely to occur within the first or second summer and an extended warranty period of two or three years may be acceptable to the principal.

Some construction specifications, such as RTA (2007a), include payment deductions schedules that may be accepted by the principal for specific non-conformities such as ride quality, compaction and percentage of binder content. Typically, payment deductions are only applied when the cost of rectification is very high, such as poor ride on an asphalt pavement, and the consequences of the non-conformity are able to be managed by the principal. In determining appropriate payment deduction schedules, sufficient payment should be withheld to compensate for any disadvantage, at the time of construction and in the future.



### 2.3.8 Corrective Action and Corrective Action Reports (CARs)

Corrective action is defined in AS/NZS ISO 9000 as the action taken to eliminate the cause of a detected non-conformity.

Corrective action reports are completed by the contractor and record the details of a deficiency and the subsequent corrective action taken to prevent its recurrence. In road construction, corrective action reports are generally used to identify trends in non-conformities and thereby improve a quality system, as opposed to non-conformance reports, which identify rectification of a particular non-conformity. Therefore, corrective action reports may or may not require principal acceptance/approval. It should be noted that, in practice, corrective action reports may be initiated by the contractor, principal or auditors.

### 2.3.9 Auditing

Auditing is defined in AS/NZS ISO 9000 as a systematic, independent and documented process for obtaining records and other information and evaluating it objectively to determine the extent to which the policies, procedures and/or requirements are fulfilled.

Audits may take the form of quality system audits, quality audits, product quality audits, or technical procedure audits and may be undertaken by an auditing team from within or outside the organisation. Audits should be conducted by a qualified auditor in accordance with AS 3911.1.

## 2.4 Surveillance of Construction Works

Surveillance is the continuing evaluation of the status of procedures, methods, conditions, products, processes and services, and analysis of records to ensure that the quality requirements are being met. Surveillance is normally undertaken by an officer appointed by the principal to observe and monitor a particular contract performance.

The amount of surveillance by the principal is a function of the confidence that the principal has in the contractor delivering a quality outcome within the specified requirements and the perceived risk of the contractor not achieving the specified quality. This requires a degree of understanding and trust between the two parties involved. Surveillance may be fairly intense until such time as both parties have developed an understanding of the other's expectations and operating characteristics and the principal is satisfied that a quality product is being delivered by the contractor.

Surveillance should be realistic in terms of the nature of the specific project. This reflects the fact that, in quality assurance contracts, the onus is on the contractor to assure quality. However, it does not mean that the principal (and principal's representatives) should abdicate responsibility for quality by being inactive in surveillance and auditing.

Surveillance of contract activities may also include separate or parallel sampling and testing by the principal to verify the accuracy of testing methods.

A further guide to surveillance in road contracts, including example checklists, is provided in the Austroads (1995).

### **Construction records**

In undertaking surveillance and management, or administration, of construction works, key project information should be recorded by the works supervisor and/or engineer, particularly for monitoring performance and progress, and verifying and/or quantifying claims. This information is typically recorded in a diary, checklist or work record sheet on at least a daily basis and may include:

- activities being undertaken on and off site (location, activity, duration)

- resources – both active and non-active (plant, equipment and labour – often cross-referenced to location, activity and duration)
- weather conditions, particularly when impacting on performance of the works (forecast weather conditions also may be recorded when appropriate such as the evening before or morning of sprayed sealing operations)
- details of any issues, contractual or otherwise, that arise
- details of any instructions issued by the principal or contractor
- details of any non-conformances observed or raised
- details of any OHS incidents
- details of any environmental incidents
- details of traffic controls in place (traffic control plan being used, time checked, location of signage, any deficiencies observed, traffic conditions).

Increasingly, digital photographs are also being used to record construction activities. Photographs provide a snapshot of the work being undertaken and can be a good reference source for determining causes of non-conformities, highlighting good (and bad) construction practices and preparing construction reports. It should be noted that the legal status of digital photographs differs between states and it is good practice, particularly when photos may be used in contractual disputes, to print photographs as soon as practical then immediately sign and date the back, verifying the image shown.

### ***Off-site surveillance and auditing***

Surveillance and auditing of off-site activities is an important aspect of quality assurance for road construction, especially as in many contracts off-site work is done by subcontractors. Generally the main contractor is responsible for evaluating and selecting subcontractors and suppliers on their ability to meet specified requirements, including quality, and to carry out the audits and surveillance of the subcontractor's/supplier's off-site activities. However, it is increasingly common for the principal to undertake additional surveillance and auditing of off-site activities, as these activities are often perceived as representing high risk.

Examples of where off-site surveillance and auditing may be required include:

- quarries
- asphalt plants
- concrete plants
- pre-cast concrete
- steel fabrication.

### 3. Construction Planning

Planning is critical to the success of pavement construction projects. Typically, planning for pavement works can be divided into the following categories:

- preliminary activities carried out well in advance of pavement construction works
- operational activities prior to commencing construction works
- operational activities during construction works
- operational activities following completion of construction works.

Preliminary activities that should be carried out well in advance of construction works include:

- site inspection to assess the site conditions, basic requirements of the job and to identify and locate existing services
- development of quality, traffic management, occupational health and safety and environmental plans for the works
- development of a works program showing the logical relationship between activities, identifying time leads and lags, resource and other constraints and the sequence of activities that constitute the critical path
- identification and development of any design requirements that are not included on the drawings and construction specifications or are an operational responsibility (i.e. earthworks layer thicknesses and rolling patterns, sprayed seal design rates for application of binder and aggregate, asphalt joint layout plans)
- selection of material suppliers, equipment, plant, stockpile sites, work sites, contract agreements
- advertisement of the start date, nature and impact of works and provision of advance warning to occupiers of adjoining properties.

Operational activities prior to commencing construction works include:

- identifying the need for any additional surface preparation work (critical for sprayed bituminous surfacings and asphalt surfacings)
- locating and preparing suitable stockpile site(s)
- identifying environmental requirements (including erosion and sedimentation controls)
- verifying the traffic control plan
- checking availability of safety equipment, properly trained personnel, plant and materials
- marking existing services
- addressing occupational health and safety issues (powerlines, overhanging trees, etc.)
- other conditions that may impinge on the works (local bylaws, time restrictions, fire bans, etc.)
- obtaining compliance certificates and test reports (materials supply, bituminous sprayer certificates, asphalt and concrete mix designs, etc.)
- undertaking trial pavement construction, where required (typical examples include stabilised pavements, asphalt pavements and concrete pavements)
- checking forecast weather conditions.

Operational activities during construction works should focus on factors that are critical to the safety and success of the operation and include:

- ensuring adequate signing of the works site is provided in accordance with the traffic management plan
- ensuring surface preparation works have been completed and are satisfactory
- marking out the works (earthworks limits, sprayer runs, asphalt milling areas, joints, etc.)
- checking weather conditions including pavement temperature, where appropriate
- managing the works activities (resource utilisation, material deliveries)
- checking conformance of placed materials (earthworks layer thicknesses, rolling patterns, compaction, binder and aggregate application spray rates)
- clean up of the work site and stockpile site(s)
- completion of daily work records.

Operational activities following completion of pavement construction works include:

- restoring stockpile sites
- correcting any non-conformities
- removal of all equipment and excess materials from the site
- collating and storing conformance records (test reports, survey reports, verification checklists)
- provision of as-constructed drawings.

**Figure 3.1: Aerial view of a typical rural construction project**



Source: RTA

## 4. Earthworks

### 4.1 Introduction

This section discusses practices relevant to earthworks construction, including:

- earthworks materials
- setting out for earthworks construction
- foundation preparation
- embankment construction
- compaction
- levelling and trimming operations.

It is acknowledged that VicRoads Technical Bulletin 39 (VicRoads 1998) and Centre for Pavement Engineering Education course notes (CPEE 2004) were useful sources of information in the preparation of this section.

### 4.2 Earthworks Materials

#### 4.2.1 Types of Earthworks Materials

The proper placing and consolidation of the materials forming an embankment are essential if the embankment is to retain its shape, height and stability. In order that an embankment can be constructed to achieve these objectives it is frequently necessary to be selective in the materials placed in the embankment.

The *Guide to Pavement Technology – Part 4I: Earthworks Materials* (Austroads 2009d) sets out the requirements for earthworks materials, however, a brief introduction to this topic follows.

#### **Sources of earthworks materials**

The main sources of earthworks materials are cuttings within the road alignment, soil and gravel pits, river beds and rock quarries. Their use for providing earthworks materials is evaluated by test and inspection. These tests determine the quality of the material, whether the material can be used in its natural state or will need processing to improve the properties and/or grading to meet the specified requirements, and whether sufficient quantities are available to meet the project requirements.

From this data the planning associated with the following operations and their costs can be evaluated:

- the methods to be used and the plant and equipment required to mine or win the source materials
- the processing methods and the associated plant and equipment required to produce the correct quality of material
- the means of transporting the material to the site and the determination of the route in relation to accessibility and distance from the material source.

Earthworks materials may include rock either in its natural state or processed. The sources are wide ranging as follows:

- cuttings within the road alignment
- pit ridge creek or water-worn gravels

- crusher rock mixed with soil binders
- decomposed granite, basalt or other igneous rock
- sedimentary or metamorphic rocks, e.g. sandstone
- fine grained materials, e.g. loam, sandy clay mixes.

Earthworks materials are normally classified into unsuitable and suitable.

### **Unsuitable materials**

Unsuitable materials are not capable of being compacted for use in the construction of embankments and need to be removed to spoil or used in non-critical areas. Such materials may include:

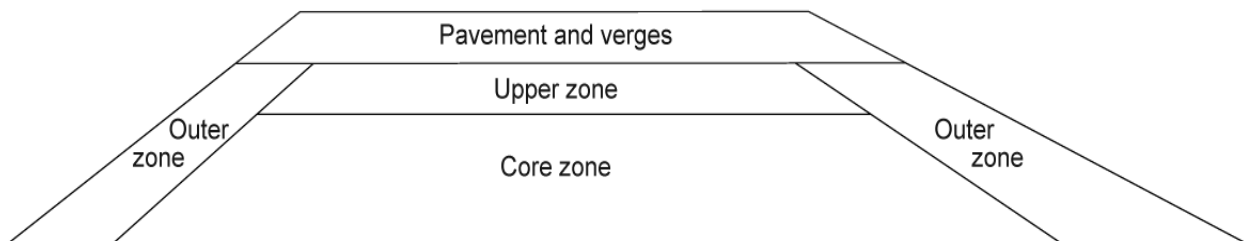
- organic soils, peats and vegetation and scrub
- contaminated materials that contain toxic and soluble substances that are not environmentally acceptable and could pollute the drainage water
- fill materials that contain demolition waste such as timber, plastics, metals and other deleterious materials
- fine-grained materials such as silts, highly expansive clays and other materials that undergo property changes when exposed to moisture such as weak sedimentary rocks, claystone, siltstone, mudstone and weakly cemented shales
- oversize materials
- water saturated soils.

### **Suitable materials**

Suitable materials are capable of being compacted for use in the construction of embankments and include most naturally occurring non-organic materials. However, the application of these materials may need consideration to ensure the stability of the structure, such as over-wet materials need to be aerated and dried out prior to placement. This could place a constraint on their use if the drying period goes beyond the programmed construction period.

Highly plastic reactive clays are prone to volumetric change with changes in moisture content and/or density. These materials may be deemed as suitable for use in embankments if placed under strict moisture and density control and only placed at lower levels and/or within the core region of the embankment (Figure 4.1).

**Figure 4.1: Zoned embankment**



Source: ARRB

### **4.2.2 Winning Earthworks Materials**

Earthworks materials are typically water worn or rounded aggregates and, in their natural state, may be suitable for embankment construction. However, where high strength materials are required they will have to be crushed to provide angular and cubic shaped aggregates.

The methods used to win earthworks materials depend on a number of factors, such as the depth and thickness of the deposit, whether different layers have to be mixed, irregularity of the layers in respect to quality of material, and the plant required to undertake the task.

Earthworks materials may be delivered to the job site with or without mixing operations being undertaken at the source.

### ***Operations requiring no mixing***

Materials which require no mixing are normally excavated by a bulldozer and pushed into windrows to be loaded by a front-end loader into trucks. If the haulage distance is less than 3 km, these materials may feasibly be excavated by scrapers and hauled to the job. For deep deposits, greater than 1 m below the surface, excavators may be used, loading directly into the trucks.

### ***Operations requiring mixing***

Full depth mixing of shallow deposits (up to 1 m deep) can be undertaken in a number of ways including:

- Bulldozer – the dozer, first cutting a slot through the varying layers to be mixed for the full length to be worked, the material is then excavated from one exposed slot face by the blade whilst mixing the material by rolling and pushing it into stockpiles at either end of the pit for loading into trucks.
- Bulldozer and grader – the dozer excavates the material into windrows and the grader mixes the materials on the pit floor. Water and/or additives can also be added during this operation if required (the methods of adding additives are discussed in Section 7). The material is then loaded from the windrows into the trucks, or picked up by scraper for delivery to the site.
- Bulldozer, loader and pugmill (this process is only used where water or additives are added to the mixed materials) – the dozer excavates and pushes the material into a stockpile which is then fed into a bin feeder, normally fitted with a grizzly to remove oversize material, by a loader, then via a conveyor into the pugmill, where it is mixed and water added if necessary to achieve optimum moisture content (OMC). Dry additives, if required, are fed in the correct proportions onto the conveyor prior to the mixing operation. Where water and liquid additives are added they are sprayed directly on to the material by a spraybar fitted in the pugmill box. The materials are then loaded from the pugmill directly into trucks and delivered to site.
- Excavator shovel – when deep deposits are being excavated from a full face and the various layers are reasonably uniform the shovel can mix the material as it works the face, prior to loading into trucks.

Deposits too deep to be worked from a full face by an excavator shovel may need to be won by dozing (or scraping) from an inclined pit floor so that each blade (or bowl) full is won from the full length of the inclined floor. If mixing is not adequate, cross mixing should be undertaken.

One way of cross mixing, especially with large deposits, is by using a scraper to work the different layers successively into a stockpile. (All loads must be spread in one direction.) A dozer works at right angles to the direction of the spreading, then moves the stockpile horizontally rolling the material on the blade to mix the layers in the desired proportions.

Alternatively, the materials may be mixed as detailed in the last two methods recommended for mixing shallow deposits. Otherwise, it may be more practical to haul the materials to the job, spread them in the correct proportions and mix them by using mix-in-place equipment or a grader.

Soft deposits typically may be won directly from the cutting/pit face or floor if a loader or scraper is powerful enough and has an effective breakout force to undertake the task.

Hard deposit materials typically need to be ripped first prior to dozing into a stockpile. Care should be taken not to rip too deeply so as to prevent incorporating unsuitable material.

### 4.2.3 Processing Earthworks Materials

Earthworks materials may have grading or material deficiencies that do not conform to the specified requirements. Common deficiencies in grading include:

- oversize particles
- deficiency in fines
- excess of fines
- excessively gap-graded mixes.

There are certain processes that can be carried out in order to rectify these deficiencies including:

- Oversize particles – removed by rock rake, grizzlies, portable screens, or fixed screens associated with mixing plant.
- Deficiency in fines – the treatment to overcome this problem consists of uniformly blending in loam, crusher dust, sand or fine gravels which are added and mixed on the cutting or pit floor or on the job site by grader or rotary mixer. Alternatively, adding them during the mixing or crushing operation prior to the screening plant or pugmill is also satisfactory.
- Excess of fines – removed by fixed or portable screens. Changing the grading by adding coarse material by either blending in the pugmill or by grader or rotary mixing at the job site is one alternative method.
- Excessively gap-graded mixes – where this situation occurs it is very difficult to correct by blending in materials by a grader or rotary mixer on site. It is more economic to incorporate the corrective material using screens and a pugmill.

Any material that has a high plasticity index (PI) should be investigated and assessed for swell potential. The stone should also be subjected to a soundness test. If this test indicates that decomposition in service is unlikely to be excessive over the service life, the method of treatment will be dictated largely by the quality and quantity of the fines, as well as the availability of corrective material. Treatment to correct PI can be undertaken in a number of ways:

- short of fines – blend in non plastic fines, sandy loams or crusher dust
- large plastic fines fraction – scalp out some of these fines prior to incorporating the corrective non plastic materials
- add lime to the material
- remove and replace materials.

In situations where a material lacks cohesion, such as gravels that are too sandy treatment by the addition of a soil binder may be appropriate provided it does not produce an excess of fines.

### 4.2.4 Delivery of Earthworks Materials

#### ***Delivery of material***

Once earthworks materials have been processed, if required, they are then delivered to the job site (Figure 4.2). Earthworks materials may be delivered in two ways:

- directly to the road or mechanical spreader for immediate spreading
- to stockpiles for temporary storage.



**Figure 4.2: Dump truck delivering fill material**



Source: RTA

### ***Loading operations***

Most loading operations are undertaken by a front-end loader. The productive output of a loader is the function of its ability to dig, manoeuvre and load in the shortest time possible. The size and type of loader will depend on the material being handled, the required output and the ground conditions. Pneumatic rubber-tyre front-end loaders are preferred due to their mobility; however, they require good ground conditions on which to operate. Where the ground surface is hard rock or soft clays tracked-tyre loaders may be necessary; alternatively the pneumatic-tyre loader is fitted with chains to maintain traction.

Where hot climatic and/or windy conditions prevail, the trucks delivering the material should be covered to prevent moisture loss in transit, and for environmental reasons.

### ***Delivery to a grader or rotary mixer***

Earthworks materials, when delivered, should not be dumped into heaps on the area to be placed, as this creates initial difficulties, and prolongs the spreading activity. All activities that involve handling the material should be kept to a minimum in order to reduce segregation. The material should be placed in even layers or windrows by running out from a delivery truck fitted with tailgate control. This method can only be undertaken with short wheelbase trucks and free flowing materials.

An alternative method is to use a spreading device, such as a box spreader, where the layer thickness can be controlled through an adjustable gate. In all cases the tipping needs to be controlled by a person on site to ensure uniformity of the tipped layer or windrow.

Where trucks and trailers deliver material to site there may be no alternative other than to dump.

### ***Stockpiling at the job site***

To ensure the continuity of paving operations, it may be necessary to stockpile material at the job site. In this event, the following points should be observed:

- Stockpiles should be located on hard, level ground.
- They should not obscure the vision of the road user.

- They should not provide any obstruction to construction operations, and their height should be restricted to 1-3 m.
- If high stockpiles are required the material should be placed in layers 1-3 m high and with edges of each layer stepped.
- For safety reasons, they should not be placed under power lines.
- The area should be stripped of all vegetable matter and unwanted materials and properly drained. It may be necessary to gravel the stockpile site, or waste the bottom 100 mm, which may become contaminated.

### **4.3 Setting Out for Earthworks Construction**

Setting out control pegs along the edge of the earthworks area enables the layer level, shape and course thickness to be regulated. The transfer of these controls is a function of the methods of construction to be used.

Before the earthworks construction begins, a control line should be established. In moderate size jobs, this is done by transferring measurements back from offset pegs. For larger work this may be undertaken by surveyors using more advanced techniques, such as total stations or global positioning system (GPS) equipment.

After the establishment of a control line, typically offset pegs are placed along the edge of the earthworks area, at specified spacing according to the work being undertaken. On smaller projects the two most common methods for controlling trimming using these offset pegs are by stringline or trim pegs. Use of laser and crossfall indicators and more sophisticated options with automatic laser or GPS control systems provide a higher level of accuracy and are typically used on larger projects to control the levelling operations of graders and other finishing machines.

#### **4.3.1 Manual Control**

##### ***Trim pegs***

Where the materials are to be placed by graders without any form of automatic control, trim pegs can be set at each chainage or offset point. The level of the top of the peg is set at the required level of the finished surface and the grader operator trims to these levels. This method however, requires another person to check the level of the pegs during construction and to check the graded surface between pegs using a straight edge.

##### ***Parallel stringlines***

The other common manual method is the use of a stringline strung tightly across the earthworks surface at a fixed height. This method sets the stringlines parallel to the crossfall and provides only one offset distance down to the surface of the placed layer. Although this method takes more time to set up, the convenience of one measurement is an advantage.

##### ***Laser measurement***

The trim peg and parallel stringline methods are now increasingly being replaced by the use of a laser unit and target. Measurements are taken by directing a laser beam from a laser unit to a target staff placed at various points. The staff beeps when the reflector is in direct line with the laser, and the level is then compared with a datum peg. A field controller directs the operator either verbally or by marking the points on the ground. This method is by far the most common method of level control for small to medium size road construction projects.

#### **4.3.2 Automatic Control**

The alternative to manual control is the use of automatic controls that are fitted to the grader and mechanical spreader.

The grader blade and mechanical paver screed can be controlled automatically by a number of systems. The most common are the stringline and sensor arm, ultrasonic transmitters and receivers and GPS.

##### ***Stringline and sensor***

The basic purpose of the stringline is to provide an accurate, parallel reference for grade and steering control of automated fine grading and paving equipment. A stringline set-up consists of a tensioned line suspended in notches on the end of steel rods attached to metal stakes. Alternatively the line is suspended in notches on the top of adjustable spikes. Because of their non-stretch properties, polythene or steel lines are used as stringlines.

Sensor devices can also utilise other datum surfaces such as kerb and channel, or the previous laid surface.

##### ***Laser levelling***

Levelling by the use of the laser or other electronic controls is undertaken in a number of ways, all of which require some form of detector fitted to the blade of the grader which can be controlled either by a laser transmitter or a control stringline or fixed reference. Once the devices are operable they assist the operator to trim to the set levels compared to a reference or datum.

The basic laser unit has a transmitter set up on a tripod and a receiver fixed either to the blade or paver control box. The laser is set to the required level then the grader blade or paver screed is set to a known level. The detector is adjusted to intersect the laser beam, and the machine is now on line. The machine is then driven and the levels are controlled by the laser unit. The laser can be brought off line when the machine turns or performs another operation, such as a grader getting rid of surplus material.

##### ***Levelling by the use of GPS***

Levelling by the use of GPS is increasingly being used on larger projects and operates in a similar manner to laser levelling without the need for controller units to be established in the field. GPS equipment may be fitted to graders, dozers and automatic spreaders and is typically controlled from total stations or central base stations.

#### **4.4 Preparation for Earthworks Construction**

Preparation for earthworks embankment construction typically incorporates clearing and grubbing of the site, which precedes all other earthworks operations (Figure 4.3). In a rural environment this generally involves removing all organic materials, vegetation, trees and scrub from the alignment and from fill material, since if allowed to remain it may decay and leave weakness planes and voids that will eventually result in settlement. Topsoil and other materials of an organic nature may be stockpiled in designated areas for reuse for better revegetation. Preparation through built up areas may present entirely different problems such as existing superstructure and surface obstructions, and both above and below ground utility services, which must be removed or relocated prior to commencement of earthworks construction.

**Figure 4.3: Clearing of earthworks alignment**



Source: RTA

Once clearing and grubbing operations have been completed the survey pegs can be re-established and earthworks foundation prepared as follows:

- All holes and depressions resulting from the site clearing operation should be backfilled with a similar material used for the embankment construction and compacted in layers not exceeding 150-200 mm.
- The site should be compacted to the standards set for the embankment construction to a depth of at least 150-200 mm before embankment construction commences. This is normally undertaken by tyning the stripped area and then compacting the loosened material. To ensure the specified compaction is achieved the moisture content of the soil should be close to OMC, otherwise it will require drying out if too wet or water to be added if less than optimum. The depth of the tyning should not be greater than that which can be compacted. The degree of compaction should be consistent with that required by the fill material.
- On sideling ground (longitudinal cut/fill zones) a series of horizontal benches will need to be prepared to found the embankment and prevent slippage occurring.
- The area should be tested by proof rolling with a rubber-tyred roller or loaded truck and any unsuitable areas identified and treated.

#### **4.4.1 Treatment of Unsuitable Material**

Where the natural soils are deemed to be unsuitable such as soft clays, silts and peats, which have high compressibility and low strength, major treatments other than the conventional operation just described are required. Treatments for unsuitable material, where it is present beneath a proposed earthworks embankment or pavement, include:

- removal and replacement of unsuitable materials
- surcharging or consolidation settlement
- vertical drains
- geotextiles, geofabrics and geogrids
- drainage
- treatment in situ.

### **Removal and replacement of unsuitable materials**

Removal and replacement of unsuitable materials is a common method where unsuitable material is shallow, up to 2 m thick. The unsuitable material is excavated down to a stable base (Figure 4.4) and removed from the site and replaced with a selected material. For isolated unsuitable areas it may not be necessary to remove the entire unsuitable material thickness and removing sufficient thickness in order to construct a stable working platform may be acceptable.

**Figure 4.4: Excavation of unsuitable material**



Source: RTA

### **Surcharging or consolidation settlement**

Surcharging or consolidation settlement is a common method for treating soft soils which involves surcharging the embankment during construction to induce consolidation settlement of the fill and underlying material which takes place over a shorter period of time. The added mass of the over-height embankment accelerates the displacement of pore water and promotes consolidation/settlement of the underlying soils. However, care must be taken when placing the fill material to ensure that the loading does not exceed the shearing strength of the natural soil. Such treatments should only be carried out after rigorous geotechnical site investigation and design as catastrophic failure of the embankment may ensue if safe design parameters are exceeded. The time required for the desired consolidation settlement to occur may exceed 12 months hence this contingency must be allowed for in the works program.

### **Vertical drains**

Vertical sand or wick drains allow the rapid consolidation of thick, unconsolidated sediments liable to consolidate when loaded. These drains are either constructed or driven down through the soft natural soil. A horizontal drainage blanket extending through to the side slopes of the fill material is constructed prior to the placement of the embankment to allow for drainage of the displaced subsurface waters. As the embankment is constructed the pressure induced by the fill forces the surplus water to flow from the natural soil allowing consolidation to take place more quickly.

### **Geotextiles and geofabrics**

Geotextiles and geofabrics are used in various applications for the treatment of unsuitable materials. They may be used as a separation layer to prevent contamination of fill materials, drainage materials (Figure 4.5) or rock fill by soft soils, or as a reinforcement layer which reduces distortion at the soil/fill interface during the spreading and placement of the initial fill layers.

**Figure 4.5: Geotextile as separation layer above and below drainage layer**



Source: RTA

### **Drainage**

Where wet conditions occur under the embankment it may be possible to intercept the groundwater flow by installing deep subsurface drains, intercepting drains and/or culverts to reduce the moisture content and flow in the soil beneath the road. For the effective interception and drainage of groundwater in flat-lying terrain comprised of fine-grained and moisture susceptible clayey silt soils, it is generally necessary to install such drainage well in advance of earthworks to allow time for the lowering of the groundwater table in soils which are typically of relatively low permeability.

### **Treated in situ**

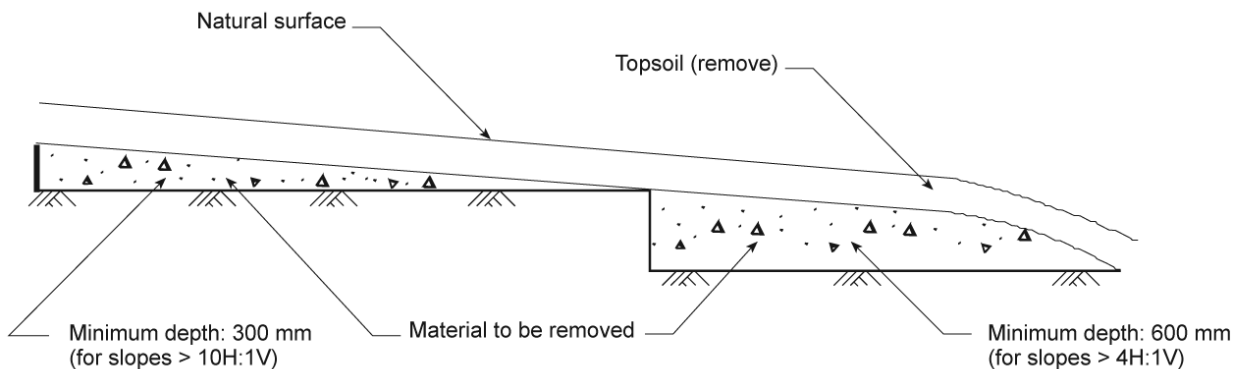
In some situations, unsuitable materials can be stabilised in situ by the addition of cementitious materials which can be used to dry back over-wet materials and/or to increase their bearing capacity such that the overlaying layers can be readily spread and compacted. Refer to Section 7 for more information on stabilisation of earthworks materials.

#### 4.4.2 Shear Key Requirements for Fills in Longitudinal Cut to Fill Zones

When constructing earthworks embankments in sideling (hillside) country, it is necessary to ensure that adequate keying of the embankment into the in situ material is accomplished to ensure the long-term stability of the embankment. This is particularly important in side cut and fill areas, where it is difficult to adequately compact the side cast material onto a steep side slope. Benching of the natural slope should be undertaken for slopes steeper than 3:1 (H:V). The removal of in situ material from below the capping layer/selected material and pavement over the full width of the formation, typically 450 mm to 600 mm in depth, is essential in these situations.

Standard drawings are available from road authorities that provide guidance on the design and construction methods to be used for fill embankments on sloping ground. Figure 4.6 provides an example cross section for the benching of an earth slope. A similar benched construction method is used for the widening of earth embankments.

**Figure 4.6: Example cross-section for benching of earth embankments**



Source: RTA (2007d)

#### 4.4.3 Cut Batter Stability

In the excavation of a cutting, an appropriate geotechnical investigation and analysis should be carried out to provide for a design slope with an appropriate factor of safety. Notwithstanding such investigation and design, progressive excavation of a cutting may reveal latent and undesirable features that were not identified during the geotechnical investigation. Features such as significant water inflow, highly fissured soils, significant clay seams in bedded rock, differential rock weathering, and undesirable orientation of discontinuities and shear zones in rock, can lead to immediate or future instability of the batter slope.

Any falls or slips of material that occur from the face should be closely inspected. If the immediate stability and safety of the slope is not compromised, such slips and falls should be removed and the area treated to prevent recurrence. If any area on cut batters becomes unstable or unsafe, suitable measures must be installed to restrict access to the area, e.g. the erection of warning signs and fencing. The affected area should be inspected and assessed by a geotechnical engineer, and made safe prior to excavation proceeding in the affected area. In the worst case, it may be necessary to redesign and flatten the cut batter slope, or to install restraining measures such as toe retaining walls, rock bolts, dowels, or cable anchors.

#### 4.4.4 Treatment of Cuttings at Cut Floor Level

Conditions at cut floor level can be variable depending on the geology and weathering grade of the materials present. Where the material at cut floor level is deemed to satisfy the earthworks specification and the project subgrade design California Bearing Ratio (CBR) requirements removal of material below the floor level is not required. In this case, the surface should be loosened to a depth of 150 mm and compacted to meet the specified requirements. However, if material at the cut floor level does not satisfy the earthworks specification and the project subgrade design CBR requirements then it will need to be removed and replaced or treated in situ as described in Section 4.4.1.

If the rock in the floor of the cut cannot be trimmed to the specified tolerances, the rocky material should be ripped to a minimum depth of 150 mm, loosened and broken such that the maximum particle size does not exceed 50 mm. Rocks or boulders larger than 50 mm should be removed and all resulting depressions backfilled with a suitable granular material (maximum particle size < 50 mm) and such backfilling, together with the loosened rocky material, must be reworked and compacted to the specified requirements.

If the rock in the floor of the cut is consistently of medium to high strength and cannot be readily ripped, the rock section may be allowed to remain subject to all loose materials and clay filled joints being removed and all resulting depressions being filled with a fully bound cement treated crushed rock or sand typically having a minimum cement content of 3% (by mass). In such cases it is usual to add a 150 mm crushed rock layer above the floor of the cutting. The cutting surface should be constructed at a sufficient grade to ensure that no water ponding occurs.

In the case of cut floors sited on jointed rock, care is required to ensure that groundwater inflows do not compromise the integrity of the earthworks and pavement layers. Such inflows are typical if there is significant catchment and water intake on the upslope side of the cutting. Remedial treatments may require the placement of a drainage blanket over the floor of the cut area affected. In some cases, the installation of cut off drains may remedy the situation.

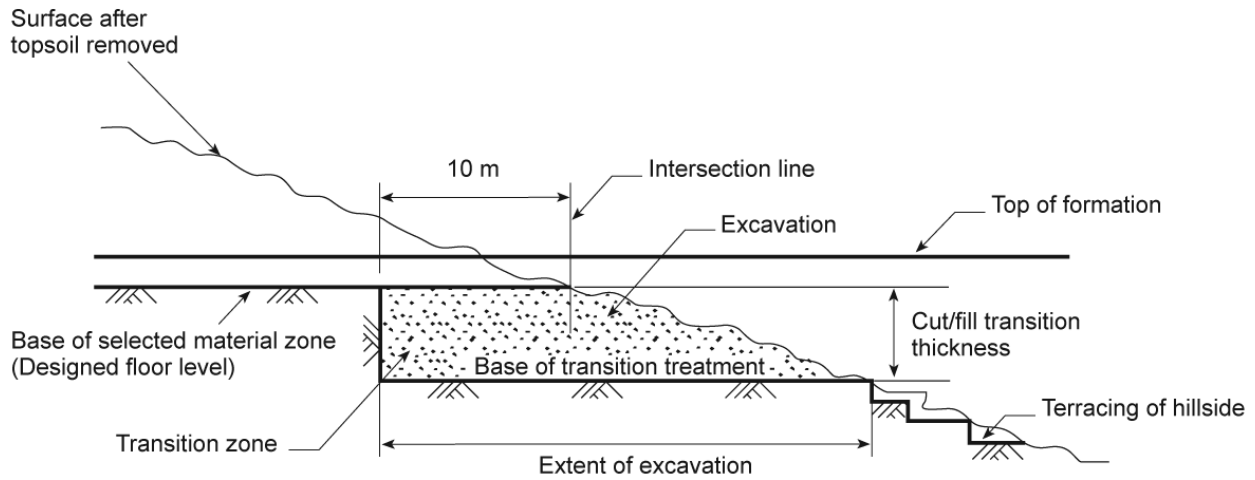
#### 4.4.5 Treatment of Cut to Fill Zones

The cut to fill zone is an area of differing materials and moisture conditions. The direction of the interface between the cut and fill areas may vary from being transverse to the centre line to being parallel to the centre line. To ensure consistency of foundation support at the cut to fill zone of earthworks, a bench should be excavated to remove all in situ material in the vicinity of the cut/fill interface to a minimum depth below the formation. The minimum depth required differs between road authorities though typically is 450 mm to 600 mm. The bench should be excavated for the full width to be occupied by pavement and verge materials, and for a distance of not less than typically 10 m to 15 m into the cut and not more than 30 m in the fill from the cut-fill line at the design top of the earthworks. Subsurface drainage will be required in most cut to fill zones and Figure 4.7 illustrates a typical treatment.

In sideling cut areas, the cut should be excavated to a depth typically 450 mm to 600 mm below the formation for the full cut width to be occupied by pavement and verge material. The excavation should be backfilled with material of a quality at least equivalent to the adjacent fill material to ensure uniformity of material strength in this zone.



Figure 4.7: Treatment of cut to fill zones



Source: Adapted from RTA (2007d)

## 4.5 Embankment Construction

### 4.5.1 In Situ Density and Bulking of Materials on Excavation

For the estimation of earthworks volumes it is necessary to have some knowledge of the bulking and compacted density factors for different earthworks materials. Table 4.1 provides indicative densities and volume change factors for a number of earthworks materials. Table 4.2 provides indicative loss factors that take into account site and environmental factors.

Table 4.1: Indicative densities and volume change factors

Materials	Density (t/m <sup>3</sup> )			Volume change factors		
	In situ	Loose	Compacted (dry)	Load	Bulking	Compaction
				Loose to in situ	In situ to loose	Compacted to loose
<b>Broken rock</b>						
Basalt	2.9	2.0	2.4	0.70	1.45	1.20
Granite	2.6	1.6	2.2	0.60	1.60	1.40
Hornfels	2.7	1.6	2.3	0.60	1.70	1.45
Sandstone	2.7	1.6	2.2	0.60	1.70	1.40
Rhyolite	2.7	1.6	2.3	0.60	1.70	1.45
Scoria	Varies	-	1.4/1.9	-	-	-
Siltstone/mudstone	2.6	1.5	1.8/2.0	0.65	1.75	1.30
Limestone	2.6	1.6	2.2	0.60	1.60	1.40
<b>Other</b>						
Clay - basaltic - dry	1.7/2.0	1.2/1.4	1.9/2.3	0.70	1.45	1.60
- wet	2.0/2.5	1.4/1.7	2.3/2.6	0.70	1.45	1.60
Clay - sandy silty - dry	1.4/1.7	1.1/1.4	1.5/1.8	0.80	1.25	1.35
- wet	1.6/1.9	1.3/1.5	1.7/2.1	0.80	1.25	1.35
Dune sand - dry	1.6	1.4	1.7	0.90	1.15	1.20
Granitic sand	2.2	1.9	2.0/2.2	0.85	1.15	1.20
Gravel	2.2	1.9	2.0/2.3	0.90	1.10	1.25
Clay & gravel - dry	1.7	1.4	2.1	0.85	1.15	1.50
- wet	1.8	1.5	2.2	0.85	1.15	1.50
Sand & gravel - dry	1.9	1.7	1.9	0.90	1.10	1.20
- wet	2.2	2.0	2.4	0.90	1.10	1.20
75% rock / 25% soil	2.8	2.0	2.4	0.70	1.45	1.20
50% rock / 75% soil	2.3	1.7	2.1	0.75	1.35	1.25
25% rock / 75% soil	2.0	1.6	2.1	0.80	1.25	1.30
Topsoil	1.4	1.0	1.5	0.70	1.45	1.50

Note: Because of variability of natural materials this table should only be used as a guide.

Source: Adapted from VicRoads (1998)

Table 4.2: Indicative earthworks loss factors

Conditions	Two-lane two-way carriageways		Divided carriageways
	Flat to undulating country	Sideling and hilly country	
Normal earthworks in favourable weather conditions	5%	10%	10%
Special conditions (percentages applicable to be added to normal earthworks figures above)			
Blasted rock <sup>1</sup>	5 – 15%	5 – 15%	5 – 15%
Adverse wet weather - surface soils & clay - ripped or blasted rock	10 – 20% No additional allowance	10 – 20% No additional allowance	10 – 20% No additional allowance

Note

<sup>1</sup> The percentage loss depends on the amount of oversize material and availability of fine granular material for blinding the rock layer. The larger the oversize without suitable blinding material, the higher the percentage loss factor adopted.  
Source: VicRoads (1998)

The percentage loss associated with oversize material may be reduced by the use of rock fills, where permitted. This involves reprocessing of the material to a size suitable for use in rock fill.

#### 4.5.2 Earthfill Embankments

Earthfill embankments (Figure 4.8) are constructed in relatively thin layers of loose soil. Each layer is compacted to the specified density before the next layer is placed, and the fill is then constructed to the desired shape and height in a series of lifts.

**Figure 4.8: Embankment construction**



Source: RTA

The depth of the loose layer normally ranges between 150-300 mm before compaction commences. The material in each layer should be as uniform as possible, and differing materials should be laid in alternating layers. Where possible, high quality materials should be reserved for the upper layers of the embankment and as selected materials for areas such as cut/fill zones, bridge abutments and materials behind culverts and retaining walls.

The maximum particle size of any rock or lumps within the layer should not exceed two thirds of the uncompacted thickness depth. It is also good practice not to include rock or similar single size materials 150 mm diameter or greater in the top 300 mm of the embankment.

The three most common placement methods for earthworks materials are:

- direct dumping and spreading of the soil by scrapers in one operation with compaction progressively being undertaken as the fill material is placed
- hauling of borrowed materials by either off-highway or highway trucks and dumping the material in heaps or windrows (Figure 4.9) to be mixed and spread to a uniform thickness by either a bulldozer or grader and then compacted
- on short fills where hauls from cut to fill are not excessive (90 m), the bulldozer working alone can economically and conceivably form the embankment.

The end dumping of material by trucks without spreading is not normally permissible as the specified compaction cannot be achieved and settlement occurs. An exception to this is when the embankment foundation is not capable of supporting construction equipment. In such cases only sufficient material is placed to permit the passage of equipment. An alternative to this is to use geofabric, which is placed directly onto the soft foundation and covered with a thin layer of selected material or rock that permits construction traffic to operate over the area.

**Figure 4.9: Dumping of material in heaps to be spread**



Source: RTA

### 4.5.3 Slopes

Permanent cuttings and embankments are generally required to be stable during their lifetime. This stability can be achieved by excavating the material to stable batter slopes or by retaining them structurally, for example using retaining walls, reinforced soil walls or shotcreting. The stability of natural slopes is controlled largely by the:

- density and strength of the materials
- groundwater conditions and pore pressures
- strength and disposition of any discontinuities.

The safe slope angle required will depend on these features and the depth of the cut or height of the embankment. However, some materials have a high propensity to erode/scour and additional treatments may be required to stabilise these materials.

Provision of benches, typically sloped back into the slope face and linked to surface drainage, is common for large cuts and fills.

Progressive topsoiling and revegetation of embankment slopes reduces (Figure 4.10) the risk of surface scouring and improve the stability of newly constructed embankments.

Figure 4.10: Progressive revegetation of an embankment with grass and straw mulch



Source: RTA

Grasses, particularly those native to the area, turf and small trees are typically used to stabilise embankment slopes. Planting large trees near or on embankments, particularly embankments consisting of expansive spoils, should be avoided as they may cause differential moisture content within the embankment and differential settlement may occur.

#### 4.5.4 Fill at Structures

Careful control must be exercised over the use of vibratory compaction equipment in close proximity of structures. This is particularly important in the case of reinforced soil structures such as shown in Figure 4.11. The particle velocity generated by vibration transmitted through the ground by vibrating compaction equipment is the critical factor in causing damage to structures. The particle velocity and amplitude depend on the frequency and amplitude of vibration, distance from the source and the nature of the material through which the vibration is being propagated. The particle velocities at most locations can be reduced by reducing the frequency and amplitude of vibration or by moving the source further away.

Figure 4.11: Hand compaction adjacent to reinforced soil wall



Source: RTA

Fill materials that are to be compacted adjacent to structures must be placed in accordance with the relevant specification requirements. An example of specification requirements for compaction of fill material at structures is shown Table 4.3.

**Table 4.3: Compaction of fill material at structures**

Non-vibrating rollers – static weight <sup>1</sup> (tonne)	Vibrating rollers – total applied force <sup>2</sup> (kN)	Minimum distance from compaction plant to side of structures (m)	Minimum distance from compaction plant to abutments, retaining walls and wingwalls (m)	Minimum cover over top of culverts (m)
Less than 2	Less than 20	0.15	0.15	0.15
2 – 5	21 – 50	0.3	0.3	0.15
6 – 10	51 – 100	1.2	1.2	0.4
11 – 20	101 – 200	2.4	1.2	0.4
21 – 35	201 – 350	2.4 or height of structure (whichever is greater)	1.2 or height of structure (whichever is greater)	0.7

*Notes*

<sup>1</sup> Includes vibrating rollers operating in non-vibrating mode.

<sup>2</sup> Total applied force is the sum of the static weight and the vertical component of the centrifugal force.

Source: VicRoads (1998)

#### 4.5.5 Completion of Formation (Surface of Earthworks)

On completion of trimming and compaction of the formation construction of the pavement should commence immediately so that the formation is not open to damage either by the weather or construction traffic. If this is not possible, to protect the formation the surface should be left high and trimmed just prior to the construction of the pavement. Any damage to the formation should be repaired prior to construction of the pavement.

### 4.6 Geotextiles and Geogrids

#### 4.6.1 Types, Specification and Supply

The *Guide to Pavement Technology – Part 4G: Geotextiles and Geogrids* (Austroads 2009c) provides advice on the types and specification of geotextiles and geogrids.

#### 4.6.2 Handling and Storage

Site handling and storage practices should minimise exposure to conditions that may reduce or alter geotextile properties. Good site handling and storage practices include:

- Store geotextiles in rolls above the ground and under cover either in the manufacturer's packaging or in shade.
- Once laid on site, geotextiles should be covered as soon as possible and at least within five days for untreated UV susceptible geotextiles and 30 days for UV treated and low UV susceptible Geotextiles.
- Adequate material cover should be provided before construction plant is allowed to traffic the geotextile. A minimum cover of 150 mm minimum is recommended for firm foundations or 300 mm minimum for soft foundations.

- If a geotextile is contaminated during installation it should be removed and replaced. An example of this may be rainfall and erosion washing clay fines onto a geotextile during construction, which may cause blockage of the pores of an uncovered geotextile.
- Relevant occupational health and safety requirements.

#### 4.6.3 Robustness Requirements

Geotextiles will not achieve their purpose if they cannot withstand construction stresses. Austroads (2009c) gives details of the robustness and survivability requirements of geotextiles used for construction applications.

Actual ground conditions have an important bearing on geotextile selection. When constructing embankments on soft compressible foundations, advantage may be taken of an existing dry crust, vegetation, root systems and burrows (crab holes, etc.) which provide some existing support to applied loads. Advantage of such surface features may be realised provided disturbance is minimised. Usually hand clearing is undertaken, leaving roots undisturbed. A geotextile used to cover such uneven surfaces must be sufficiently robust to cope with these conditions, which may exceed the necessary strength criteria established by stability considerations.

Construction vehicles should be limited in size and weight such that total rutting including construction traffic and traffic post construction does not exceed that adopted for reinforcement. Where reinforcement is not a design consideration, rutting should not exceed 75 mm. It could also be an economic decision to use a more robust geotextile and so allow site access to larger equipment.

If large depressions such as ditches and shallow creek channels are found along the embankment alignment, strips of geotextile may be unrolled directly over the existing surface depressions. Fill material can then be placed on the geotextile to raise the local grade to the approximate level of the adjacent ground surface. The underlying geotextile layer should be taken back some 2 to 3 m along the existing formation on either side of the local depression to provide for anchorage once the embankment is constructed. The primary geotextile layer is then unrolled and construction continued as usual.

#### 4.6.4 Joining

Seam and overlap requirements should be specified along with the design properties for both factory and field seams. For reinforcement, it is preferable to avoid seams in the direction of the pavement, as structural support by geotextiles is typically provided in the cross traffic direction. In these cases, for sufficiently large projects, some manufacturers can produce panels of the correct width sewn in the factory which are shipped to site, thus avoiding longitudinal joints, and limiting the number of transverse joints required on site.

As seam efficiency is much less than the host fabric and overlaps are wasteful of material, it is beneficial to use the maximum roll width suitable for the project.

The use of seams rather than overlaps can be advantageous in certain geotextile applications and several methods for seaming exist including sewing, stapling, tying, heat bonding and gluing. Except for sewing, standards have not been developed for seaming techniques. In addition, the last two seam types have questionable durability. Regardless of the type of seam, the durability of the seam should be the same as required of the geotextile. Also, for reinforcement applications, longitudinal seam strengths should exceed, with an appropriate factor of safety, the design strength required in the direction across the seam. The geotextile strength may actually have to be greater than that specified in order to provide seam strength equal to the specified tensile strength.

### Overlap and pinning

Overlaps can be used to provide continuity between adjacent geotextile rolls, through frictional resistance between the overlaps, and sufficient overlap width is required to prevent soil from squeezing into covering material at the geotextile connection. The amount of overlap depends primarily on the soil conditions and the potential for equipment to rut the soil. If the ground will not rut under construction activities, only a minimum overlap sufficient to provide some pullout resistance is required. As the potential for rutting and squeezing of soil increases, the required overlap increases.

Where joints are required in geotextiles or geogrids, either longitudinally or at the end of rolls, a minimum of 300 mm overlap should be provided. In some cases, overlaps in excess of 900 mm may be required to obtain satisfactory performance. Actual overlap requirements will depend on the particular application. Table 4.4 provides guidance on overlap requirements (AASHTO 2006).

**Table 4.4: Minimum overlap requirements**

Subgrade CBR	Minimum overlap (mm)
> 3	300 – 450
1 – 3	600 – 1000
0.5 – 1	1000 or sewn
< 0.5	Sewn
All roll ends	1000 or sewn

To maintain relative position during construction activities, geotextiles may be stapled or pinned at the overlaps, using 250 to 300 mm long, 5 mm diameter nails and 30 mm washers, placed at a minimum of 15 m centres for parallel rolls and 1.5 m centres for roll ends.

### Sewing

The specification requirement for the sewing of geotextile joints varies between road authorities; however, it is recommended that the thread be a high strength polypropylene, polyethylene or Kevlar. The Department for Energy, Transport and Infrastructure (DTEI South Australia) adds an additional requirement that a 300 tex thread be used. Some authorities such as the US Federal Highways Administration and Washington State Department of Transportation only allow for sewn joints for construction using geotextiles, although when not used specifically for reinforcement, this is considered excessive.

In larger construction projects on soft ground, sewn joints are preferred to overlapping of adjoining rolls of geotextile due to the width of overlap and associated cost implications. Ideally, the thread used in the sewing machine should be of sufficient strength to provide sewn seam strength equal to or greater than the strength of the geotextile itself, but this is not achieved in practice.

A portable field sewing machine is used to sew geotextile seams in the field and there are several methods types of jointing seam which may be used including prayer seam, J seam and butterfly seam which may be with single or double threads. Prayer seams are more suited to non-woven geotextiles, and J and butterfly to woven geotextiles. Seam efficiency is usually found to be about 60% of the base geotextile.

### Geotextile repair

Geotextile repair procedures for damaged or disturbed geotextile sections (i.e. rips, tears, clogging) are typically detailed in job specifications or the manufacturer’s documentation. Such procedures should include overlap, seaming or replacement requirements.



#### 4.6.5 Placement Requirements

##### **General**

Geotextile placement procedures should be detailed in construction specifications and typically includes grading and ground clearance requirements, cover material specifications, lift thicknesses and equipment requirements. Design considerations such as removal of wrinkles and folds or pre-tensioning the geotextile are also typically specified. Any seams should be exposed with the seam up such that the seam can be inspected and repairs easily made should faulty seams be encountered. Other special considerations such as wind conditions and underwater placement should also be addressed.

Where reinforcement is the major function of a geotextile, after the site is prepared and the geotextile delivered, the geotextile should be unrolled transverse to the embankment alignment and field-sewn at the maximum widths which can be handled by construction personnel, unless the width of the roll is compatible with the embankment or pavement width in which case it can be rolled out in the longitudinal direction.

Where filtration and separation are the required functions, it is preferable to lay the geotextile in the longitudinal direction.

Where butt seams are used to join rolls, rolls of different lengths should be used to stagger the butt seams. Where two layers of geotextiles are specified to provide high strength reinforcement in two directions, the bottom layer of geotextile may be placed with seams in one direction and the top layer placed in a perpendicular direction relative to the first layer. Because of low friction between geotextiles, the second layer should be separated from the bottom layer by 300 mm of cover material.

##### **Temporary, haul and permanent roads**

Geotextile widths should be selected such that joins of parallel rolls occur only at the centreline or shoulder, and if reinforcement is a design consideration, joins on the centreline should also be avoided in favour of construction techniques outlined for embankment construction. Joins should not occur along anticipated main wheel path locations unless account is taken of the weakened strength of the geotextile. Where a geotextile is used for reinforcement or separation it needs to extend sufficiently beyond the wheel path area to provide anchorage against the geotextile slipping.

After laying of the geotextile on the prepared or otherwise natural surface, it is essential that the cover operations be consistent with the class of geotextile selected. The minimum cover requirements and ground pressure of construction equipment must be carefully considered.

##### **Embankments**

Conventional procedures for embankment construction after placement of a geotextile usually consist of placing fill material along the embankment centreline and spreading it toward the toes. If such procedures are followed on very soft foundations, adequate geotextile anchorage cannot be developed, and the attempt to build the embankment to the design height may result in a failure.

The importance of specifying proper construction procedures for building geotextile reinforced embankments on very soft foundations cannot be overemphasised. Often a specific construction sequence is required to obtain the desired embankment behaviour. Conventional embankment construction procedures on soft foundations may inhibit or prevent the use of equipment with poor mobility. Using low ground pressure equipment, a properly selected geotextile, and the proper procedures for placement of the fill materials can minimise these problems.

Experience has indicated that construction of thin (150 to 300 mm) granular working platforms prior to geotextile placement on very soft foundations may not be cost-effective, when compared to placement of a highly robust geotextile directly on the foundation (Haliburton et al. 1980). For embankments constructed on very soft foundations, construction procedures will usually propagate a mud wave, which would destroy any granular working platform. However, a low-strength, sacrificial geotextile may be used beneath a working platform, with a reinforcing geotextile placed over the platform. This may be a good alternative where stumps and sharp sticks or rocks that could endanger the performance of the reinforcing geotextile are not removed.

A cost analysis should be carried out to see whether or not a working platform is justified. As an upper limit, if a working platform is placed directly on the foundation, the need for a geotextile may even be eliminated.

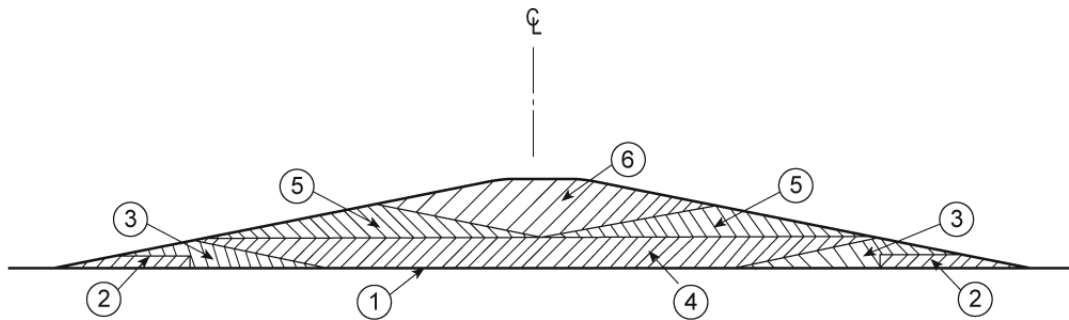
### ***Fill material placement, spreading, and compaction procedures for extremely soft foundations***

Sound geotechnical advice is required when working on soft ground to ensure the entire embankment does not slide or tilt on the soft ground conditions.

For extremely soft foundations, with CBR values less than 1, a conservative approach may be necessary in order to provide access to the site. The following remarks on placement, spreading, and compaction of the fill material apply to extremely soft foundations and may be the opposite of that used in construction without geotextiles. Figure 4.12 illustrates an example construction sequence for a geotextile reinforced embankment constructed over an extremely soft foundation, involving the following typical steps:

1. Lay geotextile of appropriate robustness in continuous strips transverse to the centreline of the embankment, and perform any necessary sewing.
2. Create haul or access roads along the embankment edges. The fill material in these locations serves to anchor the geotextile so that full tensile stresses are developed during subsequent construction phases. Place 150 to 300 mm of fill material over the initial geotextile layer using appropriate low ground pressure equipment. Fill material should be pushed out and wheel loads should not be allowed to run directly on the geotextile. Graders should not be used for this purpose unless fitted with a front push blade. Only lightly loaded trucks should be used at this stage.
3. A second geotextile layer may also be unrolled along each embankment toe parallel to the embankment alignment and an additional 150 to 300 mm of fill material placed over this second geotextile layer to make a double geotextile layer access haul road capable of sustaining fully loaded tandem axle dump trucks. If this stage is omitted, only lightly loaded trucks should be used for carting fill to Area 3 and 4 (Figure 4.13).
4. After the edge access/haul roads are constructed, fill material can be placed in Area 3 to complete the anchoring process using equipment as in step 2.
5. Fill material is then extended to Area 4 to set the geotextile, again limiting the construction equipment as in step 2.
6. Areas 5 are then filled which can be undertaken with heavier, more productive equipment if the material is front pushed. If bottom dump scrapers with centre blades are used, care must still be taken that the contact pressure is compatible with the fill height over the geotextile.
7. Finally, embankment fill material is placed on the centre section, Area 6, using the same considerations as Area 5, but heavily laden trucks can run on the filled Areas 5.

Figure 4.12: Example construction sequence for geotextile reinforced embankments



**Sequence of construction**

1. Lay geotextile in continuous transverse strips, sew strips together
2. End dump access roads
3. Construct outside sections to anchor geotextile
4. Construct interior section to 'set' geotextile
5. Construct intermediate sections to tension geotextile
6. Construct final centre section.

Source: FHWA 1998

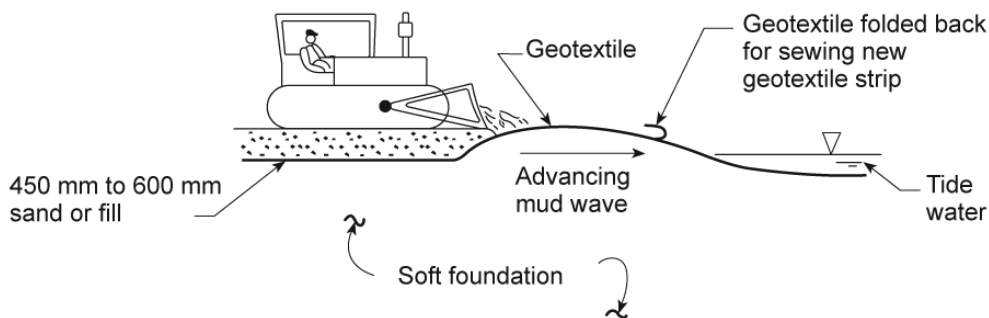
If larger trucks are used in the initial stages, the volume of fill material carried by each truck may have to be reduced to eliminate the possibility of a local bearing failure and damage to the geotextile. A minimum of 300 mm of fill material between the tyres and the geotextile should be maintained at all times. The height of the dumped fill should also be controlled to avoid local bearing capacity failures.

After the fill material is dumped, small dozer equipment and/or front end loaders may be used to spread the fill material. Typically, no additional compaction of the initial lifts is necessary as sufficient compaction can ordinarily be achieved by 'tracking in place' with the dozer or end loader. The use of heavy compaction equipment on the first lift should be avoided due to the risk of developing liquefied conditions in the natural surface. A minimum cover of between 150 and 300 mm should be maintained at all times between the spreading equipment and the geotextile.

Conventional compaction techniques can usually be resumed for the upper layers of an embankment after a good working platform, typically 600 mm to 1000 mm, is constructed above the geotextile.

In many soft foundation cases, geotextile placement is facilitated by 'working on the mud wave' as illustrated in Figure 4.13. However, geotextile layers should not be placed more than 4 to 6 m ahead of the mud wave as the geotextile or its seams may become overstressed.

Figure 4.13: Illustration of mud wave formation in a geotextile reinforced embankment



Source: Koerner (1986)

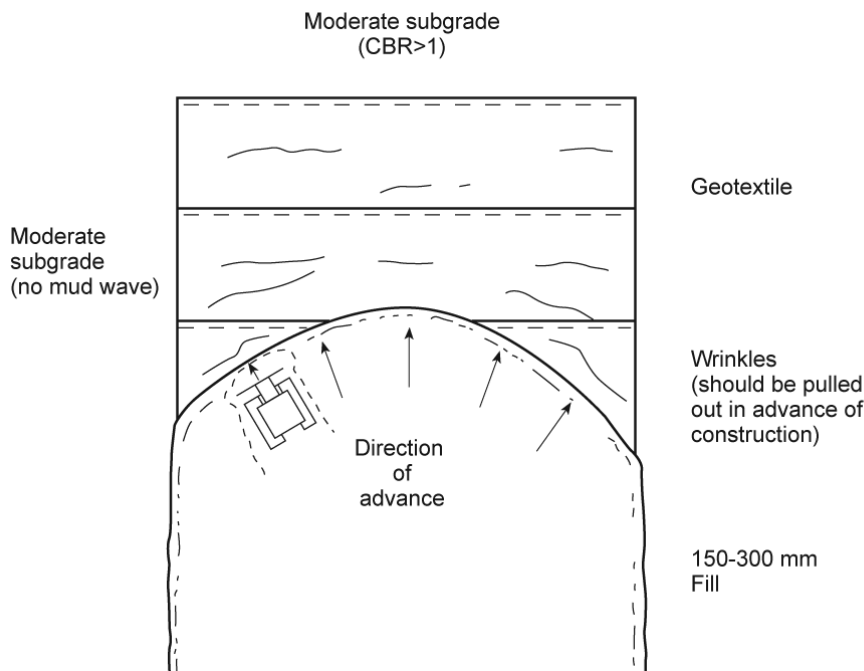
Care must be taken to ensure that construction proceeds along each embankment edge concurrently and that symmetrical fill material placement procedures are used. If the two sides are out of balance, then it is possible for the embankment to tip or slide laterally. Lateral sliding may also occur if a large ditch or channel adjacent to the alignment is encountered. In such instances, the lateral sliding stability of the embankment should be checked by conventional slope stability procedures.

**Fill material placement for moderate foundation conditions**

For less severe foundation conditions (CBR > 1), the construction procedures do not need to be nearly so restrictive. For example, larger and heavier dump trucks as well as higher contact pressure spreading and compaction equipment may be used.

For moderate foundation conditions geotextile should be placed with no wrinkles or folds and manually pulled taut prior to placement of fill material to ensure even tensile stress is developed in the geotextile. In these conditions, an inverted 'U' (convex outward) construction process as shown in Figure 4.14 has been found to work satisfactorily. Tension in the geotextile should be maintained as the fill material front moves outward.

**Figure 4.14: Procedure for placement of fill over geotextile for moderate foundation conditions**



Source: FHWA 1998

**4.6.6 Anchorage**

Anchorage of geotextiles is important if advantage is to be realised from mobilised tensile capacity, for example in foundation embankment and earth retaining reinforcement applications. It is also an important consideration for geotextiles laid on slopes to support short-term construction loading as well as the longer term application loading. Most suppliers of geotextiles provide acceptable anchorage guidance for different applications.

Anchorage is generally achieved by enveloping the ends or edge of the geotextile strip in stable earth, such as a backfilled trench or placed fill layers, to provide pullout resistance.

## 4.7 Levelling and Trimming Operations

Levelling and trimming an earthworks layer after initial compaction has been undertaken is usually accomplished by a grader (Figure 4.15) undertaking the following steps:

- The grader makes the first cut from the centreline and works to the shoulder.
- This procedure is worked up one side of the formation and down the other until the correct level is reached.
- A field offsider checks the profile and level pegs using a straight level or camber board. Alternatively, a target staff and laser optical line using a level or GPS equipment may be used.
- On completion the surface is wiped clean of windrowed material that would prevent rain water from draining away. The surface is then given a final pass or passes with the compactor to tighten the surface.

**Figure 4.15: Grader spreading earthworks material**



Source: RTA

The following should be adopted to minimise disturbance of the earthworks surface:

- Turning during trimming should be kept to a minimum. Plan to operate with passes as long as possible, 300 m or more, to ensure maximum production. If less than this distance, it is more economic to allow the grader to reverse. Also, the machine should not be turned whilst rippers or scarifiers are in the ground as this can damage both the tyne assemblies as well as the hydraulics that operate them.
- Ensure tyres are uniformly inflated to the manufacturer's specification to achieve uniform traction and level control.
- Select the speed that suits the operation and modify the cut to avoid unnecessary slip, tail sway or side slip.

The standard of finish that can be achieved is reliant on how the operator sets and uses the grader blade.

### ***Scarifying and ripping***

The other major operation in the preparation of the formation is to either scarify or rip a surface. This may be done for a number of reasons, such as to dry out wet soils or to open up dry soils so that water can be added.

The grader can be fitted with either a ripper or scarifier. A ripper is rear mounted and is used for ripping of reasonably hard ground. The number of tynes that can be used ranges from 1 to 4 depending on the power and capability of the grader. A scarifier is front-mounted and is for light ripping of soft ground, the number of tynes ranging from 1 to 13. In both cases the ripper or scarifier is lowered as the grader moves forward, to a depth that will break the ground as required or the grader is capable of. It should be noted that graders are only capable of light ripping and that heavy ripping will need to be undertaken by rippers fitted to heavy loaders or dozers.

There are also a range of other grader operations such as blade side shift enabling the blade to move around objects and pegs, blade angling and wheel leaning for drainage works, crabbing or articulating to allow the rills of loose material to pass to one side of the rear wheels so that the grader continues to operate on a level plane.

## 4.8 Compaction

Uncompacted materials are compressible and will compact/consolidate under loading over time, providing a tendency for moisture to be held in the material and consequently affecting its load bearing properties.

The objective of compaction is to improve a material's properties, in particular to increase its strength and bearing capacity, reduce compressibility and to decrease its ability to absorb water. The compaction process artificially densifies a material by pressing the particles together, expelling the air from the mass and filling the voids, thus making the material more dense, increasing its strength and hence resistance to rutting.

**Figure 4.16: Compaction of an earthworks layer**



Source: RTA

There are a significant number of construction factors that affect the degree and uniformity of compaction and the evenness of the surface of the compacted layer. These factors include:

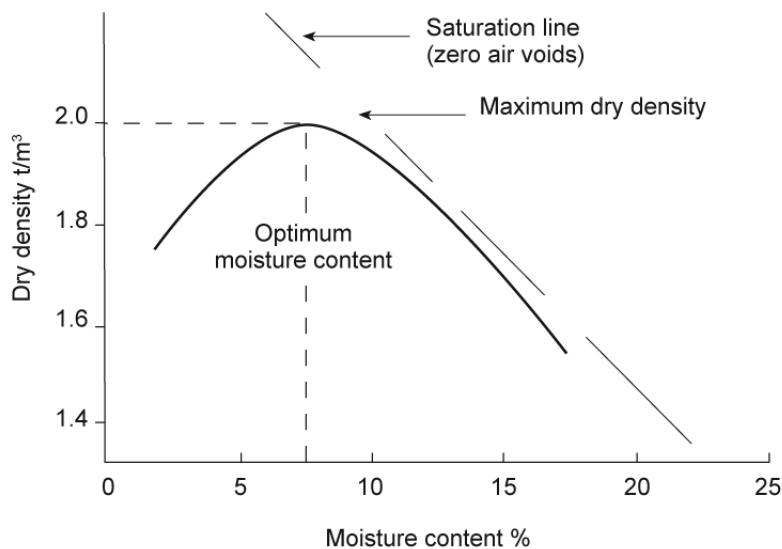
- control of moisture content
- layer thickness, type and uniformity of the material being compacted
- the combination and sequence of compaction equipment used, rolling patterns and number of passes
- environmental influences
- the condition of previously placed material and/or underlying material on which the layer is to be compacted
- supervision, testing and specified compaction levels.

### 4.8.1 Moisture Content

#### **Optimum moisture content**

If a soil is subject to a given amount of compaction over a range of moisture contents a curve similar to that shown in Figure 4.17 is obtained. It should be noted that, as the moisture content increases the soil becomes more workable. Higher densities and lower air voids are achieved until the soil reaches the maximum dry density for a particular moisture content. This value is known as the optimum moisture content (OMC). After this point, as the air content decreases, the air and water tend to keep the particles apart and thus the dry density reduces.

**Figure 4.17: Variation in dry density with moisture content at compaction**



Source: ARRB

The degree of compaction obtained in the field can be monitored by carrying out field tests to obtain the moisture content and in situ density and comparing them with the maximum dry density and optimum moisture content determined in the laboratory from samples of the material in question. The results of such testing are usually reported in terms of relative compaction, that being the recorded field density divided by the appropriate laboratory determined maximum dry density.

#### **Laboratory tests**

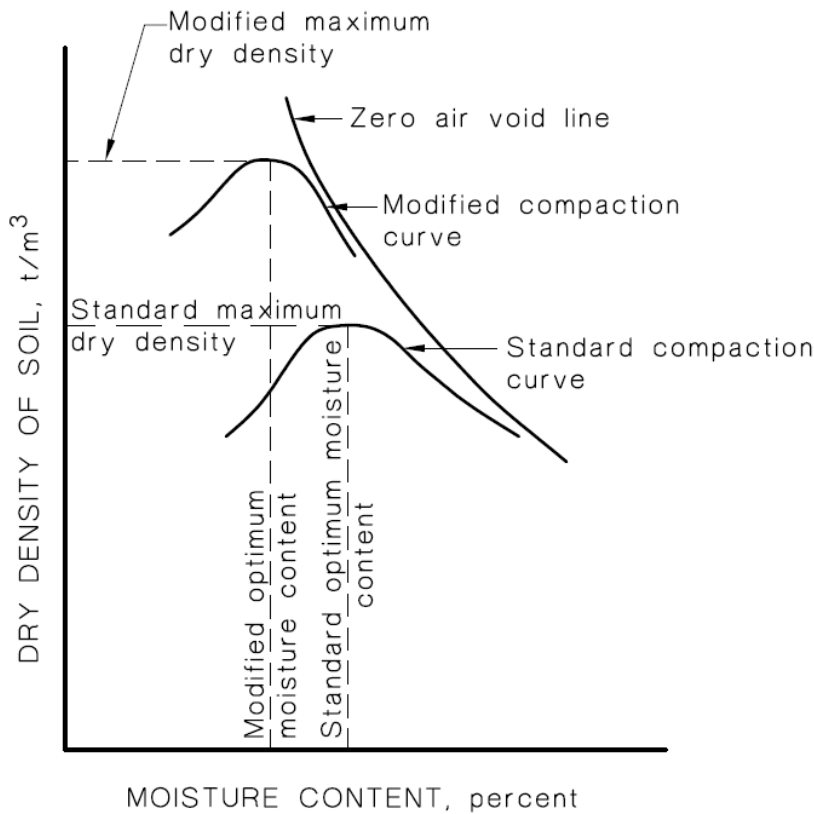
There are two tests that are used to obtain the optimum moisture content and the corresponding maximum dry density (MDD). Both tests use a falling hammer to compact the material in a mould, which roughly corresponds to the compactive effort in the field. The tests are known as:

- Standard compaction – AS 1289 5.1.1: Determination of the dry density or moisture content relation of a soil using standard compactive effort.
- Modified compaction – AS 1289 5.2.1: Determination of the dry density or moisture content relation of a soil using modified compactive effort.

The compactive effort for the standard compaction test is obtained by a 2.49 kg hammer falling 305 mm. The soil is compacted into a 101.6mm diameter mould in three layers with 25 blows of the hammer per layer. The test is repeated for a range of moisture contents, each time the mould is weighed and the bulk density is determined. Once the moisture contents have been calculated the graph of the dry density variation with moisture content is plotted. From this graph the OMC to obtain MDD can be determined.

As the weight and efficiency of compaction equipment increased it was found that densities in excess of the standard could be obtained. To take advantage of the increased compactive effort the modified compaction test was introduced. The modified compactive effort test is similar to the standard test using the same mould but applying a heavier hammer of 4.45 kg falling through 457 mm, with the soil packed in five layers each receiving 25 blows, thus resulting in an increase in compactive effort by over four times. This increase in compactive effort causes an increase in the maximum density and a corresponding decrease in moisture content.

**Figure 4.18: Standard and modified compaction curves**



Source: AS1289.0-2000

The moisture content of soils is another important factor as it affects the soil's stability, cohesion, swelling, and density.

The moisture content of a material is obtained from the test in AS 1289 2.2.1.

**Field moisture**

It should be recognised that maintaining the optimum moisture content in a cohesive material is less critical than in a non-cohesive material from which pavements are constructed. This can be demonstrated by considering the difference between the optimum moisture content of a pavement material and a natural soil material in relation to their respective grain size distribution. The optimum moisture content of a sand grain size 2 mm is in a moisture range of 4-5% whilst that of a clay grain size 0.002 mm can be 28-35%; thus the control of moisture in the construction of a pavement is more critical than in earthworks.

Further to this, there is a danger in over-compacting granular materials if they are too wet relative to the compactive effort applied. Therefore in practice, it is better to place granular materials just dry of optimum allowing for the full potential of heavy-duty compactors to be realised in providing a high strength layer.



Typically, specifications require earthworks layers to be compacted at moisture contents in the range of 60-90% of OMC for relatively non-plastic materials. However, the range specified may vary depending on the environment and material to be compacted. For example Queensland Department of Main Roads (QDMR) specifies compaction at 100-140% of OMC for highly plastic materials in high rainfall environments.

### ***Influence of moisture content***

Moisture testing is also associated with the determination of material types and their characteristics, such as the consistency values (i.e. liquid limit, plastic limit, linear shrinkage, plasticity index). It is important to know the moisture content of a material in order to assess its effect on some of the material's physical properties, such as cohesion and swelling, especially in relation to materials having a clay content, where:

- Cohesion is the mechanical cohesion between the fine fractions, which are bound by a film of moisture. The cohesive forces emanate from the surface tension at the water/air interfaces of the fines particles in the soil.
- Interaction occurs between the soil particles and water molecules.
- Swelling in clay soils is the physical effect associated with the amount of moisture present in the soil.

Cohesive materials used in the construction of earthworks, which include clays, should be compacted at the moisture content they possess in their natural state at depths unaffected by weather conditions, because with time these materials will ultimately return to their equilibrium moisture state. In practice this means that cohesive materials used in embankment construction:

- Are placed and compacted as soon after they have been excavated.
- The top of the fill is sealed by compacting the surface after each day's operations and trimmed to a crossfall to allow water to drain off the surface and not to penetrate the surface and saturate the material.
- On completion of the embankment to formation level, to prevent excessive drying out of the fill material and to maintain moisture stability, the pavement should be constructed as soon as possible. Alternatively, the formation should be left high of the finished level and trimmed back at a later stage, just prior to the construction of the pavement.

Granular materials should be compacted near OMC that will achieve maximum density in relation to the type of plant used. In practice this does not differ substantially from the natural moisture content of the material excavated above the watertable. The use of saturated materials, those excavated below the watertable, should be avoided unless the material has been dried sufficiently before being placed.

Where water is added it should be mixed in uniformly so as to achieve a moisture content at or close to the optimum or equilibrium moisture content. Where excavated materials are used they should be brought in at the equilibrium moisture content of the material being used and compacted before loss of moisture occurs.

## **4.8.2 Layer Thickness, Type and Uniformity of Material being Compacted**

### ***Layer thickness***

Material to be compacted is usually spread in layers in the range 150-300 mm (loose) but up to 500 mm is possible for some materials compacted with a heavy vibrating roller. The thickness of the layer to be compacted is a function of the roller weight and load per unit width of roll, type of soil, moisture content and specified compaction level. Most specifications nominate maximum layer thickness for different materials and locations within the formation. For example, a maximum layer thickness of 150 mm may be specified for base, subbase and upper earthworks layers while a maximum layer thickness of 300 mm may be for lower earthworks layers.

Compaction trials may be necessary, particularly in the case of highly plastic clays, to determine the maximum lift thickness, watering regime and rolling pattern required to ensure uniformity of moisture content and density to satisfy the specification requirements. Increases in layer thickness should be gradational and roller routines adjusted accordingly.

### ***Type and uniformity of material being compacted***

Earthworks in road construction rarely consist of exactly the same material type and different material types have different handling characteristics that require different compaction equipment. Some factors that affect compaction characteristics include (VicRoads 1998):

- Shear resistance – the ability of the material to resist an imposed load through friction and cohesive forces between the particles. Dry clay has very high shear resistance and therefore requires a heavy imposed load to rearrange and remould the particles, whereas a dry granular material has a low shear resistance and consequently is relatively easy to compact.
- Permeability – the rate at which water will flow through the material. Clay, if dried out is difficult to wet up again unless it is thoroughly worked and mixed.
- Volume stability – clay soils with high swell potential exhibit appreciable swelling or shrinkage with even small changes of moisture. In the longer term, to avoid distortion and loss of shape in pavements constructed over these materials, it is highly desirable to compact them in thin layers and at closely controlled moisture content. For highly plastic clays, layer thicknesses as little as 20 to 50 mm and a minimum acceptable characteristic moisture content value of 90% of standard OMC have sometimes been specified.
- Moisture content – excess moisture content does not allow the soil particles to bond and insufficient moisture content does not adequately lubricate the particles to enable them to easily remould during compaction. With respect to field compaction, while the laboratory determined OMC provides a useful guide, the required field moisture content to achieve efficient compaction may be quite different. The optimum field moisture content is best judged through field compaction trials.

In order for a given compaction process to result in uniform compaction the material being compacted should be well mixed and brought to a uniform moisture content before commencing compaction.

Non-cohesive fill materials, including sand and silt of very low plasticity, are difficult to compact and non-standard techniques, such as 'flooding' of poorly graded sands, are often employed in an effort to achieve the specified level of compaction. Where the only available earthworks material is non-plastic and non-cohesive, i.e. poorly graded dune sands, the application of foamed bitumen has been used to impart stability so that the pavement layers can be adequately compacted. The use of moisture sensitive silt as a fill material is generally not allowed and such material, along with over-wet material, is classified as unsuitable material and is allowed only in non-structural areas of earthworks, e.g. in fill batter widening, or landforms. The presence of over-size material in the form of cobbles and boulders can also inhibit effective compaction of a material, particularly if the largest particle dimension approaches or exceeds the compacted layer thickness.

A cohesive fill material aids in the production of a water-tight and regular surface as required of the platform onto which the pavement layers can be compacted. A cohesive material is also required for the construction of verges where a vertical verge/pavement interface must be freestanding for a period of time until the pavement layers are constructed. Cohesion is also required in applications where steep embankment slopes are specified.

Care must be taken not to produce excessive breakdown of coarser particles in granular materials as an increase in fines may promote deterioration in properties (e.g. lower CBR strength and increase in plasticity) leading to a non-conforming post-compaction product. Breakdown can be as a result of excessive working at the extraction site or on the roadbed. In particular, the need to rip and re-roll an already compacted layer can lead to a significant reduction in the bearing capacity of a weathered rock material.

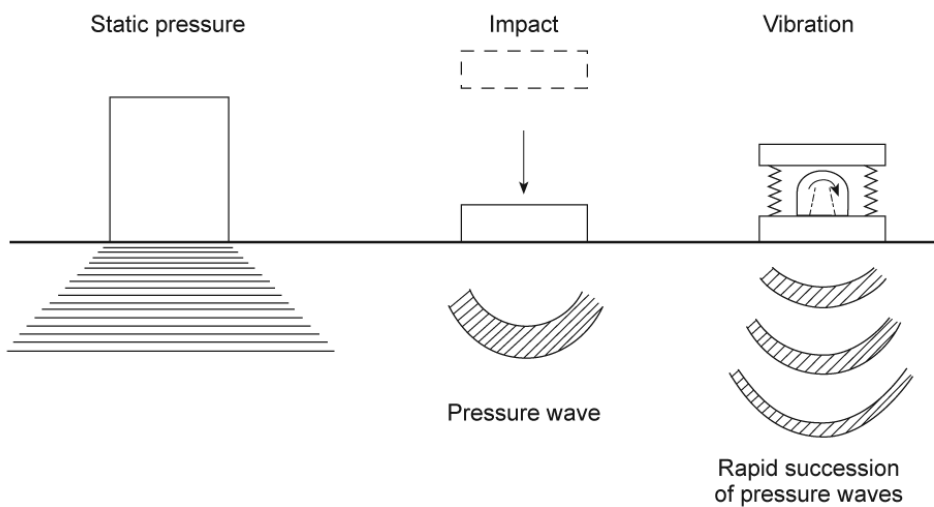
### 4.8.3 Compaction Equipment

#### **Modes of compaction**

The primary modes for the mechanical compaction of soils and granular materials are (Figure 4.19):

- static pressure
- impact force
- vibration.

**Figure 4.19: Modes of compaction**



Source: Adapted from Forssblad (1981)

Some compaction equipment combines several of the above efforts in their mode of operation.

Compaction plant that utilises these forms of compaction, either in combination or on their own, can be divided into five groups:

- Smooth drum, steel-wheeled rollers – static pressure.
- Pneumatic multi-tyred rollers – providing both a static pressure and kneading action.
- Oscillating and vibrating smooth drum, steel-wheeled and tamper rollers – combining both static and vibratory pressure.
- Padfoot, sheepsfoot and wedgefoot rollers – static pressure and impact action. These types of rollers can also be operated in a vibrating mode.
- Plate and rammers – impact pressures.

#### **Smooth drum, steel-wheeled rollers**

Smooth drum, steel-wheeled rollers apply their total weight as a static pressure through one or more drums.

There are basically two types of smooth drum roller: the tandem two axle and three drummed rollers. Both types are able to be ballasted to provide additional weight to the roller drums, the ballast being added as water in the rollers and in the body.

Smooth drum rollers are able to compact most materials. The main limitation is that they have a tendency to ride over high spots and leave adjacent areas uncompacted. They are more productive when used on prepared flat and uniform surfaces, which makes them effective finishing rollers.

### ***Pneumatic multi-tyred rollers***

Pneumatic multi-tyred rollers (Figure 4.20) compact the material by their static weight and kneading action through a flexible rubber tyre. The forward and rear sets of rubber tyre wheels are arranged so that the wheels on one axle follow the spaces between the tracks left by the other axle, thus compacting material under the full width of the machine.

**Figure 4.20: Pneumatic multi-tyre roller compacting fill material**



Source: RTA

There are differing types, both towed and self-propelled. The latter is the type most commonly used in earthworks and pavement construction; however, towed rollers may be used for proof rolling. The basic variance with the self-propelled rollers is in respect to the number of wheels, seven or eleven, and their disposition, weight and whether the wheels are rigid, whilst others are pivoted to follow surface irregularities. The size of pneumatic multi-tyred rollers used on pavement construction typically ranges between 10 and 25 tonnes fully ballasted, with tyre pressures in a range between 250 and 1050 kPa according to the ply of the tyres.

The static weight and the tyre pressure influence compaction and can be varied in two ways:

- the static weight increased by ballasting the body of the machine
- variation of the tyre pressure to provide a range of contact areas.

The ability to vary the tyre pressure and consequently the contact area and compactive effort in relation to the support capability of the material being compacted, is one of the main features of a pneumatic multi-tyred roller.

Considering this ability further, at the commencement of the compaction cycle the material is in a loose state and has little resistance to displacement allowing rutting to occur. Thus, to enable the material to support the roller at this stage without rutting and disturbance the tyre pressure is set low to provide a large contact area, allowing the wheel load to be spread creating a low compactive pressure. But as the material is compacted it gains strength and becomes capable of supporting the roller; the tyre pressure can be increased, resulting in a smaller contact area and the application of a higher compactive pressure. The tyre pressure can then be further adjusted as the conditions change until the specified density is achieved. Most pneumatic multi-tyred rollers have the ability to adjust the tyre pressure on the run, thus the operation is continuous to suit the conditions.

Another facet of the pneumatic multi-tyred roller is its ability to compact uneven surfaces, especially if it has oscillating wheels. This allows individual wheels to operate independently, riding over high spots without lifting the other wheels on the axle. This method of suspension ensures that the low spots are equally compacted.

### ***Oscillating and vibrating smooth drum, steel-wheeled and tamper rollers***

Oscillating and vibrating smooth drum, steel-wheeled and tamper rollers have the ability to impart vibratory compactive effort in addition to static pressure.

In considering compaction by vibration, there are three important components that provide the overall compactive effort:

- Total applied force, made up of centrifugal force generated by a rotating shaft and the proportion of the roller's dead weight applied to the drum.
- Frequency, which is the number of revolutions made by the eccentric shaft. This criteria controls the magnitude of the centrifugal force and hence the compactive effort.
- Amplitude of vibration, which is a function of the frequency selected and is the total vertical distance through which the drum travels.

The frequency and the amplitude produce a compaction effect that can be either an:

- impact effect – the distance the roller lifts off and falls back to the surface; this occurs at low frequency and high amplitudes; or
- contact effect – where the roller remains in constant contact with the surface; this occurs at high frequency and low amplitude.

The most common vibrating roller for the compaction of crushed and uniform grain size non-cohesive materials uses the contact effect operating at frequencies in the range 1700-3000 cycles per minute and at amplitudes of 0.6-0.8 mm. This amplitude range allows the drum to remain in contact with the surface, permitting more vibrations at a given speed to act upon a given area, overcoming the friction between particles allowing them to slide into a denser matrix. It also allows a better penetration of the smaller fractions into the voids between the larger particles providing a tight matrix of stone.

Oscillating rollers are a relatively newer form of vibrating roller, which applies a different mode of dynamic compaction. These rollers have two diametrically opposed rotating vibrator shafts that are fixed to solid diaphragm walls inside the drum and rotate with the drum. As the eccentric masses attached to the two vibrator shafts are fixed at 180° to each other, the forces are not aligned in the same plane and create a pure turning couple, which tends to rotate or oscillate the drum. The net result is that the drum oscillates at the same frequency as the rotating shafts. Oscillating rollers have a number of advantages over traditional vibrating rollers including the ability to be used closer to buildings and buried utilities than traditional vibrating rollers; they are more efficient and therefore quicker compaction of most materials and better efficiency for compaction of asphalt on bridges is achievable (Southwell 2007).

The static weight of an oscillating or vibrating roller typically ranges from 6 to 15 tonnes, providing an equivalent compactive effort of between 8 and 30 tonnes when operating in oscillating or vibrating mode.

The use of oscillating or vibrating compaction has certain advantages; it can:

- compact greater depths of granular materials
- obtain the correct density in less passes
- match the high output of mechanical pavers.

The number and size of rollers to be used for a particular operation are typically determined from compaction trials at the commencement of the work. These also set the rolling pattern. Often oscillating or vibrating rollers are used in combination with other types of compactors operated in a static mode (Figure 4.16).

**Padfoot, sheepsfoot and wedgefoot rollers**

These types of rollers consist of steel drums fitted with projecting feet (Figure 4.21). The shape of the feet varies considerably and may be circular, square or rectangular, and the feet are available in varying lengths. They may be towed or self-propelled with single or multiple axles and ballasted, as well as being either static or vibratory rollers.

**Figure 4.21: Padfoot rollers compacting earthworks fill material**



Source: RTA

**Selection of compactors**

Table 4.5 lists equipment that has generally been found suitable for various materials.

**Table 4.5: Types of compaction equipment for various materials**

Material type	Type of equipment
Heavy clay	Static tamping foot Pneumatic multi-tyred Vibrating sheepsfoot
Sandy clay	Static tamping foot Pneumatic multi-tyred Vibrating sheepsfoot
Crushed rock	Smooth steel drum Pneumatic multi-tyred Vibrating smooth drum
Sand and rock fill	Grid roller Vibrating smooth drum

Source: VicRoads (1998)

It is important to note that a roller may not necessarily increase the material density, as in some cases, inappropriate equipment or site conditions may result in the mere relocation of the material and thus cause rutting and shoving.

### ***Rolling patterns***

To ensure the formation is constructed to the specified quality standards the materials should be compacted to conform to the alignment, grades, crossfalls and thicknesses specified.

To ensure all soils receive the same number of passes a trial should be undertaken to determine the rolling pattern. This trial should use good rolling techniques as follows:

- To avoid material moving downhill and consequent loss of shape, for one-way crossfall, compaction should commence at the lower edge and progress upwards towards the high side of the formation and for crowned cross-sections, compaction should commence from the outer edge and progress inwards to the centre of the crown.
- The compactor should be operated parallel to the centre of the formation and make forward and reverse passes over the same section of layer before moving to the adjacent section.
- Each pass of the roller should overlap the previous pass so as to ensure complete coverage. Each succeeding pass should overlap by one-third to a half of the previous pass to achieve full compaction coverage.
- Where the outside edge of the layer is unsupported and squeezes out excessively, rolling should commence 200-300 mm from the edge and roll the outside strip later.

### ***The number of passes***

The rate at which an earthworks layer can be adequately and efficiently compacted relates to the number of roller passes required before subsequent passes result in very little increase in density. This is usually determined by field trials and it will be found that there is an economic limit to the number of passes to be applied. In general, the use of a heavier roller will increase the density obtained and lower the moisture content required, provided the soil has the strength to support the roller.

Normally between 8 and 16 passes of a roller are needed to adequately compact a clayey soil with heavier rollers usually performing better than light ones, other things being equal. Density increases at a decreasing rate as the number of passes of the roller is increased. Usually after 16 passes (and often even after 8 passes), there is only a very marginal gain in density. If compaction is not achieved with 8 and 16 passes, the process may need to be changed, e.g. a change in the moisture content, type of roller or layer thickness.

### ***Rate of compaction***

Roller speed is another variable, with the optimum travel speed for vibrating rollers varying with the soil type and frequency of vibration of the drum. If roller speed is increased much over 5 km/h, there are fewer impacts per m covered and the duration of impact is shorter. For the compaction of clays using relatively thin layers, the economic speed is higher than for granular or rock fills. For non-vibratory compaction equipment, it is economical to operate at the maximum speed feasible under the circumstances. However, high-speed rolling of some granular materials tends to disrupt the mechanical bond of the particles and travel speed, particularly during the initial passes, should be restricted to 5 km/h maximum.

Road Construction Authority, now VicRoads, Technical Bulletin 34 provides further guidance on the use of earthworks plant (RCA 1986).

### **Uniformity of procedures**

For uniformity and effectiveness in the compaction of an earthworks formation, compaction methods and procedures must be developed particular to the material being laid. Trials, which may include the method of preparation of the soil, lift thickness, moisture addition, compaction equipment type and the number of roller passes, are commonly used or specified in the development of a compaction or roller routine. It is critical that the compaction or roller pattern is accurately communicated and understood by all involved.

### **Rolling of the final earthworks surface**

To achieve a tight even surface, the final passes should be undertaken by a smooth-drum roller and if necessary, followed by watering and pneumatic multi-tyred rolling. Where a smooth-drum vibrating roller is used, it should be operated using low amplitude and high frequency. The sequence and type of rollers used are normally determined during the trials.

#### **4.8.4 Environmental Influences**

Environmental influences such as the in situ moisture content of the material in the borrow or cut excavation, climatic variation, and inundation of the construction site may affect the ability to provide and maintain an acceptable level of moisture content and compaction in earthworks materials. To this end, it is necessary to pay attention to drainage of the site and earthworks formation by the use of mitre drains through boxed formations, maintaining drainage outlets at low points and, if possible, by keeping run-off away from the site. Installed subsoil drains should not be utilised for construction site drainage. Construction programs that allow a compacted layer to be exposed to climatic elements for a long period without adequate cover can result in loss of density of the layer and soft areas developing, such that it must be ripped and reworked to regain the level of moisture and compaction specified (VicRoads 1998).

In hot, dry weather when the surface of the layer being compacted dries rapidly, a periodic light sprinkling with water will be necessary. If the soil becomes saturated, e.g. by rain overnight, it should be dried back to optimum moisture content.

## **4.9 Conformance and Quality Testing**

### **4.9.1 Material Supply**

The *Guide to Pavement Technology – Part 4I: Earthworks Materials* (Austroads 2009d) contains advice on the selection, testing, quality control and specification of earthworks materials.

### **4.9.2 Test (or Proof) Rolling**

Test rolling, commonly called proof rolling, with suitable compaction plant is used to initially determine whether a lot of work is at a state to be further tested for in situ density and moisture content. Test rolling is also the only practicable means available to determine the state of compaction of materials of nominal size greater than 40 mm. A test rolling procedure should be established prior to commencement of earthworks construction. Preparation of an area for test rolling should include a number of passes (not less than three) of the plant to be used for the test roll to break any set-up of the layer being test rolled.

It is essential that test rolling be carried out on material at approximately the appropriate optimum moisture content. Test rolling is an assessment of immediate stability only and is not a measure of the degree of compaction. Dry materials may display stability under test rolling at a level of compaction well below that required.



Further test rolling of a layer should be carried out if the layer has been allowed to alter its moisture content to such an extent that it no longer lies within the range of moisture contents specified. This is likely to occur if the layer is not covered by a successive layer within a reasonable period of time (the top layer may have dried out or become over wet). Prior to undertaking a further test roll of a layer, the layer should be re-prepared so that the moisture content lies within the specified range.

Observed deflections under test rolling can be approximately quantified as shown in Table 4.6:

**Table 4.6: Visible deflection under test rolling**

Deflection description	Deflection range (mm) <sup>1</sup>
No perceptible deflection	generally less than 0.65
Barely visible deflection	0.45 – 0.8
Clearly visible deflection	0.8 – 1.15
Marked deflection without any apparent permanent deflection	1.15 – 1.9
Marked deflection but accompanied by permanent deformation (rutting) and/or surface cracking	greater than 1.9

Note:

<sup>1</sup> Deflection determined by use of Benkleman Beam with a wheel load of 4080 kg.

Source: VicRoads (1998)

### 4.9.3 Level Tolerances

To ensure that the formation is constructed within specified limits so that excessive variations do not occur in overlying pavement thicknesses and horizontal grades and crossfalls, tolerances are placed on both vertical and horizontal dimensions of the finished formation.

Level tolerances may vary according to the specifying authority; however typical requirements are:

#### **Horizontal**

- where the formation is constructed close to other structures or the pavement is to be constructed between kerb and channel, +/- 50 mm
- where the formation is not adjacent to any structure the tolerance width can be more flexible, -50 mm + 250 mm.

#### **Vertical**

- a general surface tolerance of +/- 25 mm is normally applied
- to ensure that depressions do not occur, no point on the surface of any pavement layer should vary by more than 15 mm from a 3 m straight edge.

### 4.9.4 Compaction

Other things being equal, the greater the compactive effort applied to an earthworks layer, the denser (and stronger) and more impermeable that layer will be. A higher compactive effort will generally generate a higher maximum dry density (MDD) at a lower optimum moisture content (OMC). If the moisture content is greater than the OMC, water displaces soil particles that might otherwise fill voids during the densification process. If the applied energy is not sufficient to remove excess water, the density of the soil is reduced.

Compaction specifications normally specify the minimum standard of compaction to be obtained in the field as a characteristic value of relative compaction, a percentage of the maximum dry density.

**Field tests**

The common methods of measuring compaction in the field are:

- sand replacement (AS 1289.5.3.1-2004)
- nuclear densimeter.

The relative compaction in the field is obtained from the following formula:

$$RC = \frac{\text{Field dry density}}{\text{Laboratory dry density}} \times 100$$

1

Typically, the appropriate laboratory value for MDD and OMC, determined using either standard or modified compactive effort (refer to Section 4.8.1), are used to calculate the relative compaction and moisture ratio for each of the test site locations that comprise a lot. The mean, the standard deviation, and the characteristic value for both density and moisture content are then calculated (refer to Section 2.2). All test results from a lot must be included in the calculations, including those markedly different from the average. Because relative compaction varies a great deal within a lot, a range of possible values exists for the calculated characteristic, so that for work to have a high probability of acceptance, the average level of compaction within a lot must be markedly higher than the test result level contained in the specification.

**Specified compaction levels**

To achieve the desired quality of earthworks the following compaction requirements are typically specified; however, individual road authorities may specify higher or lower values depending on the location, environment, pavement design and traffic volume.

In the construction of embankments it is normal practice to bring the embankment up in uncompacted layer depths not exceeding 300 mm and then compact to a characteristic value of relative compaction, typically 95% (standard compaction) though it is common practice to specify a higher characteristic value of relative compaction for the last 300 mm, typically 97% (standard compaction).

After a cutting has been excavated the in situ material directly below the formation should be compacted to a depth of 150 mm typically to a characteristic relative compaction of 97% (standard compaction). Where the cutting is in rock or a non-ripable material and a stabilised selected material is used as a bridging layer it should typically be compacted to a characteristic relative compaction of 97% (standard compaction). If other forms of bridging layer material are used, a characteristic relative compaction value of 95% (standard compaction) is normally required.

Compaction requirements specified for embankments and cuttings may vary according to the depth of the layer from the top of formation. As an example, in VicRoads Standard Specification Section 204 – *Earthworks* (VicRoads 2008a) fill material and material within 150 mm of the design cut floor level in cuttings having a nominal size after compaction of 40 mm or less is to be compacted to comply with the requirements of Table 4.7. The VicRoads requirements are based on different scales of construction works, which are related to traffic loading or, for roads other than freeways and national highways, the volume of heavy vehicles as follows:

**Scale A:** All freeways and national highways and duplicated state highways, and main roads and tourist roads with a traffic loading in excess of  $7 \times 10^6$  equivalent standard axles (ESA).

**Scale B:** All other state highways with a traffic loading less than  $7 \times 10^6$  ESA and all main, tourist and unclassified roads with a traffic loading between  $1 \times 10^6$  and  $7 \times 10^6$  ESA.

**Scale C:** All other roads not listed above as Scale A or Scale B.

**Table 4.7: Example of compaction requirements for earthworks**

Material type and location	Scale A	Scale B	Scale C
	Minimum characteristic value of relative compaction (%)	Minimum characteristic value of relative compaction (%)	Minimum mean value of relative compaction (%)
All Type A material Type B material Top 400 mm directly beneath Type A material Ripped and re-compacted material below cut floor level	99.0	98.0	100.0
Type B material More than 400 mm below Type A material The top 150 mm of areas where fill is to be constructed	97.0	95.0	95.0
Type C material	95.0	93.0	92.0

Source: Adapted from VicRoads (1998)

The compaction test method requires that less than 20% of material being tested be retained on the 37.5 mm sieve. If this figure is exceeded then a compaction test value cannot be obtained, as there are no convenient methods for testing compaction. In materials where this occurs it will be necessary to develop a compaction routine involving spreading, watering and rolling patterns. This development may take some time and should become part of the contractor's procedures. Where compaction testing cannot be undertaken in oversize material, the characteristic moisture ratio may provide an indicator that optimum density is being achieved, but it is important that moisture ratio determinations be performed immediately after compaction, not hours or days later. In any case, the adopted routine must be performed under close surveillance to ensure compliance.

### **Repeat testing**

Repeat testing of work should not be undertaken merely because, on the basis of the results of the first testing, the lot was deemed to have failed. Repeat testing can be expected always to provide results that differ from the first result, but there is no valid reason to accept the second result in preference to the first. Further, it will also bias the distribution of risk away from an equal sharing between the two parties.

Retesting of work should be undertaken only if an entire lot has been reworked or an error is known to have occurred in the testing procedures, in which case the first set of test results should be discarded completely and the second set should be adopted to determine the acceptability of the lot concerned. Selection of the second set of site locations should be carried out by the same method as used for the first set.

### **4.9.5 Frequency of Testing**

The minimum frequency of testing should be that which is sufficient to ensure that materials and work supplied under the contract comply with the specified requirements. Testing of earthworks involves testing for both material property and compaction/moisture content compliance. The minimum frequency specified may be based on prior experience with the materials available and/or the compaction methods used. Refer to Section 6.10.6 for an example of the minimum frequency of testing requirements.

## 5. Subsurface Drainage

This section describes construction of subsurface drainage. Advice on the design of subsurface drainage is given in the *Guide to Pavement Technology – Part 10: Subsurface Drainage* (Austroads 2009g).

It is acknowledged that VicRoads Bulletin 32 (VicRoads 2004a) was a useful source of information in the preparation of this section.

### 5.1 Trenching

Trenching for subsurface drains is usually carried out by backhoe or trenching machine (Figure 5.1). The type of plant chosen for the work will depend on the ground conditions, the way in which the trench spoil is to be removed from the site and the specified tolerances on the line and grade of the trench. Subsurface drainage pipes can be laid directly behind a ripping tyne or ditch cutter by means of a guiding box. This results in high production rates but is only suitable in soils where no filter is required and where close control over line and grade is not needed.

**Figure 5.1: Trenching for subsurface drainage**



Source: VicRoads (2004a)

The degree of irregularity that can be tolerated in the trench depends on the drainage grade and type of filter proposed. The bottom of the trench will almost always require some handwork to remove irregularities. Rocks and hard bars should be removed and any excess cut in the trench base back-filled to grade with compacted natural material. If practical, the trench base may be shaped to a vee at the centre and irregularities in grade removed so that water will not pond more than 20 mm deep over more than 3 m length of trench. If these tolerances are not met, the trench may be capable of storing large quantities of water, with the potential of damaging the pavement structure. Some modern trenching machines can provide very regular, self-draining trenches requiring minimal handwork.

Where a geotextile filter is to be used in contact with the trench walls, the walls must be sufficiently uniform for the geotextile to come into close contact at all locations along the walls when the granular filter is placed. If this cannot be achieved, for example in hard ground, or where rocks or overbreak are expected, it is preferable to use a granular rather than a geotextile first-stage filter. Alternatively the use of a geotextile such as a needle-punched staple fibre with sufficiently high elongation may be used.

## 5.2 Pipes

Pipes should be laid on a compacted layer of filter material typically 25 mm or greater, except for drains designed to lower a water table. This bedding material takes out the minor irregularities in the trench, and ensures that the pipe is supported over its full length with no point loading. For drains lowering a water table, (e.g. intercepting high seepage flows) a minimum compacted bedding thickness of 75 mm is typically specified to enable the full filter to set up and protect the pipe. An alternative in these situations is to use a geotextile filter along the trench walls and base and lower the pipe to ensure as much of the water as possible is drained.

Pipes should be placed centrally in the trench on grade. Particular care should be taken when placing subsurface drains around sharp curves to ensure they remain free draining.

Pipes should be joined and grouted or wrapped as per manufacturers' specifications or as designed. When joining corrugated plastic pipe the use of purpose-built plastic joiners/connectors is advised. Similarly, risers in plastic pipes may be joined to the main pipe using plastic oblique tee connectors. For other types of pipe, special sections, if available, may be used otherwise pits should be constructed. The whole system should be flushed after pipe laying and filter compaction are complete to remove any debris from the pipe and commence the setting up of the filter.

Outlets or any other lengths of pipe that are not located in water-collecting zones are generally unslotted, smooth bore pipe of sufficient strength to prevent damage by maintenance plant at the depth they are laid.

### 5.2.1 Prefabricated Structural (Fin) Drainage

Placement of backfill against prefabricated structural (Fin) drainage must be closely monitored and mechanical compaction of the material should be minimised to prevent crushing of the plastic core or yielding and creep of the geotextile into voids in the core. Prefabricated structural drains often come in narrow widths and longitudinal joints are necessary in many applications. Where these drains are butted together the joint should be overlapped with a geotextile. Special care should be made to prevent infiltration of soil through the end of the drains. One way of doing this is to seal the end of the drain with a geotextile. Special attention to detail is also required where a prefabricated drainage system ties into a collector pipe.

## 5.3 Granular Filters

Granular filters may be placed directly from agitator trucks if only small quantities are involved. For large quantities it is generally more economical for placing to be by a horizontal discharge unit fitted to a tipper, or a horizontal discharge truck (Figure 5.2 and Figure 5.3).

Figure 5.2: Horizontal discharge unit



Source: VicRoads (2004a)

Compaction of granular filters is carried out by wetting, and use of vibrating plate or rammer type compactors and/or trench rollers. Saturation of coarse, highly permeable materials cannot be achieved in practice and the best compaction occurs in the wet state by use of high frequency, low amplitude vibrating plates. Wetting of sandy materials followed immediately by low frequency, high amplitude foot compactors should be undertaken to assist in achieving good compaction. For some filter materials, ensuring the material is saturated will significantly improve results. For narrow trenches used for prefabricated structural drains, sand filter media may be saturated and compacted from the top of the trench.

**Figure 5.3: Placement of granular material**

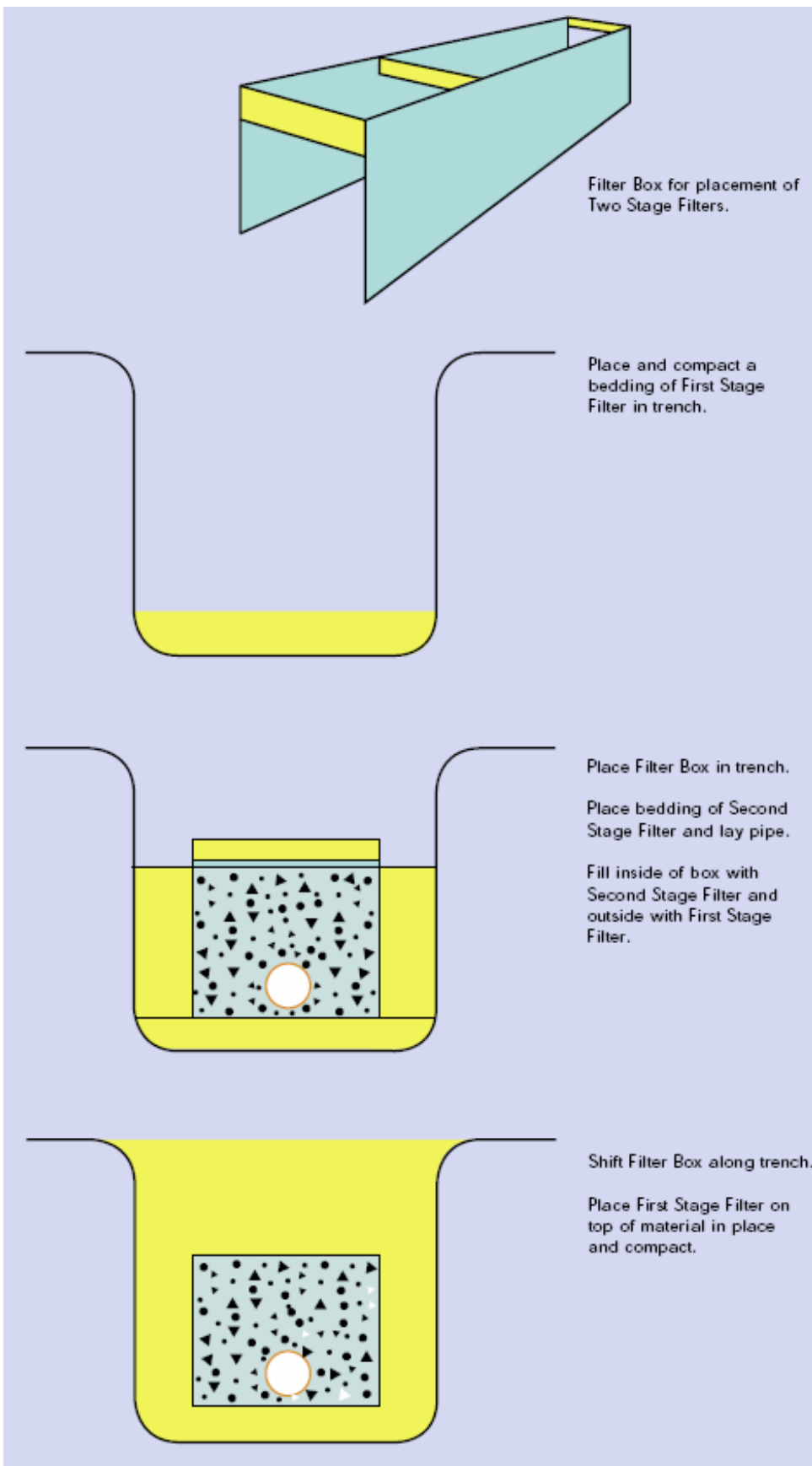


Source: VicRoads (2004a)

Compaction of granular filters may be measured using a nuclear density gauge or sand replacement test, depending on the material being compacted. However, compaction testing of single size, larger material may not be possible and conformance may rely solely on process control.

For shallow pavement subsurface drains the compaction process generally consists of three lifts. The first of these is the bedding mentioned above, the second stage is the filter around the pipe and the third is the top 300 mm lift of the filter. A split foot compactor may be useful around the pipe. The second stage may also involve the placement of the second granular filter material, if a two-stage filter is used. Figure 5.4 illustrates a method of placing the second granular filter material using a box to separate it from the first granular filter. The third stage is generally placed as either one layer or layers of 150 mm each, saturated and compacted. Additional lifts of 150 mm to 300 mm should be added for deeper subsurface drains.

Figure 5.4: Placing two-stage filters



Source: VicRoads (2004a)

As it is difficult to measure the compaction achieved in filter materials a method specification may be used. Adopting 300 mm loose lifts, three passes of a vibrating plate or other appropriate compaction unit is a typical method specification used in these situations provided the filter material is sufficiently wet. Where the relative compaction is measurable, typically a minimum characteristic value of relative compaction of 95% (modified compactive effort) or 98% (standard compactive effort) is specified for major works and a characteristic value of relative compaction of 92% (modified compactive effort) or 95% (standard compactive effort) for other works.

Every effort should be made to ensure that the filter material does not become contaminated since any fine material that enters the filter may result in a marked decrease in permeability and therefore efficiency of the subsurface drain.

Contamination usually occurs as a result of trench spoil being knocked into the trench with the filter material during its placement, by water borne materials if the filter is left open during rain or by material from plant and workers deposited on an open filter.

When constructing subsurface drains, every effort should be taken to minimise the contamination of filter by soil or surface run-off. A number of measures can be taken as follows:

- Ensure the formation is trimmed and all loose material is removed prior to installation of subsurface drainage. Material excavated during trenching should be removed from the formation prior to the installation of pipe and filter material (refer to Figure 5.1).
- Construct the pavement drains immediately prior to pavement construction.
- Over-fill with filter (for washed sand types) and trim immediately prior to pavement construction to remove contaminated material.
- Remove contaminated material and if required replace with clean filter material prior to pavement construction.

If the pavement material or bedding for kerb and channel cannot be placed immediately after the filter is compacted, a cap of minimum 50 mm of permeable pavement material or filter material should be compacted on top of the filter. This cap, together with any contaminated filter, can be removed by grader just prior to the commencement of pavement material placement.

For 'boxed in' pavement construction, boxing or mitre drains should be installed at regular intervals through the boxing to assist in the drainage of the formation.

## 5.4 Geotextile Filters

The *Guide to Pavement Technology – Part 4G: Geotextiles and Geogrids* (Austroads 2009c) provides additional advice on geotextiles and geogrids.

The handling and storage of geotextiles for drainage applications should be in accordance with Section 4.6.2.

When a geotextile is used as a first stage filter surrounding a granular filter, it is good practice to select the roll width of material that will just fit the trench, for example top, bottom and sides plus 200 mm overlap. The material should be unrolled across the top of the trench, allowed to fall down into the trench and then pushed into the corners as illustrated in Figure 5.5 and Figure 5.6. The top of the geotextile may need to be anchored by stones, small heaps of filter material or spikes to prevent it falling into the trench. Splicing of lengths of geotextile should consist of minimum 1 m of overlap secured with pins or other mechanical ties. Compaction of the granular filter should ensure that good contact is maintained between the geotextile and the trench.



**Figure 5.5: Placing a first-stage geotextile filter**



Source: VicRoads (2004a)

**Figure 5.6: Discharge pipe in trench, ready for second-stage filter**



Source: VicRoads (2004a)

Where an outlet pipe passes through the geotextile a separate piece of geotextile should be wrapped around the outlet pipe, flared against the side of the geotextile in the filled drain and secured. Similarly, geotextile should be flared, either inwards or outwards as convenient, against pits.

When pre-fitted filter socks for drainage pipes are used (Figure 5.7), extra care is required during transport and handling of the pipe to ensure the sock is not damaged.

Installation of a knitted filter sock onto pipes can be carried out using a special applicator (mandrel). Lengths of sock are stretched over the mandrel (diameter about 130 mm), and then the pipe is in turn fed through the mandrel. The sock is tied to one end of the pipe to prevent slippage. As the pipe is fed through the mandrel the sock is released onto the pipe. Long lengths of sock can be applied in one operation. The applicator must be kept smooth and free from rust and burrs.

A sock should be placed around a pipe such that it is not wrinkled, nor should it be stretched more than 5% of its manufactured length. Overlapping of the sock at joins should be a minimum of 50 mm and the cloth should be secured to the pipe such that the backfill material will not infiltrate through any cloth over-laps.

**Figure 5.7: Drain pipe with pre-fitted sock being positioned**



Source: VicRoads (2004a)

The length of sock that can be installed in one operation will vary from one material to another and must be determined by experience. To avoid damage to the sock, it should be threaded onto the pipe immediately before the pipe is placed in the trench, and the pipe should not then be dragged over the ground. Splices between lengths of sock should consist of minimum 50 mm overlaps, securely tied. Care should be taken not to apply tension to the sock during installation i.e. the sock, when installed, should sit loosely on the pipe.

All geotextiles should be stored out of direct sunlight and covered as soon after installation as possible.

## 5.5 No Fines Concrete Filters

No fines concrete is increasingly being used as a subsurface drainage filter material on large projects, under trafficked pavements and in areas where settlement cannot be tolerated.

No fines concrete is a very coarse concrete mix, typically 100% passing the 26.5 mm and greater than 70% passing the 19 mm sieve. For long-term stability a minimum 28-day compressive strength, typically 20 MPa, is specified (RTA 2005). However, where no fines concrete is used as a filter material in untrafficked areas this requirement may be reduced.

The effectiveness of no fines concrete as a drainage filter material is related to the voids within the material after placement and therefore mechanical tamping is generally not recommended (RTA 2005). In addition, the aggregate/cement ratio is required to be tightly controlled to prevent migration of cement slurry, which could result in reduced voids in lower areas of the subsurface drain.

Where no fines concrete is used as a filter material in conjunction with drainage pipes or prefabricated structural drains located within a fine silt or clay, filter sand should be placed around the drainage pipe or prefabricated structural drain prior to backfilling with no fines concrete to prevent fine silty particles from entering and blocking the pipe or drain (VicRoads 2004a).

## 5.6 Flushing

A subsurface drainage system should be flushed out during construction immediately after placing and compacting the filter material. This will remove any particles which have entered the pipe while the filter is setting up and detect any sections which are not working, for example as a result of crushed or broken pipes. Flushing should be continued until the discharge water runs clean. The response time of the system, the time between commencement of pumping into the system and first discharge, should be recorded on the as-constructed drawings.

Where it is possible to measure the volume of water used and the water recovered from the outlet this information should also be noted. The volume of water remaining in the drain (storage capacity) gives some indication of the efficiency of the original construction and irregularities in the trench bottom.

## 5.7 Finishing Details

When proof-flushing of the subsurface drainage system is complete, the pipe openings and outlets should be temporarily blocked. A can or other suitably sized object with a recovery wire can be pushed into the pipe, or a cover made from a short length of filter sock tied at one end. This prevents any foreign matter or animals entering the pipe prior to the construction of finishing details.

The excess pipe left protruding above the surface at flush-out risers should be left in place and the top opening slab formed around it. The pipe can then be trimmed and capped. Unless capped, material from construction works may readily enter the pipe as shown in Figure 5.8. The procedure for treatment of the pipe at flush-out riser is also used for kerb and channel when the kerb and channel is constructed using forms. If kerb and channel is extruded, the excess pipe should be bent aside to allow the extruder to pass, the formed section is then broken out and the pipe stood up, the channel re-finished by hand and the pipe trimmed and capped.

**Figure 5.8:** Pit with debris



Source: VicRoads (2004a)

Outlets may be pre-cast such as that shown in Figure 5.9 and grouted into place and around the pipe, or cast in situ directly around the pipe. Wire netting or a pre-formed wire plug may be used to prevent animals from entering the pipe.

**Figure 5.9: Pre-cast subsurface drain outlet**



Source: VicRoads (2004a)

All outlets and top openings should be marked as soon as they are completed. Marker posts often consist of 75 mm diameter treated timber protruding about 600 mm above ground level. Identification plates may be attached to marker posts or directly to pit covers, opening surrounds or outlet headwalls (Figure 5.10) using epoxy resin adhesive.

**Figure 5.10: Subsurface drain outlet in culvert headwall**



Source: VicRoads (2004a)

The final task of the construction personnel is to produce the drawings of the as-built subsurface drainage system. Modified original drawings, where construction details such as modifications and as-constructed levels are shown in red, are generally satisfactory for this purpose.

## 6. Unbound Granular Pavements

This section describes the materials and methods of unbound granular pavement construction. Final preparation of the surface to receive a wearing surface is also discussed.

The types of materials used for unbound granular pavements vary widely; however, in all cases, a hard, dense, tightly locked granular surface, free from lenses, laminations, compaction planes and cracking, is essential for the performance of a bitumen surface.

It is acknowledged that Centre for Pavement Engineering Education course notes (CPEE 2004) were a useful source of information in preparation of this section.

### 6.1 Introduction

The practices adopted for construction of unbound granular pavements vary depending on the size, nature and location of the project and the specified requirements (Midgley 2008). Material source, lot size, type and number of plant, production rates, testing requirements and specification tolerances may vary between projects. However, the general process for construction of unbound granular pavements consists of the following steps:

- project planning
- preparation of the underlying materials
- sourcing of granular materials
- preparation of granular material prior to delivery
- delivery of the granular material to site
- placement of the granular material via grader or mechanical spreader
- compaction of the granular material
- preparation of the pavement surface prior to surfacing.

### 6.2 Preparation of the Underlying Materials

Prior to construction of an unbound granular pavement it is important to verify that the underlying earthworks layers conform to the required specification, in terms of compaction, shape and level. Of particular importance is that there are no soft spots, which may cause premature failure of the unbound pavement, and that there are no significant high spots that reduce the granular pavement below an acceptable minimum thickness.

Any areas of the underlying earthworks that have been disturbed by construction, weather or traffic should be rectified prior to commencing pavement construction.

### 6.3 Supply of Granular Materials

Crushed rock is a product of a manufacturing process where the source or parent rock can be selected for consistency and the crushed rock properties can be closely controlled during production.

The main source of material for crushed rock is rock quarries and crushed gravels or cobbles obtained from pits and river sources.

The source rocks used to produce crushed rock pavement materials must possess characteristics which will ensure that the product will have the necessary strength and durability, both immediately and in the long term, to withstand handling during construction, weathering agents and traffic stresses (Austroads 2008a, 2008e).

The *Guide to Pavement Technology – Part 4A: Granular Base and Subbase Materials* (Austroads 2008a) and *Part 4J – Aggregate and Source Rock* (Austroads 2008e) contain advice on the selection, testing, quality control and specification of crushed rock and naturally occurring granular materials for use in pavement base and subbase construction. This guide addresses the factors that lead to the appropriate selection and specification of unbound granular materials by reference to:

- the role and function of granular materials in a pavement, including factors such as the intrinsic or manufactured properties and their relationship to in-service behaviour and performance
- the physical properties affecting material requirements, including the properties that affect structural adequacy, serviceability, durability, volume instability, permeability, compaction and handling and working
- the production of naturally occurring granular materials, crushed rock and recycled materials
- the different methods of specification, quality management, attaining required performance characteristics, and quality control and assurance.

## 6.4 Quarrying of Granular Materials

Quarrying of granular materials is a specialised operation that takes considerable planning and a full understanding of quarry plant and its operation, as well as the safe use of explosives to obtain the source rock which is fed into crushers, through screens and, where necessary, pugmills, to obtain the final specified crushed product (Figure 6.1).

**Figure 6.1: Typical pugmill used in a quarry for processing granular materials**



Source: RTA

The products produced by crushing are typically used as:

- crushed rock road pavement materials, especially for heavy duty pavements
- materials for blending with naturally occurring material and loams to produce soil aggregates
- aggregates for asphalt and other bound mixes

- aggregates for road surfacings and drainage materials.

## 6.5 Preparation of Granular Materials Prior to Delivery

The source of the granular materials, equipment used for extraction and processing, and the specification requirements will be the primary determinants of what preparation of the materials is required prior to delivery. Pre-delivery activities may include crushing, wet-mixing and/or blending.

Austrroads (2008a) contains additional advice on the preparation of granular materials prior to delivery.

## 6.6 Delivery of Granular Materials to Site

Delivery trucks usually have tipping bodies and may include semi-trailers and dog trailers. Particularly in urban area and for haulage on public roads the bodies are typically covered with tarps or similar coverings to prevent dust. Granular materials that are to be delivered straight to a mechanical spreader require delivery trucks equipped with discharge equipment that allows the material to be fed directly into a hopper without spillage and minimal segregation.

To achieve uniform spreading and compaction it is preferable that material arrives at site at or near the target moisture content, and is handled as little as possible during delivery and laying to minimise moisture loss and segregation. For delivery of wet-mixed materials with long distance hauls or delivery during high temperatures additional covering of the material should be considered to minimise loss of moisture through evaporation (Midgley 2008).

## 6.7 Placement of Granular Materials

Uniform spreading of granular materials across the pavement width to a constant depth is critical to achieving uniform compaction and layer thickness (Midgley 2008).

The placement of granular subbase and base materials can be undertaken by either:

- grader
- mechanical spreader.

Both types of plant can place either soil aggregates or crushed rock, however, the general practice is to use the grader for spreading soil aggregates, which have a degree of cohesion, and mechanical spreaders for crushed products that rely primarily upon internal friction between the aggregates for stability. The major factors in the selection of the method of placement are cost, availability and control of segregation, especially for the placement of high strength crushed rock, which may segregate when subject to the type of spreading performed by a grader.

### 6.7.1 Grader-placed Layers

A grader is typically used to place and spread materials such as soil aggregate and natural gravels, which have a degree of cohesion that reduces the possibility of segregation occurring during the spreading operation (Figure 6.2).

Figure 6.2: Grader trimming granular material



Source: RTA

The placement is normally undertaken in five stages:

#### **Stage 1 – Dumping operation**

Upon delivery the material is tipped under the control of a field person into windrows uniformly across the pavement. The total amount dumped should not exceed that which can be laid by the grader in one day.

#### **Stage 2 – Spreading operation**

The spreading operation is undertaken to provide an even distribution of material over the whole pavement. The grader commences by respreading the windrows across the formation or sub-base. These windrows should not be too high, allowing the grader to initially spread the material in about three to six movements. This work should be undertaken in a continuous cycle, and at a speed that allows for proper control, which on average is between 5 and 10 km/h.

To achieve the required level of quality spreading by grader should be undertaken as follows:

- The blade angle and pitch is adjusted until a satisfactory mixing and material flow is obtained, with the material flowing outside the rear wheels. This is normally achieved with the blade pitched forward.
- The material is spread to the required depth, crossfall and grade ready for compaction. All of these factors are controlled by either trim pegs set at the correct level on the outside and centre of pavement, by the use of stringlines or electronic control devices.
- Rills of segregated material are removed from the pavement from time to time by a quick light pass of the blade to reduce the possibility of surface blemishes during compaction.
- The spreading operation is planned to ensure the grader does not turn on freshly spread material. It may be advantageous to back the grader over the placed material to avoid turning.



### **Stage 3 – Mixing and watering**

During the spreading stage the crossfall, surface tolerance and moisture content should be checked. They can then be rectified, if necessary, by remixing of the materials or addition of water, enabling the penetration of the water and materials to be cut and re-used to fill the low spots, without lensing or lamination occurring, before the pavement becomes too tight. Remixing may also be required to address segregation of granular materials due to dumping and spreading operations. Checking should not be left until the compaction stage, as it is difficult for moisture to penetrate the compacted layer. Where water has to be added it is desirable that it should be brought up to the specified field moisture content in the pit or during manufacture in the quarry. Where water is added by water truck in the field an allowance of 1 – 2% above the specified moisture content is often made to take into account field losses due to evaporation and the laying process. The number of times the material is turned over by the grader should not be excessive otherwise segregation may occur. In practice at least three passes are required to achieve uniform mixing, but not normally greater than six as it is moved across the formation.

### **Stage 4 – Compaction**

On the completion of the initial spreading operation the material should be compacted to the correct density using the rolling pattern determined during trials, together with the grader and, when necessary, the water truck. This process is typically undertaken using smooth-drum rollers, static or vibratory, either singularly or in combination with a rubber-tyre roller. The depth of the layer is controlled by the type of compactor available to undertake the work. Compaction is discussed more fully in Section 4.8 and Section 6.8.

### **Stage 5 – Shaping**

After two to three passes of the compactor the depths, levels and crossfalls should be checked, as the material is still loose enough for any adjustments to be made by the grader before completing the compaction of the layer. The final trim ensures that the pavement is constructed to the correct levels and shape prior to the placement of the wearing surface. Where the material is a fine-grained soil aggregate, e.g. loams, sandstones and gravels, the surface is typically dampened and the top 10-15 mm cut to waste. This ensures the removal of segregated fines, crusts and compaction planes. This operation is followed by a steel-drum roller in a static mode to tighten the surface. Alternatively, a pneumatic multi-tyre roller, with its kneading action and weight, can be used in combination with the steel wheel. When the material has a coarse grading and the percentage of fines is small, trimming the surface with a grader can be difficult. A heavy steel-drum roller coupled with a drag broom to work the fines in and around the coarse particles is often used in these situations.

To achieve a successful final trim the grader should:

- have good cutting edges and tyres with uniform pressure on all wheels
- operate at a speed to avoid bounce
- use moderate blade angle and the top of the blade pitched slightly forward
- work from the centreline to the outside shoulder
- work the curves from the inside to the outside
- change the direction of travel over the same area.

On the completion of the placement operations the surface should be drag-broomed prior to surface preparation.

## 6.7.2 Mechanical Spreaders

The mechanical spreader is normally used to place bound or unbound high-strength materials and lower strength materials that have been plant mixed or processed through power screens. The use of the spreader enables the materials to be placed to uniform thickness with a minimum degree of segregation and surface blemishes occurring.

Mechanical spreaders can be classified into two broad categories:

- spreaders attached to a parent machine – the two common types available in this category are the jersey spreader that uses a tracked tractor as the parent machine, and the box spreader that can be attached to the rear of the delivery truck
- self-propelled spreaders – the two most common types available under this category are the self-propelled paver and the autograde.

### ***Spreaders attached to a parent machine***

To achieve a uniformly placed pavement when using a jersey spreader requires the selection of a tracked unit of the right weight and size to provide stability for the spreader. It also requires a skilled operator to control the screed and tipping of the material into the receiving hopper.

Spreading boxes are attached to the rear of the truck that transported the material to site. The material is tipped into the hopper as the truck moves slowly forward, the speed being set to provide an even depth of material through flow gates. The tipping operation must be controlled manually, to ensure no overspill takes place, creating an uneven running surface, which can affect the depth control of the box spreader and consequently the loose layer of spread material.

For both types of spreader the material should be delivered at the correct moisture content to ensure that segregation does not take place due to the dry nature of the material and to achieve the specified compaction. Both methods of spreading rely on manual control and cannot place the materials to a high degree of accuracy. This necessitates the use of a grader to finish the pavement to the correct shape and tolerances.

With the advent of the self-propelled pavers and other similar machines which can be controlled to place materials to close tolerances, these machines are becoming obsolete, especially the jersey spreader.

### ***Self-propelled spreaders***

The most common mechanical spreader is the self-propelled floating screed paver (Figure 6.3). This is a purpose built machine capable of laying materials to a specified thickness with a minimum of material handling and manual control. The paver uses automatic screed control systems such as a crossfall device, matching shoe, laser or beam running off the previously laid layer or control lines, to build in the crossfall and other geometric dimensions required by the designer. It can be controlled by automatic censoring systems or by manual control. A full description of a self-propelled paver is given in Section 9.4.2.

**Figure 6.3: Paver laying granular material**



Source: Austroads

Pavers are capable of undertaking the following operations:

- The placement of layers of crushed products and high quality soil aggregates with a minimum of segregation. They are also used for placing bound materials such as cement and bitumen stabilised materials and asphalt.
- Laying granular materials in homogeneous layers to controlled depths and levels.
- Attaining the specified surface and thickness tolerance without excessive manual control and handling of the material, by the use of automatic grade and sensor screed controls.

Another form of mechanical spreader is the autograde where material is fed onto the formation in front of the machine or into a receiver and then fed by an auger into a vibrating screed that lays the material to the correct thickness. The autograde machine has similar capabilities as the self-propelled paver but also has the ability to trim surfaces to achieve very high accuracy in surface level and layer thickness.

Self-propelled spreaders are generally used for heavy-duty pavements that use crushed products having a high content of coarse fractions and a few fines.

The materials are processed in a mixing plant, where moisture is added in a pugmill, after which they are delivered to site and placed, using a floating screed paver or autograde. The moisture content and grading must be consistent to achieve a uniform dense pavement. Once the material is laid, it is compacted by a vibratory smooth drum roller, operated singularly or in combination with other smooth drum or pneumatic multi-tyred rollers. The exact combination is decided by roller trials. These trials will also set the vibratory and non-vibratory periods, to ensure over-compaction does not occur.

A grader is not normally used to finish self-propelled spreader laid work.

### ***Practical considerations when using mechanical spreaders***

When placing crushed materials certain factors such as moisture and particle distribution need to be controlled to keep material segregation and surface blemishes to a minimum.

The material fed to the spreader must be at the correct field moisture content as determined by compaction trials, as the spreader has no mechanism to remix the material if the moisture content is not correct. Any variation in moisture content can cause variations in the thickness of the layer to occur due to changing the force acting on the floating screed.

When moisture is added to crushed materials it should be added through a pugmill and tightly controlled. The reason for using a pugmill is that water added at the stockpile does not always provide a material with uniform moisture content. Water sprayed on to the surface of the stockpile may only penetrate the surface to a depth of 300-400 mm, leaving the core dry. If a pugmill is not available then the material should be spread out in uniform layers from the stockpile and water added by a water truck and mixed into the material by a grader; however, there is significant risk of segregation occurring.

The operation of the spreader can be affected if the material grading is not consistently uniform. When crushed products are taken from stockpiles they normally require remixing due to segregation that has occurred during the placement of the materials into the stockpile. This operation is best undertaken using a pugmill to recombine the materials. During this mixing operation water can also be added to correct any variation in the moisture content. For other classes of material such as soil aggregates that are of a plastic and cohesive nature, they may be remixed using a grader.

To achieve a uniformly constructed pavement using a spreader there are certain construction operations that need to be controlled:

- Material delivery – to ensure that surface irregularities do not occur due to the stop-start action of the spreader, material should be delivered at a rate that allows the spreader to operate in continuous mode.
- Layer thickness – the screed should be set to a layer thickness to match the compactive effort available. This will ensure that the specified density will be achieved throughout the layer.
- Automatic controls – when control lines and sensors are used, irregularities can result if the control lines are not supported properly allowing the cant of the line to be reflected in the pavement surface.

### 6.7.3 Jointing

As with bound pavements and asphalt surfacings the location of joints can have significant impact on pavement performance. Unless otherwise specified, the layout of joints should conform to the following requirements:

- material should be spread in such a manner as to minimise the number of joints
- for all pavement layers, transverse joints in adjoining paver runs should be offset by not less than 2 m
- transverse joints should be offset from one layer to the next by not less than 2 m
- longitudinal joints should be offset from one layer to the next by not less than 150 mm
- longitudinal joints should be located within 300 mm of the planned position of traffic lane lines or within 300 mm of the centre of a traffic lane.

The exposed end of each lot and the exposed edges of any part width construction should be kept moist until spreading and compaction has been completed over the entire layer (VicRoads 2008b).

## 6.8 Compaction of Granular Materials

It is important to identify and remove any pockets of segregated material prior to compaction and to replace them with uniformly graded product. Segregation in the form of accumulation of the coarser particles often occurs as the material is unloaded from the delivery truck into a windrow and again when the material is spread across the pavement to form the required loose layer thickness (Midgley 2008).

The methods and equipment used to compact granular materials in unbound pavement construction are as outlined for earthworks materials in Section 4.8. The most common compaction equipment used for unbound granular pavements is the smooth drum steel-wheeled, pneumatic multi-tyred and vibrating smooth drum rollers. Typically a smooth drum steel-wheeled roller (Figure 6.4) in static mode is used for the initial pass after spreading followed by approximately four passes of a smooth drum steel-wheeled roller in vibratory mode operating on low frequency and high amplitude. A pneumatic multi-tyred roller operating with high tyre pressure to knead the surface into a tight, evenly textured finish is typically used for final rolling in preparation for application of the surfacing (Midgley 2008).

Knowledge of the loose to compacted thickness ratio (typically in the order of 80-90%) is important to ensure the specified pavement thickness is achieved. Loose spread thickness should be checked by manual means or use of automatic level control as discussed in Section 4.3 (Midgley 2008).

Unbound granular pavements are typically constructed in maximum 150 mm thick layers. However, maximum and minimum allowable layer thickness varies between road authorities. For example NZ Transport Agency specifies a minimum layer thickness of 2.5 times the maximum particle size and maximum layer thickness of 200 mm (Transit NZ 2005), RTA NSW specifies minimum layer thickness of 100 mm and maximum layer thickness of 150 mm (RTA 2007a) while Queensland Department of Main Roads specifies minimum layer thickness of 75 mm and maximum layer thickness of 250 mm (QDMR 1999).

Where the specified layer thickness is constructed in two or more courses, for example subbase and base, the surface of the underlying course may be lightly scarified/tyred to a depth not exceeding 25 mm (RTA 2007a).

**Figure 6.4: Smooth drum steel-wheeled roller compacting granular material**



Source: RTA

## 6.9 Preparation Prior to Surfacing

Surface preparation is a part of the base course construction process, and it is preferable to complete the finishing operation during this phase, or at least within 24 hours. However, there may be situations where this may not be possible. In these cases it is preferable to construct the base 25-50 mm thicker than the specified thickness. This will allow returning to the work, tining the surface, adding water and then undertaking the finishing operation without adverse quality impacts.

During the placement there is a tendency to draw fines to the surface; however, this practice can be detrimental to the pavement. In the first instance the material has only a certain percentage of fines; if these are drawn to the surface by compaction, then other areas of the pavement will become more open-textured, increasing the air voids with a consequent reduction in stability and strength. The other problem this can cause is that it produces a thin lens of fine material on the pavement surface, reducing the bondage between the pavement and the bituminous surfacing.

Once a pavement has been compacted and trimmed, a soil binder should never be added to the surface to fill depressions to meet surface tolerance. This again produces a weak layer or lens that will not be effectively bonded to the pavement.

Before any form of bitumen surfacing or asphalt wearing surface construction is undertaken, the new pavement surface should be inspected for any of the following:

- Corrugations – which should be removed by wetting the surface and then trimming 10 mm off the top with a grader, then recompacting the trimmed surface or replacing the layer.
- Depressions – which should be rectified by either a light scarification with a grader, new materials added if required, and then compacted ensuring no surface lenses occur, ripping, remixing and recompacting the full layer or replacing the layer.
- Surface blemishes such as coarse textured areas – which should be rectified by the addition of fines. Areas that have a fine lens of material, due to the roller pumping fines to the surface during compaction, should be removed by brooming.
- Any soft spots – which should be removed and replaced by new materials.

After final trimming the surface of the pavement layers should be uniformly tight and free of loose uncompacted material, segregated or bony material or soft, over-wet areas and free of roller indentations (DTEI SA 2007).

The surface of any compacted pavement layer should be maintained in such a way as to minimise dust (Figure 6.5), prevent ravelling, erosion, deformation or any other damage to the layer resulting from environmental conditions, traffic or construction activities. The layer should also be kept free from contamination until any subsequent pavement work is commenced (VicRoads 2008b).

Should a surface appear 'hungry', with a coarse texture, slurring up with water as part of the long grading process and rolling the moisture out using a pneumatic multi-tyred roller often achieves good results. However, over-working can sometimes produce excessive fines resulting in a 'glassy' surface finish, which is undesirable for application of a bituminous surfacing (Midgley 2008).

**Figure 6.5: Water cart ensuring surface of granular material does not dry out prior to surfacing**



Source: RTA

To reduce the risk of premature failure it is often necessary to allow the upper pavement layer to 'dry-back' prior to application of the surfacing. Depending on the climatic conditions and the compacted moisture content a period of a number of days, up to one week, may be required for the pavement to achieve the required dry-back before it is ready for surfacing (Midgley 2008).

Testing of whether an unbound granular pavement is ready for application of a bituminous surfacing is typically in the form of either non-destructively measuring the moisture content of the upper pavement layer, to gauge whether the material has dried back sufficiently, or using the ball penetration test method. Where pavement moisture content is measured it is good practice to ensure that the moisture content of an unbound pavement course prior to placing a wearing surface is less than 60-70% of OMC or 60-70% degree of saturation. Where the ball penetration test method is used pavements with measured values greater than 2 mm may result in seal flushing and pavements with a result greater than 4 mm should not be sealed and should be allowed to further dry-back until a satisfactory result is obtained (Midgley 2008).

## **6.10 Conformance and Quality Testing**

### **6.10.1 Material Supply**

Austrroads (2008a) contains advice on the selection, testing, quality control and specification of crushed rock and naturally occurring granular materials for use in pavement base and subbase construction.

### **6.10.2 Trial Pavement**

Unbound pavement construction specifications may require the contractor to construct a trial pavement using the material, equipment and construction methodology proposed in the project quality plan prior to commencing general pavement works.

The trial pavement is typically minimum 100 m long and, depending on the road authority, may or may not form part of the final works. Generally a hold point is applied, requiring submission of documentation confirming process control and conformity of the trial pavement, prior to the principal permitting further pavement construction.

### **6.10.3 Thickness and Level Tolerances**

To ensure that the pavement is constructed within specified limits so that excessive variations do not occur in pavement thicknesses and horizontal grades and crossfalls, tolerances are placed on both vertical and horizontal dimensions.

Tolerances vary according to the specifying authority; however, typical tolerances are:

#### ***Horizontal***

- +/- 50 mm

#### ***Vertical***

- a general subbase surface tolerance of +/- 15 mm at any point
- a general base surface tolerance of +/- 10 mm at any point
- +/- 5 mm of the lip of a kerb, where the pavement is to be constructed to the lip level of kerb and gutter.

#### ***Layer Thickness***

- subbase +/- 15 mm of the layer thickness specified at any point

- base +/- 10 mm of the layer thickness specified at any point.

### Shape

- no point on the surface of any pavement layer should vary by more than 5-10 mm from a 3 m straight edge (DTEI SA 2007, RTA 2007a, VicRoads 2008b)
- at no location should water pond on the surface of any pavement layer.

### 6.10.4 Compaction

To achieve the desired quality of unbound pavement a minimum characteristic value of relative compaction is typically specified; however, the minimum value differs between road authorities depending on the location, pavement design and traffic volume. Table 6.1 shows typical minimum characteristic value of relative compaction specified.

**Table 6.1: Example of acceptance criterion for compaction**

	Characteristic value of relative compaction	
	Subbase	Base
QDMR, Queensland (Type 1)	≥ 102.0% (standard)	≥ 102.0% (standard)
QDMR, Queensland (Type 2,3,4)	≥ 100.0% (standard)	≥ 100.0% (standard)
RTA, NSW	≥ 102.0% (standard)	≥ 102.0% (standard)
VicRoads, VIC (Scale A)	≥ 98.0% (modified)	≥ 100.0% (modified)
VicRoads, VIC (Scale B)	≥ 97.0% (modified)	≥ 98.0% (modified)

Source: QDMR (1999), RTA (2007a) and VicRoads (2008b)

### 6.10.5 Ride Quality

It is common for the ride quality of the finished surface of an unbound pavement to be specified. In such cases, the ride quality is typically measured within 1-2 weeks after the prime or primerseal has been applied and swept.

Typically a maximum ride quality of 60-70 NAASRA Roughness counts per kilometre or 2.3-2.7 IRI (International Roughness Index), measured using either a NAASRA Roughness Meter or laser based profilometer (Figure 6.6), is specified for new work.



**Figure 6.6: Survey vehicle featuring laser profilometer to measure ride quality**



Source: ARRB

### 6.10.6 Frequency of Testing

The minimum frequency of testing should be that which is sufficient to ensure that materials and work supplied under the contract comply with the specified requirements. The minimum frequency specified varies between road authorities as it is typically based on prior experience with the materials available and/or the compaction methods used. Some road authorities require testing of each and every lot while others have less stringent testing requirements or allow a reduction in lot testing once a certain number of concurrent conforming lots have been achieved.

Figure 6.7 and Figure 6.8 provide an example of the minimum test frequency specified for unbound pavement construction. In this case the specifying authority typically requires every lot to be tested and Figure 6.7 details the number of samples to be taken per lot. The relative compaction requirements shown are for standard compaction.

Figure 6.7: Example of sampling and testing frequency requirements (1)

**L3.1 Sampling and Testing**  
 The number of samples per lot (n) must be not less than:

Specified relative compaction (%)	Minimum testing frequency for lot area of:				
	> 5000 m <sup>2</sup>	1000-5000 m <sup>2</sup>	500-1000 m <sup>2</sup>	50-500 m <sup>2</sup>	≤ 50 m <sup>2</sup>
≤ 90.0	1 per 3000 m <sup>2</sup>	1 per 2000 m <sup>2</sup> (min. 2)	1	1	1
> 90.0 ≤ 95.0	1 per 2000 m <sup>2</sup>	1 per 1000 m <sup>2</sup> (min. 3)	1 per 250 m <sup>2</sup> (min. 3)	2	1
> 95.0 ≤ 98.0	1 per 2000 m <sup>2</sup> (min. 6)	5	4	3	1
> 98.0 ≤ 100.0	1 per 2000 m <sup>2</sup> (min. 6)	5	4	3	1
> 100.0	1 per 1000 m <sup>2</sup> (min. 10)	1 per 500 m <sup>2</sup> (min. 5)	4	3	1

Notes:

- Where the sampler/tester can assure that the work is homogeneous and has been carried out within the same day under homogeneous conditions, and:
  - where the minimum specified compaction is below 100.0%, work in separate areas, up to a total area of 1000 m<sup>2</sup>, may be considered as one lot; or
  - where the minimum specified compaction is below 98.0%, layers may be covered before testing and may be considered as one lot, subject to the following:
 

Sum Total Area of layers:	< 100 m <sup>2</sup>	101-500 m <sup>2</sup>	501-1000 m <sup>2</sup>
Maximum number of layers:	5	3	2
Maximum thickness of Lot:	600 mm	600 mm	600 mm
Minimum number of Tests:	1	2	3

 The tests must be evenly distributed throughout the layers and areas of the Lot.
- Lots less than 2 m wide must not be longer than 150 m.
- Except as stated in 1(b) above or specifically allowed by the relevant specification, the lot will only be one layer.

Source: RTA (2008a)

Figure 6.8: Example of sampling and testing frequency requirements (2)

**L1 Minimum Frequency of Testing**

The minimum frequency of testing of materials, and testing during production and end product stage are listed in Tables R71/L.1 and R71/L2 respectively.

**Table R71/L.1 – Minimum Frequency of Testing of Materials.**

Process control for construction			
7.1.2	In situ density	RTA 119, RTA T173	As per Specification Q for specified relative compaction
7.1.3	Maximum dry or wet density	RTA111, RTA 162	As per Specification Q for specified relative compaction
7.1.4	Field moisture content	RTA T120, T121 or T180	As per Specification Q for specified relative compaction
7.2	Pavement layer thickness	Testing as per Clause 7.2	At least one site per 75 metres, with a minimum of 2 per Lot
7.3	Surface level	Testing as per Clause 7.3	As per PQP
6.5	Pavement width		Minimum of 1 per 20 lineal metres
6.6	Ride quality	RTA T182 or T187	Continuous reading per lot
7.3	Deviation from straight edge	Testing as per Clause 7.3	Minimum 1 per 20 m2

Source: RTA (2007a)

## 7. Stabilised Pavements

### 7.1 General

To achieve the desired outcomes of a stabilised pavement it is essential that construction be undertaken in a sequential and methodical manner utilising equipment specifically designed for the task it is required to undertake.

Information about the quality of the proposed pavement materials is essential for stabilised pavement construction. An assessment of the material types, within and beneath the pavement, including their extent and condition, may be required for a full appreciation of pavement performance. Where in situ stabilisation of the existing pavement is an option, materials may need to be sampled from the test pits for use in laboratory mix design of the stabilisation treatment. If data from the visual survey, roughness testing and other sources suggest the subgrade may be expansive, the subgrade material should also be sampled to confirm this hypothesis. The presence of otherwise of rock bars should also be investigated as these have the potential to damage stabilisation equipment.

Prior to undertaking stabilisation it should be ensured that the climatic conditions are suitable for both the construction process and binder reactions to proceed. Australia and New Zealand have a diverse climate and it is not uncommon for pavement construction and rehabilitation to be undertaken during adverse weather conditions; however, cold, hot or wet weather may be detrimental for road stabilisation (Austroads 2002b). In addition, as with all road construction, both progress and quality are impaired if undertaken during heavy rain. As such, specifications typically preclude stabilisation works during wet or windy weather and require that spreading of the binder cease during periods when wind is sufficiently strong to cause particles of binder to become airborne and during conditions that may cause nuisance or danger to people, property or the environment. If, after mixing or during compaction, heavy rain falls, the moisture content of the material being stabilised may increase beyond an acceptable limit and under such situations consideration should be given to aerating in order to reduce the moisture content and aid in achieving compaction.

In relation to successful binder reactions occurring, high ambient temperatures accelerate the reaction of cement and cementitious binders such that there may be insufficient working time available for effective compaction and finishing of the stabilised layer. At low temperatures the binder reaction of cement and cementitious binders is slower, with the rate of strength gain of the stabilised material greatly reduced.

For bituminous binders, especially foamed bitumen stabilisation, low ambient temperatures and cold materials can impact on the foaming process and it is recommended that this process is not carried out when the pavement material is below 10 °C.

### 7.2 Specification and Supply of Materials to be Stabilised

Guidance on selection, testing, quality control and specification of materials for use in stabilised pavement construction is provided in the *Guide to Pavement Technology – Part 4D: Stabilised Materials* (Austroads 2006a) and *Part 4L: Stabilising Binders* (Austroads 2009f).

### 7.3 In Situ Stabilisation

This section has been developed from the Guide to Best Practice for the Construction of In situ Stabilised Pavements (Austroads 2003).

### 7.3.1 Production Levels

There are many factors affecting production levels; however, experience has shown that production rates for stabilisation depths up to 200 mm are 3,000 to 5,000 m<sup>2</sup> per day and 2,000 to 4,000 m<sup>2</sup> per day for depths greater than 200 mm for one set of plant. In planning an in situ stabilisation project all factors affecting production levels should be taken into account to minimise the risk of assuming unrealistic production rates.

### 7.3.2 Initial Site Preparation

Prior to stabilisation it may be necessary to:

- undertake initial dry mixing of the pavement to a depth 50 mm less than the anticipated full design depth to break down bituminous seals or oversize material
- add granular material to correct existing pavement material deficiencies or levels
- remove thick bituminous surfaces which cannot be adequately mixed
- remove thick asphalt and cemented material patches and replace with granular material of similar quality to that of the surrounding pavement
- remove a top layer of existing pavement in order to stabilise a lower layer in two layer stabilisation
- break up the pavement or subgrade to assist with removal of excessive moisture content.

In urban areas where there are fixed levels, an increase to the levels from a granular overlay will necessitate the raising of kerb and gutter, and medians as well as the adjustment to driveways, footpaths and service covers. Where kerb levels cannot be altered, incorporating a recycled pavement can still be accomplished by lowering the formation level and reconstituting a thicker overlying pavement. The pavement material is removed to one side to expose the formation which is then set at a new level followed by the existing pavement material plus overlay addition being placed and stabilised in the conventional manner.

Where a pavement has a thick bituminous surfacing, the need to remove it prior to stabilisation is generally based upon the degree by which it can be mixed and, if incorporated, its net effect on the grading of the material being stabilised.

Where asphalt patches represent a significant proportion (about 30% or greater) of the road surface area, incorporation of the recycled asphalt pavement (RAP) material may lead to a non-homogeneous base material for stabilisation. In such cases, the asphalt patches should be removed and replaced with suitable granular material.

For thick and thin asphalt surfaces on urban roads it is generally more efficient to use a pavement profiler to plane out and remove the asphalt rather than graders or front-end loaders, which lift the asphalt in slabs.

### 7.3.3 Application and Mixing of Powder Binders

Powder binders are applied by the following techniques:

- spreading the binder using a load calibrated mechanised spreader from a rear or centrally located drop chute (Figure 7.1)
- spreading the binder from a reclaimer with an integrated hopper where the binder is carried in the reclaimer and spread ahead of the mixing chamber (Figure 7.2)
- spotting bags of powder binder and hand raking is only recommended for minor patching, small remote projects (where a spreader is impractical or costly) and odd shaped sections e.g. tapers (Figure 7.3).

**Figure 7.1:** A mechanised spreader with rear drop chute



Source: Austroads (2003)

**Figure 7.2:** A reclaimer with an integrated powder binder hopper and load cells



Source: Austroads (2003)

**Figure 7.3:** Spotting cement bags and hand raking binder into position



Source: Austroads

Whilst it is common to specify binder content as a percentage of the compacted dry mass of pavement, construction adopts a spread rate in terms of  $\text{kg/m}^2$ . Two methods of verifying the binder application rate for mechanised spreaders are:

- use of trays or mats (Figure 7.4) which are weighed
- use of measurements from on-board load cells on the spreader.

**Figure 7.4 Use of trays to verify the binder application rate**



Source: Austroads (2003)

Most specifications limit the maximum application rate for a single spread and mixing pass to  $20 \text{ kg/m}^2$ . For specified application rates exceeding  $20 \text{ kg/m}^2$  binder should be spread in approximately equal rates. When using quicklime a lower maximum application rate of about  $15 \text{ kg/m}^2$  is typically used to assist in slaking of the quicklime.

As the typical density for a powder binder is about  $500 \text{ kg/m}^3$  this equates to a depth of binder of about 40 mm when spread at  $20 \text{ kg/m}^2$ . The binder does not retain a vertical face at the edge of spreading, resulting in an outwards flow. With light fluffy binders, the system of surface spraying with water over the spread binder will increase the density of the binder and can minimise the sideways movement at the edges of the spread run. With conventional spreading and mixing the front wheels of the reclaimer will cause the binder to flow to the sides.

The practical lower limit for spreading a binder is about 5 to  $10 \text{ kg/m}^2$ , and is a function of the loose density of the binder and the mixing depth. It is recommended that a trial pavement be carried out to confirm the uniformity of mixing the binder into the pavement material.

Consideration needs to be given to retaining spread binder on a steep crossfall or incline. This may be in the form of reducing the application rate to limit the potential flow, or the provision of physical barriers (such as windrows) to reduce the flow of the binder on the surface prior to mixing.

After spreading, construction equipment may move the binder sideways resulting in a variable transverse application rate (Figure 7.5). It is therefore important during construction to restrict construction plant using the road until the binder has been mixed.

Figure 7.5 Construction traffic over spread binder may alter distribution



Source: Austroads

Desirable features of mechanical spreaders are (Figure 7.6):

- On-board load cells that allow the mass of binder to be determined before and after spreading. The load cells are typically calibrated by using a weighbridge to confirm the mass of binder at various discharge levels in the tanker. Readings taken while the spreader is on steep grades or moving are not accurate because of dynamic effects on the load cells. On steep grades a tray test may need to be carried out to confirm spread rates.
- Rear drop spreaders that eliminate wheels tracking through the spread binder.
- Pneumatically operated gates to allow variable-width spreading.
- Aprons surrounding the gates to ensure the discharge of the binder is directed onto the road surface during moving operations.
- A sealed tanker to avoid binder being discharged during loading operations. Modern spreaders incorporate a filter system to minimise dust generation during the transfer of the binder from the bulk carrier to the spreader. These filter systems use filter bags and must be regularly cleaned.
- A reasonable capacity to minimise refilling operations and typically this is 15 t to 20 t for a cement type binder. Other cementitious blends may be less dense, limiting the tanker's capacity to 10 t to 15 t in the same size tanker.



Figure 7.6: Desirable features of a mechanical spreader



Source: Austroads (2003)

Agricultural spreaders and tipper style spreaders should not be used as the binder rate cannot be controlled with any accuracy.

Care must be taken when leaving binder in a spreader for prolonged periods (e.g. while travelling to the next site) as the binder will tend to compact, increasing its density and resulting in either a poor transverse distribution or a higher application rate when subsequently used.

The rotor system on the spreader must be kept clean and tight at all times as moist air will cause the hydration of cementitious material reducing the effectiveness of the cementing action due to the formation of aggregated lumps.

Control of powder binder application rates in direct injection systems can only be accomplished by measuring the amount of binder used over a specific length of pavement. As a result there is a degree of uncertainty about the uniformity of spreading (longitudinally and transversely).

Spread rates with hand spreading are variable and this method is recommended for use only with stabilised patches less than 15 m<sup>2</sup> (Austroads 2002a, AustStab 2000).

### ***In situ lime slaking***

The recommended maximum application rate for spreading and slaking of quicklime is 15 kg/m<sup>2</sup> to ensure complete hydration before mixing.

Slaking is undertaken either after spreading quicklime, by surface spraying with water and additional water added during the mixing operation, or slaking prior to spreading as a liquid binder.

Two methods to identify when quicklime has been fully slaked are:

- A thermometer to assess the maximum rise in temperature as calcium oxide is converted to calcium hydroxide.
- A visual observation of the change from a granular material to a fine powder. Confirmation can be assessed by rubbing the lime between the thumb and finger and noting if the lime is no longer gritty. A suitable type of glove should be worn when handling quicklime.

In some wet materials it is not possible to add all the quicklime in one operation as the amount of water required to slake it could exceed the material's OMC. In such cases quicklime may be added in stages provided the final addition is within 72 hours of the first application. Where this practice is adopted for base materials, unslaked quicklime may undergo further chemical reactions over some time after compaction that may lead to expansion of the material and, in particular, near the surface is likely to heave the bitumen seal. To minimise this effect it is recommended that another mixing pass with the addition of some water be carried out.

### 7.3.4 Application and Mixing of Liquid Binders

Liquid binders are either added by conventional water truck fitted with a spray bar (typically slurries and chemicals – Figure 7.7) or by directly pumping from a tanker into the mixing chamber of the recycler (typically bitumen emulsion, foamed bitumen, slurries and chemicals – Figure 7.8).

**Figure 7.7: Lime slurry spreading**



Source: Austroads

**Figure 7.8: Liquid binder (foamed bitumen) added directly through recycler**



Source: Austroads

Where liquid binders are sprayed from a tanker the binder application rate is preferably determined from a fluid meter installed on the binder delivery pipe to measure the amount consumed in a specific pavement length. For major works a calibrated sprayer is often required.

Often with chemical binders, the amount of binder is expressed in litres/m<sup>3</sup> of pavement material and the amount of binder required is added with the calculated amount of water needed to be added to reach optimum moisture content.

For liquid binders (specifically bituminous binders) directly fed to the recycler, verification of the application rate is either through frequent tanker dipping or a calibrated onboard flow meter. When using tanker dipping readings the operator should take into account the bitumen temperature, the grade and crossfall of the road and the shape of the tanker. The dipping measurement should not be carried out while the tanker is moving as it is unsafe. A dipping should be carried out on a suitable flat grade to minimise adjustments to the reading.

### 7.3.5 Adding Water During Stabilisation

Two methods by which water may be introduced into the pavement material during stabilisation are spraying from a water tanker onto the surface prior to mixing or incorporating the water directly into the mixing chamber through water jets located in the mixing chamber. The advantages and disadvantages of the two methods are listed in Table 7.1.

**Table 7.1: Advantages and disadvantages of methods for adding water during stabilisation**

Advantages	Disadvantages
<b>Water tanker approach</b>	
Visual check of extent of water being applied. Water tanker does not need to be hooked to stabiliser.	If excess moisture is applied, the distance for adjusting the water content is long, leading to a larger patch of wet area. Water may run off the pavement. If too dry after the first mixing pass, another pass would be required and thus reducing productivity.
<b>Water-in-chamber approach</b>	
Water applied during mixing and directly into material. Moisture level adjustments are quick. Even distribution of water across and within the mixing depth of the pavement material.	Water tanker needs to be hooked up to reclaimer. Space needed for both tanker and stabiliser.

### 7.3.6 Mixing

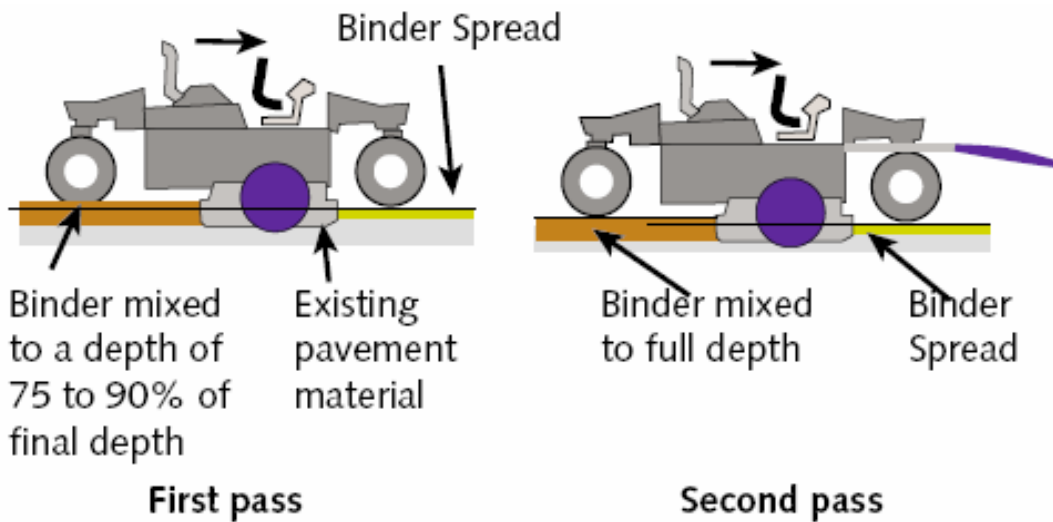
Pulverisation of the pavement and incorporation of the binder results in bulking of the pavement material. The degree of bulking needs to be taken into account to meet pavement thickness and survey levels where specified. Most stabilising operations using powder binders involve two passes of the stabiliser. In the first pass, the mixing depth is typically about 70 to 90% of the final depth to reduce the impact that bulking has on mixing to the target depth (Figure 7.9 and Figure 7.10).

Figure 7.9: Two pass mixing (1)



Source: Austroads (2003)

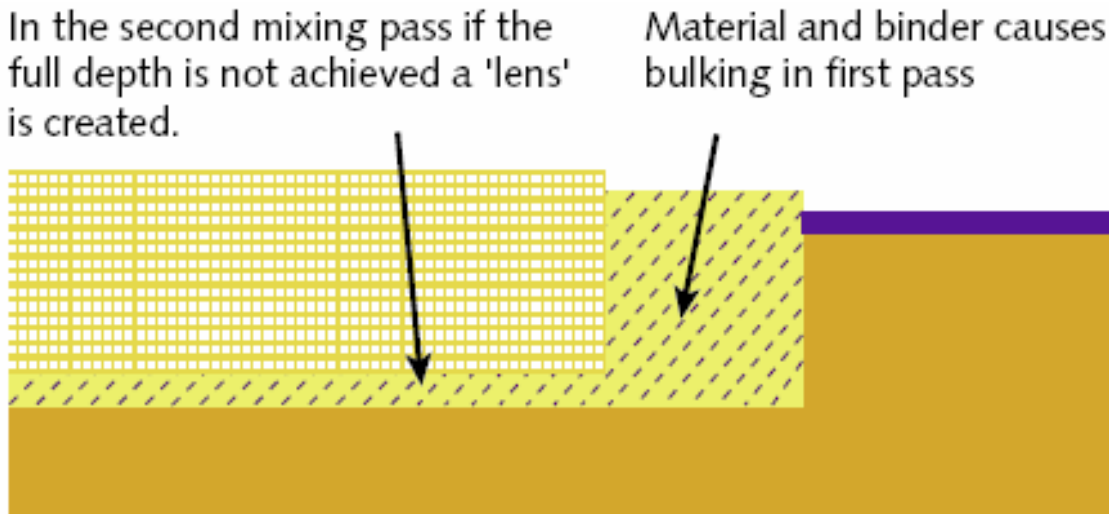
Figure 7.10: Two pass mixing (2)



Source: Austroads (2003)

Should the final stabilisation depth be carried out in the first pass, and the binder is incorporated in the second pass, if the mixing operation is at a lesser depth than the final target depth, the end result is a lack of binder in the critical lower region of the pavement layer which will impact on the performance of the pavement (Figure 7.11).

**Figure 7.11: A lens of no or little binder may occur when the first pass mixing operation is deeper than the second pass**



Source: Austroads (2003)

When the mixing chamber is centrally located between the front and rear axles of the stabiliser, the operator can optimise the mixing procedure to meet the depth requirements. Rear-mounted mixing chambers tend to lift when encountering sections of stiffer pavement material leading to lesser mixing depths. Rear-mounted mixing chambers are beneficial where there are many transitions or manholes since they allow the operator to get the mixing operations closer.

In remote areas or for small jobs, mixing binders (either liquid or powder) by windrowing with a grader is commonly undertaken specifically in unsealed road sheeting operations. When mixing powder binders, water is only added after mixing in order to shape and compact within the allowable working time of the binder. For liquid chemical binders, mixing should be completed before significant drying of the pavement material occurs and before compaction as the addition of more water may dilute the binder application rate.

Measurement of the stabilised pavement depth can be undertaken after the second or final pass of mixing by either survey measurement or by digging a small hole and, with the use of a stringline, assessing the depth of the stabilised layer as the difference between the target surface level (i.e. the level expected after compaction) and the bottom of the stabilised layer. The depth should also be checked after final trimming or after the primerseal is applied.

### 7.3.7 Joints

Vertical transverse joints in stabilised pavements occur at the start and finish of a stabilisation lot. The overlap at the commencement of work on the adjoining lot for transverse construction joints should be 1.5 m due to the size of the drum and the cylindrical shape of the drum to ensure full mixing and compaction is achieved at the joint.

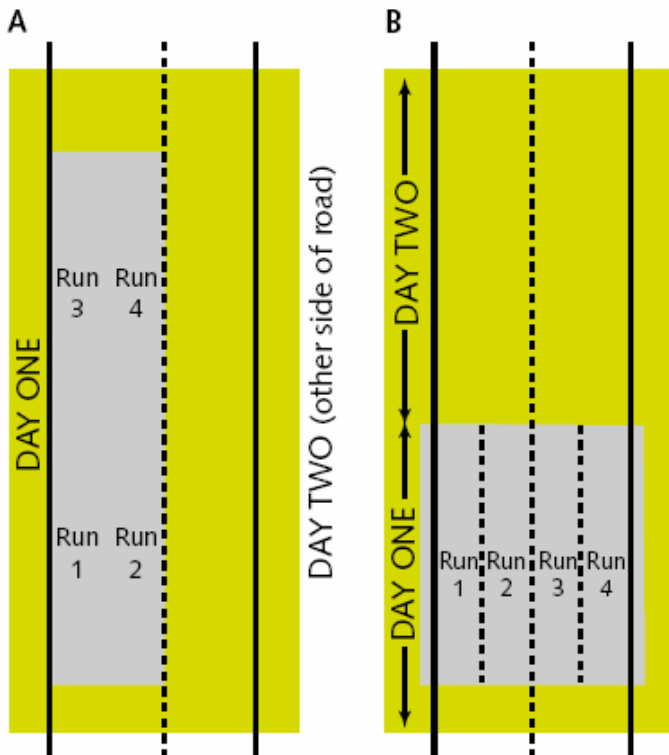
A longitudinal joint is formed when material has been compacted against an adjacent section that has previously been mixed and compacted, and the nominated working time for that section has expired. Overlap for a longitudinal joint should be at least 100 mm and longitudinal joints should be located well clear of the wheel paths.

Where a greater overlap is specified along a longitudinal joint, care must be taken not to over wet the overlap area during the mixing operation as too much moisture in the mixed material may result in a decrease in strength and induce shrinkage cracking along the joint. Equipment should have suitable switches to turn off water for particular water jets.

Where the width for compaction of several adjacent runs exceeds 5-6 m, consideration should be given to the construction of a planned longitudinal joint to avoid the possibility of the formation of an unplanned longitudinal crack.

Figure 7.12 shows typical daily operations for a lane by lane or full road-width mixing process for rural roads. The decision to use one or the other is based on the size of the project, traffic volume of the road and equipment size and configuration. In a local road full-width mixing is common as stabilisation depth is typically less than 250 mm and roads are normally closed between cross streets.

Figure 7.12: Typical spreading and mixing arrangements for half-width (A) and full-width (B)



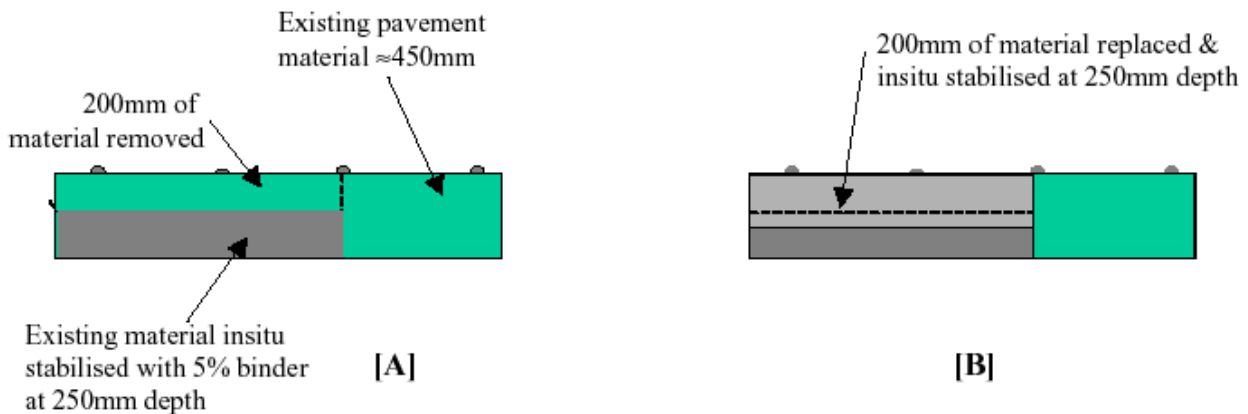
Source: Austroads (2003)

### 7.3.8 Multi-layered Stabilised Pavements

Multi-layered stabilised pavements constructed with plant-mix or in situ techniques are not common due to the risks associated with achieving a long-term bond between layers.

At sites where compaction in thinner lifts is necessary, in situ stabilisation in two consecutive layers may be carried out. Typically, the top of the existing pavement material is removed, for example to 200 mm, to allow stabilisation of the lower layer with a cementitious binder (Figure 7.13A). This is followed by replacing the top material and stabilising this layer together with the uppermost 50 mm of the lower layer in the final phase, as shown in Figure 7.13B. By following this procedure, the full depth of stabilisation is less likely to be affected by a density profile and associated modulus variations with depth.

Figure 7.13: Typical two layered in situ stabilisation of heavily trafficked highways



Source: Austroads (2003)

There is limited performance data on the multi-layered stabilised pavement process and the risks inherent with it are:

- Good mixing-depth control is required in cutting into the lower layer during the stabilisation of the upper material, otherwise an undetected lens of unstabilised material may occur.
- The binder content and mixing effectiveness can vary along the edges of stabilisation runs, and non-uniform mixing has been observed in the longitudinal joint zone of widened pavements.
- Where significant delay between placement of the lower and upper layers occurs, any shrinkage cracking in the upper material may terminate at the interface of the two bound layers, rather than continue through the lower material. This may result in the ingress of moisture to (and laterally along) the interface and may cause localised de-bonding of the bound layers.

Cameron and Mathias (2000) describe the results of various interlayer treatments to optimise bonding between cement stabilised pavement layers.

### 7.3.9 Compaction

Compaction should commence as soon as practicable after mixing. Where there are one or more adjacent runs of the stabiliser, a width of approximately 300 mm adjacent to the common longitudinal joint should remain uncompacted until the subsequent run has been completed. This procedure minimises the risk of a longitudinal crack appearing at the joint as shown in Figure 7.14.

**Figure 7.14: Longitudinal shrinkage cracks**



Source: Austroads

Compaction should be completed within the working time of the binder. An example procedure for determining the specified working time is described in VicRoads (2000). The working time in the field commences at the time of the mixing of the binder.

For deep-lift stabilised pavements, the common types of rollers used for compaction are a padfoot vibratory roller (typically minimum 18 t) and / or road compactor (34 t), and a smooth drum, steel-wheeled roller. The padfoot roller or road compactor (Figure 7.15) is most effective in the lower portion of the pavement layer and the smooth drum effective in the upper portion of the pavement. It is essential that the padfoot roller is walked out of the compacted area as compaction nears the surface to eliminate the effect of padfoot markings and the potential to induce delamination near the surface of the pavement layer.

**Figure 7.15: Static padfoot (left) vibrating padfoot (right) compactors**



Source: Austroads

A pneumatic multi-tyred roller is sometimes used as a finishing roller to knead the surface and to 'close' the surface pores (Figure 7.16).



Figure 7.16: Smooth drum and rubber tyred roller in operation



Source: Austroads

Methods of assessing when compaction has been completed and further rolling will not increase density include:

- The use of an accelerometer attached to a vibrator roller.
- The use a trial section to record the number of passes to reach the desired degree of compaction.
- Proof rolling (generally associated with allowable deflection and surface stability).
- Use of devices, such as the Clegg Hammer. (The Clegg Hammer or other devices will need to be calibrated).

The surface must not be slurried as this may cause delamination and also produce a soft crust that will cause problems with sealing adhesion.

When stabilising weak granular material, compacting at maximum dry density may cause the particles to break down. In such cases, additional compaction may not result in increased density and specifying lower relative compaction values may be more appropriate.

When using vibratory rollers greater than 18 tonnes, care should be taken to ensure that structures in close proximity to the road are not structurally or architecturally damaged or cause nuisance to residents.

### 7.3.10 Curing

Curing of a cementitious stabilised material may be achieved by:

- lightly and frequently spraying water until the bituminous surfacing or the next layer is constructed
- bituminous surfacing
- constructing the next layer.

Experience has shown that lack of curing may result in reduced pavement strength, surface cracking and subsequent ravelling under traffic if a thin wearing surface is constructed on top of the stabilised layer.

Bitumen stabilised materials do not require curing by wetting the surface.

### 7.3.11 Levelling and Trimming

The particle size distribution of granular materials is important when trimming. Where there is a high proportion of coarse aggregate, the quantity of fines may not be sufficient to fill the surface voids during trimming by a grader, resulting in a weak and 'boney' surface that may either ravel under the rollers or early trafficking.

Where recycled asphalt pavement (RAP) material is incorporated in the pavement layer, the presence of large size particles may cause drag marks in the surface during trimming.

In final trimming, material cut to waste should be minimised to ensure the specified level and thickness are met. Levels may be determined by survey or by using stringline measurements taken at close intervals (about every 25 m) to reduce longitudinal roughness.

Trimmed material must not be graded over existing low spots of fully compacted material and then compacted. This may result in delamination of the road under trafficking (Figure 7.17).

**Figure 7.17: Delamination of stabilised material**



Source: RTA

## 7.4 Plant Mix Stabilisation

### 7.4.1 Manufacture

The type of plant shown in Figure 7.18 is ideally suited to be placed in a quarry or borrow pit. It has the capability of blending a number of materials as well as the additive. The materials and additive are blended through a pugmill, which then discharges into either a truck, or off-road delivery vehicle. The blended material is normally laid on site through a paver.

The parent crushed materials (quarried or recycled) for plant mixing are typically stockpiled close to the mixing plant generally as approved or compliant stockpiles in terms of meeting crushing and screening specifications.

The stationary plant is most applicable where all the material is sourced from a single supplier, a high degree of uniformity is required and the haul to the placement area is relatively short. Care should be taken in using pugmills to ensure that the additive selected has a working time sufficient to allow for haulage to site, placement and compaction.

**Figure 7.18: Stationary mixing plant and pugmill chamber**



Source: Austroads

### 7.4.2 Placement, Compaction and Curing

The delivery requirements of plant-mixed stabilised materials depend upon the designated working time and ambient weather conditions at the time of placement.

In general, it is recommended that, regardless of the weather conditions, all plant-mixed stabilised material is delivered in covered trucks with a maximum transportation time of half the working time of the binder. In this manner, loss of moisture through evaporation or wetting due to rain is minimised during transportation and sufficient time to place and compact is available at the site.

Placement is generally undertaken by grader spreading and shaping or by paver as shown in Figure 7.19. When selecting a paver, the width should be governed by the ability of the rollers to apply full compaction within the working time without excessive drying out. In addition, for widths in excess of 5-6 m, consideration should be given to inclusion of a longitudinal joint due to the degree of shrinkage and temperature expansion/contraction in the stabilised layer.

**Figure 7.19: Paver laying stabilised material**



Source: Austroads

Moisture content checks from sampling immediately after placement but prior to compaction are recommended. To allow adequate compaction it is important that the field moisture content is close to optimum. For cementitious stabilisation, field measurement of moisture content is also important to ensure an appropriate water/cement ratio is maintained, allowing development of adequate strength.

## 7.5 Granular (Mechanical) Stabilisation

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

Granular stabilisation may be used to improve the performance of granular materials with moderate or high plasticity and/or poor grading in sealed or unsealed pavements or improve granular material in shoulder pavements to enable the application of a seal.

Granular stabilisation is not appropriate in pavements where the increased permeability of the stabilised material is incompatible with that of abutting materials and hence likely to contribute to subsequent moisture-induced distress or in pavements overlying wet or weak subgrades.

The most two common methods of granular stabilisation are via pugmill, as described in Section 7.4, or via a mechanical stabiliser, as described in Section 7.3.

## 7.6 Surface Sealing

Sealing of the surface is normally carried out after the assessment of relative compaction and other conformance criteria have been met (refer to Section 7.7).

The critical requirement for sealing is for adhesion of the seal to the surface of the stabilised layer. Some factors effecting adhesion include:

- dust on the surface
- surface porosity and voids
- ability of the bitumen to penetrate the compacted surface
- pavement surface temperature (Austroads 2002c)
- any slurried surface.

Guidelines for good sealing practice should be followed as per the *Guide to Pavement Technology – Part 4K: Seals* (Austroads 2009e), *Update of the Austroads Sprayed Seal Design Method* (Austroads 2006i) and Section 8.

While it is common to use water to cure a cementitiously stabilised pavement layer, if too much water is used during curing a slurry may form due to trafficking of the pavement. The removal of this slurry prior to bituminous surfacing should be part of the final trimming and surface preparation process.

## 7.7 Conformance and Quality Testing

### 7.7.1 Material Supply

The *Guide to Pavement Technology – Part 4D: Stabilised Materials* (Austroads 2006a) and *Part 4L: Stabilising Binders* (Austroads 2009f) contain advice on the selection, testing, quality control and specification of materials for use in stabilised pavement construction.

### 7.7.2 Thickness and Level Tolerances

Level tolerances for stabilised pavement construction are typically similar to those specified for unbound granular pavement construction, discussed in Section 6.10.3. However the requirements for stabilised pavements may be slightly more restrictive due to the greater impact of layer thickness on pavement life.

Tolerances vary according to the specifying authority; however, typical tolerances are:

horizontal

- +/- 50 mm

vertical surface tolerance

- subbase +/- 15 mm at any point
- base zero +15 mm at any point

layer thickness

- subbase +/- 10 mm of the layer thickness specified at any point
- base zero + 20 mm of the layer thickness specified at any point

shape

- no point on the surface of any pavement layer should vary by more than 5 mm from a 3 m straight edge
- at no location should water pond on the surface of any pavement layer.

It should be noted that the consequences of areas where the finished surface is higher than the specified limit are greater for stabilised pavements which have set up than for unbound pavements due to the difficulty of trimming set up bound pavements. Therefore, it is prudent to undertake level conformance checking immediately following final trimming so that any high spots can be further trimmed before the pavement has fully set.

### 7.7.3 Compaction

To achieve the desired quality of stabilised pavement a characteristic value of relative compaction between 95% and 100% is typically specified. Individual road authorities may specify higher or lower values depending on the material being stabilised, location, pavement design and traffic volume. Where the stabilised layer thickness exceeds 250 mm it is common for density to be determined over two equal depth intervals to assess the uniformity of compaction.

Holes created by density testing should be repaired using freshly mixed material of the same type as was used in the surrounding pavement and compacted to a degree similar to that of the surrounding pavement.

### 7.7.4 Ride Quality

It is common for high-speed roads to have ride quality limits set for both new construction and rehabilitation works. However, because of the need in road rehabilitation to match existing levels, the specification of ride limit may not be appropriate in the following circumstances:

- road widening adjacent to trafficked lanes where levels are required to match the adjacent pavement levels
- any road furniture or feature constraints
- half-road patches where the final trim has to match the line of the crown of the road
- close to bridge abutments where traffic guardrails are not being replaced.

Typically a maximum ride quality of 40-50 NAASRA Roughness Counts or 1.6-2.0 IRI (International Roughness Index) is specified for new work.

Some specifications, such as in RTA (2002), also include an incentive payment for ride quality less than a specified limit. In such cases a maximum limit of 30-35 NAASRA roughness counts, or 1.4 IRI, is typically set for incentive payments.

#### **7.7.5 Binder Content**

Due to the importance of incorporating the nominated proportion of binder to achieve the target modulus and hence pavement strength, stabilised pavement construction specifications typically set limits for the proportion of stabilising binder that must be in a mix relative to the nominated proportion.

Though limits differ between authorities, it is typical that a lot is deemed non-conforming when the calculated proportion of stabilising binder in the mix is less than the nominated proportion by 0.05% to 0.10%. However, it is also typical that deductions are specified for a range of variation below the nominated binder content to a minimum specified binder content, often 1.0% less than the target proportion. In such cases, any lot where the calculated proportion of stabilising binder in the mix is less than the nominated proportion by greater than 1.0% is rejected (RTA 2002).

#### **7.7.6 Unconfined Compressive Strength**

It is common for stabilised pavement construction specifications to include requirements for unconfined compressive strength (UCS) testing. Typically UCS testing is used in the mix approval stage, to determine the required proportion of binder and then during construction to confirm that the placed material conforms to the specified strength requirements.

Typically the average strength of a pair of UCS cylinders, tested in accordance with the relevant test method, at 7 days accelerated and/or 28 days normal curing, must not vary by more than a specified amount below the target UCS.

For example, RTA (2002) deems a lot of heavily-bound material with target UCS of 3 MPa to be non-conforming when the average UCS for a pair of samples is greater than 0.05 MPa below 3 MPa (i.e. less than 2.95 MPa) and recommends that a lot be rejected when the variation is greater than 1 MPa (i.e. when the average strength of a pair of UCS cylinders is less than 2 MPa).

## 8. Sprayed Bituminous Surfacing

This section gives an overview of construction practices relating to sprayed bituminous surfacings. For additional guidance refer to the *Guide to Pavement Technology – Part 4K: Seals* (Austroads 2009e), the *Guide to Pavement Technology – Part 3: Pavement Surfacing* (Austroads 2009a) and *Austroads Bituminous Materials Safety Guide* (Austroads 2008g). Many road authorities also have their own sprayed sealing guides (RTA 1997c, VicRoads 2004b), which are useful references and often include additional information relevant to the local materials, practices and specification requirements.

### 8.1 Preparation for Sprayed Bituminous Surfacing

Preparation of surfaces is critically important to achieving a consistent high standard of sprayed seal. The preparation of pavements for bituminous surfacing involves a range of activities that differ depending on the pavement to be surfaced.

#### 8.1.1 Preparation for Initial Treatments of Granular Pavements

Regardless of the type of bituminous surfacing to be applied, a granular pavement should be allowed to dry-back as discussed in Section 6.9 prior to treatment of the surface as outlined below.

##### **Priming**

Pavements to be primed should be surface dry, and no more than damp to the required depth of primer penetration. Excess moisture will inhibit the penetration of priming materials, as voids filled with moisture cannot be filled with primer.

##### **Primersealing**

Surfaces to be primersealed should be kept damp, but not wet. A wet pavement may result in aggregate embedment and flushing in wheelpaths. A dry surface may prevent the primerbinder from properly 'wetting' the surface resulting in pin-holes and a non-uniform film of primerbinder. Where the surface is excessively dry a water tanker should be used to dampen the surface.

##### **Sweeping**

The prepared pavement surface should be swept with a rotary road broom (Figure 8.1) or vacuum broom (Figure 8.2) to remove surface dust and provide a surface that is free of foreign material with the larger sized stones at the surface of the pavement exposed but not loose or dislodged.

Techniques that may be used to improve the surface of a granular base before priming or primersealing include:

Heavy brooming – sometimes excess water is used to slurry fines to produce a smooth fine-grained surface. This is considered poor practice and can result in formation of a soft surface crust that must be removed by heavy brooming or tyning and reshaping of the surface.

Vacuum broom – although less effective than heavy brooming a vacuum broom is sometimes used to remove patches of fines from the surface and between larger aggregate particles thus enabling the development of a good bond between the seal and the granular base. However, vacuum brooms may also remove poorly compacted base materials and materials with loss cohesion.

Armourcoating – involves rolling aggregate into the surface of the pavement or applying a thin layer of good quality granular material (e.g. fine crushed rock) during final preparation to reduce embedment of sealing aggregate.

Any scale, hardened mud or foreign material should be removed. Hand chipping and hand brooming may be required to supplement rotary brooming to ensure removal of all foreign materials. Hand brooming may also be required in confined areas that are not adequately swept by a rotary or vacuum broom.

The force applied to a rotary broom is dependent on the type and hardness of the pavement material and the type of broom tyne. The force should be just sufficient to flick off dust and loose stones without damaging the pavement surface. Some loosely bonded pavement materials cannot be swept at all and should be lightly watered after final preparation, just before priming or primersealing.

Very little dust is generated in sweeping damp pavement surfaces to be primersealed but may be significant when sweeping pavements with a rotary broom for priming. Care must be taken to ensure that dust is not carried back over the top of the broom, blown back across the swept work, and does not create a nuisance, or represent a danger to traffic. Where visibility is affected, the road should be closed during sweeping operations or the use of a vacuum broom should be considered.

**Figure 8.1: Typical rotary broom**



Source: Austroads

**Figure 8.2: Typical vacuum broom**



Source: Austroads

The pavement should be swept to at least 0.3 m outside the edge of the width to be primed or primersealed.

### 8.1.2 Preparation for Sprayed Seal Re-treatments

#### **General**

Sprayed seals impart little structural strength to pavements and if the pavement is deficient in strength, defects are likely to occur early in the life of the sprayed seal.

Preliminary treatment of existing pavements in preparation for re-treatment includes:

- crack sealing
- skin patching of crazed areas
- repair of any surface breaks, potholes, etc.
- repair of weak pavement areas
- removal and repair of areas of shoving or unstable materials



- shape correction of ruts or depressions
- repair of edge breaks to restore pavement width
- correction of fatty or bleeding surfaces
- treatment of porous patches
- repair of areas of non-uniform surface texture
- restoration of shoulder levels
- repair of drainage
- cleaning of mud, scale and foreign material
- removal or protection of raised pavement markers.

Existing pavement defects should be corrected well in advance of any resealing work. Advance preparation is particularly important for any work where the performance of the surfacing may be affected by curing or traffic compaction of the preliminary treatment. Some treatments involving cutback bitumen, such as cold mix patches and primerseals, may take up to six months to cure completely.

An indication of the desirable advance time for pavement repairs and corrective treatments, prior to resealing is shown in Table 8.1.

**Table 8.1: Preparation for resealing**

Maintenance treatment	Desirable time of completion before resealing
Repair of edge breaks, patching or regulating using: cold mix with fluxed bitumen cold mix with cutback bitumen binder cold mix with bitumen emulsion binder asphalt slurry surfacing.	12 months 6 months 4 months 2 months 3 months
Skin patching using: cutback bitumen binder bitumen emulsion binder.	6 months 1 month
Crack filling using: bitumen emulsion cutback bitumen products hot pour bitumen products including polymer modified binders.	2 months 6 months 2 months
Corrective treatment of flushed sprayed seal surfacings and patches using: aggregates only solvents only solvent aggregate treatments.	2 weeks 2 months 2 months
Maintenance of shoulders and longitudinal drains: before resealing after resealing.	2 weeks 2 weeks
Fresh line marking	3 to 6 months

### 8.1.3 Preparation of Concrete Surfaces

#### *New concrete pavements*

A good bond between the concrete surface and sprayed seal is essential, particularly when using polymer modified binder seals.

The type of curing compound used on newly constructed pavements needs to be considered in selecting preparation treatment. Treatments are generally applied as follows:

- No curing compound, or curing compound removed, requires a very light primer at an application rate of 0.2 to 0.3 L/m<sup>2</sup>, approximately 0.10 L/m<sup>2</sup> residual bitumen. Proprietary grades of very light primer provide improved bond and faster curing for concrete surfaces.
- Chlorinated rubber curing compound in good condition requires no additional treatment prior to sealing. Old, hardened compound can be livened up with a light application of a solvent-type reactivating agent. Worn or damaged areas should be repaired with additional curing compound.
- Wax emulsion curing compound is unsuitable for use with bitumen and should be removed by water blasting or other suitable method.

#### *Old concrete pavements*

Structural defects should be repaired prior to sealing in accordance with appropriate rehabilitation guidelines. Cracks should be filled with an appropriate crack sealant and construction joints treated with propriety geostrips prior to applying a sprayed seal using a Class 170 or a PMB as appropriate.

Relatively clean concrete surfaces should be prepared by application of a very light primer as described above. Any dirt or oil should be removed by water blasting or brooming with water and detergent solution prior to priming.

### 8.1.4 Preparation of Timber Surfaces

Timber surfaces should be prepared by application of a very light primer, similarly to the treatment of concrete surfaces. However, timber surfaces raise some additional difficulties due to seasonal movement of the timber, joints, cracks, loss of shape, etc. Some of these difficulties can be overcome by covering the timber with plywood panels before sealing.

Care should also be taken to obviate contamination of the environment with excess primer.

## 8.2 Supply of Materials

### 8.2.1 Stockpiling Aggregates

#### *Stockpile sites*

The following general rules are accepted best practice and should be followed when practical:

- The stockpile site should be as close to the spraying operation as possible. Lead times in excess of 15 minutes may affect productivity of sprayed sealing operations.
- Ensure that there is no contamination from agricultural activities.
- To allow safe access to and from the stockpile area by the aggregate trucks, sufficient sight distance should be provided either side of the access to existing roads.
- Provision should be made for crossing open drains where they exist.

- Keep the aggregates dry, especially at the base. This is facilitated by ensuring stockpiles are covered and the site is well drained, graded and firm. On large projects where large volumes of aggregate are to be stockpiled, consideration should be given to sealing the area to prevent wastage due to the bottom 100-200 mm of the stockpile becoming contaminated by soils during the loading operations.
- To assist the plant operating around the stockpile, working space should be provided at least 3 m along the sides, and a turning space of at least 10 m at the end of the stockpile.
- When the aggregate is to be precoated on site, sufficient space should be allowed to undertake this operation.
- Where the aggregate is delivered on a volume basis it should preferably be stacked to a template. Separate stockpiles for different size aggregates should be set up and identified on site.
- At the end of the job, stockpile sites should be cleared of surplus aggregate and waste materials removed from the site.
- To avoid contamination and wetting, cover with suitable waterproof material.
- The stockpile site should be located well clear of vegetation, overhanging branches and power lines.

### ***Pre-treatment of aggregates***

Aggregates that have been stockpiled for a long period of time and/or in a dusty location should be re-screened in a dry state. Screening of aggregates when wet or damp will remove few, if any, of the fines.

To counter the adverse adhesion influences of dust and moisture on the surface of aggregates, it is good practice to precoat the surface of sealing aggregates. Done properly, it tends to be the most efficient means of ensuring good adhesion between aggregates and binder. The best adhesion occurs between dry, clean precoated aggregate and binder.

Precoating in advance of sealing operations is generally termed plant precoating while precoating immediately prior to use is generally termed field precoating. These terms are not strictly correct as both forms of precoating can employ various combinations of plant and field procedures, but are reasonably well accepted by field personnel.

Plant pre-coated aggregates are generally intended to be used within about four weeks of pre-coating but may be stockpiled for longer periods if covered. Pre-coated aggregates that have been stockpiled for long periods should be assessed for deterioration prior to use and, if necessary, rejuvenated with a light coating of kerosene.

Precoating materials consist of:

- bitumen based materials
- oil based materials
- water based materials.

Bitumen based materials are generally used for plant pre-coating in advance of spraying operations, oil based materials are mainly used for pre-coating for immediate use, and water based pre-coating materials are generally used for field pre-coating immediately prior to use. It should be noted that water based pre-coating materials rely on a combination of adhesion agent and a uniform surface-damp condition that can be difficult to maintain in the field, hence restricting the materials to limited applications.

Bitumen/flux oil mixtures generally require a curing period of at least a week, prior to use, to avoid tackiness of aggregates or softening of sealing binders. Bitumen emulsion based pre-coating materials do not require a curing period.

Field precoating is undertaken while loading the spreading trucks, using a loader able to load and precoat in one operation. The precoating agent used in field precoating is normally flux oil (industrial diesel fuel) or equal mixtures of flux oil and cutter oil. In cooler weather conditions, cutter oil only may also be used.

Purpose-built machinery may be used for both plant and field precoating of aggregate or, where only small volumes are required to be treated, a front-end loader (Figure 8.3) and spray lance. However, it can be difficult to get a uniform coverage using a spray lance.

**Figure 8.3: Loading precoated aggregate with a front-end loader**



Source: Austroads

Typical precoating rates are shown in Table 8.2 and may vary according to the condition, type (degree of porosity) and size (surface area) of the aggregate.

**Table 8.2: Typical precoating rates**

Aggregate quality	Bitumen based, including bitumen emulsions L/m <sup>3</sup>	Oil based L/m <sup>3</sup>
Clean	6 to 12	4 to 10
Dirty	8 to 12	6 to 12

Adhesion agents are used to change the surface chemistry of the aggregate/bitumen interface to improve adhesion in the presence of moisture or for aggregates that have a poor affinity for bitumen.

Use of adhesion agents is generally only necessary when:

- using polymer modified binders
- using acidic (quartz rich or siliceous) aggregates
- using damp aggregates
- priming or primersealing.

Adhesion agents may be used as an additive:

- to the precoating material
- in the binder.

Adhesion agents should be used in primerbinder to assist in adhesion to damp base materials.

## 8.2.2 Supply and Storage of Bituminous Materials

### ***Bitumen hazards***

All personnel involved in the supply and storage of bituminous products should be familiar with the associated risks and adequately trained. Two persons should be in attendance at all times when heating on site or handling hot bitumen.

Special attention should be given to the quality and site safety plan and any special requirements noted in the relevant materials safety data sheets. All plant, drums and storage units should clearly display emergency management information in the form of labels or signs (such as HAZCHEM signs) in accordance with the *Australian Code for the Transport of Dangerous Goods by Road and Rail* (NTC 2007) or the New Zealand Hazardous Substances and New Organisms Act 1996 as appropriate.

General guidance on safety considerations when working with bituminous materials is given in Austroads (2008g) however; individual jurisdictions may have additional requirements that should be followed.

### ***Transport of bituminous materials***

Bituminous materials may be transported in the following ways:

- Road tankers (Figure 8.4), including towed trailers – used for supply direct to depots and worksites. They are normally well insulated and fitted with heating and pumping equipment to maintain the temperature and affect the transfer of bitumen.
- Rail tank cars – well insulated and will transport hot bitumen with a loss of 1 °C per hour. They are fitted with heating tubes for direct flame heating without circulation.
- International Standards Association (ISO) containers fitted with heating equipment (without circulation).
- Fuel trailer tankers – they are not insulated or heated and are used for the supply of cutter oil, flux oil, and precoating materials.
- 900-litre bulk palletised containers – used in some locations for supply of bitumen emulsions and adhesion agents.
- 200-litre drums – used for flux oil, cutter oil, cutback bitumen, bitumen emulsions, precoating materials, adhesion agents and other liquids that do not require heating.
- 20-litre drums – for adhesion agents and other additives. Other size packaging may also be used.
- Packaged bitumen – depending on the use these may be supplied in cardboard containers, steel drums or plastic bags. These need to be split open and the contents heated in open bowl heaters.
- Bags – usually 25 kg. Used for crumb rubber for on-site production of crumb rubber binders. Crumbed rubber can also be supplied in 900 kg bulk bags. Some pelletised forms of adhesion agent may also be supplied in bags.

**Figure 8.4: Road ranker (at rear) delivering binder to site**



Source: Austroads

### **Storage of hot materials in bulk.**

Bitumen and cutback bitumen may be stored in insulated storage tanks for ready use at or near working temperatures. Fixed storages should have bunds enclosing the storage area to minimise contamination in the event of a spillage.

In the transport and storage of hot bituminous products, an allowance must be made for expansion. Bitumen can expand by 10% in volume when the temperature is raised from 15 °C to 180 °C. Field produced crumb rubber binder can expand by up to 30% during the mixing operation.

Bitumen should not be stored at high temperatures for long periods of time as this may cause premature ageing of the bitumen and non-compliance with specification values. If materials are not required for some time, they should be allowed to cool and then carefully reheated when required. For guidance on reheating bituminous materials refer to Austroads (2008g).

Polymer modified binders are also subject to degradation when held at elevated temperatures for extended periods. It is important to follow the manufacturer's recommendations for maximum storage temperature and storage time. AAPA (2003) provides a guide to heating temperatures and storage times for these materials.

### **Storage of cold materials in bulk**

Some bituminous materials flow readily at ambient temperatures and can be stored cold. These include light grade cutback bitumen and emulsions.

There are certain procedures that should be followed:

- gently circulate or agitate at regular intervals
- always circulate before being drawn from storage
- always check that the material is fluid at ambient temperature and suitable for placing in cold bulk storage (i.e. may be stored in non-insulated tanks).

The following procedures should be observed when storing emulsions in bulk as these materials will only remain stable whilst the fine particles of bitumen remain uniformly suspended in the water base:

- Check compatibility between newly delivered emulsion and stored emulsion before pumping the new material into the storage tank. Emulsions of the same type and grade may be different chemically and in performance.
- Anionic and cationic emulsions should never be mixed.
- Emulsions should be held between 15 °C and 60 °C or the temperature specified for the grade. Freezing of the water content of the bitumen emulsion at temperatures below 0 °C or vaporisation above temperatures of around 80 °C will cause premature breaking and changes in the consistency of the emulsion that cannot be reversed.
- Emulsions held in bulk storage should be gently circulated every 1 to 2 weeks, or as recommended by the manufacturer, to prevent build up of sediment bitumen particles.
- Spreading a thin layer of kerosene (0.5 L per m<sup>2</sup>) on top of the emulsion in the storage tank may assist in prevention of scum formation.

### **Storage in drums**

Cutback bitumen, bitumen emulsion and various oils and additives such as adhesion agents may be stored in drums. Supply and storage of cutback bitumen and bitumen emulsion in drums is only practical for small projects and isolated locations where hot bulk materials are unavailable. When storing in drums, the following procedures should be observed:

- similar materials should be stored in the same location
- drums should be stacked in a single layer with bung uppermost
- drums should be stored undercover
- drums containing emulsions should be placed in an upright position, above ground level and protected from low temperatures
- drums should be properly chocked to prevent movement
- drums should be used in order of delivery, the oldest first
- loading ramps and lifting machinery should be used where possible to avoid handling the drums manually.

### **8.2.3 Field Preparation of Bitumen Binders**

#### **General**

The effective spraying of primer, primerbinder or sealing binder requires the following:

- Sufficient fluid for pumping and spraying. Correct functioning of spraying nozzles requires a viscosity in the range of 0.05-0.1 Pa.s (equivalent to Class 170 bitumen at a temperature of 165 °C to 180 °C).
- Sufficient fluid, after spraying onto the pavement, to penetrate the surface (primers and primerbinders) or wet and adhere to the aggregate (primersealing and sealing binders). At this point, primersealing and sealing binders must also be sufficiently viscous to hold the aggregate while being rolled and withstand initial trafficking.

To achieve these requirements, one or more of the following processes can be used, singly or in combination:

- heating
- fluxing and/or cutting back
- emulsifying

- foaming.

### ***Heating and transfer of bitumen***

Applied heat should be carefully monitored to ensure that overheating, leading to oxidation of the bitumen, does not occur. Unless specifically designed for transfer of heat by convection, most units require the contents to be circulated while heating and for a further 20 minutes after burners are turned off before transfer or emptying of tanks. The following rates of heating should not be exceeded:

- bitumen, 40 °C per hour
- cutback bitumen, 30 °C per hour
- bitumen emulsions, 15 °C per hour.

Care must be taken when loading hot bitumen to avoid risks associated with ignition of the flammable vapours or presence of water. Even a small amount of water can cause a rapid expansion and foaming of the hot bitumen resulting in 'boil over' and increase the risk of fire from contact with heating equipment. Particular care must be taken when changing over from use of bitumen emulsion to hot bitumen products. Care must also be taken to ensure that cutter or flux oils are not contaminated with water; if in doubt check with water-finding paste. Refer to NTC (2007) and Austroads (2008g) for further details.

Heated bitumen is loaded into the bitumen sprayer through the sprayer's pumping and filter system. The filters prevent the entry of solid particles that could cause a blockage of the spraying nozzles. The transfer should be by suction where possible.

### ***Fluxing and cutting back***

Fluxing is used to provide a reduction in binder viscosity to improve durability of seals on lightly trafficked pavements. Cutting is used to provide a temporary reduction in viscosity of bitumen binders to improve wetting of aggregates in cool conditions. Cutting is also used to decrease bitumen viscosity for priming and primersealing applications.

The proportion of cutter oil added to hot bitumen for sealing work is a function of pavement temperature, size of aggregate, traffic volume, prevailing weather conditions, condition of aggregate and condition of aggregate precoat. Guides to the cutting back of bitumen for sprayed sealing work are provided in Austroads (2005a) and Austroads (2006d).

### ***Addition of cutter***

It is preferable to add the majority of the cutter to the sprayer before loading the bitumen. The cutter/flux oil should be added to the bitumen by suction through the pump and should never be added to the top of the bitumen in the tank because of the risk of fire and explosion. Loading operations should be monitored for any indication of water contamination. If foaming occurs loading should be stopped until foaming subsides. Loading should only be resumed in small quantities until the water has evaporated.

The mixture of bitumen and cutter/flux oil should be circulated at a rate of at least 1,000 litres per minute until the sprayer contents have been circulated twice, typically at least 20 minutes, to ensure thorough and uniform blending.

No smoking or lighting of naked flame should take place within a radius of 15 m during the loading and mixing operations.



Cutback bitumen should not be heated unless the temperature is below that required for spraying. Extreme care must be taken when heating cutback as the volatile vapours given off can ignite. Cutback bitumen should only be heated in equipment fitted with a circulation system and circulated after the burners have been turned off at a rate of at least 1,000 litres per minute until the sprayer contents have been circulated twice, typically at least 20 minutes.

### ***Addition of adhesion agents***

Adhesion agents are used to change the surface chemistry of the aggregate/bitumen interface to improve adhesion in the presence of moisture or improve adhesion of aggregates that have a poor affinity for bitumen.

The typical concentration of adhesion agent in precoating materials is 1% by volume but can be up to 2% by volume.

Care needs to be taken in proportioning the agent as adding too much can be detrimental, resulting in aggregate stripping and adversely affecting the ability of the binder to hold the aggregate in place.

The effectiveness of an adhesion agent may be destroyed when combined for a prolonged time in hot bitumen (more than a few hours). Addition of adhesion agent to hot bitumen is generally done in the sprayer and circulated for a minimum of 20 minutes.

Adhesion agents may be supplied as a liquid, pellets or paste.

### ***Addition of crumb rubber***

Crumb rubber is added through a measuring and mixing box set in the bitumen line from the supply tank to the sprayer. This allows partial distribution of the rubber into the bitumen. Final mixing is undertaken in the sprayer by using the circulating pump, at temperatures ranging from 190 °C to 200 °C. The process of adding crumb rubber to bitumen should be undertaken at least 45 minutes before spraying, and prior to the addition of any cutter oil and adhesion agents in order to allow for full digestion of the rubber into the bitumen. Cutter oil and adhesion agents should not be incorporated until at least 20 minutes after the burners have been turned off and whilst the mixture is still circulating.

### ***Anti-foaming agents***

Anti-foaming agents are used to minimise the potential of a 'boil over' of hot bitumen from small quantities of water which can arise from condensation in tanks after periods of disuse, incomplete cleaning of tanks used for bitumen emulsion or contamination of drummed materials.

Anti-foaming agents are normally mixtures of fluid silicone and diesel oil or distillate. The standard proportion is 1 to 5,000 of anti-foaming agent to bitumen at 15 °C. This should not be exceeded as additional quantities of anti-foaming agent may reduce the adhesion properties of the bitumen. Anti-foaming agents should be poured into the tank prior to loading the bitumen. Anti-foaming agents should not be used with polymer-modified binders.

## 8.3 Placement of Sprayed Bituminous Surfacing

### 8.3.1 Preliminary Activities

#### *Inspection of pavement surface*

To achieve a high standard of sealed work it is essential for the pavement surface to be structurally sound, properly compacted, shaped, dust free, uniform in texture and have good riding qualities. Sprayed seals cannot correct shape or pavement deficiencies.

Before any work is undertaken on site, a final inspection of the surface should be undertaken. Inspection of the pavement surface is required to ensure that preparation activities have been carried out correctly and that the pavement is in a suitable condition for sealing. Inspection should also be used to confirm the assumptions in the sprayed seal design.

If the surface of a new granular pavement is too damp it may prevent penetration of the primer into the pavement and should be dried out before any primer is applied. This may delay the operation, especially if the moisture present is excessive.

For primerseals, if the surface is too dry it may require dampening to reduce the dust hazard and assist uniform coverage and penetration of the primerbinder. Primerbinders applied to dry surfaces may result in the bituminous material contracting into small balls leaving incomplete coverage and poor penetration.

It is good practice to prepare and finish the surface of a new pavement to receive the bituminous surfacing as early as possible after the construction of the pavement. However, if the pavement will be opened to traffic before priming or primersealing, the preparation of the finished surface should be left until just before priming or primersealing.

Any defects in an existing surface, including depressions in granular pavements, should be remedied before commencing the sealing operation. Dampening of the surface should occur after brooming and marking out have been completed.

#### *Brooming*

Immediately prior to priming, primersealing or sealing, the pavement surface should be swept to remove any loose material, dust, dirt and foreign matter. Mechanical rotary brooms are most commonly used, supplemented as necessary by hand sweeping.

Specific procedures for the sweeping of new granular pavements include:

- Remove unbonded fine material, foreign matter or clay to avoid eventual breakdown of the pavement surface.
- The pressure applied by the broom should be set to flick the loose or unwanted material so as to uncover, but not dislodge the pavement surface. The pressure that should be applied to the rotary broom will depend on the tightness of the pavement surface.
- Sweeping should commence at the upwind end of the work and progress downwind to avoid the dust blowing over zones already swept. Dampening of the surface may assist brooming and reduce the dust hazard. Brooming should extend at least 300 mm clear of the area to be primed or primersealed and, where possible, should commence in the middle of the pavement.

### Marking out

After the pavement has been swept, guidelines for the sprayer should be marked out. These marks allow the sprayer operator to align the guide rod chain to pass over the marks. Marking out can be achieved by:

- paint marks
- guide pegs, not more than 6 m apart
- stringlines.

The location of the marks should be set:

- such that the net width of the spray is not less than the design or the existing width of the pavement; full width spraying is desirable for road widths up to 7.5 m
- to ensure that any joints are along a lane boundary or pavement centre line
- to allow primer/primerbinder to be sprayed at least 150 mm, and typically 300 mm, wider than the width of the seal.

As a general rule, longitudinal joints should coincide with lane markings. This provides a neat appearance and ensures any minor overlap is placed in an area of least traffic and is therefore less likely to flush. Transverse joints should avoid areas within 50 m of intersections or pedestrian crossings, within curves and approaches and exits from curves.

Joints between adjacent sprayer runs should be marked out to provide binder overlap prior to spreading the aggregate. To achieve a seamless joint requires selection of the correct spray nozzles and width of overlap. Table 8.3 indicates the correct overlap for commonly used nozzle types.

**Table 8.3: Binder overlap**

Configuration	Overlap required
Overlapping adjacent sprayer runs prior to spreading aggregate: <ul style="list-style-type: none"> <li>• using EAN 36 (EA4) nozzles or jig-set intermediate nozzles</li> <li>• using AN 18 (A4) intermediate nozzles only</li> </ul>	50 mm 200 – 300 mm
Overlapping paved and previously sealed surface	20 – 50 mm (EA4 or jig-set intermediate nozzles)

If the overlapping operation is not carried out correctly it may result in the application of insufficient binder causing a potential for stripping of the aggregate, or alternatively too much binder may cause flushing of the surface.

It is also important to use the same type of nozzle (i.e. end nozzles or intermediate nozzles) on both overlapping edges. Overlap between a binder sprayed with an intermediate nozzle and an end nozzle can result in a variation of up to double the designed rate being sprayed in the overlap area. Spray bars with variable width require special consideration.

### Protection of pavement structures

Items such as kerbs, concrete drainage channels, bridge rails and fences should be protected from spray. This can be achieved by masking items with spray paper, plastic sheet or other protective covering. End shields on spraybars also assist in minimising overspray onto adjoining surfaces and structures. Spraying in urban areas on windy days should be avoided to minimise potential for spray drift and damage to buildings, structures, parked cars, etc.

### ***Starting and finishing paper***

Spraying should start and finish on a strip of protective paper. This is to ensure that there is a square and even start and finish to each run, and the transverse joins are not fatty due to overspray. The protective paper should be placed at right angles to the direction of the spray run, which on most jobs is the centreline of the road. The leading edge of the spray sheet should be placed at least 15-25 mm over the edge of the previous run and firmly held in place with cover aggregate. Protective paper normally has a minimum width of 1.2 m and a mass 120 g/m<sup>2</sup>.

### ***Determination of the target bitumen application rates and quantities***

Calculation of bitumen application rates is based upon volume of bitumen at a standard temperature of 15 °C. When determining the quantity of material to be sprayed, allowance must be made for expansion in volume at spraying temperature and the proportion of cutter oil or other additives including water content of bitumen emulsions.

Design application rates for sealing binders are based on residual binder while design application rates for primers and primerbinders are based on the total volume, including cutter oil.

To determine the actual volume of hot bitumen to be loaded into the sprayer for a particular area and application rate at 15 °C, the volume is multiplied by a conversion factor. Additional information on bitumen conversion factors is provided in AAPA (2004), DTEI SA (2004) and VicRoads (2004b).

When determining the quantity of material to be loaded into the sprayer, an allowance of approximately 10% should be included to ensure that there is sufficient material to finish a sprayer run.

### ***Setting up the sprayer***

The bitumen sprayer (Figure 8.5) is an important piece of plant used for sealing work. The purpose of the sprayer is to uniformly apply bituminous material to a prepared surface at a specified rate. The general performance requirements for bitumen sprayers are described in NAASRA (1989).

Most spraybars are of the circulating type. Individual operating valves (taps) are used for each spraying nozzle, or each pair of nozzles. This enables simultaneous operation of the nozzles across the full spraying width. The spraybar design ensures that the material in the bar is at the correct pressure and the valves open and shut simultaneously.

Binder application rates are achieved by a combination of a spray bar that provides a constant output of binder across the width of the bar and varying the forward speed of the sprayer. Precise control of both functions is required in order to achieve accurate application rates.

Specifications typically require that a current calibration certificate is kept with the bitumen sprayer at all times. Calibration procedures are set out in (Austroads 2005d, 2005e, 2005f, 2005g, 2005h, 2006g, 2006h) and include the following:

- inspection and report on instruments and operating equipment including calibration of instruments and the tank dipstick
- volumetric calibration of pump output
- transverse distribution test
- road speed and distance calibration.

Figure 8.5: Bitumen sprayer



Source: Austroads

The width of the spraybar should be adjusted to suit the width of the pavement to be sealed. The number and type of nozzles, pump revolutions for the number of nozzles and the forward speed of the spraybar are determined from the sprayer table that should accompany each bitumen sprayer.

### 8.3.2 Priming

#### General

The operations for priming are similar to the application of a primerbinder. The principal difference is that the operations for priming do not involve aggregate loading, spreading or compaction equipment and are generally undertaken with a much smaller crew than that required for sealing operations.

#### Spraying primer

The total width to be primed should extend beyond the width to be sealed. This width can range from 100-150 mm outside of each edge of the subsequent seal.

Spraying primer should not proceed if rain is imminent, and/or if the pavement temperature is less than shown in Table 8.4. Primer that is not fully cured may be washed from the pavement by rain and may contaminate the surrounding area.

Table 8.4: Minimum pavement temperature for priming.

Type of primer	Minimum pavement temperature (°C)
Cutback bitumen	10
Special priming grade cationic bitumen emulsion	5

Bitumen emulsion primer is not as susceptible to weather conditions as cutback bitumen and damp pavement is often desirable. However, heavy rain may be detrimental to an emulsion primer if it has not broken or cured.

On dusty pavements a light spraying of water prior to priming may assist in achieving a uniform coverage. Dusty surfaces may cause primer to contract into small balls resulting in uneven coverage and reduced penetration. Equally, primers may not penetrate deeply into glassy surfaces that have been prepared by working a water/fines slurry to the surface.

The temperature of the primer at the time of spraying should be in the range shown in Table 8.5.

**Table 8.5: Typical primer temperatures for spraying**

Type of primer	Grade	Temperature range for spraying (°C)
Cutback bitumen	AMC00	10 – 20
	AMC0	35 – 55
	AMC1	60 – 80
Bitumen emulsion		Manufacturer's recommendations

Where run-off of a primer is likely to occur during the spraying operation, the primer should be applied in two applications, the first application being slightly heavier than the second. The second application should not be sprayed until the first has penetrated into the pavement.

### **Drying**

The primed section should be closed to traffic until dry, determined as being not tacky to touch. Typical drying times are shown in Table 8.6. Where it is necessary to provide limited access to traffic the surface should be blinded with fine grit or sand. Dust must not be used, as it will form a skin that cannot be swept off.

**Table 8.6: Typical drying time of cutback primers**

Weather condition	Drying time
Hot weather	6 – 12 hours
Cool weather	12 – 24 hours
Cold cool damp weather	24 – 48 hours
Bitumen emulsion	2 – 4 hours

### **Curing**

In the case of cutback bitumen primers, following priming the application of the bituminous seal should not be undertaken until the prime has dried out and cured. Volatile cutter oil should be allowed to escape from the primer as any residual volatile can affect the subsequent treatment. The rate at which volatiles escape depends upon temperature and time. The period required to allow volatiles to escape is referred to as curing and the suggested times to be allowed for this process are:

- light to medium primers – minimum of one week
- heavy primers – minimum of two weeks.

### **Inspection**

An inspection of the primed area should be undertaken within 2-4 hours of spraying. Where the surface is hungry, the area should be re-primed. Pools of free primer on the surface should be broomed over adjacent areas.

### Blinding

Where it is absolutely necessary to allow traffic onto sections of the primed area, for example to access properties, a light uniform application of 5 mm or 7 mm aggregate may be spread over the surface. All loose aggregate must be removed before the next treatment. Blinding is generally only suitable for low trafficked situations.

### Life expectancy of a primed surface

A number of factors, such as weather conditions, nature and volume of traffic and condition of the pavement influence the life expectancy of a primed pavement. The typical life expectancy of primed surfaces, subject to light traffic and without further treatment other than maintenance, is shown in Table 8.7. It is advisable to undertake sealing work within these periods.

**Table 8.7: Typical life expectancy of a primed pavement**

Grade of primer	Life expectancy
Light	up to 4 weeks
Medium	up to 6 weeks
Heavy	6 to 10 weeks

### 8.3.3 Primersealing

#### General

The operations for primersealing are similar to those for sealing (refer to Section 8.3.4).

The spraying temperature of the primerbinder should be within the range shown in Table 8.8 and pavement temperature should not be less than 10 °C.

**Table 8.8: Primerbinder temperatures for spraying**

Type of primerbinder	Grade	Temperature range for spraying (°C)
Cutback bitumen	AMC2	75 – 100
	AMC3	95 – 110
	AMC4	110 – 135
Bitumen emulsion		Manufacturer's recommendations (but always less than 80 °C)
Proprietary product		Manufacturer's recommendations

The primerbinder should be covered with aggregate directly after the spraying operation. Immediately after the aggregate has been spread, initial rolling should take place. Drag brooming, if necessary to ensure uniform coverage, should be withheld until the primerbinder has firmly bonded to the aggregate.

Rolling should be undertaken by pneumatic, multi-tyred rollers closely following behind the spreader. The number of rollers required will depend on the rate of progress of work and the volume of traffic.

#### Drying

Primerbinders need to dry to retain aggregate. Therefore, traffic should be strictly controlled during the first 4 to 6 hours. Where the primerseal is not trafficked immediately, traffic should also be controlled when subsequently opened, particularly during cold or wet periods to minimise the risk of stripping.

If the primerseal has been subject to wet weather during or after placement, and the primerbinder emulsifies, the surface should be covered by fine aggregate to prevent pick up by tyres. In these circumstances traffic should be strictly controlled, or excluded if possible, until sufficient drying and curing of the binder has occurred.

### ***Curing***

Like priming, for cutback primerseals it is necessary for the volatile cutter oil to escape from cutback bitumen primerbinders as any residual volatile can affect subsequent treatments. The rate at which the volatiles escape depends upon primerbinder grade, application rate and temperature. The minimum time period for curing may vary from three months in warm conditions to two years in cooler conditions. Minimum curing times do not apply to bitumen emulsion primerbinders, which may be resurfaced immediately after initial curing has taken place.

### **8.3.4 Sealing**

#### ***General***

General requirements for preparation for sealing and resealing, and spraying of binder, have been covered in previous sections. This section refers to those additional activities involved in spreading and rolling of aggregate.

#### ***Coordination of aggregate supply with sealing operations***

Sealing operations require planning to ensure that the sprayer, aggregate spreaders, rollers and brooms are available to operate in the correct sequence. This will require the aggregate trucks, spreaders and rollers to be in position at the commencement of the spraying operation.

The capacity of trucks used to carry aggregate from the stockpile should be accurately known and displayed on the side of the truck. For a balanced operation, it is preferable that all the trucks have the same capacity. A sufficient quantity of aggregate should be available from the stockpile to ensure a continuity of supply, before spraying commences.

To accurately check the spread rate, trucks should be fully loaded and aggregate levelled to the top of the tray or sideboards.

#### ***Spreading aggregate***

It is essential that the aggregate is spread immediately after spraying. Spreading trucks (Figure 8.6) or self-propelled spreaders (Figure 8.7) should be in position to commence spreading as soon as the area is sprayed.

The spreading rate of box spreaders is achieved by controlling the speed at which the truck reverses and the opening of the spreader gate. Similarly, self-propelled spreaders are controlled by a combination of forward speed and gate openings.



**Figure 8.6: Truck mounted box spreader**



Source: Austroads

The aggregate should uniformly cover the binder without any excess. The actual spreading rate should be checked regularly by measuring the quantity of aggregate applied and the area covered. This should be compared with the design spread rate for the area. If differences in excess of 5% occur, action should be taken to ensure tighter control over the spreading operation, prior to commencing subsequent runs.

After the aggregate has been spread it is necessary to inspect the surface to ensure uniform coverage. Skill and experience should be applied to visually assess the rate of cover compared to design rates. If there are any deficiencies the following action should be undertaken to rectify them whilst the binder is still soft:

- Under-spreading – if excessive areas of binder are visible, additional aggregate should be spread as soon as possible and the area broomed by hand, prior to the commencement of rolling.
- Over-spreading – excess aggregate in heaps or rills should be removed prior to rolling, as they tend to crush under the action of rolling. Where the excess is not great it can be dispersed over adjacent areas by either hand broom or a light drag broom.
- Damp aggregate – where the aggregate is visibly damp or wet after spreading, rolling should be delayed until the aggregate is dry.

**Figure 8.7: Aggregate spreader**



Source: Austroads

## Rolling

The performance and life of a sprayed seal is dependent on the bond between the cover aggregate and the binder and the level of the binder up the aggregate. To achieve good bond, rolling should commence immediately after the aggregate has been spread or when rectification work is complete, and before the binder becomes too viscous. This is to ensure full embedment of the aggregate into the binder, reorientation of the aggregate and a good aggregate mosaic.

The principal type of roller used in sealing work is the self-propelled pneumatic multi-tyred roller (Figure 8.8). Smooth steel wheeled rollers are not used on sprayed seal work due to the risks associated with breakdown and crushing of aggregate, particularly when using softer aggregates, and on uneven surfaces due to potential for variable contact pressure. Rollers should be operated smoothly to avoid tearing and damaging the work, particularly when changing direction, to prevent scuffing of the surface.

**Figure 8.8:** Pneumatic multi-tyred roller



Source: Austroads

The initial rolling should commence at a speed of between 5-10 km/h immediately behind the spreader without overlapping, to ensure the aggregate is pressed into the binder before it hardens. After the surface has been rolled once, rolling should proceed at a speed of 15-25 km/h to move and reorientate the aggregate. Rolling should also follow a set pattern, overlapping each proceeding pass by about one third of the effective rolling width, commencing at the edges and working towards the centre while ensuring full cover is achieved.

Rolling should be continuous and the full width of the seal should receive an equal number of roller passes. Adequate time must be allowed at the end of a day's work to ensure all materials spread have received the same number of passes of the rollers.

The number of rollers selected will vary with the area covered, size of aggregate and traffic conditions. The general rule is one roller hour per 1,500-2,000 L of binder sprayed. This requirement is normally met by a minimum of two rollers, one working immediately behind the spreader and the other supplementing the total number of passes required. In cool weather or where polymer modified binders are used, it may be necessary to employ sufficient rollers initially to cover the full width of the spray run in one pass.

A more detailed guide to roller hours is provided in Table 8.9.

**Table 8.9: Area effectively rolled per hour by a self-propelled multi-tyre roller**

Aggregate size (mm)	Traffic volume (vehicles per lane per day)		
	< 300	300 – 1,200	> 1,200
	Area – m <sup>2</sup> per roller hour		
7 or smaller	4,000 – 4,500	5,000 – 5,500	6,000 – 6,500
10	3,000 – 3,500	3,500 – 4,000	4,500 – 5,000
14	2,500 – 3,000	3,000 – 3,500	3,500 – 4,000

On heavily trafficked roads, compaction is assisted by controlled use of traffic. As lack of rolling may cause early stripping, particular attention should be given to areas such as the centre of the pavement and roads that are subject to light trafficking. Rolling of lightly trafficked roads should continue for at least 4-5 hours after initial compaction. On deviations constructed without traffic, back rolling may be necessary for a number of days.

Traffic should not be allowed onto the work until at least two passes of the roller have been made. When opened to traffic, speed, turning and overtaking should be controlled to prevent damage to the seal.

If rain occurs during rolling, the amount of rolling should be reduced while the aggregate is wet as adhesion between the binder and aggregate is adversely affected. Normal rolling should resume once the aggregate is dry. The traffic using a new seal under these conditions should be controlled or excluded to reduce damage.

### ***Drag brooming***

Drag brooming may be used in conjunction with rolling and initial trafficking to correct small variations in spread rate of aggregates. The action of drag brooming tends to set in motion any loose aggregate particles sitting on the surface. Particles that subsequently lodge in spaces where they come in contact with the binder assist in completing the uniformity of aggregate coverage.

### ***Removal of loose aggregate***

In sprayed sealing work, a small proportion of aggregate is not incorporated into the surface, generally due to slight overspreading of aggregates to compensate for small variations in the uniformity of spreading. Early removal of the surplus aggregate is desirable in order to reduce the risk of loose aggregate to the road user. Surplus aggregate can also be crushed or ground by traffic, resulting in fines and dust that may fill surface voids and cause flushing or partial stripping.

The timing, type of sweeper and method of removing loose aggregate vary according to the traffic volume/road class, type of binder, ambient temperature and, to some degree, the size of aggregate.

High traffic volumes rapidly reorient the aggregate so removal of surplus stone may commence within a few hours of spreading. On lighter traffic roads this period increases up to 48 hours. Polymer modified binders develop early cohesion enabling the aggregate to be removed the same day. Emulsion binders develop cohesion at a much slower rate and up to 48 hours curing may be needed before sweeping is carried out. Where air temperatures are in excess of 30 °C it is preferable to remove surplus aggregate in either the early part of the day or the evening.

To ensure the safety of the road user, signing and any necessary traffic control measures should remain in place while removal activities are taking place and remain until there is no further hazard from loose aggregate. This removal process may have to be repeated several times as traffic generates new loose aggregate.

Surplus aggregates removed from sealed surfaces are generally not suitable for re-use as sealing aggregates due to contamination with dust and foreign material and breakdown of aggregate particles.

When a standard rotary broom is used for removing loose aggregate from a sealed surface, the broom pressure should be less than for normal sweeping. The bristles should be new or nearly new and be long enough to clear the surface for the width of the brush and provide a flicking action. As they usually operate at an angle, the bristles wear at an angle, and this can render them ineffective. Sweeping should commence at the centre of the sealed surface and move towards the edge, particularly on wide pavements. To ensure the surface is not damaged this operation should be undertaken in a number of light passes. It is also desirable to follow brooming with a multi-tyred roller to ensure that aggregate that is disturbed by brooming is rolled back in place. Any loose material should be placed in a windrow clear of the surface, to be picked up and removed from site.

Suction sweepers that operate in conjunction with a rotary broom have the advantage of complete removal of aggregate, which is desirable in urban areas to avoid aggregates accumulating in channels or being thrown onto verges and footpaths or back onto the sealed surface. They are particularly effective in removal of aggregates from channels or windrows.

Vacuum brooms are often preferred to rotary brooms as they are capable of sweeping, lifting loose aggregate, retaining the loose aggregate and transporting it from site, which eliminates the need for a separate operation. In addition, the amount of dust produced by a vacuum broom is significantly less than for rotary brooming. However, care should be exercised when pavement temperatures exceed 30 °C as a vacuum broom may damage seals. The vacuum broom should not stop with the suction operating as this can cause damage to the surface.

### **8.3.5 Other Seal and Reseal Treatments**

#### ***Polymer modified binder (PMB) seals***

Polymer modified binders are generally used in the prevention of reflective cracking and provision of tougher binders for high stress seal situations.

Polymer binders increase more rapidly in viscosity after spraying onto the road surface than standard bitumen and cutback bitumen binder. As a consequence, there is a shorter time available between the application of the binder and effective incorporation of the aggregate and particular care must be taken to ensure that the spreading of the aggregate and rolling follow immediately after the spraying operation.

Special requirements apply to storage, transport and spraying of PMB and it is important to obtain the recommendations provided by the manufacturer. Additional guidance on polymer modified binders is provided in AAPA 2003, (Austroads 2006d) and (Austroads 2006e).

#### ***Crumb rubber bitumen***

Crumb rubber is another form of PMB used for similar applications to those referred to above. The field blending of crumb rubber into the bitumen has been discussed in Section 8.2.3. Crumb rubber blends may also be supplied as manufactured products (Austroads 2006e).

#### ***Reinforced seals***

Two types of reinforced seals are geotextile and fibre reinforced seals. Geotextile reinforced seals are used to provide a strong waterproof membrane over cracked and weak pavements. They may also be used for protecting low quality, moisture sensitive pavement materials in areas where higher quality materials are not available. Fibre reinforced seals are a proprietary system that are also used where a tougher membrane is required than that provided by conventional bitumen binder.

Construction procedures for reinforced seals are as follows:

- Geotextile reinforced seals – geotextile fabric is rolled out on to a surface to which a bond coat has been applied and then covered by a sprayed bituminous seal, usually a single/single or double/double seal. The geotextile fabric is typically placed using a roller dispenser attached to the front of a tractor, loader or pneumatic multi-tyred roller. The dispenser controls the tension, providing a smooth, non-wrinkle surface. For small jobs the geotextile fabric may be placed manually. Creasing of the geotextile fabric may occur when attempting to place the fabric around bends. This is normally accommodated by cutting and butting the wrinkles in the geotextile.
- Fibre reinforced seals – glass fibres are incorporated into the seal via a purpose built sprayer in a single pass operation.

For further information on geotextile reinforced seals refer to (Austroads 1990), (Austroads 2005i), RTA (1992), RTA (1997a), VicRoads (2001) and VicRoads (2008c) and for fibre reinforced seals refer to (Austroads 2005b), (Austroads 2006d) and RTA (1997b).

### ***Foamed bitumen***

In the foamed bitumen process, water is added in a purpose-built sprayer causing the binder to foam. The foam only lasts for a short period of time; however, the improved wetting properties facilitate the incorporation of the aggregate into the binder. The foam then subsides and the binder achieves full strength in a shorter time than that for cutback bitumen and bitumen emulsion. Spraying of foamed bitumen has been largely discontinued in Australia but foamed bitumen is still used in stabilisation, asphalt recycling and warm mix asphalt applications.

#### **8.3.6 Handwork**

Handwork should be kept to a minimum and restricted to those areas where variations in binder applications will have a minimal effect on the quality and appearance of the completed work.

Handwork should not be undertaken with crumb rubber binders because of the high pressures required in the hand lance.

#### **8.3.7 Cleaning Up**

Upon completing sealing, it is necessary to ensure the works, stockpile and bitumen storage areas are cleaned, including removal of protective papers, made environmentally safe and there are no hazardous conditions or materials left.

Plant and equipment used for the works should be cleaned and made ready for the next sealing operation.

## **8.4 Conformance and Quality Testing**

### **8.4.1 Material Supply**

The *Guide to Pavement Technology – Part 4F: Bituminous Binders* (Austroads 2008d), *Part 4J – Aggregate and Source Rock* (Austroads 2008e) and *Part 4K – Seals* (Austroads 2009e) contains advice on the selection, testing, quality control and specification of materials for use in sprayed bituminous surfacing.

### **8.4.2 Bitumen Application Rate**

Bitumen application rates should be checked after every run. The following should be recorded and checked:

- temperatures and volume of all deliveries to site and converting to mass at 15 °C

- quantities to be loaded using volume conversion tables and dipping quantities in the sprayer, including any adjustment for part loads due to materials remaining in the sprayer (if relevant)
- quantities sprayed by dipping the tank after each run
- area sprayed
- quantity of aggregate spread.

Actual application rates of binder and aggregate should be compared against design rates and adjustments made, as required. Where the actual bitumen application rate for each of three consecutive runs differs by more than 5% from the target application rate, the sprayer should not be used until a new sprayer certificate has been obtained (RTA 2006).

It is common for deductions to be applied when the actual bitumen application rate differs from the target application rate by more than a certain percentage, typically 5%, for each sprayer binder run greater than 1000 L. A maximum allowable deviation in binder application rate, typically 10%, above which the lot is rejected, may also be specified.

### 8.4.3 Assessment of Loose Aggregate

Sweeping is unlikely to remove all loose aggregate. Acceptable standards for removal will depend upon traffic volume, road class and specifying authority; however, Table 8.10 may be taken as a guide to the acceptable quantity of loose aggregate particles remaining for 10 mm and 14 mm seals after completion of sweeping.

**Table 8.10: Indicative acceptable quantities of loose aggregate particles**

Road type/traffic	Loose particles
Urban areas	20 particles /m <sup>2</sup>
Other medium to high traffic (> 250 v/l/d)	30 particles /m <sup>2</sup>
Low traffic (< 250 v/l/d)	40 particles /m <sup>2</sup>

### 8.4.4 Riding Quality

Sprayed bituminous surfacings are not considered to have the ability to improve ride quality. Therefore ride quality is a function of the underlying layer and sprayed sealing specifications should not include ride quality requirements. However, specifications may require final measurement of the ride quality of an unbound or stabilised pavement following application of a seal.

### 8.4.5 Treatment of Non-conformities

#### **General**

To allow early identification of premature deterioration, new seals should be kept under observation in the first few days, particularly in hot weather, heavy traffic, or if rain occurs or there is a sudden drop in night-time temperatures soon after placement. If premature deterioration is identified corrective treatments may be required.

#### **Flushed, fatty or bleeding surfaces**

Bleeding in new seals may be a result of:

- excess binder through poor design assessment or inaccurate sprayer application
- excess cutter oil or hotter than anticipated conditions

- heavier traffic than anticipated, particularly effects of stopping and turning vehicles
- aggregate embedment into a primed surface, into a fresh primersealed surfacing, into a fresh slurry seal or into fresh asphalt/coldmix patching
- aggregate breakdown.

Bleeding surfaces may be treated by an application of clean grit (fine aggregate). Prompt gritting may prevent binder pick up or rolling over of aggregate and reduce the risk of serious damage to the new seal.

### ***Aggregate plucking***

Plucking refers to loss of aggregate with a small amount of binder adhering to the aggregate. Rolling of aggregate with binder attached leaves inadequate binder to hold aggregate in place as well as increasing the risk of plucking. Plucking may be reduced using a small sized, pre-coated aggregate applied as a scatter coat to lock aggregate particles in place.

### ***Rain on new work***

If rain occurs on new work, traffic speed should be reduced and strictly controlled, or preferably excluded, to avoid loss of aggregate from the seal. If traffic can be controlled until drying takes place, normal adhesion between the binder and the aggregate should be achievable.

### ***Stripping***

Stripping in new seals may be a result of:

- insufficient cutter oil for ambient conditions (e.g. shaded areas)
- dirty or poorly pre-coated aggregate
- inaccurate aggregate spreading application
- rain immediately following application (within 6-12 hours)
- inadequate allowance for turning of heavy vehicles
- insufficient binder through incorrect design assessment or inaccurate sprayer application.

Remedial action for stripping in new seals includes:

- additional rolling and control of traffic to improve aggregate embedment into the binder
- application of a scatter coat to lock aggregate in place until adhesion is obtained
- application of a surface enrichment
- application of a correction seal (small aggregate)
- treatment of areas of exposed binder as for flushed, fatty or bleeding surfaces.

For additional information on the treatment of bituminous sprayed seal non-conformities refer to AAPA/Austrroads (1998), (Austrroads 2008f), VicRoads (2007a) and VicRoads (2007b).

## **8.4.6 Audit and Surveillance of Sprayed Bituminous Surfacing Work**

Additional guidance on the audit and surveillance of sprayed bituminous surfacings work is provided in (Austrroads 2005c) which includes a systematic framework for evaluating the adequacy of a contractor's quality system and project quality plan as well as its application.

## 9. Asphalt Pavements and Surfacing

This section gives an overview of construction practices relating to asphalt pavements and surfacings. For additional guidance on asphalt pavements refer to the *Guide to Pavement Technology – Part 4B: Asphalt* (Austroads 2007), the *Guide to Pavement Technology – Part 3: Pavement Surfacing* (Austroads 2009a), *AP-T64/06: Asphalt Manufacture* (Austroads 2006b) and *AP-T65/06: Asphalt Paving* (Austroads 2006c).

### 9.1 Preparation for Asphalt Construction

#### 9.1.1 Site Planning

Detailed site planning for asphalt pavement and surfacing construction should include:

- positioning of longitudinal joints
- planning the optimum length of each run to minimise transverse joints;
- method of level control
- optimum use of labour, equipment and delivery trucks
- adoption of good practices in all aspects of work
- planning for traffic management
- sketches and/or plans of the work, if appropriate.

#### 9.1.2 Consideration of Climatic Conditions

Asphalt cools rapidly in thin layers and when pavement and ambient temperatures are low. Wind and excessive moisture will also increase the cooling rate. Unless quickly and adequately rolled, these conditions may result in a low degree of compaction being achieved and a subsequent reduction in the service life of the asphalt. Ideally, paving operations should be planned to take advantage of warm to hot temperature conditions when rain is not expected before completing the compaction. When paving at night, adequate artificial lighting should be provided.

Generally, asphalt layers of less than 40 mm thickness should not be placed when pavement temperatures are less than about 10 °C (15 °C for open graded asphalt and mixes containing polymer modified binders). Higher minimum pavement temperatures are desirable where cooling rates are increased by wind. Paving at lower pavement temperatures will generally be satisfactory for layers of 40 mm and thicker. However, mix temperatures and compaction densities should be closely monitored to ensure that compaction standards are achieved.

When paving, the pavement should be dry, except for granular surfaces that may be slightly damp. Work may be permitted on a slightly damp bituminous surfaced pavement, provided it was previously tack coated. Paving should not proceed if rain appears imminent.

Ideal conditions do not always exist and at times decisions have to be made as to whether paving should proceed. In these instances a risk management approach should be adopted. Paving under adverse conditions will involve some risk of poorer density, surface finish and reduced service life, particularly where handwork is involved. This may lead to a reduction in the life expectancy and early replacement of the asphalt pavement. In adverse conditions particular attention should be paid to planning and execution of work to ensure that required standards of density, joint construction and handwork are achieved. The additional effort involved, and potential risks, should be balanced against the additional benefits to the community.



### 9.1.3 Surface Preparation

#### ***Cleaning***

Before asphalt is placed, the existing surface should be dry, and thoroughly swept to remove any loose stones, dirt and foreign matter. Sweeping should be carried out with a rotary road broom (Figure 9.1) or suction cleaner. Sweeping should extend at least 300 mm beyond each side of the area to be paved.

Any foreign matter adhering to the pavement and not swept off by broom should be removed by other means. Any existing asphalt areas affected by minor oil contamination should be cleaned by an approved method. Any area significantly affected by oil, and which has softened to an appreciable degree or has ravelled, should be removed and reinstated with asphalt.

**Figure 9.1: Rotary road broom**



Source: Austroads (2006c)

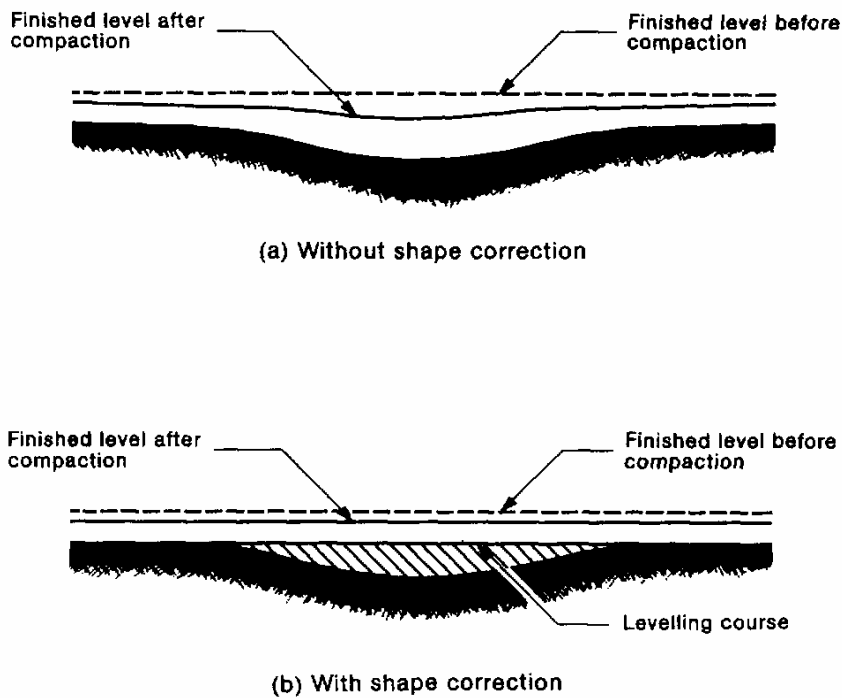
#### ***Correction of defects***

Prior to commencement of the paving operation, defects should be corrected, as follows:

- filling of potholes and depressions with asphalt or approved patching material. Cold-mix should not be used as it may lead to bleeding or flushing
- removal of excess binder from fatty patches
- crack filling
- repair of edge breaks
- cleaning and repairing of any joints
- removal and replacement of unstable materials
- removal and replacement of cold-mix patches
- shape correction.

Where the surface is badly out of shape, a corrective (regulation) course of asphalt should be placed (Figure 9.2). This will reduce the effects of differential compaction of subsequent layers and enable the best possible riding quality to be achieved. Alternatively, cold milling may be used to correct surface shape, as well as remove any unsuitable or unstable materials.

Figure 9.2: Shape correction



Source: Austroads (2006c)

Further information on maintenance of asphalt pavements is given in (Austroads 2006f).

**Preparation of concrete pavements**

Prior to placing asphalt over concrete pavements, remedial action may be required to ensure that concrete slabs are firmly supported and that the joints are in good condition as joints and cracks in the slabs may be reflected in the asphalt overlay. Slabs should be firmly supported by the subbase, and if necessary corrected by slab jacking, grout injection or other appropriate means.

Joints should be sealed with suitable hot bituminous filler. Techniques for control of reflection cracking include the use of a bandage of fabric impregnated with bituminous materials over the joints, or saw cutting of the asphalt overlay and formation of a sealed joint in the asphalt.

**9.1.4 Cold Milling**

Planing or milling of the surface is used to remove existing asphalt that is unstable, poorly shaped or where the new asphalt must match existing road levels, kerb and gutter, etc.

Common cold milling applications include:

- removal of surface that is uneven or rough
- removal of rutted, unstable or fatty materials
- restoring desired profile by removing excess crown or crossfall
- excavation of areas to be patched
- creation of tapers for smooth transition or matching of levels of adjoining work
- texturing as a bonding technique or for improving surface texture depth.

The benefits of cold milling include:

- provides an edge or key against which to compact new asphalt
- old pavement material is removed, eliminating build-up
- the need for levelling or regulating courses or material may be reduced or eliminated
- differential compaction problems from uneven bases are eliminated
- it may provide a source of recyclable reclaimed asphalt pavement
- it can be done quickly with minimum inconvenience to the traffic flow
- it allows neat and speedy excavation for patching.

The depth of milling (removal of material) will depend on the purpose and the material in the existing pavement. Depths of up to 150 mm are possible in one pass, except for the smallest machines (cutting width typically less than 500 mm) that are generally limited to a maximum depth of 100 mm.

Where possible, the size of the profiler should be matched to the size and productivity of the job. Table 9.1 provides a general guide to suitable profiler size based on typical production rates for a depth of cut of up to about 80 mm and appropriate job size that is suited to the productivity of the machine.

**Table 9.1: Guide to selection of profiler**

Width of profiler (mm)	Production rate (m <sup>2</sup> /h)	Suitable job size (m <sup>2</sup> )			
		0 – 500	500 – 1,000	1,000 – 2,000	> 2,000
150	10 – 30				
350	40 – 60				
500	60 – 80				
600	70 – 100				
1,000	200 – 500				
1,900	300 – 600				
2,000	400 – 700				
2,100	500 – 900				

Source: Austroads (2006c)

### 9.1.5 Pre-treatment of Granular Pavements

Prior to placing asphalt on a crushed rock or natural gravel pavement, a sprayed bituminous prime or primerseal should be applied. Although desirable, a primer or primerseal is not always necessary where the asphalt thickness is in excess of 100 mm.

A prime is used to penetrate the surface to protect the base against weather and assist in achieving a bond between the granular pavement and the asphalt layer.

A primerseal is used where it is desired to run traffic on the granular pavement before placing asphalt.

Sufficient time should be allowed for curing of primerseals prior to paving. A cutback bitumen primerseal may require up to about 12 months curing for evaporation of the cutter oil. Without proper curing, cutter oils may bleed through asphalt surfaces causing softening of the layer or carrying of binder to the surface.

Bitumen emulsion primerseals contain little or no cutter and do not require extended curing periods.

### 9.1.6 Pre-treatment of Asphalt Surfaces

A tack coat is a light application of bituminous binder that provides a bond between an existing asphalt surface and the new asphalt layer. Surfaced pavements require tack coating before commencing asphalt paving except when placing asphalt over a freshly placed, untrafficked, asphalt or clean pre-treated (prime or primerseal) surface. All contact surfaces (e.g. kerbs, pit edges, etc.) should be tack coated, including cold joints.

A tack coat should not be applied directly to natural gravel or crushed rock surfaces because of its inability to penetrate the surface and the likelihood of pick up of tack coat and underlying granular material by vehicles and paving equipment.

Tack coating should not be applied if the pavement surface is wet.

Generally, rapid setting cationic bitumen emulsion is used for tack coating although medium setting grades and anionic emulsions may be used in dry conditions. Asphalt should not be placed until the emulsion has fully broken; i.e. the tack coat has turned from its original brown colour to a shiny black.

The tack coat should be applied by a mechanical sprayer with a spray bar to ensure an even application (Figure 9.3). Hand spraying or brushing may be used but this is undesirable and should be limited to small irregular shaped areas inaccessible to a mechanical sprayer.

**Figure 9.3: Tack coat sprayer in operation**



Source: Austroads (2006c)

Generally, a bitumen emulsion tack coat is applied at a rate of 0.15 to 0.30 L/m<sup>2</sup> (60% bitumen content) of residual bitumen. The application rate is often doubled for joints and chases.

Some tack coating material may be picked up on truck tyres. If the pick up is excessive, a light application of coarse sand or 5 mm aggregate should be spread across the tack-coated areas traversed by construction traffic.

### 9.1.7 Pre-treatment of Concrete Surfaces

For asphalt overlays on concrete pavements the use of a primer is generally recommended to ensure an adequate bond to the existing surface is achieved. A tack coat may also be required, but by itself will lack the penetration into the concrete necessary to provide a good bond.

Concrete bridge decks often require special treatments in addition to priming or sealing before overlaying with asphalt.

### **9.1.8 Protection of Public Utilities**

The levels of public utility surface fittings (covers, access points, etc.) should be adjusted to match the proposed surface levels prior to paving or masked and clearly marked and recorded for immediate recovery after the asphalt work.

## **9.2 Transport of Asphalt**

Asphalt should be transported in such a way as to minimise loss of heat, segregation of the mix or contamination from foreign matter.

Delivery trucks should have clean, smooth, metal bodies and may include semi-trailers and dog trailers. The size and number of trucks are important to both the smooth running of the job and the quality of the work.

The mix should be delivered at a uniform rate, within the capacity of the spreading and compacting equipment, to enable a continuous paving process. To reduce cooling of the mix, deliveries should be made by the shortest practical route. Waiting time and delays on site should also be minimised.

During transportation, the asphalt should be covered with canvas or other similar waterproof cover. Transporting asphalt over long distances may require heavy-duty covers and, in some instances, insulation of truck bodies using oiled plywood or other suitable material to minimise heat loss.

## **9.3 Transfer of Asphalt from Trucks to Paver**

For most applications, asphalt is tipped directly from trucks into the front hopper of the paver.

Care should be taken when tipping the mix into the paver hopper to avoid spilling mix onto the pavement in front of the paver or jarring the paver.

Trucks should reverse to a position just short of the paver to allow the paver to make contact with the stationary truck and push it forward. Trucks should only apply brakes sufficiently to maintain the truck in contact with the paver.

Where project circumstances are suitable, a materials transfer device may be used to improve control over feeding asphalt materials to the asphalt paver. In this case, delivery trucks tip asphalt into a materials transfer device that then feeds it into the paver hopper by means of a conveyor belt.

Materials transfer devices (Figure 9.4) generally hold about 20 to 25 tonnes of asphalt.

**Figure 9.4: Materials transfer device**



Source: Austroads (2006c)

Use of materials transfer devices provides advantages in terms of:

- minimising the paver being bumped by trucks
- acting as a surge bin for asphalt delivery to minimise unplanned paver stops resulting from interruptions to asphalt supply
- reducing segregation of the asphalt in the paver hopper
- the ability to incorporate remixing facilities to reduce the influence of mix segregation and temperature variation during loading and transport.

If used correctly, materials transfer devices can lead to improved uniformity and smoothness of the paved finish.

## **9.4 Placement of Asphalt**

### **9.4.1 General**

Asphalt should be spread and compacted uniformly in order to:

- limit segregation
- produce a homogeneous product
- achieve a density that delivers the intended design performance of the asphalt
- provide the specified thickness of asphalt
- achieve the specified riding quality.

Spreading may be carried out by hand spreading, grader/bobcat or self-propelled paver.

Hand spreading should be kept to a minimum and generally used only for very small asphalt quantities, in areas where pavers cannot access and around services. Wooden lutes are most commonly used for hand screeding. Their light weight enables smooth screeding of hot materials. Where practicable, the screeding should be done with a head of material in front of the lute, and using a single pass that leaves a uniform surface of fresh asphalt. Excessive working of the surface leads to separation of coarse materials, and should be avoided.

Spreading by grader or bobcat may be used in special circumstances such as small quantities and asphalt patches that are not wide enough to allow paver access. Spreading by grader in particular should be discouraged because of difficulty in achieving compaction and ride quality requirements and risk of segregation of the asphalt. Graders and bobcats should only be used for applications such as temporary access roads and patching work where it is not practical to operate a paver.

Wherever possible, self-propelled pavers should be used as they provide greater control during spreading and a superior surface finish (Figure 9.5). Particularly for larger projects, paver operations are also quicker and more economical.

### 9.4.2 Spreading by Paver

Spreading is designed to be a continuous operation. The rate of delivery of the asphalt should be arranged so that the paver can operate at a uniform speed. Paving should not commence until sufficient asphalt is on site to ensure continuous operation.

Pavers operate on the floating screed principle, spreading material in a uniform layer to a desired thickness and longitudinal and transverse shape. During paving, the screed plate is supported by the asphalt being spread. This also provides partial compaction of the asphalt. The thickness of the asphalt layer is determined by the height of the towing arm and angle of attack of the screed plate. Changes to thickness are achieved by changing the height of the tow point and/or changing the angle of the screed plate relative to the towing arm. Changing the angle of attack of the screed plate produces a gradual change in asphalt thickness until a new equilibrium state is achieved.

**Figure 9.5: Asphalt paver**

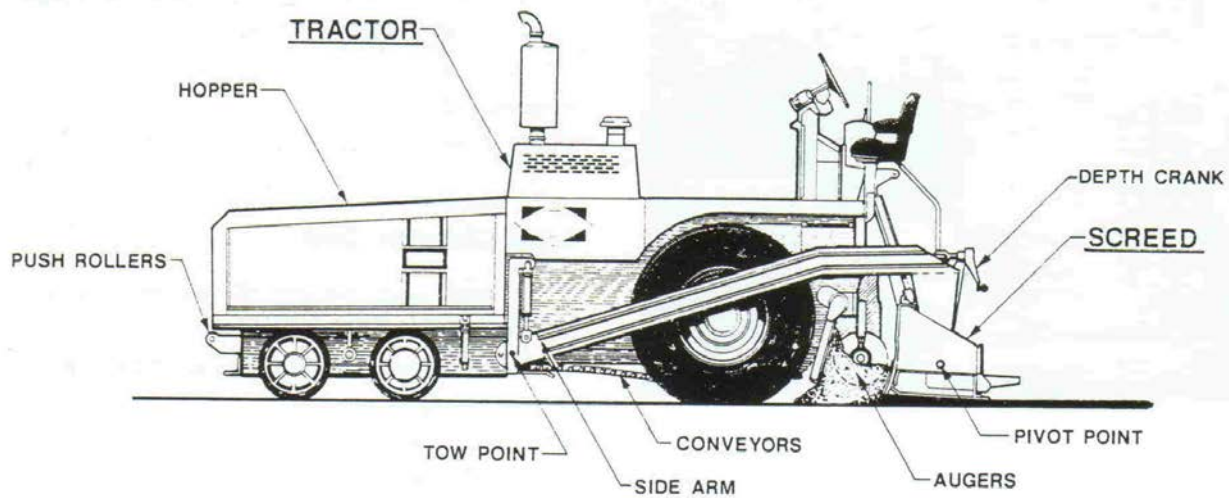


Source: Austroads (2006c)

Adjustments to the tow point and screed plate angle (Figure 9.6) may be made manually, or controlled automatically by sensors referenced to an adjoining paved surface, levelling (or averaging) beam attached to the paver, fixed reference line or by computer programmed level data.

Minor changes to the finished level will also be introduced by changes in the head of the material in front of the screed, adjustments in paver forward speed, and variation in asphalt stiffness due to changes in mix temperature. Smooth, continuous operation is therefore an important factor in achieving high standards of ride quality.

Figure 9.6: Typical paver components



Source: Austroads (2006c)

The width of paving may be varied using extensions to the screed. These may be rigid boxes that are bolted to the screed, or hydraulically operated extensions (Figure 9.7). Paving widths of up to 8 m are possible using screed extensions, although widths greater than 5 or 6 m are not commonly used due to the difficulty of maintaining a consistent head of material across the full width of the screed.

Figure 9.7: Paver with hydraulic screed



Source: Austroads (2006c)

**Paver stoppages**

When the paver stops the equilibrium conditions of the screed can change due to:

- a slight settling of the screed
- cooling of the material in front of the screed.



When the paver starts moving again, the screed will rise or fall to achieve a new equilibrium position. These effects can be eliminated or minimised if the paver moves continuously. Sometimes it is not practical to keep the paver moving continuously. In these cases the stopping and accelerating of the paver should be achieved quickly but smoothly. During prolonged paving stoppages, settling of the screed may cause a permanent depression of unacceptable depth in the pavement surface. In such circumstances the mat should be cut back to remove the depression and a transverse joint constructed.

### **Automatic sensing and levelling equipment**

Automatic sensing and levelling equipment is used to control the operation of the screed unit in a pre-determined relationship relative to either:

- the existing surface (Figure 9.8)
- an adjoining finished surface (Figure 9.9)
- a fixed reference line.

These controls are used to maintain levels, mat thickness and crossfall within required limits.

There are a number of types of sensing and levelling equipment, including:

- joint matching shoe (Figure 9.8)
- levelling (or averaging) beam (Figure 9.9)
- fixed wire
- crossfall control
- computerised level control
- laser control.

**Figure 9.8: Joint matching shoe**



Source: Austroads (2006c)

**Figure 9.9: Levelling beam**

Source: Austroads (2006c)

### 9.4.3 Joints

Construction joints in an asphalt layer are planes of weakness and imperfection that can be the first locations to deteriorate. Generally, this is due to initial separation of the joint, the ingress of water, and cracking and ravelling. However, joints that are more permeable than the rest of the asphalt mat may also lead to the ingress of moisture into the pavement during periods of wet weather.

The number and extent of joints in asphalt layers should be kept to a minimum and the paving pattern should be designed accordingly in advance of the work.

#### ***Longitudinal joints***

Correct jointing techniques ensure that the two mats are joined in such a way as to minimise differences in density, texture, shape and level and also minimise the extent of joints. Generally, longitudinal joints should be placed away from wheel paths and offset between layers.

In some cases, before the longitudinal joint between two adjacent paver lanes is constructed, the edge of the previously placed mix is cut back to remove material that may have a lower density than the main portion of the mat. This is typically accomplished with a cutting wheel attached to a roller (Figure 9.10). The cut face should be lightly tack coated before the adjacent lane is placed.

**Figure 9.10: Cutting disc attached to steel roller**



Source: Austroads (2006c)

Cutting back of the exposed edge is generally not necessary if adequate joint construction procedures are followed.

Tack coating is generally not required for clean, untrimmed edges.

A hot joint is one where both the new mat and the adjacent mat are still workable and have not been compacted. Hot joints should be constructed by leaving an uncompacted strip approximately 150 mm wide along the edge of the first placed mat until the subsequent mat has been laid. Both sides of a hot joint should then be rolled simultaneously. Hot joints are preferable to cold joints but are usually only possible when using two pavers in echelon, e.g. on large construction works (Figure 9.11). In such cases, the distance between pavers operating in echelon should not exceed 80 m.

**Figure 9.11: Paving in echelon (hot joints)**



Source: Austroads (2006c)

### ***Transverse joints***

Transverse joints are construction joints in the asphalt paving, at right angles to the direction of paving, and are formed:

- at the start and finish of each paving run
- when the work is disrupted causing cooling of the asphalt and/or settlement of the screed
- when resuming on the next day.

Transverse joints can be a common cause of poor riding qualities in a finished asphalt wearing surface. The importance of correct transverse joint preparation and formation techniques cannot be over-emphasised.

Transverse joints should be staggered by at least 1 m between successive layers and between adjacent runs to avoid planes of weakness and possible water entry through the whole asphalt thickness.

A paving run may be finished against a timber bulkhead to ensure a straight, vertical, well compacted edge, or may be feathered out (ramped) and compacted. For ramped material, the transverse joint is formed by the subsequent trimming back to a line where the minimum layer thickness exists.

When finishing flush against an existing surface, the paver should maintain sufficient material in front of the screed to pave to the end of the run. It is poor practice to finish machine spreading several metres before the end of the run, lifting the screed, tipping asphalt from the paver, and then hand spreading the remaining material to finish the run. This hand-spread material rarely matches the surface finish and uniformity of the machine laid material.

Trimming of transverse joints, and edge cutting, may be carried out using:

- a cutting disc attached to a steel wheel roller (Figure 9.10)
- a jackhammer with spade attachment
- milling machine.

#### **9.4.4 Layer Thickness**

The thickness of asphalt layer(s) within a pavement should normally be determined by the structural requirements of the pavement. For initial treatments, the determination of thickness is part of the structural design. For re-treatments, the thickness will depend on the amount of surface correction and structural strengthening required. If this is considerable, it may be necessary to lay one or more corrective courses before the final course.

The nominal size of an asphalt mix is an indication of the maximum particle size present and is usually expressed as a convenient whole number above the largest sieve size to retain more than 0% and less than 10% of the aggregate material.

The selected nominal size of mix will typically be determined by:

- location of the asphalt course in the pavement
- proposed compacted thickness of the layer
- functional requirements of the asphalt layer.

Generally, asphalt should be placed in layers with a compacted thickness of not less than 2.5 times the nominal size of mix in order to:

- prevent the mix tearing during laying
- assist the compaction process by allowing the aggregate particles to mechanically interlock.

Table 9.2 provides a guide to appropriate mix sizes for ranges of course thickness.

**Table 9.2: Typical asphalt layer thickness**

Nominal mix size (mm)	Compacted layer thickness (mm) <sup>1</sup>
5	15 – 20
7	20 – 30
10	25 – 40
14	35 – 55
20	50 – 80
28	70 – 110
40	100 – 160

The minimum thickness may need to be increased when placing thin layers in cool conditions or using less workable mixes. The maximum thickness may be exceeded provided that asphalt surface shape requirements and specified compaction standards over the total thickness of the layer can be adequately achieved.

The thickness of the uncompacted mat depends on the degree of compaction achieved by tampers and screed. It may be up to 20 to 25% thicker than the required compacted thickness. Field experience will determine the required uncompacted thickness.

## 9.5 Compaction

### 9.5.1 General

Adequate compaction of asphalt is essential to ensure that the design performance of the mix and expected service life are achieved. The compaction should be uniform and achieve a high density.

The main factors influencing the successful compaction of asphalt are:

- type and numbers of rollers or other compaction equipment – requirements vary with rate of spreading, layer thickness, rate of cooling and workability of asphalt mix
- rolling procedures and techniques – rolling sequence, rolling speed and number of passes
- temperature of the mix – compaction must be completed while asphalt remains workable. Asphalt will cool more rapidly in thin layers and cool ambient conditions
- mix properties – workability is a function of internal friction of mix and binder stiffness
- soundness and stiffness of the underlying base – must adequately support compaction equipment without distortion of asphalt layer.

For paving operations, compaction is a two-stage process as follows:

- primary compaction by the paver screed
- secondary compaction by rollers.

### 9.5.2 Compaction Equipment

The equipment used for compaction of asphalt work includes:

- oscillating and vibratory steel-wheeled rollers – most commonly used for initial rolling and basic compaction (Figure 9.12)

- non-vibratory (static) steel-wheeled rollers – may be used for initial rolling, although largely superseded by vibratory steel-wheeled rollers
- pneumatic multi-tyred rollers – used for secondary compaction and sealing of the surface (Figure 9.13)
- impact compactors such as vibratory plates, hand tampers, etc. – generally only used for areas inaccessible to rollers and minor works.

**Figure 9.12: Vibratory steel-wheeled roller**



Source: Austroads (2006c)

Rollers should have good brakes and smooth transmission systems to prevent shoving damage to the mix when starting, stopping and changing direction. The steering should also be in proper adjustment. To prevent pick up of the mix, rollers should be fitted with watering systems and fibre mats or scrapers. Care should be taken not to use excessive amounts of water, which may cool the asphalt. Once the tyres of pneumatic multi-tyred rollers have heated up, water may not be required. Light applications of diesel oil on this compaction equipment may also be used to prevent pick-up. The use of excessive diesel should be avoided, as it will soften the asphalt.

**Figure 9.13: Pneumatic multi-tyred roller**



Source: Austroads (2006c)

Vibratory plates, mechanical tampers, hand guided rollers and hand tampers are used mainly on small jobs or in areas where it is difficult to obtain access for larger rollers, e.g. median nosings.

### 9.5.3 Roller Numbers and Speed

The minimum number of rollers required to achieve adequate compaction depends on:

- depth and width of layer
- placement temperature and rate of cooling of mix
- rate of spreading (output)
- mix type and binder type
- compactability of mix.

Table 9.3 provides a guide to the minimum number of rollers required to provide adequate compaction based on rate of spreading, or output. Actual types and numbers of rollers should be determined for each project.

**Table 9.3: Number of rollers (dense graded asphalt)**

Range of output		Alternative combinations of rollers		
Tonnes per hour	Tonnes per day	Steel static <sup>3</sup>	Steel vibrating <sup>4</sup>	Pneumatic multi-tyred <sup>5</sup>
< 20	< 160	1	–	1
		–	1	1
20 – 45	160 – 360	1	–	1
		–	1	1
45 – 85	360 – 680	1	–	2
		–	1	1
		–	2	1
85 – 120	680 – 960	1	–	3
		2	–	2
		–	2	1

Notes:

1. For thin layers in cold weather, additional rollers will be required.
2. Thicknesses assumed for the purpose of this table are up to 60 mm. For greater thicknesses, the numbers of rollers may be reduced, e.g. for rate 45 t/h to 85 t/h and depth of 100 mm, only one steel vibrating or static roller and one pneumatic multi-tyred roller would be required.
3. The steel static rollers are assumed to be self-propelled and of 6 t to 12 t mass.
4. The steel vibrating rollers are assumed to be self-propelled tandems of a minimum of 6 t mass.
5. The pneumatic multi-tyred rollers are assumed to be of 10 t to 20 t ballasted mass. Multi-tyred rollers are not used on open graded asphalt, ultra thin open graded asphalt, stone mastic asphalt and many minor works.

Rollers should travel at uniform speed, which is sufficiently slow to prevent displacement of the mix. Acceleration and braking should be carried out as smoothly as possible.

Roller speeds should be in the following ranges:

- steel wheeled rollers – not exceeding 5 km/h
- vibratory rollers – 8 to 10 km/h
- pneumatic multi-tyred rollers – 6 to 10 km/h.

If the mix is being adversely affected by rolling, the speeds should be reduced to prevent excessive displacement.

If transverse cracks appear in the surface, the roller should be removed until the mix is sufficiently stable to support the roller.

#### 9.5.4 Rolling Procedures

Rolling should be carried out as soon as possible after placing the mix but should not commence before deficiencies in spreading of the mix are corrected.

The rolling of a freshly laid asphalt mix should be carried out in the following order:

- transverse joints
- longitudinal joints (when adjoining a previous run)
- outside edge
- remainder of the mat.

The sequence of rolling the remainder of the mat is:

- initial rolling
- intermediate rolling
- final rolling.

Initial rolling compacts the asphalt to obtain most of the final density. Intermediate rolling increases density and seals the surface. Final rolling removes roller marks and other blemishes left by previous rolling.

A typical rolling sequence is shown in Table 9.4.

**Table 9.4: Typical rolling sequence**

Rolling phase	Roller type
Initial rolling	Non-vibratory steel roller or vibratory roller in static mode
Intermediate rolling	Vibratory steel roller in vibratory mode plus pneumatic multi-tyred roller
Final rolling	Steel roller

Best results are obtained when rolling is performed in a definite pattern that provides a uniform coverage of the run. A rolling pattern should be developed to suit the particular location, and operators should be required to follow it.

To ensure the required results, the whole rolling procedure must be carefully observed, by both operators and supervisors, so that any adjustments to the rolling technique to adapt to conditions can be made immediately.

The rolling pattern and number of passes should be selected carefully when using vibratory rollers. Overuse of vibration or number of passes can lead to de-compaction and delamination of the mix. This results in a rapid lowering of density, which is extremely difficult to correct with falling temperature.

Heavy equipment including rollers should never stand on a recently compacted surface before it has cooled sufficiently to resist deformation under the static load.

The length of a rolling lane should normally be about 30 to 50 m, depending on ambient conditions.



Sharp turns should be avoided and any change from forward to reverse should be made smoothly. Vibratory rollers should not be stopped or reversed while in vibratory mode. Lateral changes in the direction of rolling should be made on previously compacted mix.

An alternative technique allows for a sweeping turn at the end of each pass equivalent to the width of an adjacent pass, during the initial rolling. This method results in a diagonal roller stop depression (and ridge of uncompacted material) rather than it being 'square'. When the roller passes over this diagonal stop, the progressive (rather than sudden) transition from the compacted mat to the uncompacted material, results in a better distribution of the material and a reduced depression. Therefore, this technique aims at a better surface finish with less noticeable roller stop depressions.

### 9.5.5 Mix Temperatures for Placing

Temperature is by far the most important factor and a key element in the compactability of the asphalt mix. The mix should not, however, be so hot as to result in a low viscosity of binder and insufficient cohesion in the mix to adequately support rollers without excessive displacement.

Temperature of asphalt during placing depends on the initial spreading temperature and rate of cooling.

The rate of cooling is a function of:

- layer thickness
- road surface temperature
- ambient conditions – air temperature, wind, and moisture.

Thicker layers retain heat longer resulting in more time for compaction and/or a reduced number of passes. Retention of heat in thick layers may also necessitate delaying placing of subsequent layers until the lower layer has cooled sufficiently to provide a solid working platform.

Asphalt will cool more rapidly at low base temperatures. Wind and rain will further increase the rate of cooling.

Asphalt mixes containing modified bitumen binders generally require compaction to be completed at temperatures that are typically 5 °C to 10 °C higher than mixes with conventional binder as the binder stiffens more quickly. Most manufacturers of PMB provide guidelines for their products.

All these factors are inter-related and influence:

- conditions under which asphalt work should proceed
- minimum temperature for delivery and spreading of asphalt
- time available for compaction and hence choice of rolling equipment and compaction techniques.

Asphalt construction specifications typically limit the minimum pavement temperature, maximum wind speed and/or minimum air temperature for asphalt paving.

Placing of thin layers of asphalt at low temperatures may allow less than five minutes before the asphalt mix temperature falls below the minimum required for effective compaction. For thicker layers placed at low temperatures the period for compaction may be up to 10 minutes. These times apply to dry conditions and wind speeds less than about 10 km/h. Higher wind speeds or presence of moisture will further reduce the time available for effective compaction.

## 9.6 Conformance and Quality Testing

### 9.6.1 Material Supply

The *Guide to Pavement Technology – Part 4B: Asphalt* (Austroads 2007) contains advice on the selection, testing, quality control and specification of materials for use in asphalt pavement construction.

### 9.6.2 Trial Pavement

For larger works, particularly project work using transportable mixing plants, it may be appropriate to carry out paving of trial areas. This can assist in assessing:

- suitability of proposed mix in terms of workability, ease of compaction, and surface texture (including check for segregation)
- adequacy of mixing plant to supply asphalt at the rate required for continuous paving
- adequacy of the proposed transport to ensure the mix arrives on site at the required temperature and rate of supply
- adequacy of spreading equipment, and associated techniques to achieve the required rate of paving and to produce the required quality of asphalt pavement
- suitability of the spreading and rolling pattern including location of joints
- suitability of compaction equipment and procedures
- allowance in thickness to be made for roller compaction.

Generally, a trial section of one lane width between 50 and 100 m in length must be constructed in one continuous operation.

Typically trial pavements should be assessed for conformity using the same requirements as for the main works (i.e. thickness, levels shapes, compaction, etc.). However, the frequency of testing may be greater than otherwise specified.

If significant changes are made in the equipment, materials or plant or if asphalt placed in the main works fails substantially to comply with the specification further trial pavements may be required.

### 9.6.3 Thickness and Level Tolerance

The average total compacted thickness of the combined asphalt courses should be not less than the specified thickness. The thickness of any individual asphalt course should be not less than the specified thickness by more than 10 mm.

Each course should be finished to a plane surface, parallel to the finished surface of the wearing course, so that subsequent courses can be of uniform thickness.

The level at the top of each course of asphalt should not differ from the specified level by more than 10 mm, except that where asphalt is placed against kerb and channel, the surface at the edge of the wearing course should be flush with, or not more than 5 mm above the lip of the channel.

An open-graded wearing course, which is designed to allow free drainage from the lowest edge, needs to be finished with the entire layer thickness above the lip of the channel unless special edge drainage is provided.

Surface shape is generally assessed as the deviation measured between any two points under a 3 m straight edge. Straight edge checking may be carried out at any stage of construction. Checking should be performed at closely spaced intervals and at all joints.

Commonly specified tolerances for shape are indicated in Table 9.5.

**Table 9.5: Typical permissible tolerances in shape**

Course	Deviations from 3 m straight edge (mm)			
	Airports, rural highways where traffic speed > 70 km/h	Urban highways, classified roads where traffic speed < 70 km/h	Minor roads, residential streets	Parking areas, driveways
Correction course	10	15	20	25
Base course/intermediate course	5	10	12	15
Wearing course	3	5	7	10

### 9.6.4 Compaction

#### General

Testing of density is usually undertaken on a lot-by-lot basis. Lot sizes are generally one shift production of the same mix and layer. Lots must be essentially homogeneous sections of completed pavement. Defective areas of pavement showing cracking, bony or fatty material should be rectified before being tested separately.

Density testing should be carried out as soon as practicable after completion of work using either core samples or nuclear gauge testing of in situ materials. Location of test sites should be determined by a suitable method of random sampling.

Density testing is usually not undertaken on lots of less than 50 t, layers with a nominal thickness less than 30 mm, or layers with a nominal thickness less than 2.5 times the nominal mix size.

Relative compaction is the percentage ratio of the in situ density of the compacted asphalt to the reference density of the asphalt of a particular lot. Characteristic values of relative compaction are calculated using statistical procedures.

Two methods for determination of reference density are in common use. They are:

- maximum density
- laboratory compacted bulk density.

Use of maximum density as the reference density allows results to be directly converted to in situ air voids (i.e. 100 – percentage relative compaction). The required value will vary according to mix type and application. Typical minimum characteristic values for air voids for dense graded asphalt work are shown in Table 9.6.

**Table 9.6: Typical in situ air voids (dense graded asphalt)**

Layer thickness/application	Maximum characteristic value	
	Heavy traffic	Light/medium traffic
30 – 50 mm	8	9
> 50 mm	7	8
High fatigue base	6	–

Typical characteristic values of relative compaction based on laboratory bulk density are shown in Table 9.7.

**Table 9.7: Typical relative compaction (bulk density)**

Layer thickness	Minimum characteristic value of relative compaction
30 – 50 mm	95%
> 50 mm	96%

**Testing of density using nuclear density gauge**

Two forms of nuclear gauge that are used on asphalt work are the standard backscatter gauge and ‘thin lift’ gauge. The thin lift gauge uses a dual detector set that enables the operator to select the measurement depth over a range of 25 to 100 mm whereas the standard gauge is not suitable for layers less than 50 mm in thickness.

Gauges are calibrated using blocks of materials of known density. It is also recommended that field density offsets be established by comparison with core samples. Care is required to ensure accurate seating of the meter in order to achieve accurate results.

Nuclear density testing is quick and non-destructive (Figure 9.14). A nuclear gauge gives an indirect measure of field density and hence requires calibration in accordance with AS 2891.14.3 or AS 2891.14.4 as appropriate. Nuclear density gauges may also be used in the development of rolling patterns, in particular the number of roller passes.

**Figure 9.14: Nuclear density testing**



Source: Austroads (2006c)

### 9.6.5 Riding Quality

The finished surface should have a smooth longitudinal profile for good riding quality. For general asphalt work, the application of shape standards as referred to above, together with the use of good placing practices, should provide adequate surface smoothness and ride quality.

Measurement of pavement smoothness may be applicable to high class facilities where the importance of the resulting ride quality justifies both the cost of testing and the additional cost of work procedures that may be required to achieve the required standard, for example, freeways and major arterial roads with posted speed limits of 80 km/h or more.

The standard of ride quality that can be achieved will depend on the roughness of the surface on which the asphalt layer is to be placed, and the extent of shape correction and additional asphalt layers that may be applied prior to the final layer.

Measurement is usually made as the average of three replica runs. Generally, each lane is divided into homogeneous sections 100 m long. Start and finish joints of the project are generally excluded.

Typical values are as follows:

**New Work:** A maximum of 40-50 NAASRA roughness counts or 1.6-2.0 IRI is generally applicable where the contractor has control of the shape of base and wearing course layers. Higher standards are achievable but the cost of obtaining higher ride standards through the use of special techniques and equipment needs to be balanced against the benefit likely to be derived from the attainment of that higher standard of pavement smoothness.

**Resurfacing:** The standard of ride quality achievable is influenced by the shape of the existing surface and the number of layers, or extent of any regulation and shape correction, prior to placing the wearing course layer. Specifications typically set out the calculation method used to determine the ride requirement for asphalt overlays. As a general guide, a suitable target for improvement for each layer is to achieve a NAASRA Roughness Count of 60% of the existing conditions plus 5. For example, for an existing roughness count of 80:

$$\text{Target} = 0.60 \times 80 + 5 = 53$$

For measurement in IRI, a similar calculation is performed to determine the ride requirement following an asphalt overlay. For example VicRoads uses  $\text{IRI (after overlay)} = 0.3 + (0.667 \times \text{IRI before overlay}) - (0.0109 \times \text{placed asphalt thickness})$ .

### 9.6.6 Audit and Surveillance of Asphalt Paving Contract Work

A guide to audit and surveillance of asphalt paving contract works is included in (Austroads 2006b) which provides checklists for paving activity as well as general guidelines for the preparation of quality plans, inspection and test plans and manufacture of asphalt. Detailed checklists for asphalt paving activity and a troubleshooting guide are included in (Austroads 2006c).

## 10. Concrete Pavements

### 10.1 General

Design of concrete pavements is covered in the *Guide to Pavement Technology – Part 2: Pavement Structural Design* (Austroads 2008b) while additional detailed advice on concrete is provided in the *Guide to Pavement Technology – Part 4C: Materials for Concrete Pavements* (Austroads 2009b).

It is acknowledged that the RTA *Concrete Pavement Manual* (RTA 1991) and Centre for Pavement Engineering Education course notes (CPEE 2006) were useful sources of information in the preparation of this section.

#### 10.1.1 Sub-base Types

For concrete pavements the sub-base is typically defined as the layer between the formation or select material zone and the base concrete.

The purpose of the sub-base is to provide uniform support to the base concrete layer and avoid erosion type distress under transverse joints and cracks.

Sub-bases may be constructed of granular materials, asphalt, cement stabilised material or lean-mix concrete. However, a bound or lean-mix concrete sub-base is recommended (Austroads 2008b) under a concrete pavement for the following reasons:

- to resist erosion of the sub-base and limit 'pumping' at joints and slab edges
- to provide uniform support under the pavement
- to reduce deflection at joints and enhance load transfer across joints (especially if no other load transfer devices are provided, such as dowels)
- to assist in the control of shrinkage and swelling of high volume-change subgrade soils.

Lean-mix concrete (LMC), which is the most common sub-base used for concrete pavements in Australia, typically has a characteristic 28-day compressive strength of not less than 5 MPa and is designed to have low shrinkage, typically less than 450 microstrain. There are typically no sawn or formed joints in a lean-mix concrete sub-base as it is anticipated that fine random cracks are produced during shrinkage of the lean-mix concrete sub-base. In Australia, contractors generally prefer to slipform lean-mix concrete sub-bases due to the resulting high productivity levels and the accurate final surface levels of the completed sub-base. In other countries, a rolled lean-mix concrete or a cement stabilised plant-mix layer is sometimes used to construct the sub-base.

#### 10.1.2 Base Types

The principal types of concrete pavements are:

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

Joints in PCP and JRCP may be skewed or square and may be dowelled or undowelled.

### 10.1.3 Wearing Surface

The wearing surface texture specified for the road should take into consideration the traffic speed, grade, crossfall, carriageway width, and rainfall. For further details refer to (Austroads 2009a).

Other types of concrete surfaces (e.g. stamped and stencilled) can be produced for residential streets and medians. Road Note 43 (C&CAA 1994) provides guidance on the specification and construction of these surfaces.

For the purpose of the base design thickness, wearing surface layers of asphalt or concrete segmental paving are deemed not to contribute to the strength of the pavement.

### 10.1.4 Construction Process

The construction procedure for a concrete pavement generally follows the steps shown in Table 10.1.

**Table 10.1: Typical construction procedure for concrete pavements**

Stage	Description of construction stage
1	Prepare and evaluate trial concrete mixes
2	Set up concrete batch plant
3	Set up paver for sub-base placement
4	Construct sub-base
5	Saw cut into sub-base, excavate and construct slab anchors
6	Install reinforcement for JRCP and CRCP
7	Construct base pavement. Two or more paving runs may be required according to the size of the paver and specification limitations
8	Construct base hand paving slabs – typically at ramps and crossovers
9	Seal all joints
10	Construct verges
11	Install guardrails and other safety devices
12	Install lane markers and lines

Subsoil pipe installations are not listed in Table 10.1 but are generally installed at the end of the earthworks and before paving commences.

In a rural divided road carriageway (Figure 10.1), concrete shoulders greater than 1.0 m in width are typically constructed separate to the main carriageway due to the combined width, the location of the longitudinal joint and the ability of saw cutters to create an induced joint when the slab dimension ratios, ratio of width to length of a panel surrounded by joints, are about 1 to 2.

**Figure 10.1 Typical concrete road**



Source: RTA

## 10.2 Surface Preparation

Concrete roads are designed and built on the premise of uniform support of the base layer. Therefore, construction practices are aimed at ensuring initial and long-term support of the base layer. Adequate drainage paths and removal of excess water from the selected material zone and formation assist in providing uniform support to the subbase. It is also important to effectively compact materials when backfilling trenches and adjacent to bridge support structures to minimise the effect of uneven support over time.

## 10.3 Concrete Production and Delivery

### 10.3.1 Concrete Batch Plants

The process of concrete production is an important element in successful paving, and the following components should be considered:

- adequate access and space for the storage of raw materials
- clean storage of raw materials
- access to quality water supply
- cool storage or other devices to keep water cold for summer paving
- access to electricity or the use of a generator
- sufficient mixing drum capacity to meet paver output
- a run-off containment plan to meet environmental regulations.

Figure 10.2 shows a typical concrete plant, which is very common in Australia and is favoured by most contractors.



**Figure 10.2: Concrete batch plant**

Source: RTA

Three or more bays are typically used to store fine and coarse aggregate at the concrete plant. The surrounding area and bin floor must be kept relatively clean as clay or soil from the tracks or buckets of loaders and delivery trucks may contaminate stockpiles.

During summer or windy/warm weather, aggregate stockpiles may become dry or hot. This creates problems in achieving the correct concrete slump and may give rise to high concrete temperatures before placement. Protecting the water tank with a tarpaulin and cooling the water by refrigeration helps reduce the concrete temperature during summer concrete placement.

The storage capacity for raw materials is a function of the paver production rate or alternatively, the paver speed may be limited by the production capacity of the plant. If the paver is expected to average 1 m/min for a 10-hour day shift, the total concrete production for a 10 m wide pour with a 230 mm thick pavement is 1,380 m<sup>3</sup>. If the quantity of cementitious material is 0.35 t/m<sup>3</sup> of concrete, the cementitious material used per day is 483 t. As a standard cement delivery by bulk tanker is about 30 t, some 16 tanker deliveries are required per day, or on average one every 40 minutes.

An experienced plant designer and operator must consider the batching sequence of the cement (possibly general purpose cement and fly ash), sand (fine aggregate), coarse aggregate, water and admixtures. If using steel fibres and admixtures, the hatching sequence of the materials is critical (Hodgkinson 1994). When using admixtures, especially with a combination of products, the manufacturer's recommendation for dosing the water should always be sought. Some contractors have encountered problems when one admixture has reacted with another, forming a gel that resulted in a blocked supply line to the mixing chamber.

The mixing time of the raw materials in the mixing drum is typically over one minute and usually confirmed by a mixer uniformity test. The mixing time and size of the mixing chamber will sometimes control the size of the paving run.

### 10.3.2 Delivery of Concrete

The objectives for transporting concrete from the plant to the site are:

- to prevent segregation or loss of materials in the mix, such that the mix is homogeneous when delivered to the paver or into forms for fixed-form construction
- to ensure that the concrete is workable at the time of placing.

For slipform paving, the concrete must have the specified slump after discharging from the truck in front of the paver. Should the concrete undergo substantial changes during transport, the concrete is likely to produce paths in the paved concrete that may impact on concrete characteristics such as texturing and strength.

The two methods of transporting concrete from the plant to site are by transit mixer or tipper trucks (Figure 10.3).

**Figure 10.3: Tipper trucks being used on a typical PCP project**



Source: RTA

From established urban premix plants, transit mixers are commonly used as they perform the mixing process in the truck rather than in a drum. In using transit mixers for slipform paving a number of factors need to be considered. A typical slipform paver moves constantly at about 1 m/minute and the required quantity of concrete at the slipform paver is 2.5 m<sup>3</sup>/minute based on 10 m wide and 250 mm thick pavement. Given that an average transit mixer can discharge its load at about 2 m<sup>3</sup>/minute, the minimum number of trucks requiring access at the front of the paver would be two. Consideration would also have to be given to the confined space between stringlines, simultaneous discharging of two or more trucks and road access in the area.

It is common for the operator of a transit mixer to remix the concrete before discharging the fresh concrete. The duration of the remixing varies according to the properties of the mix, but typically one minute is used.

The use of tipper trucks is more common in rural sites with dedicated batching plants. A split-drum mixer of 4 to 6 m<sup>3</sup> capacity produces a load of about 10 to 15 t. It is essential that axle loads do not overload the sub-base or base concrete with respect to its age. These limitations are set by the road agency and may be calculated by the thickness design method using an appropriate flexural concrete strength, load safety factor and axle load distribution for loaded and unloaded trucks. Trafficking of the sub-base by concrete delivery trucks may be permitted over limited lengths when the in situ compressive strength of the sub-base has reached 4 MPa.

To ensure that construction traffic does not cause distress to the sub-base, specifications limit access for base construction to approximately 300 m immediately ahead of the paver. Vehicles with a gross mass of less than 1.5 t are typically allowed to use the base and sub-base without limitation after minimum concrete strengths are achieved.

Another consideration for the delivery of concrete is the distance between the plant and site, and whether road or construction traffic may delay delivery of concrete to the paver. Delivery time is limited by the specification and is usually set at 90 minutes for transit mixers and 45 minutes for tipper trucks. However, prevailing weather conditions can shorten the time in warm weather.

The number of trucks in a fleet for paving operations is determined by the mixing time, travel time to the site and the discharge of concrete from the tipper or transit mixer. Traffic delays must also be taken into consideration for travel time and the number of trucks per hour should be greater than the average speed of the paver. In most cases, the production capacity of the concrete plant will also impact on the whole operation.

### 10.3.3 Retempering

Concrete that is delivered by other than a mobile batch mixer must not have water (retempered) or any other ingredient added to the mixed batch.

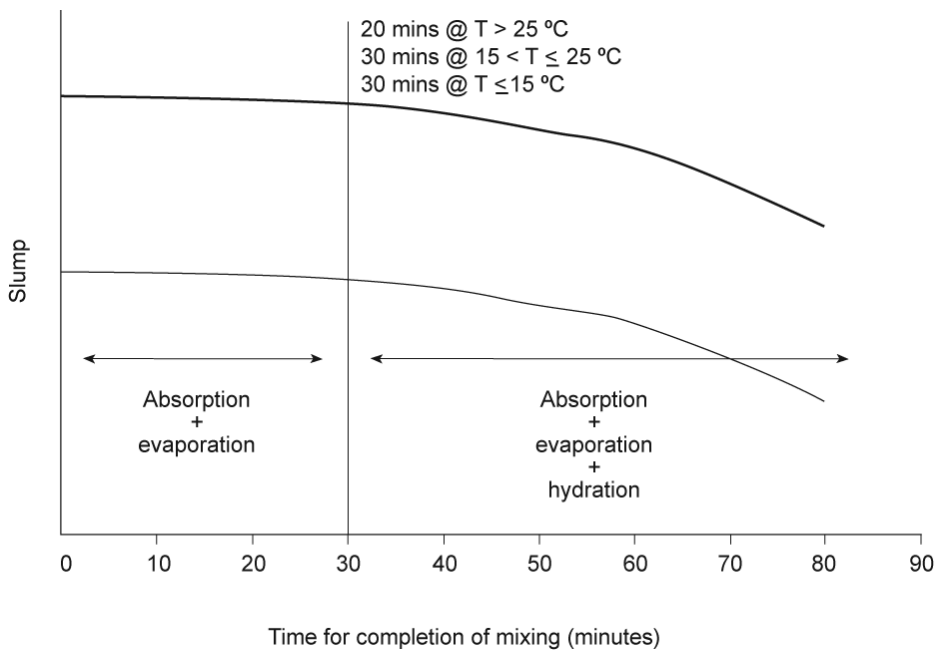
Concrete that is delivered by mobile batch mixer may be retempered prior to the completion of discharge of the batch provided the following conditions are met:

- Immediately after retempering, the mixing mechanism must be operated at mixing speed for not less than the mixing time displayed on the mixer's compliance plate, or such additional time as may be necessary to re-establish uniformity of the mix.
- The retempering and the quality of the water added (accurate to one litre) should be recorded on the identification certificate for that batch.
- The slump should be checked for compliance immediately after mixing the retempered batch.
- The quantity of water added must be such that the maximum water to cement ratio, if specified, is not exceeded.
- Retempering is within 30 minutes ( $> 25\text{ }^{\circ}\text{C}$ ) or 40 minutes ( $\leq 25\text{ }^{\circ}\text{C}$ ) from the time of batching.
- Retempering takes place only at either the batch plant, the testing station or the point of placement.
- Test cylinders for compressive strength are made from the retempered mix.

Any water added to the transit mixer must be measured and thoroughly mixed for a minimum of 2 minutes. It is also noted that it is good construction practice to mix the concrete, with or without retempering, for a minimum of 2 minutes prior to discharging.

The limits on retempering outlined in most specifications are based on the commencement of cement hydration as shown in Figure 10.4.

Figure 10.4: Limit of retempering



Source: CPEE (2006)

## 10.4 Steel Reinforcement

Detailed advice on steel reinforcement (Figure 10.5 and Figure 10.6) for concrete roads is provided in the (Austroads 2009b); however, a brief description is given here.

Figure 10.5: CRCP reinforcement



Source: RTA

Steel bars are used in construction of concrete pavements in the following manner:

- steel deformed bars for CRCP and JRCP
- mesh for JRCP
- steel deformed bars for tie bars in tied longitudinal joints
- plain round bars for dowels

- steel bars for slab anchors.

In JRCP and CRCP the reinforcement is provided to manage the shrinkage of concrete and thereby limit crack widths to allow shear transfer by aggregate interlock.

In Australia the location of the base reinforcement has generally been in the upper half of the base. The Australian practice is to allow 70 mm cover for most highway type pavements. In thinner concrete pavements, the cover may be less provided sufficient cover is given to maintain durability of the concrete. In NSW, the Roads and Traffic Authority (RTA) recommends the position of longitudinal reinforcement above mid-depth to minimise crack width and maximise compaction of concrete below the reinforcement level.

Steel reinforcement cannot be supplied in infinite lengths and needs to be lapped or spliced to ensure adequate load transfer.

For welded wire fabric, continuity is achieved by ensuring a two-wire overlap for adjoining sheets. Although most fabric currently produced is made with deformed wire, no anchorage rules that take advantage of the deformations are available. Where slab lengths are less than 12 m, it may be possible to have the reinforcement supplier manufacture the fabric to the exact length (minus allowance for cover). This offers substantial opportunity to save on waste from lapping of fabric. Where individual reinforcing bars are used, lap reinforcements to the requirements of AS 3600 should be followed. It is also good practice to stagger laps to avoid a plane of weakness.

In Australia, 16 mm diameter bars are generally selected as the longitudinal bar and the spacing is varied according to the base thickness, concrete properties and other factors as outlined in the Austroads design procedure. Size 16 bars are typically lapped at 450 mm and if a skewed arrangement is adopted, the step is half the lap length (i.e. 225 mm). Another method of lapping permitted by the RTA is to lap every third bar and have alternate staggered laps at 1500 mm. It is important that laps are displaced (i.e. typically 2 m) from high stress zones, such as transverse construction joints or terminal anchors, to ensure the reinforcement is effective.

**Figure 10.6: Erection of CRCP reinforcement**



Source: RTA

For reinforcement to be effective in controlling shrinkage cracking, it needs to be maintained in its correct location during placement of concrete and while the concrete hardens. The most effective means of achieving this is to use bar chairs either plastic or steel to support the reinforcement. For further information on use of bar chairs Smorgan ARC (1989) is a useful reference.

## 10.5 Placement of Concrete

### 10.5.1 Placement by Slipform Paver

Concrete for pavements is typically placed using a slipform paver (Figure 10.7 and Figure 10.8). Homogeneity is critical for concrete pavements and can only be achieved by a homogenous product in front of the paver and a uniform process at the paver.

The overall features of pavers are:

- powered by diesel engine
- adjustable tractor frames which may be hydraulic telescopic members and/or bolted, rigid structural extensions for various pavement width configurations
- zero or minimum clearance configuration for paving in tunnels or against retaining walls
- spreader augers with variable controls for speed and direction
- metering strike-off (operated control) for material flow through
- immersion-type 'L-shaped' hydraulic vibrators, variable in frequency and hydraulically adjustable for depth
- tamper bar, speed controlled for stroke frequency
- profile pan, allowing for various profiles and width configurations
- side forms, providing longitudinal joint profile for load transfer
- 'over build' adjustment system to limit edge-slump
- rigid float pan for secondary surface control.

**Figure 10.7: Components of a slipform paver**

*Source: CPEE (2006)*

The maximum paving depth for most slipform pavers is about 450 mm and in practice, generally limited by the weight and power of the machine and the availability of the concrete to the paver to ensure a uniform paving speed.

The size of the paver is important in preparing the earthworks, planning the construction of ramps on freeways and proximity limits to bridge abutments. The distance from the inside face of the track to the edge of the sub-base and base should also be taken into consideration. The size of a slipform paver varies according to the manufacturer and model number.

Figure 10.8: Slipform paver



Source: RTA

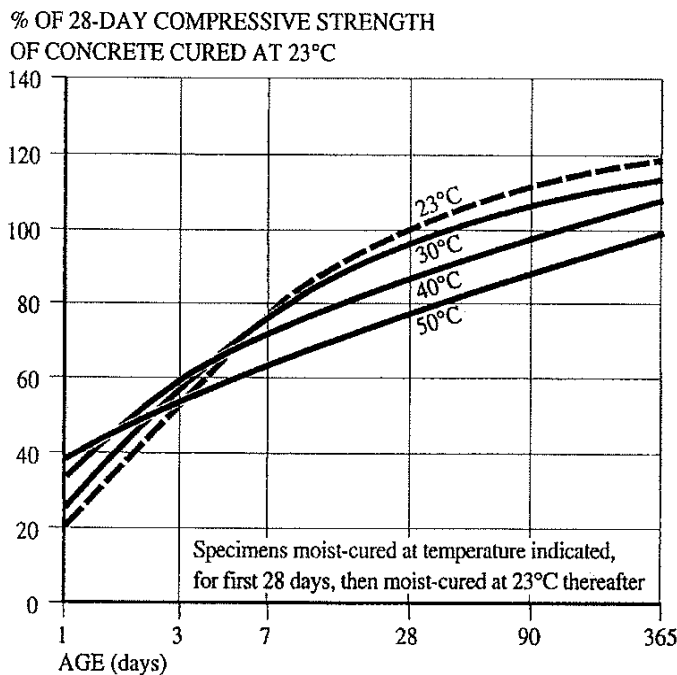
An important challenge for slipform paving is coping with changes in the pavement surface profile during slipforming. The pavement may change from single to dual crossfall, and the profile pan needs to cope with this situation without compromising surface smoothness and productivity.

Two-track and four-track pavers are generally used for single and multilane paving respectively. Two-track pavers require more track-line and clearance from verticals and are more sensitive to changes in track-line and concrete uniformity owing to their two-point stance. Four-track machines have adjustable legs permitting greater flexibility and have greater stability (and less sensitivity) due to the longer track base and four-point stance.

### 10.5.2 Consideration of Climatic Conditions

Concrete, by the nature of its constituents, should only be placed when the air temperature ranges from 5 to 35 °C. Figure 10.9 shows how 28-day compressive strength declines as the concrete temperature at placement increases from 23 to 50 °C.

Figure 10.9: Effect of temperature on concrete compressive strength development with age



Source: CPEE (2006)

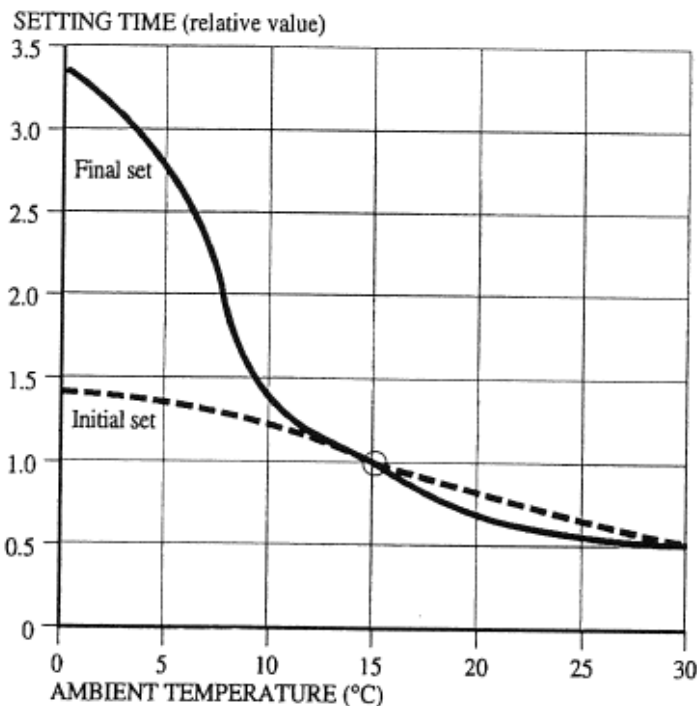
Road construction specifications set limits on the placing of concrete in hot and cold weather to minimise the effect of hot or cold temperatures on the development of strength, unplanned cracking and durability of the concrete. The following sections detail the principles for cold weather and hot weather concreting.

### Concreting in cold weather

The main characteristics of placing concrete at low temperature are:

- a decrease in the rate at which the concrete sets and gains strength, with a resultant increase in the time taken to finish the concrete (Figure 10.10)
- at temperatures below freezing, there is physical damage to the concrete in the form of surface scaling or bursting, and the cessation of hydration.

Figure 10.10: Effect of temperature on concrete setting time



Source: APMCA (1996)

Cold weather concreting requires special construction operations, and practical considerations during cold weather concreting include:

- the need to finish the surface after concrete placement (i.e. texture)
- allowance for curing
- protection of base concrete, ensuring the concrete temperature does not fall below 5 °C during the first 24 hours after placement or as required by the specification, to minimise the effect of surface damage.

Blankets, usually consisting of tarpaulins or recycled carpet, are sometimes used to protect concrete pavements from adverse conditions. However, with the high output of concrete paving during a full day's production, it is often not practical for contractors to cover the paved area with blankets. In addition, cold weather is often accompanied by wind, making it difficult to fasten the covers to the pavement and allow the saw cutting operations to proceed efficiently.



Lean-mix concrete sub-bases are especially prone to cold weather distress if placed on cold ground with low cement content and hence, little potential for hydration to commence. If cold weather concreting is anticipated for LMC sub-base, the trial pavement should assess any potential problems which may prevent the concrete from developing sufficient hydration; otherwise strength gain may not be possible.

### ***Concreting in hot weather***

The effects of placing concrete during high temperatures are more prevalent in the Australian climate and include:

- shorter setting times and early stiffening that may present problems with texturing and sawcutting
- increased rates of hardening that may lead to unplanned cracking
- possible strength loss (Figure 10.9)
- increased tendency for prehardening cracking (Figure 10.11)
- increased risk of early full depth cracking of CRCP due to non-uniform sub-base friction and low early-strength of the concrete
- danger of cold joints in hand placing
- increased risk of unplanned cracking on the paving operation.

Additional precautions should be taken when paving at ambient temperatures expected to exceed 30 to 35 °C, which may include:

- Cooling concrete ingredients, especially cooling the water.
- Restricting concrete placement to night-time when ambient temperatures are generally lower. However, this may not be feasible in urban areas due to noise constraints.
- Completing the transporting, placing and finishing of concrete as rapidly as practicable.
- Spraying a fine film of aliphatic alcohol over the exposed concrete surface immediately after the initial finishing operation to limit evaporation and help control plastic shrinkage cracking.
- Immediately spraying a curing compound onto the concrete surface after texturing is completed.

Hydration will take place rapidly during hot weather, which may reduce the workability of a concrete mix. Though addition of extra water to the concrete mix may improve workability this practice is generally not permitted unless consistency of the mix can be accurately controlled. Retempering of concrete supplied to the site by transit mixer is generally permitted under limited conditions as described in Section 10.3.3. Wetting the top of the tipped concrete should not be permitted as consistency of the mix cannot be controlled and may lead to patches of weak concrete.

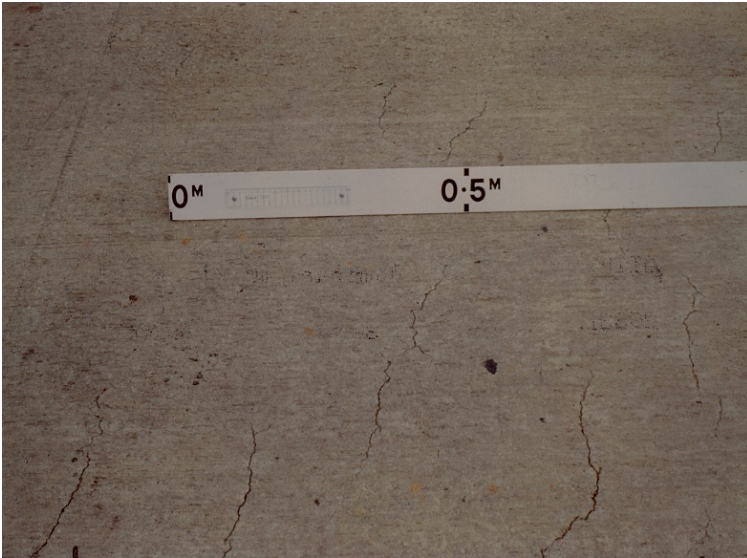
### ***Pre-hardening cracks***

Pre-hardening cracking in concrete may be the result of:

- plastic shrinkage cracking formed after the concrete starts to harden as a result of tension induced by evaporation
- plastic settlement cracks after the bleed water evaporates
- there is a loss of total volume causing the non-hardened concrete to settle
- formwork movement cracks due to faulty construction of fixed forms.

Plastic shrinkage cracks, which are the most common form of pre-hardening cracking, are usually 25 mm to 2 m in length without any definite surface pattern (Figure 10.11). These cracks are generally caused by a high rate of evaporation that exceeds the rate at which bleed water rises to the surface. The condition is more likely to occur during hot weather or mid-season on windy days as a result of poor protection of the pavement surface.

**Figure 10.11: Example of plastic shrinkage cracks**



Source: ARRB

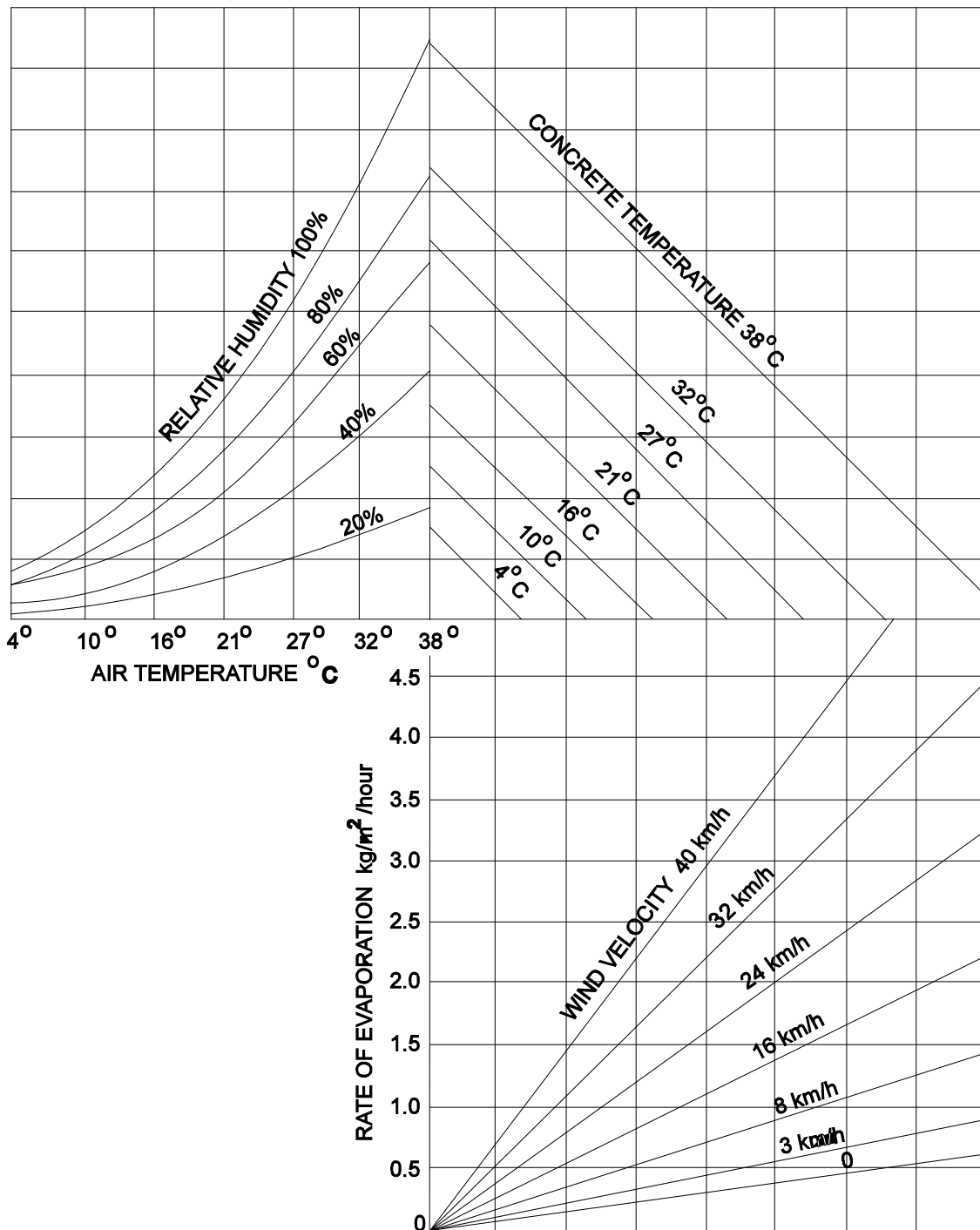
There is no limit placed on the moisture loss or evaporation rate in the current Australian state road authority specifications, although the American Concrete Institute (ACI 1992) suggests a figure of 1 kg/m<sup>2</sup>/hr. In Australia there seems to be an evaporation rate range of 0.5 to 1 kg/m<sup>2</sup>/h, where the contractor should be cautious. After 1 kg/m<sup>2</sup>/hr it is highly likely that cracking will occur.

An estimate of the evaporation rate that may cause plastic shrinkage cracking is shown in Figure 10.12. To use the chart, follow these steps:

- from air temperature (horizontal) mid-axis move up to the relative humidity line
- move to the right horizontally to the concrete temperature line
- move down to the wind velocity line
- move horizontally left to the rate of evaporation (vertical axis).

For example, if the air temperature is 27 °C, relative humidity at 40%, concrete temperature at 27 °C, and a wind velocity of 26 km/hr; the rate of evaporation would be 1.6 kg/m<sup>2</sup>/hour according to Figure 10.12.

Figure 10.12: Rate of evaporation of surface moisture from concrete



Source: RTA (2007b)

Some of the preventive measures to minimise pre-hardening cracking include (Ho & Lewis 1992):

- using cohesive concrete mixes
- using air-entrainment admixtures
- reducing evaporation by the use of sprayed aliphatic alcohol
- retrowelling plastic shrinkage cracks though cracks may not be closed throughout their depth.

## 10.6 Compaction of Concrete

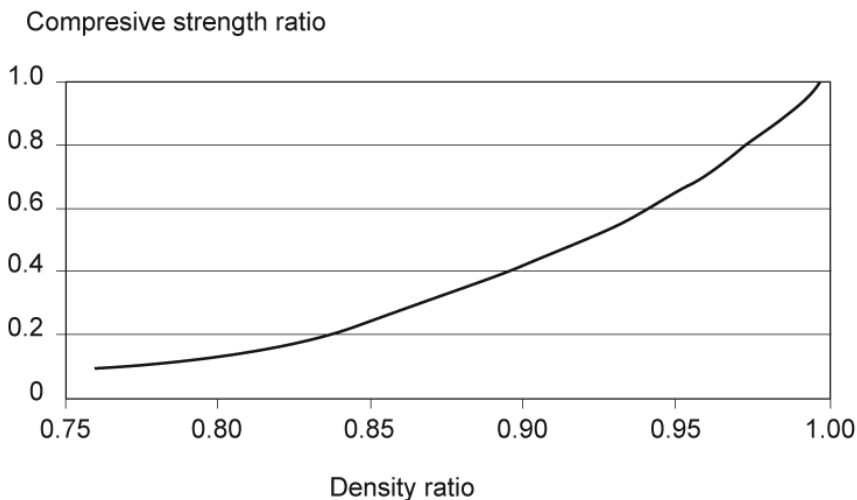
The purpose of compacting concrete is to increase the density of concrete by packing the aggregate particles together and minimising the entrapped air content, which is different from entrained air. Entrained air is air voids (typically) between 10 µm and 1 mm in diameter and spherical while entrapped air voids are 1 mm or more in size and irregular in shape (ACI 1984).

Increasing the density of concrete has the following benefits:

- improves strength
- improves pavement life
- minimises shrinkage and creep characteristics
- improves durability
- improves abrasion resistance
- enhances the bond with reinforcement.

One dramatic effect of reduced density is loss of concrete strength. Figure 10.13 shows the effect of a 5% loss in density, which reduces compressive strength by 37%.

**Figure 10.13: Compressive strength versus density or air voids content**



Source: Neville (1975)

Compacting concrete is a two-stage process in which the particles are first set in motion by the vibrator and liquefaction of the concrete occurs. In the second stage, the bubbles entrapped in the concrete rise to the surface, allowing the aggregate to pack tightly. The first stage of the compaction takes 3 to 5 seconds with the whole process taking just 10 to 20 seconds.

Good compaction is achieved by getting the optimum concrete mix, the best mechanical equipment and following correct procedures.

Methods of compacting concrete by slipform paver and by hand placement are different and the following sections provide guidance on how optimum compaction may be achieved.

### 10.6.1 Compaction by Hand

The two types of vibrators used to compact hand placed concrete are immersion (internal) vibrators and vibrating screeds. The immersion vibrator compacts at depth throughout the slab and the vibrating screed compacts the upper zone. Whilst vibrating screeds should always be used on concrete pavements, their maximum effective operating depth is only about 200 mm (or less when reinforcement is present in the slab) and therefore are typically used in tandem with immersion vibrators.

A shortcoming of the vibrating screed is its inability to effectively compact concrete at the edges as the formwork absorbs most of the compaction energy. Immersion vibrators should therefore be used along all edges of the formwork to ensure the concrete is thoroughly compacted. In addition, the vibrating screed is only effective if used for two passes and moves at a speed of 0.5 to 1.0 m per minute.

The characteristics of immersion vibrators have been well documented (ACI 1993) and the recommended operating values for concrete pavement type mixes are as follows:

- diameter of head – 50 to 90 mm
- recommend frequency in concrete – 130 to 200 Hz
- average amplitude 0.6 to 1.3 mm
- radius of action (e) – 180 to 360 mm
- rate of concrete placement 4.6 to 15 m<sup>3</sup>/hr per vibrator.

The distance between immersions should be between  $e\sqrt{2}$  and  $e\sqrt{3}$  for square and offset immersion patterns respectively (ACI 1993). On average, the spacing for immersion would therefore be 300 to 450 mm. For a 4.2 m wide and 235 mm thick pavement, at best one vibrator could only compact a 15 m length of the pavement in one hour. To improve productivity, at least two or three vibrators are consequently required to successfully compact concrete pavements. It is good practice, and a requirement of some concrete paving specifications, to have at least one spare vibrator on site at all times.

### 10.6.2 Compaction by Slipform Paver

For compaction by slipform paver the following procedures are recommended:

- the preferred lateral spacing (i.e. parallel to road alignment) of vibrators is 400 mm and not greater than 450 mm
- the tail end of the vibrator should be tilted down 7° to 10°
- the tail end of the vibrators should be as near as possible to the mid-depth of the PCC or just above the reinforcement
- the vibrator frequency is in the range 5,000 to 8,000 rpm.

#### ***Internal vibrations using gang-mounted vibrators***

The centrifugal force and vibrator spacing should be based upon the aggregate to be used, mixture characteristics, rate of concrete delivery, method of reinforcement placement, and paver speed. The location of the outside (edge) vibrators is critical, especially in slipform paving and separate edge compactors may be required.

When non-uniformity or mortar streaking occurs in vibrator paths while operating at normal paving speeds the vibrators should be lowered in the concrete, their angularity changed, the frequency increased or decreased, the amplitude changed (usually by changing the eccentric weight), or additional vibrators added until the streaking is eliminated. Proper consolidation is generally achieved when the concrete surface has a uniform texture and sheen, with coarse aggregate particles barely visible on or immediately below the surface.

For pavements less than 250 mm thick, vibrators should be operated parallel with, or at a slight angle to, the sub-base. For thicker non-reinforced pavements, vibrators should be angled toward the vertical, with the vibrator tip preferably about 50 mm from the sub-base, and the top of the vibrator several centimetres below the pavement surface.

A 100 to 150 mm surcharge of concrete should be carried over the vibrators during the placing operation. Greater surcharge loads are likely to cause surging behind the screed or extrusion plate and prevent full release of entrapped air.

For a reinforced pavement with a thickness less than 250 mm vibrators should be parallel with the sub-base above, and as near as practical, to the reinforcement but at least two vibrator diameters below the surface. When the reinforcement is close to the surface, the concrete should be placed in multiple passes to permit consolidation. If inadequate consolidation is discovered at the bottom of the slab under the steel, space the vibrators closer together, increase the vibratory effort, or decrease the paver speed. Since it is common practice to attach the vibratory unit to the equipment carrying the first transverse screed, the proper adjustment of the vibrators will depend on the forward speed of this equipment.

Reinforced slabs, in which the reinforcement is placed by vibration after full-depth concrete placement, require initial consolidation prior to steel placement. In continuously reinforced pavements, where the steel is placed on chairs prior to concrete placement, care should be taken to ensure that the concrete below the steel is receiving adequate consolidation. For reinforcement placed with a mesh depressor, less vibration will normally be required than for mesh placed on chairs or for concrete placed in two courses. For reinforced slabs placed in two courses, the vibrators should be used in both courses.

### ***Surface vibration***

The vibratory-pan unit should be positioned behind the surface strike-off equipment. The vibration frequency should be set in accordance with the forward speed of the equipment on which it is mounted. A surcharge should not be allowed to build up in front of the pan because it will dampen the vibrations. An internal vibrator may assist in consolidating concrete along each form.

It is usually advisable to make two passes of the screed or roller. The first strikes off and consolidates the concrete, and the second provides the surface finish. Maximum frequency should be used on the first pass and a reduced frequency on the second. In this case, surface appearance is not a satisfactory criterion of the adequacy of consolidation. An understanding of the effectiveness of consolidation below the surface is required.

### ***Transition zones***

It should be noted that there are a number of areas where typically vibrators attached to slipform pavers cannot adequately, and consistently, compact concrete. These include transition zones, where a concrete paver starts and finishes a run and either side of some joints. In these locations manual compaction, as described for hand placed concrete, is required.

Ayton (2001) discussed in more detail the requirements and specification of concrete compaction.

## **10.7 Finishing and Curing**

Unlike flexible pavements where a wearing course is constructed above the base layer, the top of the concrete base is generally the wearing surface.

As concrete changes from its plastic to hardened state the finishing and curing of the concrete is vital to ensure the required long-term properties of the concrete are achieved.

Finishing concrete, in the context of construction, refers to the production of the surface texture. The functions of the surface of the base and sub-base concrete are different and procedures have been developed to meet their specific design requirements.

As concrete hardens, good curing procedures minimise the effects of environmental loading onto the early-age concrete. Poor or no curing will cause surface defects and strength deficiency.

### 10.7.1 Finishing

Finishing concrete involves the production of the surface texture and differs for the subbase and base layers.

#### **Sub-base**

Debonding of the base from the sub-base is critically important. Concrete sub-bases should have a uniform and formed smooth finish and generally no texture requirements are specified on the surface of the sub-base.

Interlayer bonding impacts on the following key aspects of pavement design (Aytton & Haber 1997):

- the amount of longitudinal reinforcement in CRCP
- crack patterns in CRCP
- the amount of tie bars in longitudinal tied joints
- the risk of reflective cracking
- the magnitude of concrete stresses induced by contraction strains.

A debonding treatment is therefore required to ensure that the base concrete does not bond to the top of the sub-base during placement.

#### **Base**

Surface texture of a base layer provides the following functions:

- a smooth and uninterrupted surface for safe travel
- sufficient skid resistance
- minimisation of road/tyre interaction noise
- sufficient surface slope to remove surface water and minimise aquaplaning.

In Australia concrete pavements for major roads are constructed with transverse and longitudinal texture. Transverse texture is generally not applied if the pavement has an asphalt surface layer or the vehicle speeds are below 70 km/h.

Table 10.2 notes the specified average texture depth for RTA NSW Specifications R83 (RTA 2007b) and R84 (RTA 2007c). The data in this table indicates that the texture depth specified for the transverse direction is greater than for the longitudinal direction.

**Table 10.2: Typical texture depth requirements**

Direction	Texture depth (mm) RTA T192 or T240
Longitudinal	0.40 or 0.55 + 0.05
Transverse	0.45 or 0.65 + 0.05

Source: RTA (2007b) and RTA (2007c)

Areas of the surface with less than specified transverse texture may need to be saw-grooved or diamond ground to improve the texture depth.

Longitudinal texture is applied by means of dragging hessian behind the slipform paver (Figure 10.14) or by hand operation using a hessian cloth attached to an aluminium pipe. Various forms of longitudinal texture have been tested including artificial turf and the results are noted in Table 10.3 (Nichols & Dash 1993).

**Table 10.3: Longitudinal texture drag characteristics**

Type	Characteristics of noise and friction
Light hessian	Noisier than combination with light transverse tyning. Inadequate surface friction is used without tyning.
Coarse hessian	Noisier than combination of light hessian and light transverse tyning. Adequate surface friction when used without tyning.
Light artificial turf	Noisier than combination of light hessian and light transverse tyning. Adequate surface friction when used without tyning. Uniform appearance,
Coarse artificial turf	Same as for light artificial turf except noisier and possibility of tram tracking for motor cycles.

Source: Nichols & Dash (1993)

**Figure 10.14: Damp hessian being dragged behind a slipform paver**



Source: RTA (2000)

Transverse tyning (Figure 10.15) is produced by rectangular shaped steel tynes approximately 0.6 mm thick and 3 mm wide. The current approach to tyning is to create a random pattern such that a harmonic vibration is unlikely to occur in the tyre wall resulting in a whine from the road/tyre surface. The spacing of the tynes is in the range of 10 to 21 mm and is in a random order.



Transverse tining is usually carried out soon after the bleed water has left the surface of the concrete. If the variation on the mix is such that there are pockets of bleed water on the surface, the tining effect will appear inconsistent.

**Figure 10.15: Tyned concrete surface**



Source: RTA (2000) and RTA

Additional guidance on concrete finishing is provided in (Austroads 2009a).

### 10.7.2 Curing

For most modern concrete mixes used in paving there is generally sufficient post-bleed water in the mix for full hydration of the concrete. It is necessary, however, to ensure that this moisture is either retained or supplemented to ensure the cement hydration continues for the concrete to reach its intended strength.

Curing is a procedure that controls moisture loss from concrete after it has been placed. For concrete roads the curing operation should occur immediately after finishing and should include all exposed concrete surfaces. The use of plastic sheeting or fine water spray is generally impractical on road sites and hence, a sprayed on curing membrane is most commonly used. The most common curing method used for paving operations is to seal the moisture in the concrete by coating the concrete with membrane curing compound such as wax emulsion or hydrocarbon resins.

If asphalt surfacing is proposed, the use of a bitumen emulsion curing compound is recommended to minimise debonding between the concrete and asphalt.

Pigmented curing compounds provide a simple visual assessment that the membrane is applied uniformly over the surface. White pigmented wax emulsions are particularly useful because their light reflectance reduces surface temperature variations in the concrete.

With emerging emphasis on both environmental and occupational health and safety issues the use of chlorinated rubber membranes on concrete road pavement sites has been eliminated in Australia.

Ayton and Haber (1997) provide information on the relative performance of curing materials under Australian conditions to prevent moisture loss from concrete.

### ***Application rate***

The application rates of curing compounds are tied to the water retention efficiency index test in AS 3799 and are generally in the range of 0.2 to 0.4 litres/m<sup>2</sup>. There are also separate specification requirements for curing compound application rates for lean mix concrete subbase and concrete base construction.

In subbase construction, there is minimal surface texture, since a smooth surface is sought and the application is therefore a single treatment of about 0.2 litres/m<sup>2</sup>. For base concrete there are generally two applications, separated in time by 15 to 30 minutes, each of 0.2 litres/m<sup>2</sup>, which assists in coating the sides of the tined concrete surface.

The average application rate is assessed by placing three felt mats, each about 0.25 m<sup>2</sup> in area, randomly on the surface to be cured and weighing the mats following application of the curing compound.

For multiple-lane paving (i.e. greater than 4.5 m wide), the curing membrane is usually sprayed from a full width, multiple nozzle, powered spray bar mounted on a self-propelled frame. For single-lane paving (i.e. 2.5 to 4.5 m wide), a spray bar fitted with multiple nozzles is required. For very small areas (i.e. widths less than 2.5 m) hand held single nozzle powered spray devices are commonly used and the target application rate is typically increased by 25%.

### ***Timing of application***

The optimum time for commencement of the curing process is a balance between the evaporation of any bleed water and not damaging the surface of the concrete. With sub-base concrete there is usually more bleeding due to the nature of this material as well as a longer period to initial set and subsequently a longer delay period. Water-based emulsion curing compounds cannot be applied until all the bleed water has evaporated otherwise the emulsion is immediately destroyed. However, performance is often improved for solvent-based compounds by early application.

As base concrete will have a textured surface of some type, the application of the curing membrane should follow as soon as practicable after the texturing is completed. Depending on the concrete and site conditions the timing of the application could be from 20 to 60 minutes after texturing. In the case of the self-propelled frames, both texturing and curing equipment may be mounted on the frame.

### ***Duration of curing***

Various research papers recommend a curing duration in the range of 3-10 days. Given the rate of strength gain of contemporary base concrete and the limited strength requirements of lean mix concrete sub-base most specifications require a minimum seven-day curing period.

More information on Australian specifications and practices for curing and interlayer debonding is provided in Ayton & Haber (1997).

### ***Curing compounds and linemarking***

In the mid-1990s pavement line marking on some projects had a limited life and investigations showed that poor long-term adhesion of the line markings was due to the curing compounds used. As such most specification now require verification that the curing compound is suited to the application of linemarkings.

### ***Early trafficking of pavement***

During construction the base concrete often needs to be trafficked by various vehicles to complete joints, linemarkings and prepare the verge adjacent to the shoulders.

It is common for specifications to allow trafficking of the base by the following types of vehicles with the following restrictions:

- concrete saws and coring machines subject to a 0.5 t limit on any item
- construction vehicles once 20 MPa compressive strength is reached and all joints have been permanently sealed
- steel implements such as grader blades and loader buckets should not be allowed to impact joints or edges of the base
- compaction of granular verge material against the edge of base should only be allowed once compressive strength has reached 20 MPa and all joints have been permanently sealed, including the vertical faces.

Construction vehicles carrying fresh concrete or soil to various locations on the site commonly may also use the finished base as a means of access subject to the restrictions above; however, when tracked vehicles are used the concrete must be protected from surface damage.

While the concrete is developing its strength, the following axle limits are recommended:

- axle loads – 5.0 t single axle, 8.0 t tandem axle, 9.0 t triaxle
- tracked vehicles – 15 t/m<sup>2</sup> pressure over the track area.

Early trafficking requirements for sub-base construction are different since the concrete strength in the subbase will not have achieved 20 MPa. The subbase should generally not be trafficked until the in situ compressive strength has reached at least 4 MPa.

To minimise the risk of cracking of the thin sub-base layer, only vehicles with a gross mass less than 1.5 t and construction equipment used to apply the surface debonding treatment and base paving, should be permitted on the sub-base concrete.

Tipper trucks needing access to the sub-base in front of the paver to deliver concrete are typically limited to a distance of up to 300 m immediately ahead of the paver. Fully laden tipper trucks can also damage the debonding treatment and this is more evident with spray seal treatments on hot days.

Most specifications do not allow traffic onto the pavement until the concrete compressive strength reaches a given strength, which may be lower than the design strength. For example trafficking may be permitted once a concrete pavement reaches 20 MPa even though the design strength is 32 MPa. However, as the forces imposed by trucks are very high, in terms of the flexural stress in the base concrete, it is important to minimise any reduction in fatigue life of the concrete caused by trafficking when the early strength is below its design target. Trafficking of the base should also be controlled to minimise damage to the curing compound.

## 10.8 Joints

Joints in concrete pavements provide an essential method of managing the expansion and contraction of the pavement, subject to daily temperature cycles and the shrinking and drying of concrete. In concrete pavements, joints:

- control transverse and longitudinal cracking from restrained contraction and the combined effects of traffic loading, curling and warping
- divide the pavement into practical construction increments, such as traffic lanes
- allow the concrete to expand and contract with seasonal temperature variations
- provide load transfer between adjacent slabs.

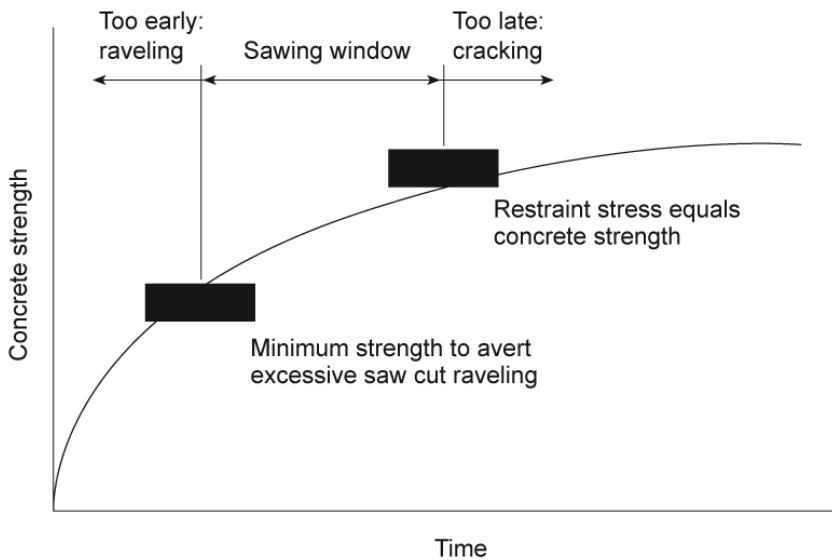
### 10.8.1 Sawn Joints

Sawn joints, also known as induced joints, are made by diamond-disc cutting of the upper portion of the base concrete, in order to create a potential weak plane such that a full-depth crack will be induced during the concrete shrinkage process.

Although the distance between saw cuts is dependent on a number of variables, 4 to 5 m is common. Experience has demonstrated that longer spacing between sawn joints increases the potential of an unplanned or uncontrolled crack due to shrinkage.

The optimum time to saw contraction joints in new concrete pavements is defined as the sawing window and is depicted in Figure 10.16. This window is a short period after placement, during which the concrete can be cut successfully before it cracks. It begins when the concrete strength is acceptable for joint sawing without excessive raveling along the cut (i.e. chipping of one or both edges of the sawn joint) and ends when concrete shrinkage produces a restraining force greater than the concrete strength and induces uncontrolled cracking.

**Figure 10.16: Sawing Window**



Source: Voight (1997)

Under normal weather conditions and for typical pavement designs, the window should be long enough to complete sawing. However, certain design features and weather conditions can shorten the window considerably, as noted in Table 10.4.

**Table 10.4: Typical factors the shorten the sawing window**

Factor	Effect on sawing window
<b>Weather</b>	
Sudden temperature drop or rain shower	End of window
Sudden temperature rise	End of window
High winds and low humidity	Start of window
Cool temperatures & cloudy	End of window
Hot temperatures & sunny	Start and end of window
<b>Subbase</b>	
High friction between underlying subbase and concrete slab	End of window
Bond between underlying subbase and concrete slab	End of window
Dry top surface	Start of window
Porous aggregate subbase materials	Start of window
<b>Concrete mixture</b>	
Rapid early strength	Start and end of window
Slow early strength	Start and end of window
Retarded set	Start of window
Coarse aggregate	Start and end of window
<b>Miscellaneous</b>	
Paving against or between existing lanes	End of window
Saw blade selection and technique	Start of window
Delay in curing protection	Start of window

Source: Voight (1997)

### **Conventional sawing**

The standard method of inducing a planned crack in pavements is by power sawing. These saws are typically small, single-blade units with solid wheels. However, some sawing contractors use multiple in-line saws with blades traversing under a truss frame suspended over the pavement. These frames are often shrouded to minimise noise emission when used in urban environments.

Current practice is for both the transverse and longitudinal sawing to be done at the same time. As expected the process is noisy, and saw cutting close to residential housing requires diplomacy to ensure that nuisance noise is minimised.

The selection of saw blade, in terms of diamond fineness and density and the bonding of the diamond to the blade varies depending on the aggregate type used in the concrete. A relatively finer cutting medium on the blade is used for river gravels, which are more prone to spalling during sawing due to inherent hardness and shape (Hodgkinson 1994).

For a single-lane width of paving to cut and set up again at the next transverse joint using a single blade saw takes 3 to 4 minutes. At 4.2 m joint spacing it would take more than 6 hours, excluding down time, to complete cutting 400 m of single-lane paving. Note, it is not suggested by these calculations that one saw is sufficient, as in warm weather and with the paver moving at 1 m per minute, it would be more than necessary to have two saws, and preferably one breakdown spare, to complete sawing and prevent early-age cracking of the concrete. If sawcutting work falls behind the strength development of the concrete, sawing every second transverse joint then returning and sawing all the intermediate transverse joints may minimise potential unplanned cracking.

Immediately after sawing the groove needs to be washed clean of debris, which is usually done with compressed air so as to not harm either the concrete or the curing membrane. A temporary seal, in the form of a spline rubber strip, is then typically firmly engaged at the base of the cut to inhibit the intrusion of compressible fines before the main joint sealing can be installed.

In order to limit spalling and achieve a suitable sealant reservoir groove width, a double-cut sawing method is now standard practice within most concrete pavement specifications. The initial cut is typically made with a standard nominal 3 mm thick blade allowing the second widening cut, using two blades, to be done later at a more convenient time.

The second widening cut will generally remove most if not all the spalling from the initial cut. Typical joint detail drawings show the second widening cut at a depth and to one side of the initial cut to form the sealant reservoir. Under field conditions and once the widening blades are engaged in the initial cut, the reservoir may be to one side or centrally over the initial cut. This latter occurrence does not affect sealant or pavement performance.

On steep gradients and when combined with superelevation, the weight of the saw tends to cause it to drift slightly down the gradient. Under these conditions, very slight curvature may occur in the saw path, as once the blade is fully engaged in the concrete it is not practicable to try to straighten it up. Provided the deviations are not significant it should have little effect on the pavement surface.

### ***Early age sawing***

Early age saw cutting, undertaken as soon as the concrete can take foot traffic without marking, involves use of purpose-built early age sawing equipment and may reduce uncontrolled cracking of concrete roads in some instances. The timing for early age sawing usually coincides with completion of finishing. Early age saws range in size from approximately 10 kg to 150 kg and often use 65 mm wide wheels made of a relatively soft plastic material, which can be taken on concrete pavements significantly earlier than conventional saws. In addition, many early age saws have what is referred to as a skid plate, which effectively supports the concrete on either side of the saw blade thereby preventing plucking of aggregate and ravelling of the joint edges.

Research has shown that early age sawing with sawing depths of less than  $0.25 d$  ( $d$  = base thickness) should provide better crack control than standard methods with depths of  $0.25 d$  or  $0.33 d$  (Zollinger 1994).

Practical experience with early age sawing equipment has led manufacturers and road authorities to recommend a minimum sawcut depth, generally one tenth of the pavement thickness, with the qualification that the depth of cut should not be less than the maximum size of aggregate used in the concrete mix.

Saws are typically used to cut a joint approximately 3 mm wide and thereby maximise the control of early cracking from changes in pavement temperature. If the sawn joint is to be sealed it will generally require widening (and cleaning) using a conventional saw at a later stage.

### 10.8.2 Crack Inducers

Transverse and longitudinal joints can be effectively induced by various forms of top and bottom located inserts, usually made from different grades of plastic. In slipforming operations in the 1980s the use of ribbon crack inducers was common for longitudinal joints. However, they tended to wander around and not induce a crack in the desired location and are typically no longer permitted.

In general, top crack inducers are only suited to low speed roads such as ramps, roundabouts and residential streets or where the concrete base is to be surfaced with asphalt. Top crack inducers will disturb the surface of the fresh concrete due to the method of installation. This results in considerable hand finishing, which introduces pavement roughness. With slipform concrete mixes using a typical slump in the range of 30 to 50 mm, the surface disturbance and effort required in finishing the surface are considerable and therefore unlikely to meet current specifications.

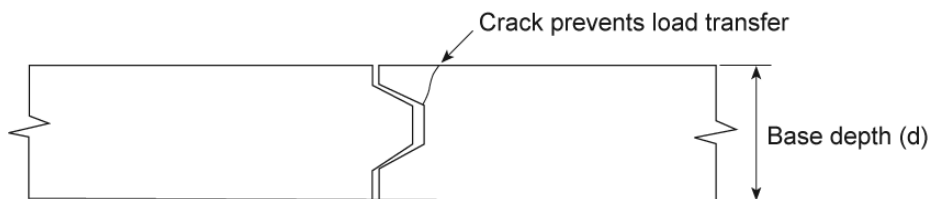
Joint induction is also possible via bottom crack inducers attached to the sub-base. However, little is known of their use and success in major road construction. With access to the sub-base pavement important for the delivery of concrete to the paver, performance of these types of crack inducers, in terms of their ability to withstand construction wheel loading, must be assessed prior to use.

#### Formed joints

Formed joints are either transverse construction joints or longitudinal joints. For longitudinal formed joints that require load transfer, a corrugation is preferred to a keyed joint. Keyed joints are known to fail, as shown in Figure 10.17, due to heavy axle loading. Once the key action in the joint is inoperative, there is no load transfer resulting in edge loading conditions and therefore, incorrect base thickness. Also, repairs to keyed joints are extremely difficult to achieve in the field and the female half of the key is difficult to form using slipform pavers.

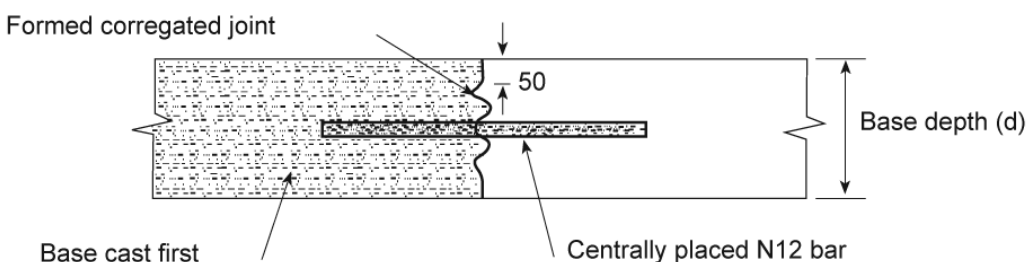
In longitudinal tied joints, the edge of the paved area is debonded before the concrete is placed adjacent to the joint to allow the joint to rotate. The debonded vertical surface allows the tie bar, shown in Figure 10.18, to work in the joint, rather than create the potential for a crack to form at the ends of the tie bars.

Figure 10.17: Failure of keyed joint



Source: CPEE (2006)

Figure 10.18: Tied transverse construction joint



Source: CPEE (2006)

Isolation joints are formed joints that are debonded but not tied to allow movement of the concrete. These joints are formed with vertical smooth faces free of honeycomb or gaps in the surface.

### 10.8.3 Dowel and Tie Bars

Dowel and tie bars are used in joints for specific purposes and are generally not interchangeable. It is common for dowel and tie bars to be confused; however, the simplest way to remember is:

- dowels transfer load and allow horizontal opening/closing in the joint
- tie bars tie together two adjacent slabs.

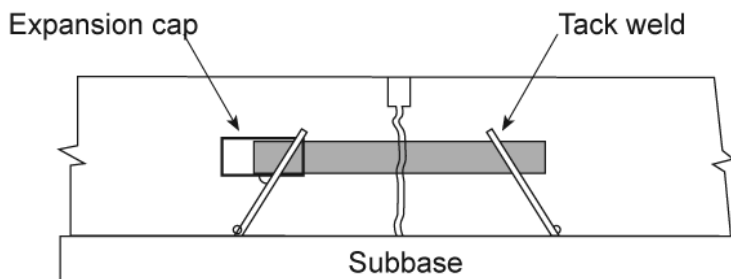
Tie bars are commonly used in longitudinal joints as shown in Figure 10.19 and dowels are used in transverse joints as shown in Figure 10.20.

**Figure 10.19:** Tie bars protruding from concrete slab at longitudinal joint



Source: RTA

**Figure 10.20:** Dowel bars located in specially prepared cages



Source: CPEE (2006)



For CRCP, the shoulders are typically constructed in plain concrete and transverse joints are installed at about 2 m spacings. Since the main carriageway in CRCP will crack at variable spacing, it is important to position the tie bars in a cluster in the longitudinal joint protruding from the CRCP section. The cluster of tie bars is midway between joints in the shoulder to allow the longitudinal joint to move longitudinally between cracks and transverse joints, while keeping the shoulder tied to the main carriageway.

#### 10.8.4 Sealants

The role of sealants in joints is to prevent incompressible materials from entering the joint and prevent water infiltration that assists pumping in the subbase and formation. When incompressible materials enter a contraction or expansion joint, the joint may spall locally or a crack may appear orthogonal to the joint. Moreover, the presence of incompressible materials in the joint may prevent the joint functioning as designed and ultimately lead to joint blowout.

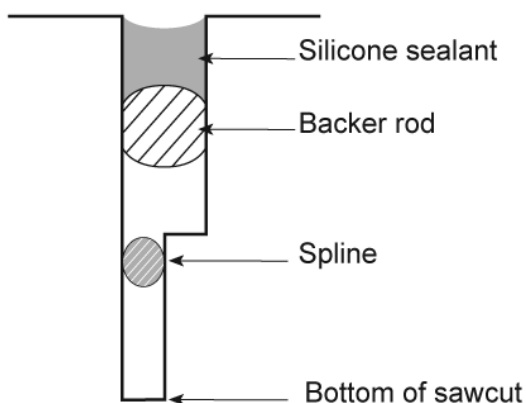
There are two types of sealants, namely:

- preformed sealants, such as pre-compressed neoprene elastomeric seals and self-expanding cork
- field moulded sealants that are poured or gunned into a sealant reservoir.

Preformed seals were used extensively for pavement construction until the 1990s. A structural grade silicone sealant (i.e. field moulded sealant) is now widely used for sealing joints. Hot poured sealants and other field sealants are not covered here due to their infrequent use in the construction of new concrete roads.

Silicone sealants require the installation of a backer rod for their successful operation as shown in Figure 10.21. The purpose of the backer rod is to define the shape of the sealant (including controlling the shape factor), optimise the quantity of the sealant, and prevent the sealant from adhering to the bottom of the joint.

**Figure 10.21: Joint sealant with backing rod**



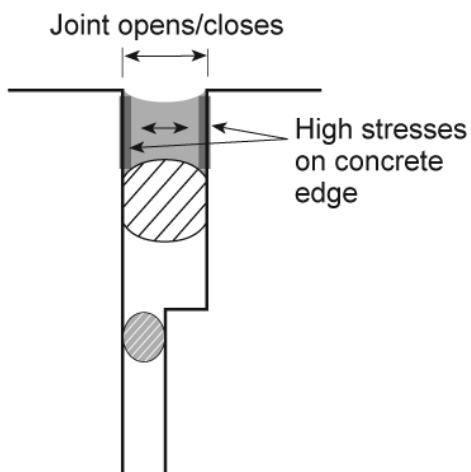
Source: RTA (2000)

In Australia, closed-cell polyethylene backer rods are used as they do not absorb water and being moderately compressible, assist in defining the shape for the sealant. The backer rod diameter is typically 25% greater than the width of the joint reservoir to ensure a tight fit and once the sealant has cured has no further role.

The sealant must also remain below the road surface to prevent damage by tyres. Neoprene seals used in the 1970s and 1980s are mostly intact today; however, where the sealant has risen above the road surface, the seal has been immediately damaged by traffic.

The shape factor, defined as the width divided by depth, is important for the long-term success of a sealant. The joint opens and closes with temperature changes in the base concrete and this movement places stress on the sealant and the interface between the edge of the joint and sealant. These stresses can be excessive if the shape factor is not appropriate and the sealant may tear leading to loss of function (Figure 10.22).

Figure 10.22: Joint sealant loads



Source: CPEE (2006)

Typical joint dimensions for field-moulded sealants are noted in Table 10.5. These values should be evaluated against those applying to the specific sealant proposed by the contractor, taking into account requirements stipulated by the sealant manufacturer.

Table 10.5: Joint dimensions of field moulded sealants

Mean slab length (m)	Sealant width (mm)	Sealant depth (mm)
4.2	8	15
5.0	10	15
7.5	15	15
10	15	15
12	20	20
15	20	20

Source: CPEE (2006)

It is essential that the vertical faces of transverse joints are sealed to prevent ingress of soil into the seal reservoir and the crack formed below a sawcut as this may result in distress of the concrete adjacent to the joint (Figure 10.23). The same extent of sealant should be applied when replacing neoprene seals in transverse joints with silicone sealants.

**Figure 10.23: Sealant extending down the side of the base concrete**



Source: RTA (2000)

At isolation and expansion joints, the sealant in the joint must be able to cope with large lateral changes in the joint opening. In these joints, it is important to ensure the sealant is not deformed during joint closing to prevent it appearing above the joint and exposing it to wheels. Because these joints are designed to take large movements, joint filler consisting of a backer rod with sealant, or alternatively, a filler-sealant, such as self-expanding cork may be used.

## 10.9 Conformance and Quality Testing

### 10.9.1 General

Quality assurance associated with concrete pavement construction applies well before the commencement of paving and continues through the various stages to beyond the completion of the work. Simple methods such as visual inspection are often both efficient and effective without the need for sophisticated automated techniques. However, a combination of both specialised techniques and simple procedures usually provides the best overall quality control.

The key stages of concrete pavement construction requiring quality control are:

- assessment of materials and concrete mix
- trial concrete mix
- evaluation by trial pavement
- quality control at batching plant
- transport and discharge of concrete
- supervision during paving
- post-setting inspection and checking (within 4 to 12 hours)

- post-hardening inspection and checking.

### 10.9.2 Material Supply

The *Guide to Pavement Technology – Part 4C: Materials for Concrete Road Pavements* (Austroads 2009b) contains advice on the selection, testing, quality control and specification of materials for use in concrete road pavement construction.

### 10.9.3 Trial Pavement

It is common practice for a trial concrete pavement to be constructed prior to construction of any permanent section of the works. Generally, a trial section of between 50 and 100 m in length must be constructed in one continuous operation. Separate trials are typically required for each paver to be used on a project. For manual paving, a trial section of between 15 and 50 m in length is typical.

Trial pavements should be assessed for conformity using the same requirements as for the main concrete paving (i.e. slump, compaction, strength, thickness, levels, shape, cracking, etc.). However, the frequency of testing may be greater than otherwise specified.

If significant changes are made in the equipment, materials, plant or rate of paving or if placed concrete pavement fails substantially to comply with the specification further trial pavements may be required.

### 10.9.4 Location of Steel Reinforcement

Concrete paving specifications typically specify a number of requirements for the placement of steel reinforcement, including:

- location of reinforcement (for example for plain concrete pavements steel reinforcement should be within  $80 \pm 15$  mm of the finished surface of the base slab and clear of all joints and edges by  $80 \pm 20$  mm)
- length, location and alignment (vertical and horizontal) of tiebars
- length, location and alignment (vertical and horizontal) of dowels
- splice length for lapped bars

Additional information on the typical requirements for steel reinforcement is provided in RTA (1991, 2007b and 2007c).

### 10.9.5 Thickness and Level Tolerances

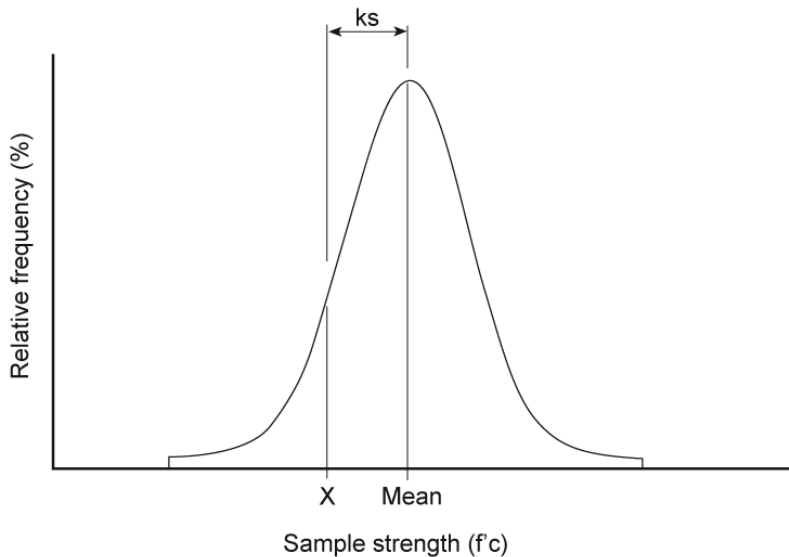
Example thickness and level tolerance requirements for concrete road pavements are (RTA 2007b):

- The level at any point on the top of the base must not vary by more than 20 mm above or 0 mm below the design level (+20, -0 mm).
- The mean thickness of a concrete pavement, as measured by survey and/or coring, should be greater than the specified thickness and at any point should be within 5 to 10 mm of the specified thickness. It should be noted that in RTA Specification R83 (RTA 2007b) a statistical assessment of thickness results is applied to determine whether a lot complies, is non-conforming with specified deductions or is non-conforming and rejected.
- Deviations under a 3 m straight edge laid in the transverse direction should not exceed 5 mm, except for areas within 10 mm of superelevation transitions where deviations should not exceed 3 mm.
- Deviations under a 3 m straight edge laid in the transverse direction should not exceed 5 mm.

### 10.9.6 Statistical Methods for Concrete Quality

The normal distribution curve is commonly used in sampling and is regarded as a satisfactory method to predict the relationship of production characteristics to the percentage of defects from a typical manufacturing facility. The distribution of concrete test results reasonably approximates a normal distribution curve (AS 3600) as shown in Figure 10.24. Most testing requirements for concrete involve the normal distribution curve (using the mean, standard deviation and area under the curve).

**Figure 10.24: Normal distribution of concrete strengths**



Source: CPEE (2006)

The measurement of characteristic strength of concrete ( $f'c$ ) is typical of how statistics and the normal distribution curve are used in concrete pavements, where  $f'c$  is defined as the strength that would be exceeded by 95% of the sample strengths (AS3600). For example, if 100 samples were taken from a batch, it would not be unusual to find five samples to have 28 day strength less than the specified  $f'c$ . For this situation to occur, the value of  $f'c$  would not be the mean value of the samples, but 1.65 times the standard deviation less than the mean. If  $f'c$  was 25 MPa and the standard deviation was 3 MPa, the mean strength from the test results would be  $25 + 3 \times 1.65 = 30$  MPa.

A value greater than 1.65 may be required if the manufacture of concrete is in small quantities or the level of testing is small (AS 1379). Obviously, the lowering of the standard deviation would reduce the mean strength.

For a more detailed introduction to the practical implementation for earthworks and pavement construction of basic concepts of quality control and compliance judgement refer to Gray and Robinson (1980). NAASRA (1989) defines the basic parameters and presents numerical data for implementation of various compliance judgement schemes.

### 10.9.7 Lot Definition for Concrete Paving

Typically, transition zones, where a concrete paver starts and finishes a run and either side of some joints, require significant hand compaction and finishing. Therefore, they are not homogeneous with the main paving run and should be designated as separate lots to the remainder of the paving run (RTA 2007b). For the main paving run, lots are typically defined as a continuous pour of 30 m<sup>3</sup> (hand-paved) and 50 m<sup>3</sup> (slipform paver placed) as opposed to a full shift's production.

### 10.9.8 Concrete Slump

Test samples for slump testing should be obtained in accordance with AS 1012.1 and should be tested in accordance with AS 1012.3.1. Slump testing should be conducted for every batch of concrete until a series of consecutive conforming loads are tested then the testing frequency may be reduced. For example RTA (2007b) specifies that every batch of concrete is slump tested until 8 consecutive conforming loads are tested then the testing frequency may be reduced depending on the standard deviation of the 8 conforming loads.

The slump should be within a specified limit from the nominated slump, typically  $\pm 10$  mm for slipformed concrete and  $\pm 15$  mm for manually placed concrete.

### 10.9.9 Compaction

Concrete compaction is determined from moulding and testing of cylinders, which are used to determine the unit mass reference values, and core specimens. Concrete core specimens are typically nominal diameter 75 or 100 mm, cut from the full depth of the concrete layer. AS 1012.12.2 and AS 1012.14 provide detailed information on the moulding and testing of cylinders and core specimens respectively.

A lot is deemed to conform for compaction if:

- relative compaction is at least 98.0%, determined as the percentage ratio of the core unit mass of the lot to the rolling cylinder unit mass (RCUM) for the lot
- the within-core variability does not exceed  $40 \text{ kg/m}^3$ , determined in accordance with the relevant specification.

For nonconforming lots:

- if the relative compaction is between 97.0% and 98.0%, the lot is typically accepted if the 28-day compressive strength of core specimens from that lot conforms
- if the relative compaction is less than 97.0%, the lot is typically required to be removed and replaced
- if the only nonconformity is the within-core variability, the lot is typically accepted subject to corrective action being taken in the compaction process.

### 10.9.10 Compressive Strength

Concrete compressive strength is determined from cylinder strength testing and core strength testing.

#### *Test cylinders*

The compressive strength of concrete can be determined using moulded 28-day test cylinders of 100 mm nominal diameter with compaction by internal vibrator. All specimens of a set should be moulded from the same sample of concrete. Sampling and testing should be in accordance with AS 1012.1 and AS 1012.9. If the age of the test specimens is greater than 28 days at the time of compressive testing, the test results should be adjusted for age.

For each lot, two pairs of cylinder test specimens should be moulded for compressive strength testing, one at 7 days and the other at 28 days.

A lot is typically deemed to comply if the 28-day compressive strength of test cylinders for any lot is greater than  $f_{cmin}$ . Concrete with a 28-day cylinder strength of between  $0.9 f_{cmin}$  and  $f_{cmin}$  is typically accepted subject to a deduction, provided it represents less than 5% of the area of the appropriate slipformed, hand-paved or transition zones placed up to and including that lot. Concrete with a 28-day cylinder strength of less than  $0.9 f_{cmin}$  should be rejected and the appropriate section of concrete paving removed and replaced.

### **Core strength testing**

Where coring is required, it should be carried out in accordance with AS 1012.14 and the relevant concrete paving specification. The test results should be adjusted for age and shape.

A lot, if required to be core strength tested, is generally deemed to comply if the strength, corrected for age, is greater than or equal to  $f_{cmin}$  for all core specimens from that lot.

Lots with core specimens with corrected strength less than  $0.9 f_{cmin}$  should be rejected and the appropriate section of concrete paving removed and replaced.

Core holes should be cleaned and restored with low-shrink cementitious concrete having a compressive strength not less than that of the base. The approved base mix is often used. The surface of the restored hole should be similar in colour to the surrounding surface. Prior to trafficking, the concrete in the core should be cured sufficiently to achieve an estimated compressive strength of 15 MPa.

### **10.9.11 Cracking**

Following construction it is common for the location, size, direction and apparent depth of observed cracks to be recorded. Specifications may set maximum limits on cracking length, depth or quantity.

For example RTA (2007b) requires that plain concrete pavements 'contain only plastic shrinkage cracks with a cumulative length of one metre or less in any slab' and specifies that 'all other cracked slabs must be removed and replaced'.

### **10.9.12 Ride Quality**

Measurement of ride quality for concrete road pavements is conducted using the same methods as used for asphalt pavements described in Section 9.6.5.

Typically, a maximum ride quality of 40-50 NAASRA Roughness Counts or 1.6-2.0 IRI is specified depending on the type of concrete pavement being constructed.

## References

- AAPA/Austrroads 1998, *Treatment of bleeding or flushed surfaces*, Austrroads Pavement Reference Group pavement work tips no. 7, Australian Asphalt Pavement Association, Kew, Vic.
- AAPA 2003, *Guide to the selection, heating and storage of binders for sprayed sealing and hot mixed asphalt*, Advisory Note no. 7, Australian Asphalt Pavement Association, Kew, Vic.
- AAPA 2004, *National sprayed sealing specification*, no. SS-1, Australian Asphalt Pavement Association, Kew, Vic.
- AASHTO 2006, *M288-06: Standard specification for geotextile specification for highway applications*, American Association of State Highway and Transportation Officials.
- ACI 1984, *Cement and concrete terminology*, ACI committee report ACI 116, American Concrete Institute, Chicago, IL., USA.
- ACI 1992, *Standard practice for curing concrete*, ACI committee report ACI 308, ACI manual of concrete practice, part 2, American Concrete Institute, Chicago, IL., USA.
- ACI 1993, *Behaviour of fresh concrete during vibration*, ACI committee report ACI 309, ACI manual of concrete practice, part 2, American Concrete Institute, Chicago, IL., USA.
- APMCA 1996, *Cold weather concreting*, technical bulletin 96/2, Australian Premixed Concrete Association, Sydney, NSW.
- APMCA 1996, *Cold weather concreting*, Technical Bulletin 96/2, Australian Premixed Concrete Association, Sydney, NSW.
- Austrroads 1990, *Guide to Geotextiles*, NTR-10, Austrroads, Haymarket, NSW.
- Austrroads 1995, *Guide to Field Surveillance of Quality Assurance Contracts*, AP\_38/95, Austrroads, Sydney, NSW.
- Austrroads 2002a, *2002 Austrroads Pavement Rehabilitation Guide (Final Draft) for Public Comment*, AP-T15/02, Austrroads, Sydney, NSW.
- Austrroads 2002b, *Effect of Design, Construction and Environmental Factors for Long-Term Performance of Stabilised Materials*, APRG Technical Note 12, Austrroads, Sydney, NSW.
- Austrroads 2002c, *Practitioner's Guide to the Design of Sprayed Seals; Revision 2000 Method*, AP-T17/02, Austrroads, Sydney, NSW.
- Austrroads 2003, *Guide to Best Practice for the Construction of In Situ Stabilised Pavements*, AGPT04/06, Austrroads, Sydney, NSW.
- Austrroads 2005a, *Sprayed Seal Cutting Practice*, AP-T39/05, Austrroads, Sydney, NSW.
- Austrroads 2005b, *Fibre-Reinforced Seals*, AP-T38/05, Austrroads, Sydney, NSW.
- Austrroads 2005c, *Audit and Surveillance of Sprayed Sealing Contract Works* by J Rebbechi, AP-T40/05, Austrroads, Sydney, NSW.
- Austrroads 2005d, *Calibration of Bitumen Sprayers, Part 1: Volumetric Calibration*, AG:PT/T531, Austrroads, Sydney, NSW.
- Austrroads 2005e, *Calibration of Bitumen Sprayers, Part 3: Transverse Distribution by RTA field method*, AG:PT/T533, Austrroads, Sydney, NSW.
- Austrroads 2005f, *Calibration of Bitumen Sprayers, Part 4: Transverse Distribution by BCA field method*, AG:PT/T534, Austrroads, Sydney, NSW.
- Austrroads 2005g, *Calibration of Bitumen Sprayers, Part 5: Road Speed Calibration*, AG:PT/T535, Austrroads, Sydney, NSW.
- Austrroads 2005h, *Calibration of Bitumen Sprayers, Part 6: Viscosity of Test Fluid Determination*, AG:PT/T536, Austrroads, Sydney, NSW.



- Austrroads 2005i, *Geotextile Reinforced Seals*, AP-T37/05, Austrroads, Sydney, NSW.
- Austrroads 2006a, *Guide to Pavement Technology – Part 4D: Stabilised Materials*, by B Andrews, AGPT04D/06, Austrroads, Sydney, NSW.
- Austrroads 2006b, *Asphalt Manufacture*, by J Rebbechi, AP-T64/06, Austrroads, Sydney, NSW.
- Austrroads 2006c, *Asphalt Paving*, by J Rebbechi, AP-T65/06, Austrroads, Sydney, NSW.
- Austrroads 2006d, *Maintenance of Asphalt Surfacing*s, by J Rebbechi, AP-T67/06, Austrroads, Sydney, NSW.
- Austrroads 2006e, *Guide to the Selection and Use of Polymer Modified Binders and Multigrade Bitumens*, by P Tredrea, AP-T42/06, Austrroads, Sydney, NSW.
- Austrroads 2006f, *Specification Framework for Polymer Modified Binders and Multigrade Bitumens*, by P Tredrea, AP-T41/06, Austrroads, Sydney, NSW.
- Austrroads 2006g, *Calibration of Bitumen Sprayers, Part 1: General Introduction, List of Methods and Calibration Test Certificate*, AG:PT/T530, Austrroads, Sydney, NSW.
- Austrroads 2006h, *Calibration of Bitumen Sprayers, Part 2: Transverse Distribution by Fixed Pit Facility*, AG:PT/T532, Austrroads, Sydney, NSW.
- Austrroads 2006i, *Update of the Austrroads Sprayed Seal Design Method*, by A Alderson, AP-T68/06, Austrroads, Sydney, NSW.
- Austrroads 2007, *Guide to Pavement Technology – Part 4B: Asphalt*, by J Rebbechi, AGPT04B/07, Austrroads, Sydney, NSW.
- Austrroads 2008a, *Guide to Pavement Technology – Part 4A: Granular Base and Subbase Materials*, by B Vuong, G Jameson, K Sharp & B Fielding, AGPT04A/08, Austrroads, Sydney, NSW.
- Austrroads 2008b, *Guide to Pavement Technology – Part 2: Pavement Structural Design*, by G Jameson, AGPT02/08, Austrroads, Sydney, NSW.
- Austrroads 2008c, *Glossary of Austrroads terms*, 3<sup>rd</sup> edn., Austrroads, Sydney, NSW.
- Austrroads 2008d, *Guide to Pavement Technology – Part 4F: Bituminous Binders*, by J Rebbechi, AGPT04F/08, Austrroads, Sydney, NSW.
- Austrroads 2008e, *Guide to Pavement Technology – Part 4J: Aggregate and Source Rock*, by B Vuong, G Jameson & B Fielding, AGPT04J/08, Austrroads, Sydney, NSW.
- Austrroads 2008f, *Guide to Pavement Technology – Part 5: Pavement Evaluation and Treatment Design*, by G Jameson & M Shackleton, AGPT05/08, Austrroads, Sydney, NSW.
- Austrroads 2008g, *Bituminous Materials Safety Guide*, AP-G41/08, Austrroads, Sydney, NSW.
- Austrroads 2009a, *Guide to Pavement Technology – Part 3: Pavement Surfacing*s, Austrroads, Sydney, NSW.
- Austrroads 2009b, *Guide to Pavement Technology – Part 4C: Materials for Concrete Pavements*, Austrroads, Sydney, NSW.
- Austrroads 2009c, *Guide to Pavement Technology – Part 4G: Geotextiles and Geogrids*, Austrroads, Sydney, NSW.
- Austrroads 2009d, *Guide to Pavement Technology – Part 4I: Earthworks Materials*, Austrroads, Sydney, NSW.
- Austrroads 2009e, *Guide to Pavement Technology – Part 4K: Seals*, Austrroads, Sydney, NSW.
- Austrroads 2009f, *Guide to Pavement Technology – Part 4L: Stabilising Binders*, Austrroads, Sydney, NSW.
- Austrroads 2009g, *Guide to Pavement Technology – Part 10: Subsurface Drainage*, Austrroads, Sydney, NSW.
- AustStab 2000, *Stabilisation patch width*, AustStab construction tip no.5, Australian Stabilisation Industry Association, Artarmon, NSW.

- Ayton, GP & Haber EW 1997, 'Curing and interlayer debonding', *International Purdue conference on concrete pavement design and materials for high performance, 6<sup>th</sup>, Indianapolis, Indiana*, Purdue University. School of Civil Engineering, West Lafayette, IN, USA, vol. 2, pp. 63-86.
- Ayton, G 2001, *A recipe for compaction of concrete*, Proceedings of the 7<sup>th</sup> International Conference on Concrete Pavements, Orlando, USA.
- Cameron DA & Mathias, CL 2000, 'Bonding of cement treated pavement layers', *ARRB Transport Research Ltd conference, 20<sup>th</sup>, Melbourne, Victoria*, ARRB Transport Research, Vermont Sth., Vic., 25pp.
- C&CAA 1994, *Surface features of concrete pavements*, road note 43, Cement and Concrete Association of Australia, North Sydney, NSW.
- CPEE 2003, *Course notes: CPEE 650: Introduction to pavements*, Centre for Pavement Engineering Education, Kew, VIC.
- CPEE 2004, *Course notes: CPEE 652: Flexible pavement construction*, Centre for Pavement Engineering Education, Kew, VIC.
- CPEE 2006, *Course notes: CPEE 660: Rigid pavement construction*, Centre for Pavement Engineering Education, Kew, VIC.
- DTEI SA 2004, *Application of sprayed bituminous surfacing: specification: part 226*, Department for Transport, Energy and Infrastructure, Adelaide, SA.
- DTEI SA 2007, *Construction of unstabilised granular pavement: specification: part 221*, Department for Transport, Energy and Infrastructure, Adelaide, SA.
- FHWA 1998, *Geosynthetic design and construction guidelines – participant's notebook*, Federal Highways Administration, Washington DC, USA.
- Forssblad, L 1981, *Vibratory soil and rock fill compaction*, Dynapac, Solna, Sweden.
- Grant, EL 1964, *Statistical quality control*, 3<sup>rd</sup> ed., McGraw Hill, New York.
- Gray, WJ & Robinson, GK 1980, 'Basic ideas of acceptance control in road construction', *Australian Road Research Board (ARRB) conference, 10<sup>th</sup>, Sydney, New South Wales*, Australian Road Research Board, Vermont Sth., Vic, vol. 10, no. 2, pp. 65-79.
- Haliburton, TA, Lawmaster, JD & King, JK 1980, *Potential use of geotextile fabric in airfield runway design*, Air Force Office of Scientific Research, School of Civil Engineering, Oklahoma State University, Stillwater, OK, USA.
- Ho, DWS & Lewis, RK 1992, 'Research into curing techniques and their effectiveness: the major findings', *Concrete in Australia*, vol.18, no.2, pp. 3-7.
- Hodgkinson, JR 1994, 'Construction: joint induction and sealing', ACPAVE Concrete Pavements Workshop, Yass, NSW.
- Koerner RM 1986. *Designing with geosynthetics*. Prentice-Hall, New Jersey.
- Midgley, L 2008, 'Ingredients of an unbound granular pavement for a successful sprayed seal', *International sprayed sealing conference, 1<sup>st</sup>, Adelaide, South Australia*, ARRB Group, Vermont Sth, Vic., 14pp.
- NAASRA 1989, *Pavement materials: statistical assessment of quality*, NAASRA Technical Report NTR-09, National Association of Australian State Road Authorities, Milsons Point, NSW.
- Neville, AM 1975, *Properties of concrete*, Pitman, London, UK.
- Nichols J & Dash D 1993, 'Australian developments to reduce road traffic noise on concrete pavements', *International conference on concrete pavement design and rehabilitation, 5<sup>th</sup>, Purdue University, Indiana*, Purdue University. School of Civil Engineering, West Lafayette, IN, USA, pp. 99-106.
- NTC 2007, *Australian code for the transport of dangerous goods by road and rail (Australian Dangerous Goods Code) 7<sup>th</sup> edn.*, National Transport Commission, Melbourne, Vic.
- PIARC 1995, 'Concrete roads: report of the Committee C7', *World road congress, 20th Montreal, Canada*, Permanent International Association of Road Congresses (PIARC), Paris, France, 68pp.

- QDMR 1999, *Standard specification – unbound pavements*, MRS11.05, 12/99, Queensland Department of Main Roads, QLD.
- RCA 1986, *Proper and economical use of plant*, technical bulletin no. 34, Road Construction Authority, Kew, Vic.
- RTA 1991, *Concrete pavement manual: design and construction: edition 2*, Roads and Traffic Authority, Sydney, NSW.
- RTA 1992, *Guide to the design, construction, maintenance and management of clay pavements with geotextile reinforced seals*, Roads & Traffic Authority, Sydney, NSW.
- RTA 1997a, *Geotextile reinforced seals (GRS)*, RTA Technology Directorate Directions Preferred Practice 97/3, Roads & Traffic Authority, Sydney, NSW.
- RTA 1997b, *Sprayed bituminous surfacing (with fibre reinforcement)*, RTA Technology Directorate Directions Preferred Practice 97/16, Roads & Traffic Authority, Sydney, NSW.
- RTA 1997c, *Sprayed sealing guide: edition 2*, Roads & Traffic Authority, Milsons Point, NSW.
- RTA 2000, *Guide to maintenance of concrete pavements*, Roads and Traffic Authority, Sydney, NSW.
- RTA 2002, *Heavily bound pavement course (plant mixed using slow setting binders: specification R75*, edn.1 / revision 0, Roads and Traffic Authority, Sydney, NSW.
- RTA 2005, *Subsurface drainage materials: specification R32*, edn. 3 / revision 1, December 2005, Roads and Traffic Authority, Sydney, NSW.
- RTA 2006, *Sprayed bituminous surfacing (with cutback bitumen): specification R106*, edn., 4 / Revision 0, Roads and Traffic Authority, Sydney, NSW.
- RTA 2007a, *Unbound and modified pavement course: specification R71*, edn., 3 / revision 1, Roads and Traffic Authority, Sydney, NSW.
- RTA 2007b, *Plain concrete base: specification R83*, edn. 2/revision 6, Roads and Traffic Authority, Sydney, NSW.
- RTA 2007c, *Continuously reinforced concrete base: specification R84*, edn. 2/revision 6, Roads and Traffic Authority, Sydney, NSW.
- RTA 2007d, *Earthworks: specification R44*. edn. 3/revision 7, Roads and Traffic Authority, Sydney, NSW.
- RTA 2008a, *Quality management system (type 6): specification Q6*, edn.1 / Revision 7, Roads and Traffic Authority, Sydney, NSW.
- Smorgan ARC 1989, *A guide to using chairs and spacers for reinforcement*, Smorgan ARC, West Footscray, Vic.
- Southwell, G 2007, 'Oscillating roller – a new form of compaction', *Queensland Roads*, no. 3, March 2007.
- Transit NZ 2005, *Specification for construction of unbound granular pavement layers*, TNZ B/02:2005, Transit New Zealand, NZ.
- VicRoads 1998, *Guide to general requirements for unbound pavement materials*, technical bulletin no. 39, VicRoads, Kew, Vic.
- VicRoads 2000, *Determination of working time using cementitious binders*, Code of Practice RC330.02, VicRoads, Kew, Vic.
- VicRoads 2001, *Guide to geotextile reinforced sprayed seal surfacing*, technical bulletin no. 38, VicRoads, Kew, Vic.
- VicRoads 2004a, *Drainage of subsurface water from roads*, technical bulletin no. 32, VicRoads, Kew, VIC.
- VicRoads 2004b, *Bituminous sprayed surfacing manual*, technical bulletin no. 45, VicRoads, Kew, VIC.
- VicRoads 2007a, *Selecting a treatment for flushed surfacing*, technical note no. 85, VicRoads, Kew, VIC.
- VicRoads 2007b, *Bituminous surfacing: stripped seals*, technical note no. 87, VicRoads, Kew, Vic.

VicRoads 2008a, *Standard specifications for roadworks and bridgeworks: section 204: earthworks*, VicRoads, Kew, Vic.

VicRoads 2008b, *Standard specifications for roadworks and bridgeworks: section 304: unbound flexible pavement construction*, VicRoads, Kew, Vic.

VicRoads 2008c, *Geotextile reinforced seals*, Technical note no. 14, VicRoads, Kew, Vic.

Voight, GF 1997, 'Specification synthesis and recommendations for repairing uncontrolled cracks that occur during concrete pavement construction', *International Purdue conference on concrete pavement design and materials for high performance, 6<sup>th</sup>, Indianapolis, Indiana*, Purdue University. School of Civil Engineering, West Lafayette, IN, USA, pp. 13-28.

Wadsworth HM 1989, *Statistical methods for engineers and scientists*, McGraw Hill.

Zollinger, D, Tang, T & Xin D 1994, 'Sawcut depth considerations for jointed concrete pavement based on fracture mechanics analysis', *Transportation Research Record*, no. 1449, pp. 91-100.

### **Australian and New Zealand Standards**

AS 1012.1-1993, *Methods of testing concrete: sampling of fresh concrete*.

AS 1012.3.1-1998, *Methods of testing concrete: determination of properties related to the consistency of concrete: slump test*.

AS 1012.9-1999, *Methods of testing concrete: determination of the compressive strength of concrete specimens*.

AS 1012.12.2-1998, *Methods of testing concrete: determination of mass per unit volume of hardened concrete: water displacement method*.

AS 1012.14-1991, *Methods of testing concrete: method for securing and testing cores from hardened concrete for compressive strength*.

AS 1289.0-2000, *Methods of testing soils for engineering purposes: general requirements and list of methods*.

AS 1289.2.2.1-1998, *Methods of testing soils for engineering purposes: soil moisture content tests: determination of the total suction of a soil: standard method*.

AS 1289 5.1.1-2003, *Methods of testing soils for engineering purposes : soil compaction and density tests : determination of the dry density /moisture content relation of a soil using standard compactive effort* .

AS 1289 5.2.1-2003, *Methods of testing soils for engineering purposes : soil compaction and density tests : determination of the dry density or moisture content relation of a soil using modified compactive effort*.

AS 1289 5.3.1-2004, *Determination of the field density of a soil – sand replacement method using a sand-cone pouring apparatus*.

AS 1379-2007, *Specification and supply of concrete*.

AS 2891.14.3-1999, *Methods of sampling and testing asphalt : field density tests :calibration of nuclear thin-layer density gauge using standard blocks*.

AS 2891.14.4-1999, *Methods of sampling and testing asphalt : field density tests: calibration of nuclear surface moisture-density gauge: backscatter mode*.

AS 3799-1998, *Liquid membrane-forming curing compounds for concrete*.

AS 3905.2-1997, (Superseded), *Quality system guidelines: guide to AS/NZS ISO 9002, and AS/NZS ISO 9003 for construction*.

AS 3911.1-1992, (Superseded), *Guidelines for auditing quality systems: auditing*.

AS 3600-2001, *Concrete structures*.

AS/NZS ISO 9000:2006, *Quality management systems: fundamentals and vocabulary*.

AS/NZS ISO 9001:2000, *Quality management system: requirements*.

AS/NZS ISO 9004:2000, *Quality management and quality systems: guidelines for performance improvements.*

Austrroads' **Guide to Pavement Technology Part 8: Pavement Construction** provides advice on the general requirements for the management of quality assurance, construction planning, earthworks, subsurface drainage, unbound pavements, stabilised pavements, sprayed bituminous surfacings, asphalt pavements and surfacings, and concrete pavements.

## Guide to Pavement Technology Part 8



*Austrroads*

Austrroads is the association of Australasian road and transport agencies.

[www.austrroads.com.au](http://www.austrroads.com.au)