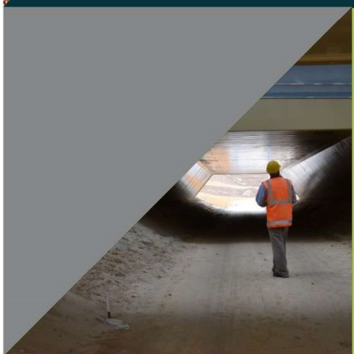
WARRIP
WESTERN AUSTRALIAN ROAD RESEARCH
AND INNOVATION PROGRAM



Investigation of Tonkin, Reid and Kwinana Trial Sections-Stage 3

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AN INITIATIVE BY:

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for Main Roads Western Australia

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SUMMARY

The investigation of several trial sections located within the Perth metropolitan region was undertaken as part of Stages 1 and 2 of the WARRIP project *Investigation of Tonkin Highway and Reid Highway Trial Sections*. To enable the validation of the Stage 2 findings, a comprehensive review of selected granular pavements with thin asphalt surfacings located within the Perth metropolitan area was undertaken, including:

- design pavement profile and construction data
- subsurface and climatic conditions
- traffic history analysis
- pavement maintenance history
- performance/condition data including historic Falling Weight Deflectometer (FWD), roughness and rutting data.

Data was collected at 17 sections across six sites, which all comprised a thin asphalt surfacing (≤ 60 mm) and either a bitumen-stabilised limestone (BSL) or crushed rock base (CRB) basecourse, overlying a crushed limestone subbase and sand subgrade.

Analysis of the data validated the following Stage 2 findings:

- The current mechanistic-empirical design method does not replicate observed performance and typically underestimates asphalt fatigue life.
- The typical observed asphalt fatigue life of granular pavements with compliant granular thickness as per the Austroads empirical design method is at least 15 years regardless of the 40 year design traffic.
- The modulus of the granular base materials increases with time and may be modelled using short-term and long-term strength characteristics.
- The limestone subbase has a higher modulus than currently assumed in the design model and has a consistent modulus throughout its in-service life.
- The yellow and white sand subgrades (Perth sands) have higher moduli than currently used design values.

The findings in Stage 3 confirm the Stage 2 recommendation that MRWA consider revising the Main Roads Engineering Road Note 9 (ERN9) design procedures for thin asphalt-surfaced granular pavements. These findings, together with those of the WAPARC Finite Element Pavement Model project, suggest the following areas for revision:

- Amend ERN9 to provide a higher predicted asphalt fatigue life across all design traffic loadings and remove the 5 year design period for less heavily-trafficked roads.
- Implement either a short-term and long-term asphalt fatigue design method – which accounts for the increased design modulus of granular materials as in-service life progresses, or an increased laboratory-to-field shift factor in conjunction with the current fatigue design method.

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1 INTRODUCTION

The investigation of several trial sections located within the Perth metropolitan region was undertaken as part of Stages 1 and 2 of the WARRIP project *Investigation of Tonkin Highway and Reid Highway Trial Sections*. The trial sections investigated comprised granular pavement systems surfaced with a thin asphalt (≤ 60 mm). The investigation was limited to pavements with either crushed rock or bitumen-stabilised limestone (BSL) base materials overlying a crushed limestone subbase (CLS) and sand subgrade. These pavement systems and materials are typical of the Perth metropolitan region; historically, the performance of these pavements over a range of traffic applications, including urban freeways, highways and arterial routes, is good.

Hydrated cement-treated crushed rock base (HCTCRB) was also a typical base material used extensively in conjunction with thin asphalt granular pavements throughout the early 2000s due to its superior strength and indifference to moisture. However, premature failure of several HCTCRB pavements has since led to the exclusion of this material from the MRWA specification 501 (Main Roads Western Australia (MRWA) 2018). Therefore these sections were not included as part of this overall investigation.

The first two stages of the project focussed on: (1) collating observations and historic performance and monitoring data, and (2) a critical review of the current Main Roads pavement mechanistic-empirical (ME) design procedure for granular pavements with thin asphalt surfacings (MRWA 2013).

Findings from the previous investigations included:

- the design modulus of BSL, CRB and CLS materials may be increased
- the design modulus of sand subgrades may be increased
- observations of short- and long-term design moduli differences for granular materials
- subsequent amendments to the Main Roads Engineering Road Note 9 (MRWA 2013) and Austroads (2018) design methods for thin asphalt surfacings over granular pavements.

Aligning design outcomes with observed behaviour and measured performance may allow design conservatism to be reduced. This may result in increased implementation of these cost-effective pavements systems across a wider range of traffic scenarios throughout the Perth metropolitan area.

The aim of Stage 3 of the project was to further investigate, and ultimately validate, these observations through the investigation of other sections of road, using similar sets of observational and historic performance and monitoring data. Stage 3 investigated the performance of pavements which were constructed either as part of Design and Construct, Construct only, or Alliance contracts, rather than pavement specifically constructed as MRWA trials. The objective was to provide a vital insight into the applicability of the previous findings to pavements constructed to a more representative, standard quality level.

2 VALIDATION CHARACTERISTICS

2.1 Introduction

The collation and review of nine MRWA trial sections in Stage 2 of the project enabled a review of the current ERN9 (MRWA 2013) design method to be conducted and the mechanisms behind the ongoing better-than-expected performance of these sections to be explored. These observations were based on three separate trials with varying traffic levels, climatic characteristics, subsurface geology, pavement material sources, year of construction, and specification conformance. All have shown similar performance trends throughout their in-service life.

The following sections describe the validation sites which were investigated in Stage 3.

2.2 Performance Phases

During the Stage 2 investigation, similar performance phases were observed through the investigation in terms of deflection and curvature and through timeline representations of back-calculated granular moduli. These phases were as follows:

- Phase 1 (opening to traffic to end of 1st year): pavement system strengthens with the application of the first year of traffic. The initial strength on opening to traffic represents the worst-case design scenario over the design life.
- Phase 2 (end of first year to around end of fifth year): pavement systems continue to strengthen, with the rate of increase in modulus gradually reducing.
- Phase 3 (end of fifth year to end of fifteenth year): pavements continue to strengthen, but at a lower rate.
- Phase 4 (beyond fifteenth year): beyond the standard ERN9 15-year asphalt fatigue design period: pavement systems show continuing decreases in strength.

These four phases reflect the changes in granular modulus with loading/time. It was concluded in Stage 2 that the ME design process could be improved by allowing for the changes in granular moduli with loading/time.

An investigation to understand if these performance phases exist in pavements that were not built to trial standards was undertaken in Stage 3 through the analysis of deflection and curvature data in addition to back-calculated granular moduli.

2.3 Granular and Subgrade Moduli

The findings regarding granular moduli from the Stage 2 investigations were as follows. These observations were based on back-calculation data in addition to repeated load triaxial (RLT) data.

- The design modulus for the BSL could be increased based on:
  - in-service observations in which the BSL demonstrated equal or superior performance to the CRB
  - moduli back-calculated from measured FWD deflection bowls
  - ongoing good performance of trial sections.
- The design modulus for the CRB could be increased based on:

    - the results of RLT testing conducted at base confinement stress levels
    - back-calculated moduli
    - ongoing good performance of the trial sections.
- The design modulus of the limestone subbase could be increased based on:
    - the results of RLT testing conducted at subbase confinement stress levels in addition to the outcomes of the finite element pavement design modelling project (Jameson et al. 2017)
    - back-calculated moduli
    - ongoing good performance of trial sections.
- The design moduli for the sand subgrades could be increased based on:
    - CBR results, both current and historic
    - the results of RLT testing conducted at various confinement stress levels in addition to the outcomes of the finite element pavement design modelling project (Jameson et al. 2017)
    - back-calculated moduli.

A similar investigation of granular moduli was undertaken in Stage 3.

2.4 Short-term and Long-term Design

The observed performance phases in Stage 2 formed the basis of a draft design method which provided for the short-term (first year) and long-term (> 1 year) thin asphalt fatigue modelling to better reflect observed in-service performance. This draft design method considered two sperate stages of modelling to account for the increase in strength of a granular pavement with age and loading: the short-term fatigue performance encompassing Phase 1, and the long-term performance encompassing Phases 2 and 3.

Stage 2 of the project recommended the following design considerations for each of these two stages:

- The short-term asphalt fatigue damage can be evaluated using the current ME method and current granular design moduli, as this represents a conservative method to model the most detrimental loading period.

- The long-term asphalt fatigue damage may be modelled using the same procedure but with increased design moduli of the granular base and subbase layers and sand subgrade.

3 CHOOSING SECTIONS TO BE INVESTIGATED

3.1 Introduction

Similarly to the first two stages of the project, the first phase of Stage 3 involved the identification of investigation sections and associated historic pavement performance and observational data. The preliminary criteria for choosing the Stage 3 investigation sections included the pavement location, pavement type (thin asphalt over granular) and the materials used for the surfacing, base, subbase and subgrade. Specifically, these requirements included the following:

- located within the Perth metropolitan area
- met the material requirements of ERN9 Clause 1.2 (c), including a well-drained subgrade comprising Perth sand, a subbase comprising crushed limestone and a basecourse comprising BSL or CRB
- pavements surfaced with a thin asphalt surfacing comprising either a single dense-graded asphalt (DGA) layer not exceeding 40 mm in thickness, or an open-graded asphalt (OGA) layer overlying a DGA layer with a combined thickness of 60 mm.

Secondary site selection requirements considered the existence and consistency of historic performance and observational data, including:

- data relating to the original pavement design, including design traffic and design layer thicknesses
- data relating to the as-constructed pavement, including date of opening to traffic, constructed thicknesses, in situ densities and dryback specification requirements and subsequent conformance to these specifications
- dates of resurfacings (if present)
- data relating to any asphalt fatigue related distress (if present)
- have not been widened or modified other than resurfacing
- do not include bridge decks
- network or project level FWD measured surface deflection data
- roughness and rutting data to aid in identifying performance trends and possible resurface/modification dates
- located between traffic entry and exit points to ensure traffic volume consistency over the entire investigation length.

Identifying pavements which complied to these site selection criteria was difficult due to a lack of robust data or inconsistencies in available data. Widening works and the use of alternative base materials such as HCTCRB also ruled out a significant number of possible investigation sections.

3.2 Definition of Observed Fatigue Life

In order to compare the output of the current design system with the observed, in-service performance of granular pavements with thin asphalt surfacings, the predicted allowable traffic loading to asphalt fatigue is compared to the actual traffic loading leading to a terminal condition of asphalt fatigue cracking.

A major limitation of this project is that cracking data was not available to determine the cumulative traffic loading to a terminal cracking condition. MRWA has confirmed that re-surfacing works are typically undertaken not long after fatigue cracking is observed. Therefore, for this comparison observed fatigue life had to be inferred to correspond to the cumulative traffic loading when the DGA layer was first replaced. The roughness and rutting data collected on resurfaced portions of a section prior to the resurfacing works being carried out was analysed to ensure that cracking was indeed the trigger for the resurfacing works.

For pavements with a composite surfacing of OGA overlying DGA, further investigation was required to understand if re-surfacing works of these sections related to distress of the OGA layer only, or included DGA distress. The typical life of an OGA wearing course in the Perth region is approximately 10 to 15 years, after which time these wearing courses start to ravel and the functionality characteristics of the surfacing degrades, requiring replacement of the OGA layer whilst the underlying DGA continues to perform. This was observed along several of the investigation sections where only the OGA layer had been replaced and the underlying DGA was still performing and therefore had not reached the fatigue life.

The presence of a geotextile reinforced seal (GRS) was also noted at one of the investigation sections during the replacement of the OGA layer. This suggests that the underlying DGA along this investigation section was cracked and assumed to have reached its design fatigue life.

4 DETAILS OF INVESTIGATION SECTIONS

4.1 Introduction

Investigation sections complying to the specified criteria as outlined in Section 3.1 were identified along major metropolitan freeways and highways. Details are presented in Table 4.1. Each of these locations are discussed further in the subsequent sections.

Table 4.1: Details of investigation sections

Road name	Road ID	Section ID	Direction	Lane	Date opened to traffic	Pavement age at Jun-19 (years)
Graham Farmer Freeway	H020	G1	EB	L1	Apr-2000	19.0
		G2	EB			
		G3	WB	R1		
		G4	WB			
Kwinana Freeway	H015	KF1	SB	L1	Sep-1994	24.5
		KF2	SB			
		KF3	SB			
Mitchell Freeway	H016	M1	SB	R1	Jul-1998	31.0
		M2	SB			
		M3	SB		Dec-1999	19.5
		M4	NB	L1	Aug-2017	2.0
Reid Highway	H021	RH1	EB	L1	Nov-1994	24.5
		RH2	WB	R1		
Roe Highway	H018	RO1	NB	L1	Dec-1984	34.5
Tonkin Highway	H017	TH2	NB	R1	Jul-1984	35.0

4.2 Graham Farmer Freeway

4.2.1 Overview

MRWA Contract 19/95 covered chainages 0.00 to 6.50 SLK of the Graham Farmer Freeway and corresponding city bypass tunnel. It was opened to traffic in April 2000. The portion of the carriageway through the tunnel is a concrete pavement with the remainder of the external carriageway comprised of a granular pavement with a thin asphalt surfacing. The majority of the granular pavement is comprised of a BSL base overlying CL subbase and sand subgrade plus an OGA over DGA surfacing system. A small area of CRB with 2% cement located on the eastbound entrance to the tunnel was excluded from the study. The posted speed limit is 80 km/h.

As-constructed and design thicknesses were available for these sections of interest. The design modulus used in the original ME design, the design traffic loading, measured traffic, roughness, rutting and FWD data at both the network and project level were also sourced. In situ densities and dryback conformance information could not be sourced. However, it was assumed that non-conformances would have been identified and addressed prior to practical completion.

4.2.2 Sub-sections of Interest

Four sub-sections along the Graham Farmer Freeway, two eastbound (G1 and G2) and two westbound (G3 and G4), were chosen for further investigation based on compliance with the site selection criteria discussed in Section 3. These sections comprise a total length of 2.8 km and relate to the outer, slow lane and designated L2 in the eastbound direction and R2 in the westbound direction. The details of these sections, which were accessed through MRWA's corporate Integrated Road Information System (IRIS) on 17 April 2019, are summarised in Table 4.2.

Table 4.2: Details of Graham Farmer Freeway investigation sections

Section ID	Direction	Start chainage (km)	End chainage (km)	Length (m)	Constructed material type			
					Surfacing	Basecourse	Subbase	Subgrade
G1	EB	2.95	3.32	370	OGA/DGA	2% BSL	Crushed limestone	White sand
G2	EB	4.19	5.24	1050				
G3	WB	2.50	3.32	820				
G4	WB	4.50	5.00	500				

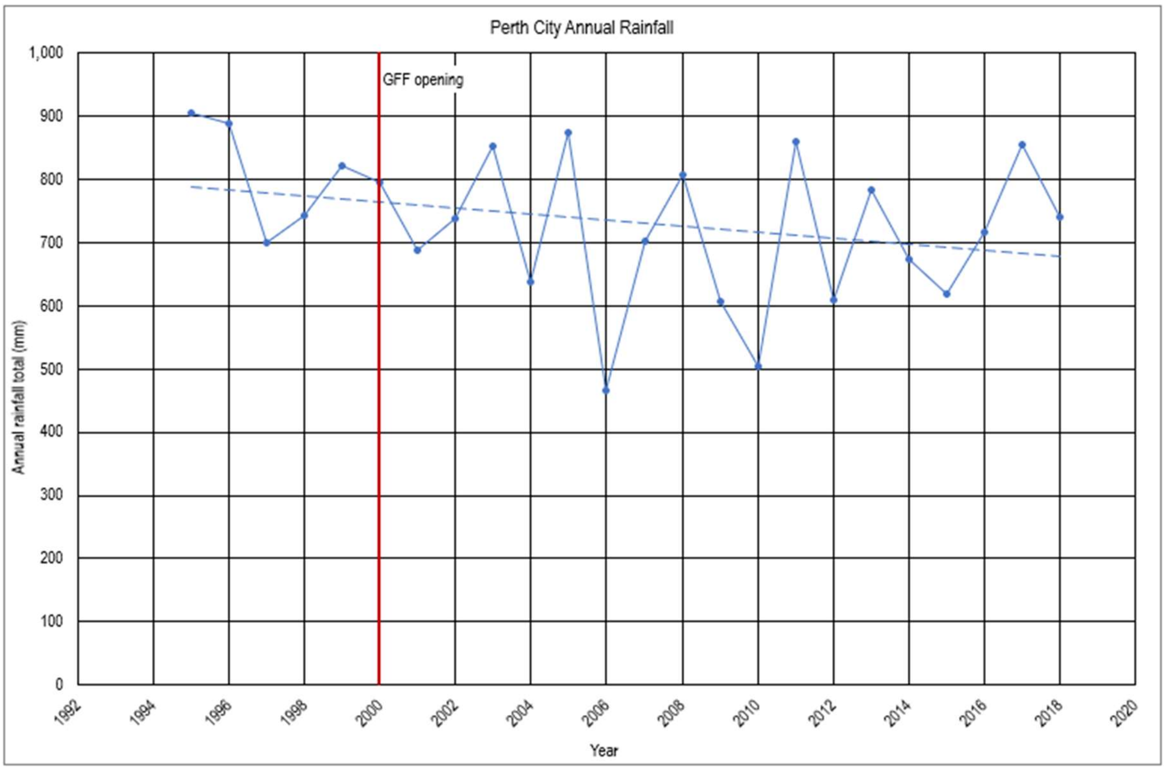
Subsurface Conditions

The 1:250 000 Environmental Geological Map series produced by the Geological Survey of Western Australia Perth sheet (Geological Survey of Western Australia 1980a) indicated that the natural subsurface material underlying the Graham Farmer Freeway investigation sections was quartz sands derived from coastal Tamala limestone.

Climate Data

The annual average rainfall from the Perth City weather station, which is in close vicinity to the Graham Farmer Freeway, was collated to demonstrate the climatic history during the in-service life of the pavements. The data for five years prior to construction through to 2018 obtained from the Bureau of Meteorology (Bureau of Meteorology (BOM) 2013) is presented in Figure 4.1. The plot demonstrates a clear downward trend of annual rainfall for this location.

Figure 4.1: Graham Farmer Freeway historic annual rainfall data



Source: BOM (2013).

4.2.3 Design and Construction Data

Design and construction data for sub-sections G1, G2, G3 and G4 was sourced from various documentation and correspondence relating to Contract 19/95. Specific pavement dipping depths were not available and specified construction target thicknesses were taken from as-constructed contract drawings. This data is summarised in Table 4.3.

Table 4.3: Graham Farmer Freeway – design and specified target thicknesses

Identification number	Layer	Material	Design thickness (mm)	Specified target thickness (mm)
G1, G2, G3, G4	Asphalt	10 mm OGA	30	–
		10 mm DGA	30	–
	Basecourse	2% BSL	130	150
	Subbase	Crushed limestone	220	170
	Subgrade	White sand	–	–

The subgrade design vertical modulus for the Graham Farmer Freeway sections of interest was documented as 120 MPa, derived from a design CBR of 12%.

Details of the original design moduli are given in Table 4.4.

Table 4.4: Graham Farmer Freeway – original design moduli

Identification number	Material	Design modulus (MPa)
G1, G2, G3, G4	10 mm OGA	2500
	10 mm DGA	2800
	2% BSL	450
	White sand	120

4.2.4 Traffic Loading

The 40-year design traffic loading and the asphalt fatigue design traffic loading in equivalent standard axles (ESAs) and associated traffic parameters are summarised in Table 4.5.

Table 4.5: Graham Farmer Freeway – original design traffic

Section ID	Design traffic (ESAs)	Design period (years)	Design traffic growth rate (%)	Per cent heavy vehicles (%)	Asphalt fatigue design traffic (ESAs)
G1	1.0×10^8	40	2.0	5	1.0×10^7
G2					
G3					
G4					

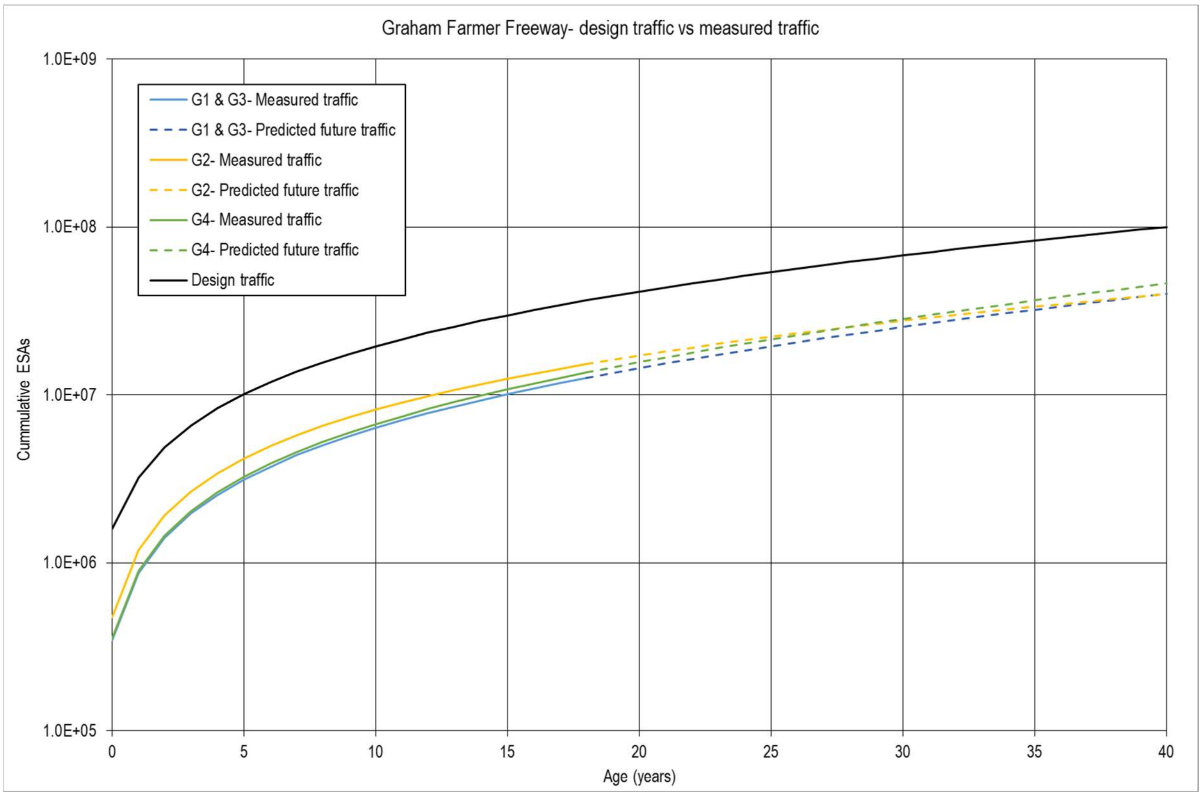
The actual traffic data was extracted from IRIS in April 2019. Using the available measured traffic data, the average annual traffic growth rate was calculated for each investigation section. The first year of traffic loading, and the cumulative number of ESAs from opening until June 2019, were subsequently estimated using the back-calculated average growth rate and the available traffic data sets. This calculated traffic data is presented in Table 4.6.

Table 4.6: Graham Farmer Freeway – actual traffic loading

Section ID	First year of traffic data available	Most recent year of traffic data available	Average growth rate (%)	First year traffic (ESAs)	Cumulative traffic from opening to Jun-19 (ESAs)	Pavement age in Jun-19 (years)
G1	2008	2018	3.0	8.7×10^5	1.4×10^7	19.2
G2	2001	2018	1.6	1.2×10^6	1.6×10^7	
G3	2008	2018	2.9	8.8×10^5	1.4×10^7	
G4	2001	2018	3.5	9.0×10^5	1.5×10^7	

The cumulative ESAs for each of the four sections G1, G2, G3 and G4 are shown in Figure 4.2 in addition to the future predicted traffic and the original design traffic.

Figure 4.2: Graham Farmer Freeway – traffic data



4.2.5 Pavement Maintenance

It is important to identify any maintenance/rehabilitation conducted on the investigation sections as this would influence the overall performance of the pavements. Pavement and surface detail data was extracted from IRIS which documents the most recent historic structural changes or major resurfacing works that have been undertaken.

Pavement detail data for the Graham Farmer Freeway was extracted from the IRIS database in April 2019. This data indicated that part of section G1 and all of section G2 had undergone resurfacing in 2014, and the majority of section G3 in 2017 and all of section G4 in 2013. IRIS also recorded that a GRS was placed along investigation section G2 after the resurfacing in 2014.

Historic aerial images were subsequently used to identify previous maintenance or rehabilitation works which were not listed in IRIS. Roughness and rutting timelines were also used to corroborate the possible works identified by historic images, with a decrease typically demonstrated after works were undertaken.

The aerial image review identified resurfacing of section G1 in March 2014, section G2 in December 2014, section G3 in December 2010 and March 2017, and section G4 in February 2013.

Table 4.7 presents the rutting and roughness data measured before the resurfacings along each of the investigation sections. Also presented are the rutting and roughness values along the sub-section which was resurfaced. For section G2 and section G4 where the entire investigation section was resurfaced, the resurfacing sub-section is the same as the investigation section. By comparing the condition of the resurfaced sub-section to the overall section, reasons for the resurfacing besides from cracking may be identified.

Considering the data in Table 4.7, the rutting data had no measurements above 10 mm and the sub-sections were not much different to the whole investigation section. Similarly, the roughness data for the chainages resurfaced was also well below the typical intervention level of 110 counts/km and condition was similar to the entire section. This suggests that high roughness or rutting was not the reason for the resurfacing of the sub-sections.

Table 4.7: Graham Farmer Freeway – roughness and rutting before resurfacing

Section ID	Resurface date	Full investigation section				Resurfaced sub-section			
		Chainage	Average roughness and survey date (counts/km)	Average rutting and survey date (mm)		Chainage	Average roughness (counts/km)	Average rutting (mm)	
				IWP	OWP			IWP	OWP
G1	Mar-14	2.95 – 3.32	30.7 (Nov-10)	2.4 (Mar-13)	1.3	3.14 – 3.32	27.8	3.7	1.6
G2	Dec-14	4.19 – 5.24	21.8 (Nov-10)	2.1 (Mar-13)	2.1	4.19 – 5.24	21.8	2.1	2.1
G3	Dec-10	2.5 – 3.32	48.3 (Nov-09)	1.4 (Dec-08)	1.1	2.50 – 3.14	49.6	1.3	1.0
	Mar-17		N/A[1]	2.4 (Mar-13)	2.1	2.65 – 3.14	N/A[1]	2.6	2.2
G4	Feb-13	4.5 – 5.0	27.7 (Nov-10)	1.8 (Dec-08)	2.1	4.50 – 5.00	27.7	1.8	2.1

Notes:

1. Roughness data only available up to 2010.

The inclusion of a GRS after the 2014 resurfacing along section G2 indicates that this section would have demonstrated a high amount of cracking. This cracking may be due to shrinkage of the underlying BSL, or fatigue cracking of the DGA. As this information is not available it has been assumed that the DGA in this section had fatigue cracked.

Using the traffic data calculated from the measured data sets, the approximate cumulative traffic levels applied to each resurfacing were calculated These are shown in Table 4.8.

According to this data, asphalt fatigue cracking was observed along section G2 only, approximately 15 years after the surfacing was first constructed.

It is postulated that the replacement of the OGA in sections G1, G3 and G4 may have been due to ravelling as the age at resurfacing is within the typical replacement age of OGA wearing courses.

Table 4.8: Graham Farmer Freeway – resurfacing dates and traffic loading

Section ID	Resurface chainage	Date of 1st resurfacing and surfacing age (years)	Cumulative traffic at resurfacing (ESAs)	Layer removed and replaced[1]	Subsequent resurface chainage	Subsequent resurfacing dates and age	Cumulative traffic at subsequent resurfacings (ESAs)	Layer removed and replaced[1]
G1	3.14 – 3.32	Mar-14 (13.9)	9.2×10^6	OGA	N/A			
G2	4.19 – 5.24	Dec-14 (14.7)	1.2×10^7	OGA/DGA				
G3	2.50 – 3.14	Dec-10 (10.6)	6.9×10^6	OGA	2.65 – 3.14	Mar-17 (6.9)	4.8×10^6	OGA
G4	4.50 – 5.00	Feb-13 (12.8)	8.9×10^6		N/A			

Notes:

2 Layer replaced at resurfacing advised by MRWA.

4.2.6 Performance Monitoring

Deflection and curvature

Both project and network level FWD data was available for the Graham Framer Freeway sections as detailed in Table 4.9. Full FWD data sets are presented in Appendix A.

Table 4.9: Graham Farmer Freeway – available FWD data

Section ID	Years of network level FWD	Age at network level FWD	Years of project level FWD	Age at network level FWD
G1	2002-2005, 2007, 2008, 2010	2 – 5, 7, 8, 10 years	2006, 2008, 2009	6 , 8, 9 years
G2	2002 – 2008 (inclusive)	2 – 8 years	N/A	
G3	2002 – 2008, 2010	2 – 8, 10 years	2006, 2008, 2009	6 , 8, 9 years
G4	2002 – 2008 (inclusive)	2 – 8 years	N/A	

Deflection results

A summary of the Graham Farmer Freeway mean deflection data from the network level and project level FWD surveys are presented in Table 4.10 and Table 4.11 respectively. This data has been corrected from the measured surface temperature to the Weighted Mean Annual Pavement Temperature (WMAPT) (29°C for Perth) using the method described in Austroads (2011) assuming the DGA temperature is the same as the surface temperature. The network level data is also presented in Figure 4.3. Deflection trend statistics are presented in Table 4.12.

Table 4.10: Graham Farmer Freeway – network level maximum deflection

Section ID	2002	2003	2004	2005	2006	2007	2008	2009	2010
G1	0.53	0.53	0.49	0.58	–	0.52	0.42	–	0.46
G2	0.41	0.39	0.34	0.35	0.32	0.32	0.41	0.43	–
G3	0.52	0.56	0.49	0.51	0.46	0.52	0.54	0.34	0.53
G4	0.40	0.41	0.33	0.38	0.35	0.35	0.33	0.38	–

Table 4.11: Graham Farmer Freeway – project level maximum deflection

Section ID	2006	2008	2009
G1	0.39	0.37	0.32
G3	0.44	0.39	0.34

Figure 4.3: Graham Farmer Freeway – Mean maximum deflection

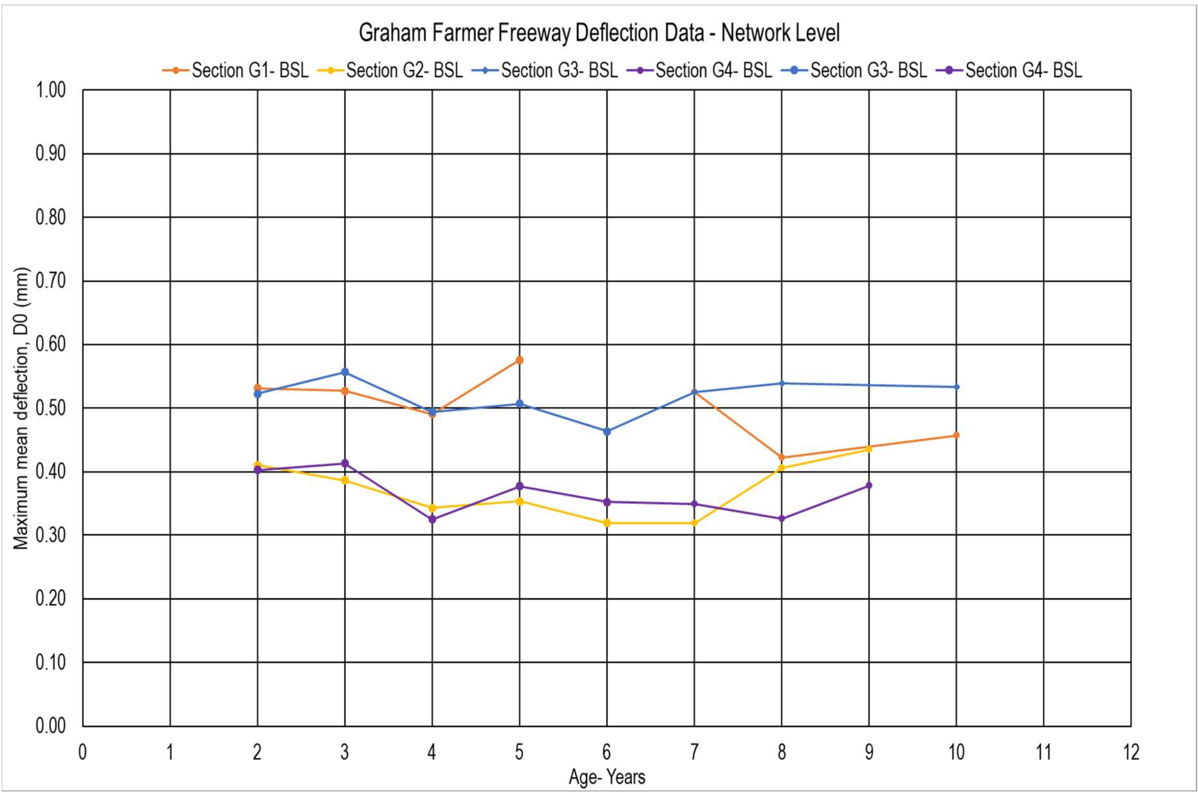


Table 4.12: Graham Farmer Freeway – network level deflection trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 2 (up to 5th year)	G1	0.13	0.01	0.6
	G2	0.81	–0.02	0.1
	G3	0.27	–0.01	0.5
	G4	0.29	–0.02	0.5
Phase 3 (end of 5th year onwards)	G1	0.25	–0.02	0.7
	G2	0.88	0.04	0.06
	G3	0.52	0.01	0.3
	G4	0.11	0.01	0.7

Curvature results

A summary of the Graham Farmer Freeway mean curvature data from the network level and project level FWD surveys are presented in Table 4.13 and Table 4.14 respectively. This data has been corrected from the measurement surface temperature to the WMAPT (29°C for Perth). The network level data is also presented in Figure 4.4. Deflection trend statistics are presented in Table 4.15.

Table 4.13: Graham Farmer Freeway – network level curvature

Section ID	2002	2003	2004	2005	2006	2007	2008	2009	2010
G1	0.16	0.19	0.15	0.16	-	0.16	0.13	-	0.19
G2	0.15	0.14	0.12	0.12	0.11	0.11	0.14	0.11	–
G3	0.17	0.18	0.16	0.15	0.18	0.20	0.18	–	0.16
G4	0.14	0.14	0.11	0.12	0.11	0.11	0.12	0.10	–

Table 4.14: Graham Farmer Freeway – project level curvature

Section ID	2006	2008	2009
G1	0.16	0.14	0.11
G3	0.16	0.15	0.12

Figure 4.4: Graham Farmer Freeway – curvature

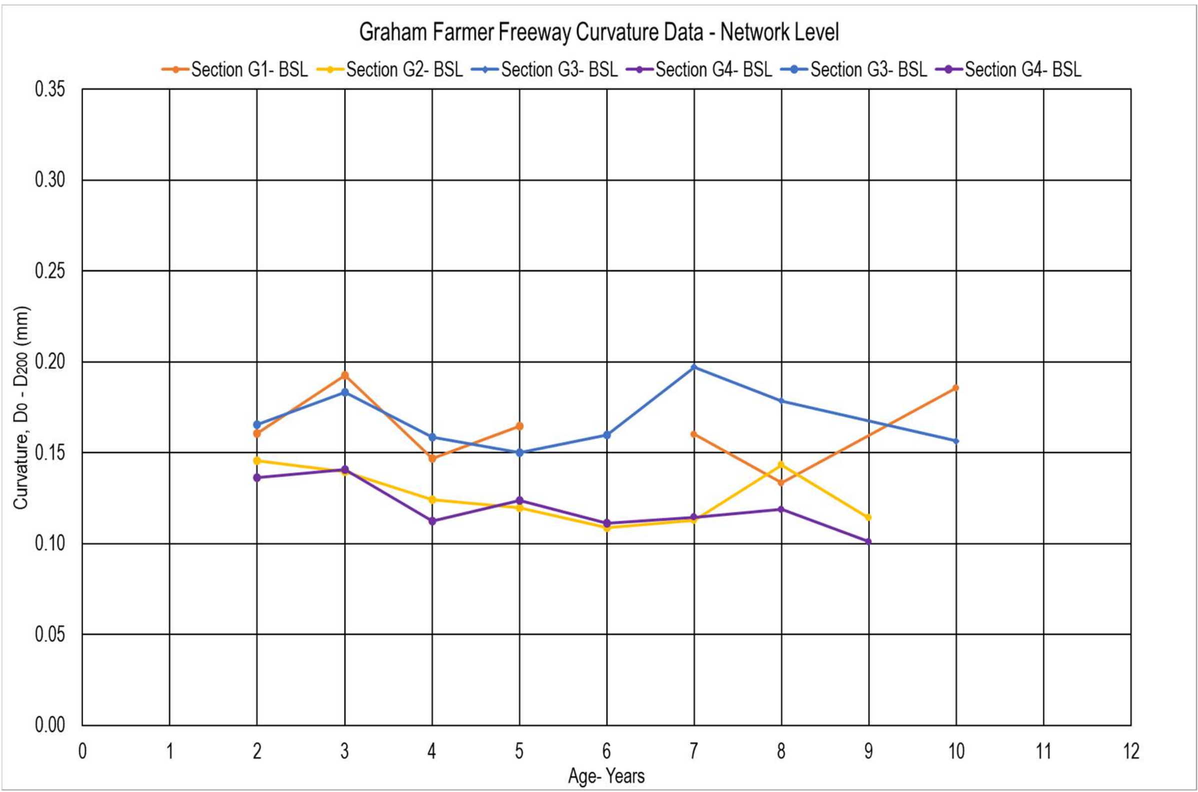


Table 4.15: Graham Farmer Freeway – network level curvature trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 2	G1	0.05	–0.003	0.8
	G2	0.95	–0.01	0.02
	G3	0.42	–0.01	0.4
	G4	0.44	–0.01	0.3
Phase 3	G1	0.42	0.01	0.6
	G2	0.15	0.005	0.6
	G3	0.11	–0.004	0.7
	G4	0.20	–0.003	0.6

Trends in deflection and curvature

Both the deflection and curvature data for the Graham Farmer Freeway investigation sections were consistent over the eight years of data with little change in measured values.

For both deflection and curvature measurements, the project level data was consistently lower than the network level data. The project level data also showed a consistent decrease in deflection and curvature for G1 and G3 over the three-year data period.

As discussed in Section 2, four similar phases of performance were identified from the deflection and curvature data of the trial sections investigated previously in Stage 2. The deflection and curvature data for the Graham Farmer Freeway investigation sections cover the proposed second and third phases of these previously-identified performance trends.

From Figure 4.3 and Figure 4.4 and Table 4.12 and Table 4.15, the following trends within these two previously-identified phases can be inferred:

- Phase 2: end of first year to around the end of 5th year:
  - Deflection is typically constant with the exception of G2 which shows an increase. Curvature is also typically constant with the exception of G2 which shows a slight increase.
- Phase 3: end of fifth year to end of 15th year
  - Deflection and curvature of all sections remains constant and shows little change.

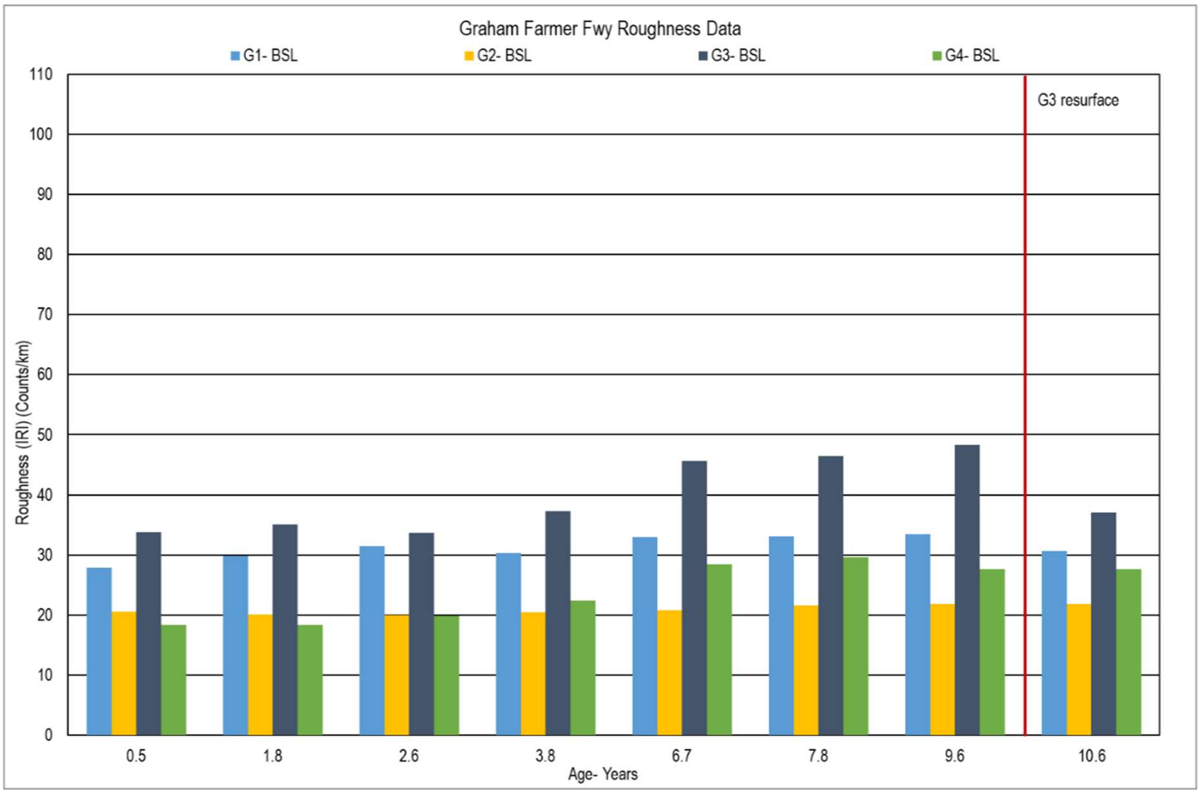
The observations during Phase 2 are typically consistent with the trends identified from the Stage 2 trial data. However, Stage 2 identified slight increases in deflection and curvature during Phase 3. This was not identified from this data; rather, the measurements stayed constant.

Roughness data

Roughness data over the first 11 years of service life on the Graham Farmer Freeway investigation sections was extracted from the IRIS database.

Figure 4.5 shows the roughness progression of each section over time. All sections were under the typical intervention level of 110 counts/km.

Figure 4.5: Graham Farmer Freeway – roughness data



Rutting data

Rutting data for the Graham Farmer Freeway investigation sections was sourced from the IRIS database. The mean rut depth using a 2 m straight edge was reported for both the inner wheelpath (IWP) and the outer wheelpath (OWP). The IWP and OWP rutting data is presented in Figure 4.6 and Figure 4.7 respectively.

For all sections, the IWP rut depth was typically greater than that in the OWP. Rut depths along G1, G2, G3 and G4 were all below 10 mm.

Figure 4.6: Graham Farmer Freeway – rutting data (2 m straight edge, left lane, IWP)

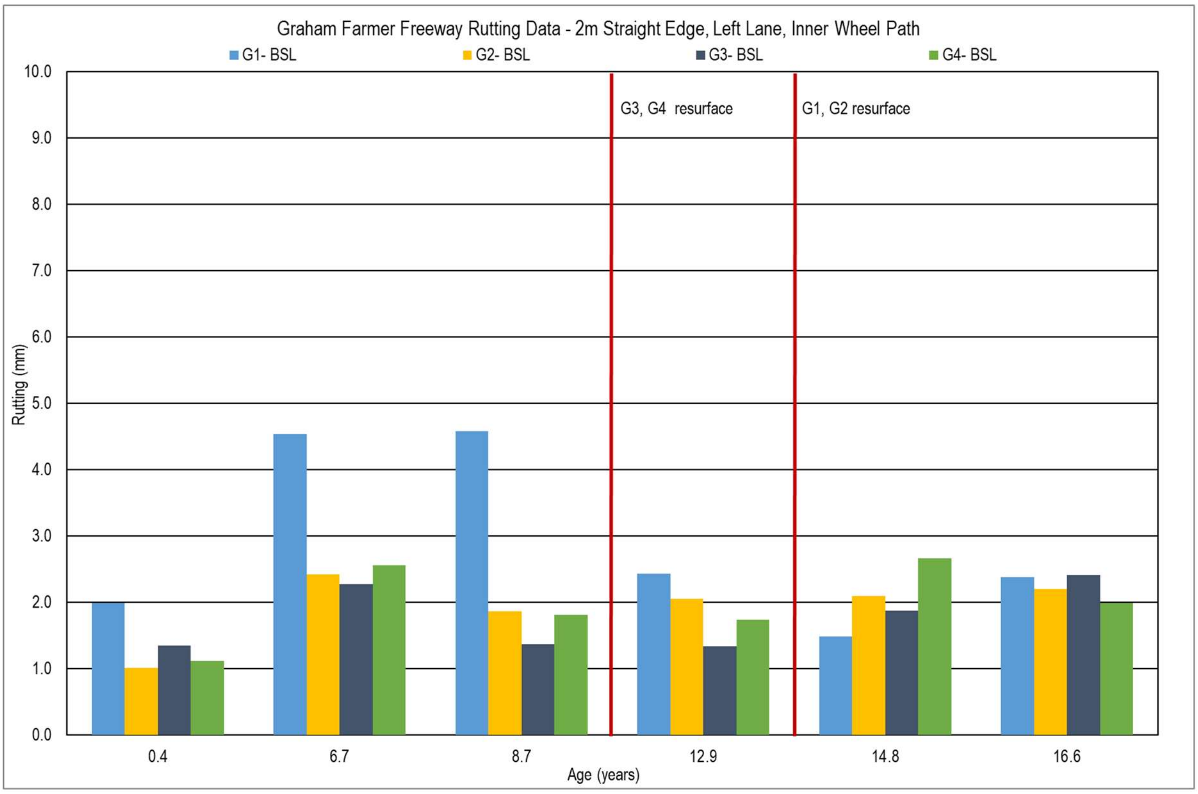


Figure 4.7: Graham Farmer Freeway – rutting data (2 m straight edge, left lane, OWP)

4.3 Kwinana Freeway

4.3.1 Overview

MRWA Contract 140/92 covers chainages 19.08 to 30.77 SLK of the Kwinana Freeway between Beeliar Drive and Thomas Road. It was opened to traffic in September 1994. The investigation sections of Kwinana Freeway are comprised of OGA/DGA surfacing on a CRB, CL subbase and a sand. However, it is important to note that when the sections were constructed in September 1994 the surfacing was originally a sprayed seal; the OGA/DGA surfacing was paved in March 2001. The posted speed limit is 100 km/h.

As-constructed and design thicknesses were available for these sections of interest. Design traffic loading, measured traffic, roughness, rutting and FWD data at both the network and project level were also sourced. As in situ densities and dryback conformance information could not be sourced, it was assumed that non-conformances would have been identified and addressed prior to practical completion.

4.3.2 Sections of Interest

Three sub-sections along the Kwinana Freeway southbound (KF1, KF2 and KF3) were chosen for further investigation based on compliance with the site selection criteria discussed in Section 3. These sections comprise a total length of 3.73 km and relate to the outer, slow lane designated L2 in the southbound direction. Table 4.16 summarises the details of the Kwinana Freeway investigations sections accessed through IRIS on 17 April 2019.

Table 4.16: Details of Kwinana Freeway investigation sections

Section ID	Direction	Start chainage (km)	End chainage (km)	Length (m)	Constructed material type			
					Surfacing[1]	Basecourse	Subbase	Subgrade
KF1	SB	24.45	25.25	800	OGA/DGA	CRB	Crushed limestone	Sand
KF2	SB	25.65	27.65	2000				
KF3	SB	27.72	28.65	930				

Notes:

1 Placed in 2001, 7 years after opening to traffic.

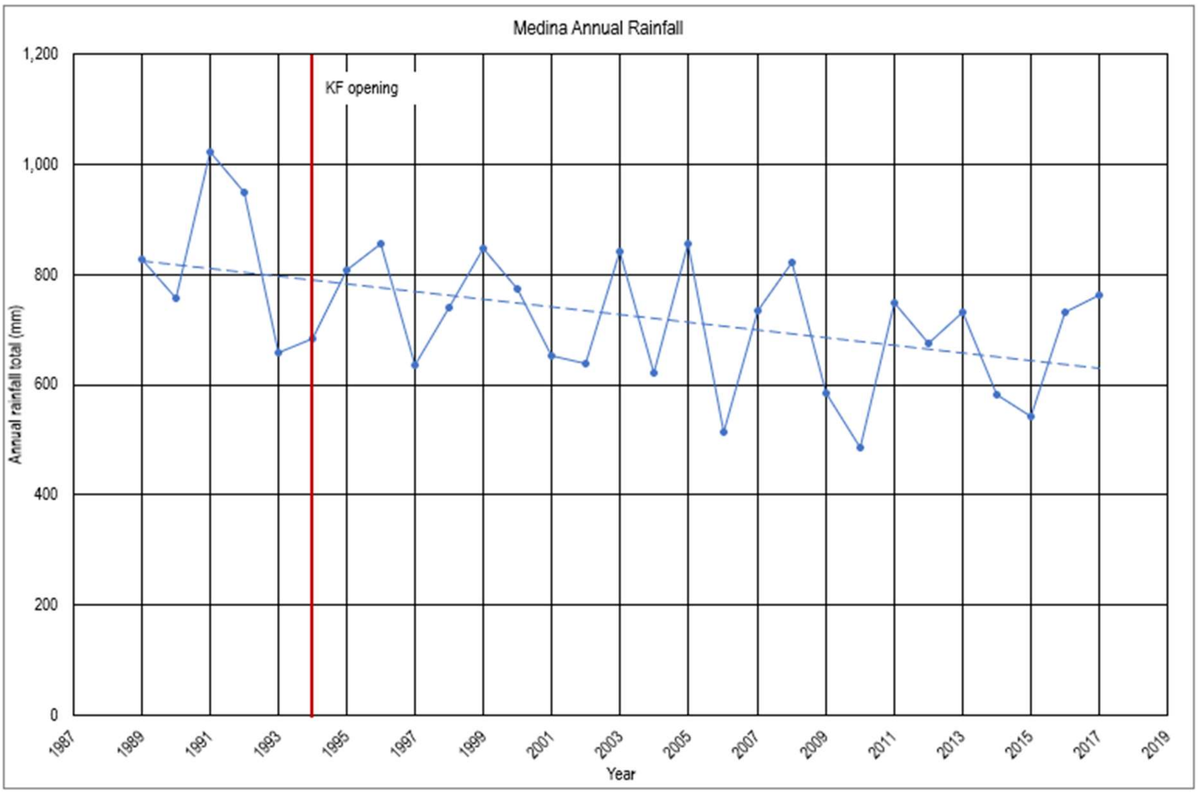
Subsurface Conditions

The 1:250 000 Environmental Geological Map series produced by the Geological Survey of Western Australia Pinjarra sheet (Geological Survey of Western Australia 1980b) indicated that the natural subsurface material underlying the Kwinana Freeway investigation sections was Bassendean dune quartz sands.

Climate Data

The annual average rainfall from the Medina weather station, which is in close vicinity to the Kwinana Freeway investigation sections, was collated to demonstrate the climatic history during the in-service life of the pavements. Data from five years prior to construction through to the most recent data recorded in 2017 was obtained from the Bureau of Meteorology (BOM 2013) and is presented in Figure 4.8. The plot demonstrates a clear downward trend of annual rainfall for this location.

Figure 4.8: Kwinana Freeway – historic annual rainfall data



Source: BOM (2013).

4.3.3 *Design and Construction Data*

Design and construction data for each of the Kwinana Freeway investigation sections was obtained from various sources relating to the MRWA Kwinana Freeway Contract no. 140/92. Pavement dipping depths were not available and specified construction target thicknesses were taken from as-constructed contract drawings. This data is summarised in Table 4.17.

Table 4.17: Kwinana Freeway – design and specified target thicknesses

Identification number	Layer	Material	Design thickness (mm)	Specified target thickness (mm)
KF1, KF2 & KF3	Sprayed seal	Two coat primerseal, 10/5 mm diorite aggregate	–	–
	Basecourse	CRB	170	190
	Subbase	Crushed limestone	200	210
	Subgrade	White sand	–	–

The subgrade design CBR for the Kwinana Freeway sections of interest was documented as 12 %.

4.3.4 *Traffic Loading*

The 40 year design traffic loading (ESAs), and the design traffic growth rate for each of the Kwinana Freeway investigation sections are summarised in Table 4.18. No fatigue design was undertaken for these sections.

Table 4.18: Kwinana Freeway – traffic design data

Section ID	Design traffic (ESAs)	Design period (years)	Design traffic growth rate (%)
KF1	1 x 10^8	40	7.0
KF2			
KF3			

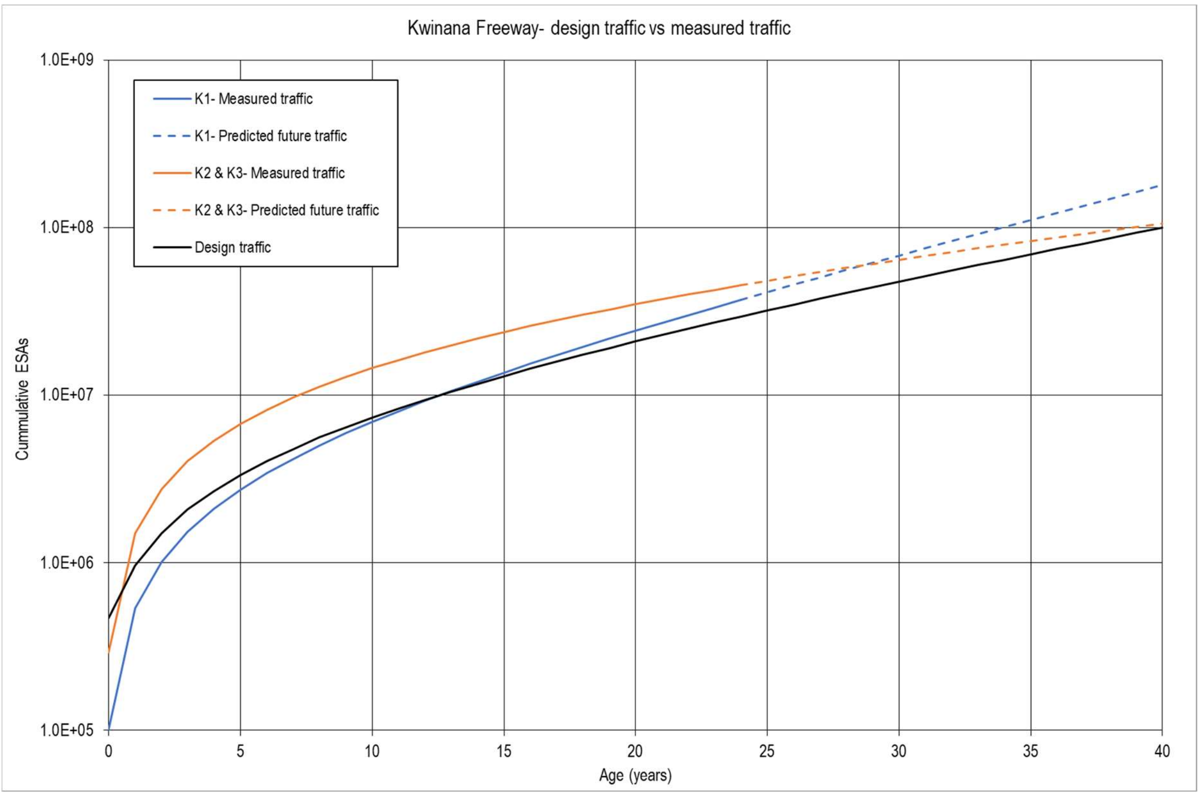
The actual traffic data was extracted from IRIS in April 2019. Using the available measured traffic data, the average annual traffic growth rate was calculated for each investigation section. The first year of traffic loading, and the cumulative number of ESAs from opening up until June 2019, were subsequently estimated using the back-calculated average growth rate and the available traffic data sets. This calculated traffic data is presented in Table 4.19.

Table 4.19: Kwinana Freeway – calculated traffic data

Section ID	First year of traffic data available	Most recent year of traffic data available	Average growth rate (%)	First year traffic (ESAs)	Cumulative traffic from opening to Jun-19 (ESAs)	Pavement age in Jun-19 (years)
KF1	2007	2017	3.3	2.1 x 10^6	6.3 x 10^7	24.8
KF2	2011	2017	3.5	1.5 x 10^6	4.8 x 10^7	
KF3	2011	2017	4.0	1.1 x 10^6	3.8 x 10^7	

The cumulative ESAs for each of the five sections K1, K2, K3, K4 and K5 are shown in Figure 4.9, in addition to the future predicted traffic and the original design traffic.

Figure 4.9: Kwinana Freeway – traffic data



4.3.5 Pavement Maintenance

Pavement maintenance and resurfacing detail data for the Kwinana Freeway investigation sections extracted from the IRIS database on 17 April 2019 indicated that sections KF1 and KF2 had undergone resurfacing in 2015 and KF3 in 2017.

The aerial image review identified the 2001 replacement of the sprayed seal with OGA/DGA surfacing, in addition to works along the entire length of KF1, KF2 and KF3 in 2011. Secondary resurfacing of KF2 was undertaken in early 2016, and of KF1 in early 2017. Roughness and rutting timelines were also used to corroborate the possible works identified by historic images, with a decrease typically demonstrated after works were undertaken.

Table 4.20 presents the rutting and roughness data measured before the resurfacing dates along each of the investigation sections.Considering the data in Table 4.20, the rutting data had no measurements above 10 mm and the roughness data was also well below the typical intervention level of 110 counts/km. This suggests that high roughness or rutting was not the reason for resurfacing.

Table 4.20: Kwinana Freeway – roughness and rutting before resurfacing

Section ID	Resurface	Chainage	Average roughness and survey date (counts/km)	Average rutting and survey date (mm)	
				IWP	OWP
KF1	Apr-11	24.45 – 25.25	20.6 (Dec-07)	2.4 (Dec-09)	1.9
	Feb-17		N/A[1]	2.4 (Nov-16)	1.9
KF2	Apr-11	25.65 – 27.65	23.1 (Dec-07)	2.9 (Dec-09)	1.9
	Apr-16		N/A[1]	3.2 (Dec-14)	1.8
KF3	Feb-11	27.72 – 28.65	29.1 (Dec-07)	4.0 (Dec-09)	2.4

Notes:

1 Roughness data only available up until 2007.

It was noted that a GRS was placed along KF1 and KF2 as part of the 2017 and 2016 resurfacing respectively.

Using the traffic data calculated from measured data sets, the approximate cumulative traffic levels applied to each resurfacing were calculated. These are shown in Table 4.21. The age and cumulative traffic on the sprayed seal has not been included in the resurfacing data and the 1st resurfacing dates were calculated from March 2001.

According to this data, asphalt fatigue cracking was observed along KF1 and KF2, approximately 15 years after the surfacing was first constructed.

Table 4.21: Kwinana Freeway – resurfacing dates

<table>
<tr><th>Section ID</th><th>Date of 1st resurfacing and surfacing age (years)[1]</th><th>Cumulative traffic at resurfacing (ESAs)</th><th>Layer replaced[1]</th><th>Subsequent resurfacing dates and age</th><th>Cumulative traffic at subsequent resurfacings (ESAs)</th><th>Layers Replaced[2]</th></tr>
<tr><td>KF1</td><td>Apr-11 (9.6)</td><td>2.4 x 10^7</td><td rowspan="3">OGA</td><td>Feb-17 (5.8)</td><td>1.8 x 10^7</td><td rowspan="2">OGA, DGA</td></tr>
<tr><td>KF2</td><td>Apr-11 (9.6)</td><td>1.8 x 10^7</td><td>Apr-16 (5.0)</td><td>1.2 x 10^7</td></tr>
<tr><td>KF3</td><td>Feb-11 (9.4)</td><td>1.4 x 10^7</td><td colspan="3">N/A</td></tr>
</table>

Notes:

1 Age begins from placement of OGA/DGA in March 2001.

2 Layer replaced at resurfacing advised by MRWA.

4.3.6 Performance Monitoring

Deflection and curvature

Both project and network level FWD data was available for the Kwinana Freeway sections as detailed in Table 4.22. Full FWD data sets are presented in Appendix A.

Table 4.22: Kwinana Freeway – available FWD data

<table>
<tr><th>Section ID</th><th>Years of network level FWD</th><th>Age at network level FWD</th><th>Years of project level FWD</th><th>Age at network level FWD</th></tr>
<tr><td>KF1</td><td>2006 – 2009 (inclusive)</td><td>12 – 15 years</td><td>2007</td><td>13 years</td></tr>
<tr><td>KF2</td><td>2002 – 2009 (inclusive)</td><td>8 – 15 years</td><td colspan="2">N/A</td></tr>
<tr><td>KF3</td><td>2003 – 2009 (inclusive)</td><td>9 – 15 years</td><td>2007</td><td>13 years</td></tr>
</table>

Deflection results

A summary of the Kwinana Freeway mean deflection data from the network level and project level FWD surveys are presented in Table 4.23 and Table 4.24 respectively. This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth) using the method described in Austroads (2011) assuming the DGA temperature was the surface temperature. The network level data is also presented in Figure 4.10. Deflection trend statistics are presented in Table 4.25.

Table 4.23: Kwinana Freeway – network level maximum deflection

Section ID	2002	2003	2004	2005	2006	2007	2008	2009
KF1	–	–	–	–	0.41	0.40	0.47	0.52
KF2	0.40	0.34	0.35	0.35	0.36	0.37	0.37	0.42
KF3	–	0.43	0.39	0.48	0.44	0.51	0.45	0.54

Table 4.24: Kwinana Freeway – project level maximum deflection

Section ID	2003	2007	2014
KF1	–	0.24	–
KF3	–	0.31	–

Figure 4.10: Kwinana Freeway – Mean maximum deflection

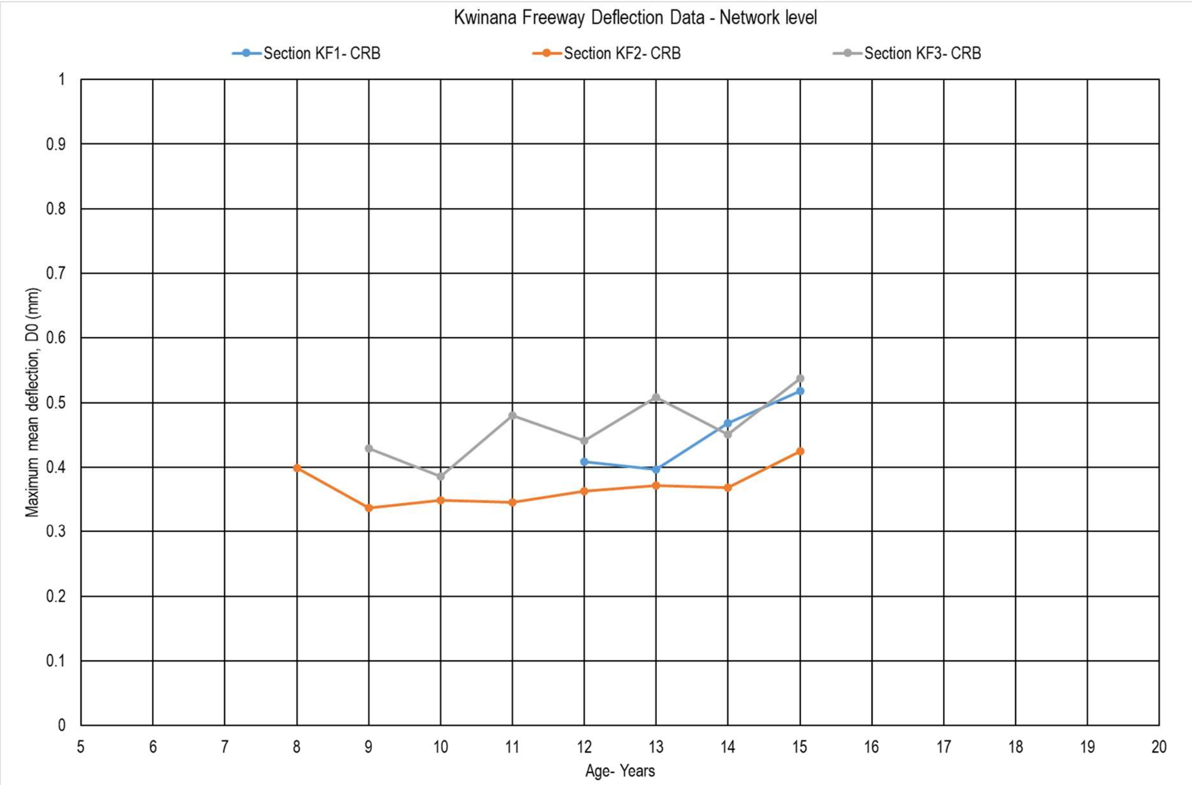


Table 4.25: Kwinana Freeway investigation sections – network level deflection trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 3 (end of 5th year onwards)	KF1	0.8	0.04	0.08
	KF2	0.2	0.01	0.3
	KF3	0.5	0.02	0.06

Curvature results

A summary of the Kwinana Freeway mean curvature data from the network level and project level FWD surveys are presented in Table 4.26 and Table 4.27 respectively. This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth) using the method described in Austroads (2011). The network level data is also presented in Figure 4.11. Curvature trend statistics are presented in Table 4.28.

Table 4.26: Kwinana Freeway – network level curvature

Section ID	2002	2003	2004	2005	2006	2007	2008	2009
KF1	–	–	–	–	0.10	0.10	0.21	0.20
KF2	0.09	0.11	0.13	0.10	0.12	0.12	0.13	0.14
KF3	–	0.09	0.11	0.15	0.14	0.15	0.16	0.18

Table 4.27: Kwinana Freeway – project level curvature

Section ID	2003	2007	2014
KF1	-	0.05	-
KF3	-	0.07	-

Figure 4.11: Kwinana Freeway – curvature

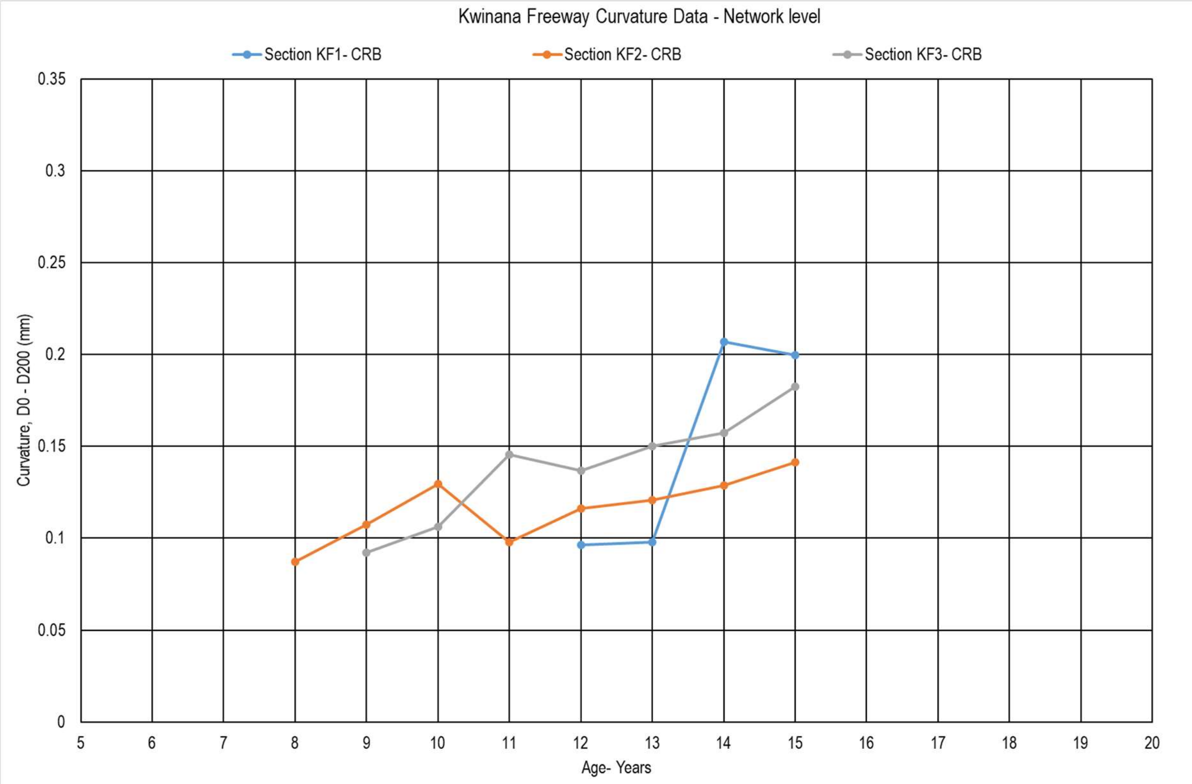


Table 4.28: Kwinana Freeway – network level curvature trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 3 (end of 5th year onwards)	KF1	0.8	0.04	0.1
	KF2	0.6	0.01	0.02
	KF3	0.9	0.01	0.001

Trends in deflection and curvature

Both the deflection and curvature data for the Kwinana Freeway investigation sections typically demonstrated an increase over the seven year monitoring period.

As discussed in Section 2, four similar phases of performance were identified from the deflection and curvature data of the trial sections investigated previously in Stage 2. The deflection and curvature data for the Kwinana Freeway investigation sections cover the proposed third phase of these previously identified performance trends.

From Figure 4.10 and Figure 4.11 and Table 4.25. and Table 4.28, the following trends within these two previously-identified phases can be inferred:

- Phase 3: end of 5th year to end of 15th year
  - Deflection and curvature of all sections show an increase over this period.

This observation is consistent with the trends identified from the Stage 2 trial data for this phase.

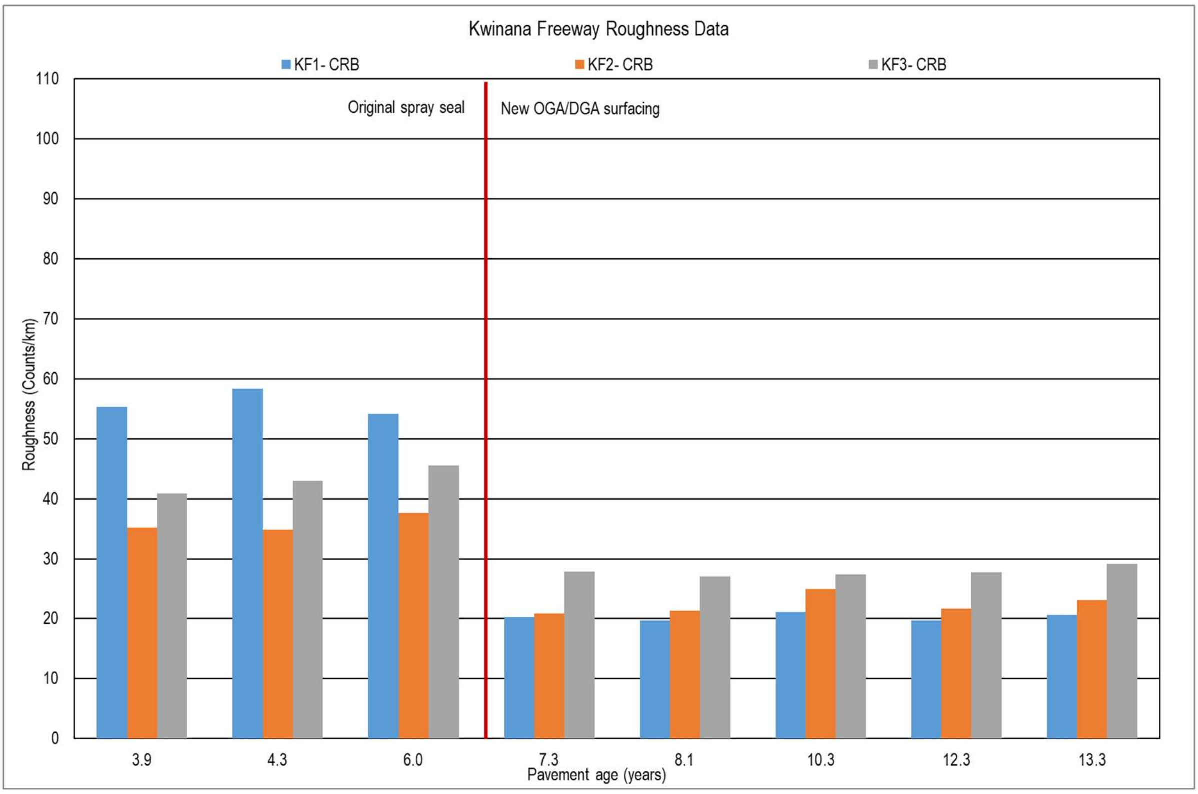
Roughness data

Roughness data over the first 13 years of service life on the Kwinana Freeway investigation sections was extracted from the IRIS database.

Figure 4.12 presents the roughness progression of each section over time. All sections were under the typical intervention level of 110 counts/km.

After the replacement of the sprayed seal with OGA over DGA thin asphalt in 2001, which can be clearly inferred from the roughness progression, roughness measurements along KF1 were consistent with not much change, whilst the other sections continued to increase.

Figure 4.12: Kwinana Freeway – roughness data



Rutting data

Rutting data for the Kwinana Freeway sections was extracted from IRIS. This included a mean value of rutting using the 2 m straight edge for both the IWP and OWP which are shown in Figure 4.13 and Figure 4.14 respectively.

For all sections, the IWP rut depth was typically greater than that in the OWP. Rut depths were all below typical intervention level of 10 mm.

Figure 4.13: Kwinana Freeway – rutting data (2 m straight edge left lane, IWP)

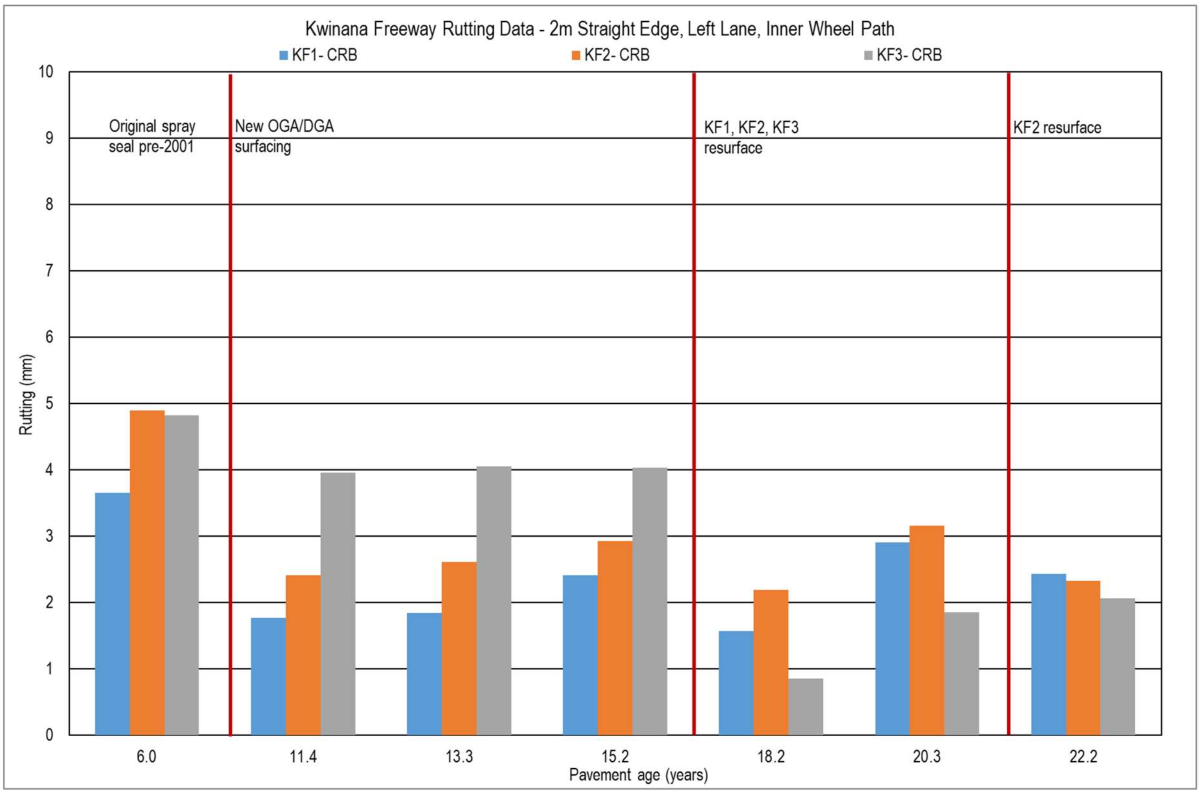
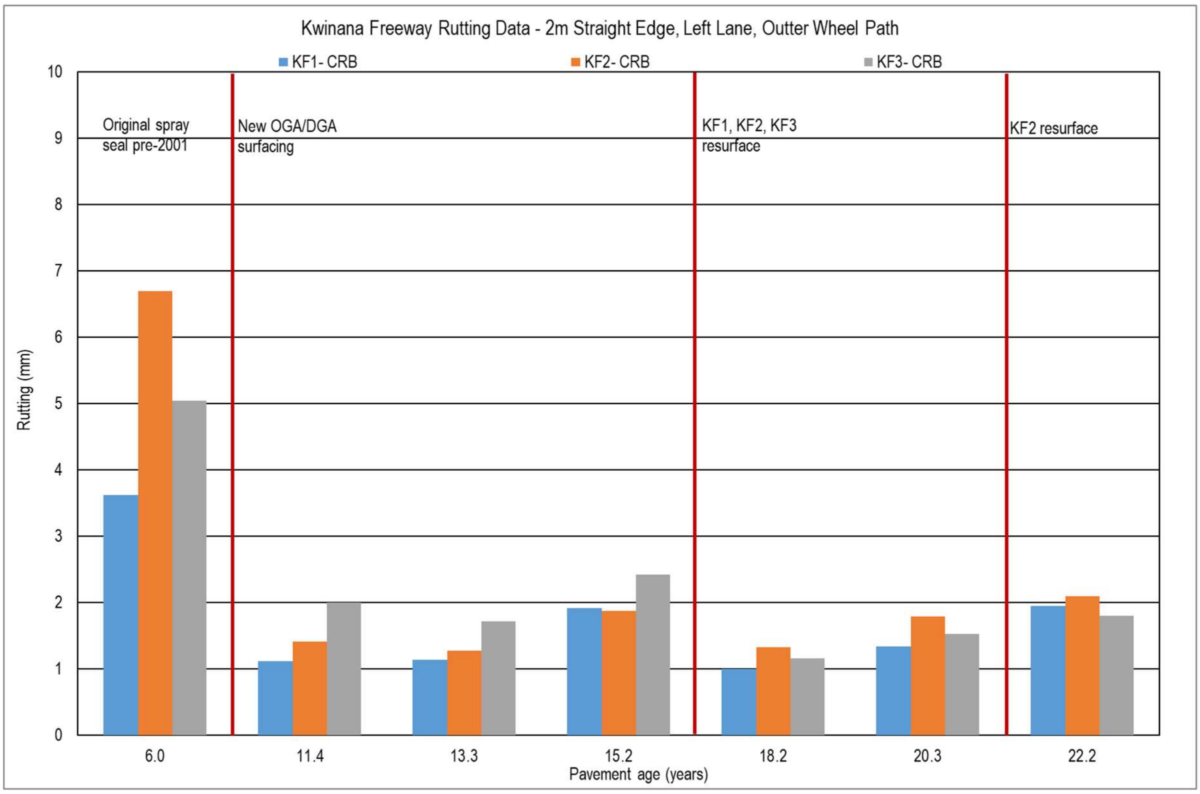


Figure 4.14: Kwinana Freeway – rutting data (2 m straight edge left lane, OWP)



4.4 Mitchell Freeway

4.4.1 Overview

Mitchell Freeway Stage VII covers chainages 18.40 to 22.73 SLK between Hepburn Avenue and Ocean Reef Road. It was opened to traffic in July 1988. The pavement along this section of the Mitchell Freeway comprises an OGA/DGA surfacing on CRB, CL subbase and a sand subgrade.

Stage VIII covers chainages 22.73 to 25.0 SLK between Ocean Reef Road and Hodges Road. It was opened to traffic nine years later in December 1999. The pavement along this section comprises OGA/DGA surfacing on BSL base, crushed limestone subbase and a sand subgrade.

The most recent extension of the Mitchell Freeway between Burns Beach Road and Hester Avenue covers chainages to 29.15 to 35.0 SLK. It was opened to traffic in August 2017. The pavement along this section is of similar composition to Stage VII, comprising an OGA/DGA surfacing on CRB, crushed limestone subbase and sand subgrade.

The posted speed limit along all of these sections of the Mitchell Freeway is 100 km/h.

As-constructed and design thicknesses were available for these sections of interest. Design modulus assumed for the ME design, Design traffic, measured traffic, roughness, rutting and FWD data at both network and project level were also sourced. In situ densities and dryback conformance information could not be sourced. However, it was assumed that non-conformances would have been identified and addressed prior to practical completion.

4.4.2 Sections of Interest

Four sections of the Mitchell Freeway within the chainages listed previously were chosen for further investigation based on compliance with the site selection criteria discussed in Section 3 These investigation sections include six southbound sections (M1, M2 and M3) and one northbound section (M4). The lanes of interest are R1 in the southbound direction and L1 in the northbound direction. Table 4.29 summarises the details of the Mitchell Freeway investigations sections accessed through IRIS on 11 October 2018.

Table 4.29: Details of Mitchell Freeway investigation sections

Section ID	Direction	Start chainage (km)	End chainage (km)	Length (m)	Constructed material type			
					Surfacing	Basecourse	Subbase	Subgrade
M1	SB	19.00	19.60	600	OGA/DGA	CRB	Crushed limestone	Sand
M2	SB	21.00	22.40	1400				
M3	SB	23.60	24.94	1340		BSL		
M4	NB	33.00	33.98	980		CRB		

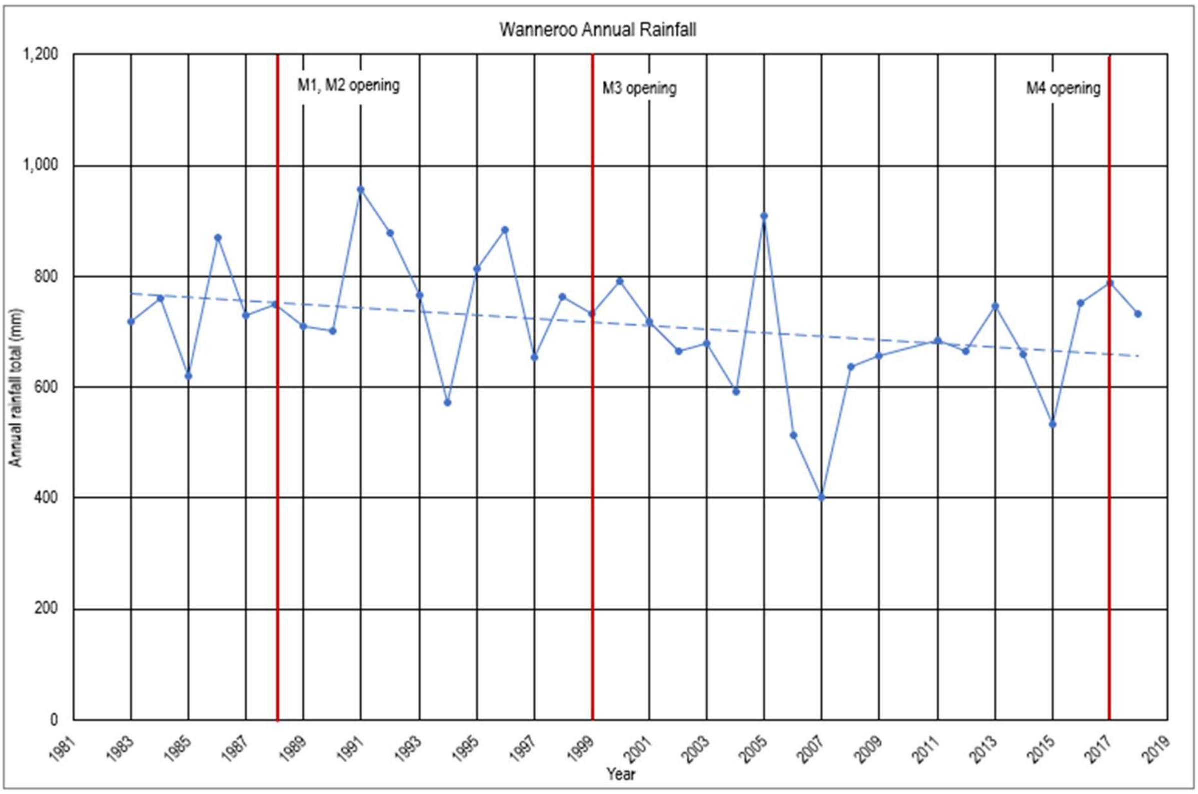
Subsurface Conditions

The 1:250 000 Environmental Geological Map series produced by the Geological Survey of Western Australia Perth sheet (Geological Survey of Western Australia 1980a) indicates that the natural subsurface material comprises quartz sands derived from coastal Tamala limestone underlying M1, and calcarenite underlying M2, M3, and M4.

Climate Data

Annual rainfall averages were obtained from Wanneroo weather station, located approximately 3.5 km from the Mitchell Freeway investigation sections, was used to demonstrate the climatic history of the area. Data from five years prior to construction through to 2018 obtained from the Bureau of Meteorology (BOM 2013) is presented in Figure 4.15. The plot demonstrates a clear downward trend of annual rainfall for this location.

Figure 4.15: Mitchell Freeway – historic annual rainfall data



Source: BOM (2013).

4.4.3 Design and Construction Data

Design data for the Mitchell Freeway sections of interest was obtained from various sources relating to the MRWA contracts. Specific pavement dipping depths were not available and specified construction target thicknesses were taken from as-constructed contract drawings. This data is summarised in Table 4.30.

The subgrade design California Bearing Ratio (CBR) for the Mitchell Freeway sections of interest was documented as 15 % for sections M1, M2, and M3, and 12% for sections M4.

Original design moduli were only available for section M4 and are summarised in Table 4.31.

Table 4.30: Mitchell Freeway – design and specified target thicknesses

Identification number	Layer	Material	Design thickness (mm)	Specified target thickness (mm)
M1 and M2	Asphalt	10 mm OGA	30	30
		10 mm DGA	30	30
	Basecourse	CRB	75	75
	Subbase	Crushed limestone	200	200
	Subgrade	Sand	–	–
M3	Asphalt	10 mm OGA	30	30
		10 mm DGA	30	30
	Basecourse	2% BSL	110	110
	Subbase	Crushed limestone	170	170
	Subgrade	Sand	–	–
M4	Asphalt	10 mm OGA	30	30
		10 mm DGA	30	30
	Basecourse	CRB	200	250
	Subbase	Crushed limestone	230	180
	Subgrade	Sand	–	–

Table 4.31: Mitchell Freeway – design moduli M4

Identification number	Material	Design moduli (MPa)
M4	10 mm OGA	2500
	10 mm DGA	2720
	CRB	600
	Crushed limestone	–
	White sand	120

4.4.4 Traffic Loading

The design traffic loading (ESAs) and the design traffic growth rate for each of the Mitchell Freeway investigation sections is summarised in Table 4.32.

Table 4.32: Mitchell Freeway – traffic design data

Section ID	Design traffic (ESAs)	Design period (years)	Design traffic growth rate (%)	Asphalt fatigue design traffic (ESAs)
M1	6.6×10^6	20	3.0	N/A
M2				
M3	3.0×10^7	40	3.0	
M4	2.7×10^7		2.4	2.2×10^6

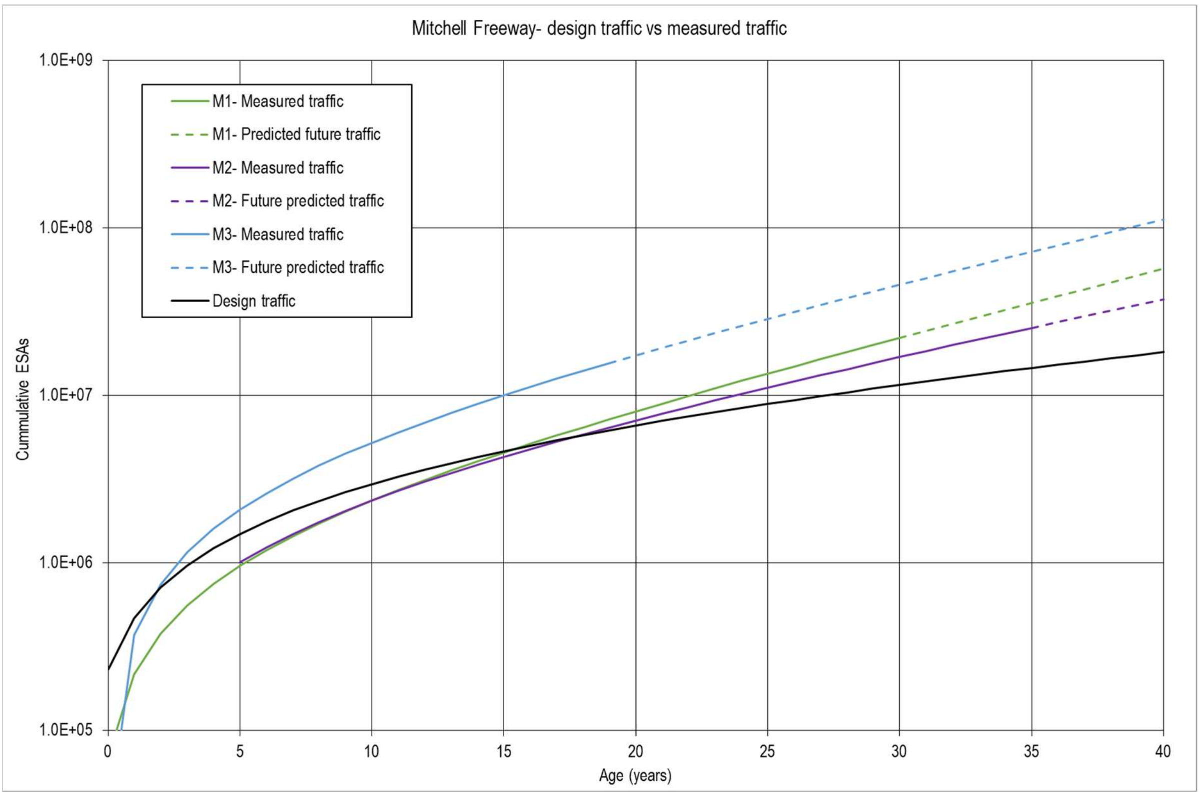
The Mitchell Freeway traffic data was extracted from the MRWA IRIS database in April 2019. As discussed previously, the average annual traffic growth rate was calculated for each investigation section. The first year of traffic loading in addition to the cumulative number of ESAs from opening until June 2019 were subsequently estimated using the back-calculated average growth rate and the available traffic data sets. There is currently no data available for section M4. The calculated traffic data is presented in Table 4.33.

Table 4.33: Mitchell Freeway – calculated traffic data

Section ID	First year of traffic data available	Most recent year of traffic data available	Average growth rate (%)	First year traffic (ESAs)	Cumulative traffic up to Jun-19 (ESAs)	Pavement age in Jun-19 (years)
M1	2007	2014	9.5	2.2×10^5	2.4×10^7	31.0
M2	2006	2017	7.6	2.4×10^5	1.8×10^7	31.0
M3	2011	2014	8.9	3.7×10^5	1.7×10^7	19.6

The cumulative ESAs for each of the three sections M1, M2 and M3 are shown in Figure 4.16, in addition to the future predicted traffic and the original design traffic.

Figure 4.16: Mitchell Freeway – traffic data



4.4.5 Pavement Maintenance

Pavement maintenance and resurfacing detail data for the Mitchell Freeway investigation sections extracted from the IRIS database in April 2019 indicated that section M1 was resurfaced in 2006. No other information was available.

The aerial image review identified the 2006 resurfacing of M1 as indicated in IRIS in addition to a resurfacing in early 2010 of M2. No resurfacing works were identified along M3 or M4. Roughness and rutting timelines were also used to corroborate the possible works identified by historic images, with a decrease typically demonstrated after works are undertaken.

Table 4.34 presents the rutting and roughness data measured before the resurfacing dates along M1 and M2.

Considering the data in Table 4.34, the rutting data had no measurements above 10 mm and the roughness data was also well below typical intervention level of 110 counts/km. This suggests that high roughness or rutting was not the reason for resurfacing.

Table 4.34: Mitchell Freeway – roughness and rutting before resurfacing

Section ID	Resurface date	Chainage	Average roughness and survey date (counts/km)	Average rutting and survey date (mm)	
				IWP	OWP
M1	Mar-06	19.00 – 19.60	43.4 (Feb-06)	4.2 (Oct-00)	2.2
M2	Jan-10	21.00 – 22.40	35.5 (Nov-09)	1.4 (Nov-09)	2.0

Using the traffic data calculated from measured data sets, the approximate cumulative traffic levels applied to each resurfacing was calculated. These are shown in Table 4.35.

According to this data, asphalt fatigue has not yet been observed along these sections.

Table 4.35: Mitchell Freeway – resurfacing dates

Section ID	Date of 1st resurfacing and surfacing age (years)	Cumulative traffic at resurfacing (ESAs)	Layer replaced[1]	Subsequent resurfacing dates and age	Cumulative traffic at subsequent resurfacings (ESAs)
M1	Mar-06 (17.7)	6.2×10^6	OGA	N/A	N/A
M2	Jan-10 (21.5)	8.3×10^6			
M3	N/A				
M4	N/A				

Notes:

1 Layer replaced at resurfacing advised by MRWA.

4.4.6 *Performance Monitoring*

Deflection and curvature

Both project and network level FWD data was available for the Mitchell Freeway sections as detailed in Table 4.36. Full FWD data sets are presented in Appendix A.

Table 4.36: Mitchell Freeway investigation sections – available FWD data

Section ID	Years of network level FWD	Age at network level FWD	Years of project level FWD	Age at network level FWD
MF1	2002 – 2009	14 – 21 years	N/A	
MF2				
MF3		3 – 10 years		
MF4	N/A		Aug-17, May-18	Pre-traffic (0), 0.8 months

Deflection results

A summary of the Mitchell Freeway mean deflection data from the network level and project level FWD surveys are presented in Table 4.37 and Table 4.38 respectively. This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth). The network level data is also presented in Figure 4.17. Deflection trend statistics are presented in Table 4.39.

Table 4.37: Mitchell Freeway – network level maximum deflection

Section ID	2002	2003	2004	2005	2006	2007	2008	2009
MF1	0.25	0.35	0.24	0.26	0.25	0.27	0.29	0.30
MF2	0.18	0.21	0.24	0.33	0.20	0.29	0.35	0.44
MF3	0.39	0.36	0.27	0.31	0.34	0.29	0.28	0.34

Table 4.38: Mitchell Freeway – project level maximum deflection

Section ID	2017	2018
MF4	0.39	0.29

Figure 4.17: Mitchell Freeway – Mean maximum deflection

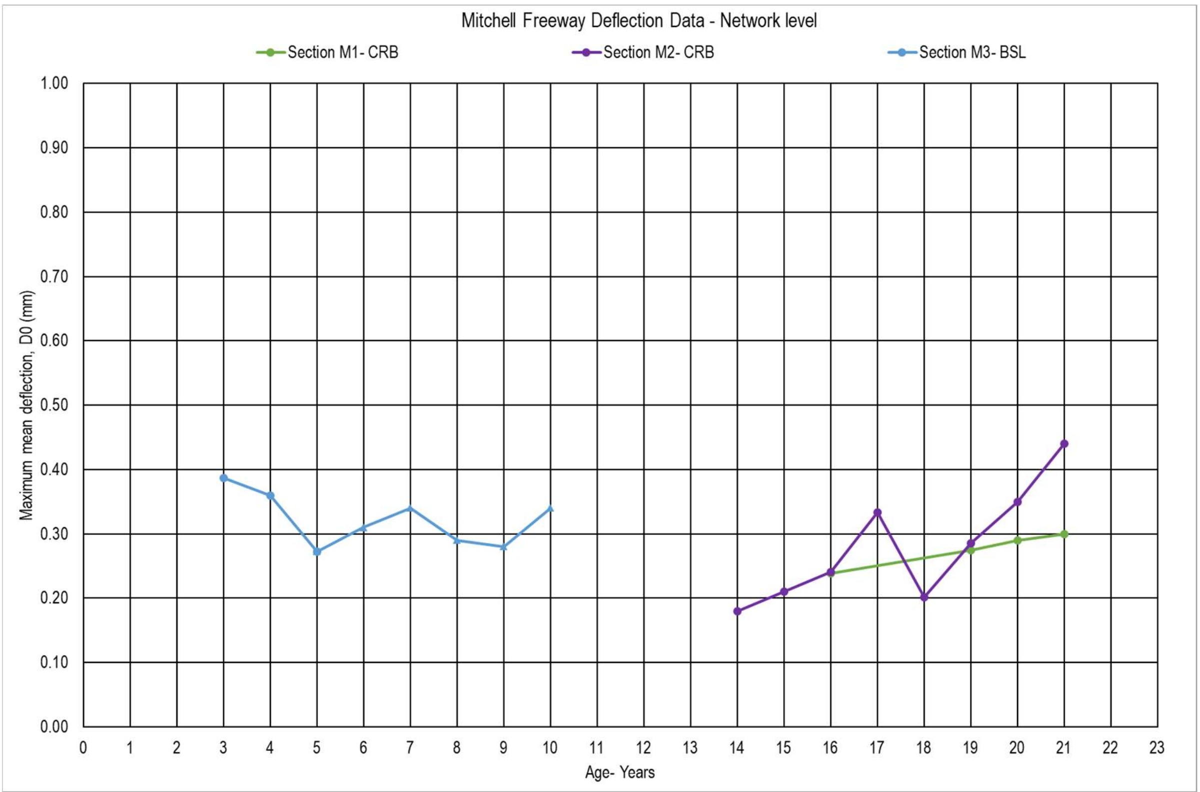


Table 4.39: Mitchell Freeway – network level deflection trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 2 (up to 5th year)	M3	0.9	-0.06	0.18
Phase 3 (5th year to 15th)		0.001	-0.001	1.0
Phase 4 (15th year onwards)	M1	1.0	0.01	0.001
	M2	0.7	0.03	0.01

Curvature results

A summary of the Mitchell Freeway mean curvature data from the network level and project level FWD surveys is presented in Table 4.40 and Table 4.41 respectively. This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth). This network level data is also presented in Figure 4.18. Curvature trend statistics are presented in Table 4.42

Table 4.40: Mitchell Freeway – network level curvature

Section ID	2002	2003	2004	2005	2006	2007	2008	2009
MF1	0.08	0.11	0.09	0.09	0.09	0.10	0.10	0.09
MF2	0.07	0.08	0.08	0.09	0.07	0.10	0.12	0.13
MF3	0.14	0.11	0.10	0.10	0.10	0.10	0.09	0.09

Table 4.41: Mitchell Freeway – project level curvature

Section ID	2017	2018
MF4	0.14	0.10

Figure 4.18: Mitchell Freeway – curvature

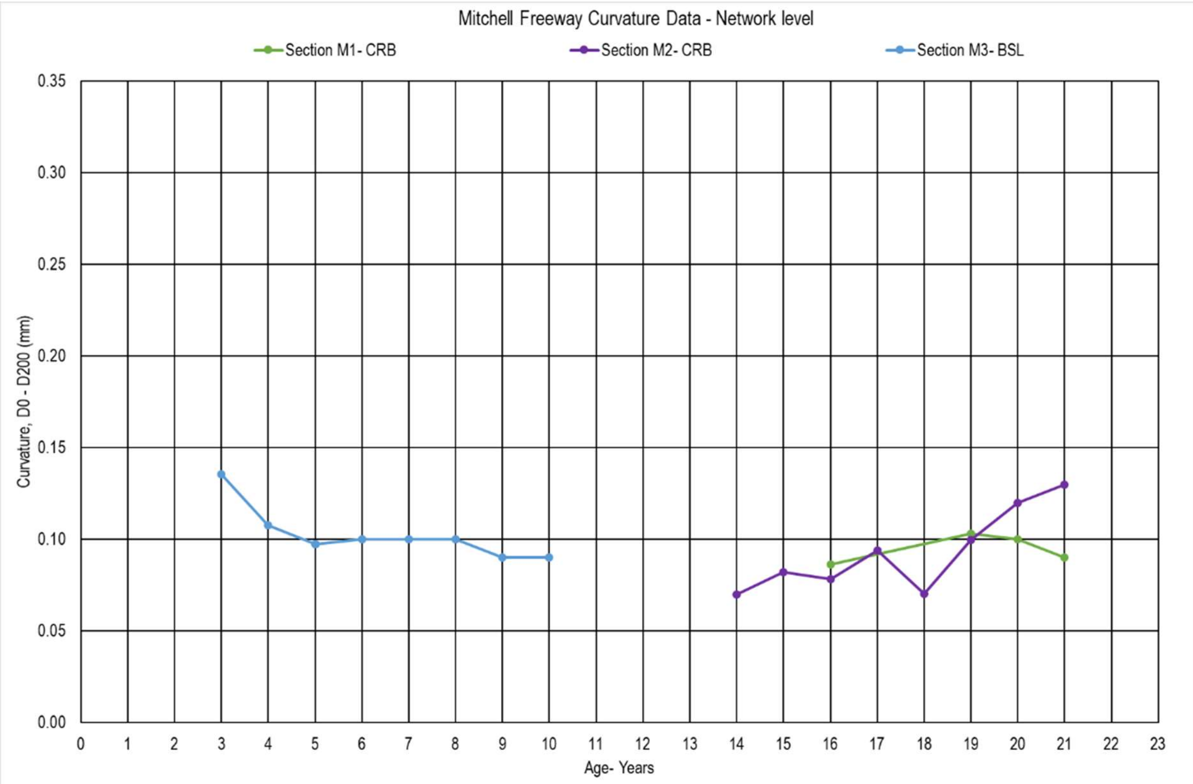


Table 4.42: Mitchell Freeway – network level curvature trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 2 (up to 5th year)	M3	0.9	–0.02	0.2
Phase 3 (5th year to 15th)		0.5	–0.002	0.2
Phase 4 (15th year onwards)	M1	0.0	0.001	0.8
	M2	0.7	0.01	0.01

Trends in deflection and curvature

As discussed in Section 2, four similar phases of performance were identified from the deflection and curvature data of the trial sections investigated previously in Stage 2. The deflection and curvature data for the Mitchell Freeway investigation sections covers the proposed first phase (M4), second phase (M3), third phase (M3) and fourth phase (M1 and M2) of these previously-identified performance trends.

From Figure 4.17 and Figure 4.18 and Table 4.39 and Table 4.42, the following trends within these four previously-identified phases can be inferred as follows:

- Phase 1: Pre-traffic to end of 1st year
  - Deflection and curvature decrease below the pre-traffic measurement, but data was limited to Section M4 (Table 4.38 and Table 4.41).
- Phase 2: end of first year to around the end of the 5th year:
  - Deflection and curvature did not show a strong increase or decrease, being typically consistent throughout this period. Data was limited to Section M3.
- Phase 3: end of fifth year to end of 15th year:
  - Deflection and curvature did not show a strong increase or decrease and were typically consistent throughout this period. Again data was limited to Section M3.
- Phase 4: end of 15th year onwards
  - Deflection and curvature typically increased with the exception of the curvature along section M1 which remained constant.

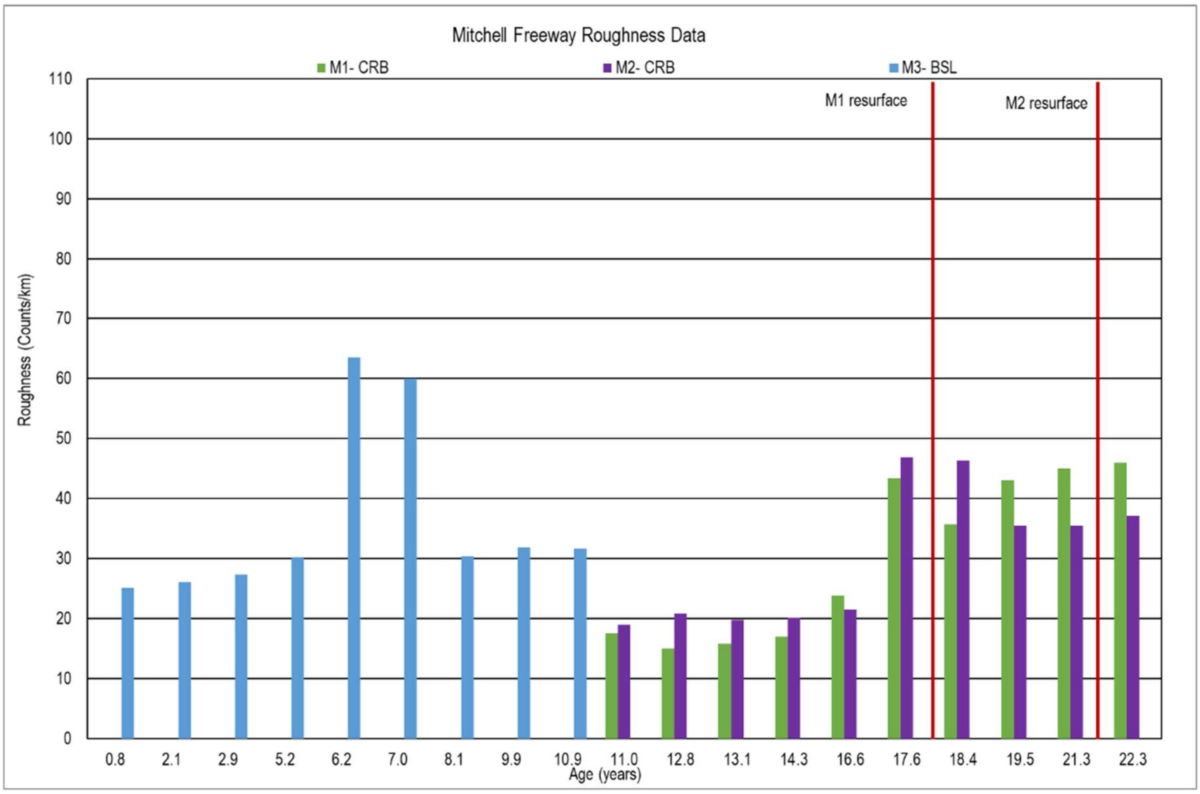
These observations are typically consistent with the trends identified from the Stage 2 trial data for these phases.

Roughness data

Roughness data over the Mitchell Freeway investigation sections was extracted from the IRIS database. Figure 4.19 presents the roughness progression of each section over time. No roughness data was available for M4. All investigation sections demonstrated an increase in roughness over time, with all sections having similar levels of roughness.

A large increase is seen in the roughness of M3 at 6.2 and 7.0 years. These ages correspond to data collected in February 2006 and December 2006 and are inferred as outlying values.

Figure 4.19: Mitchell Freeway – roughness data



Rutting data

Rutting data for the Mitchell Freeway sections was extracted from IRIS. This included a mean value of rutting using the 2 m straight edge for both the IWP and OWP which are shown in Figure 4.20 and Figure 4.21 respectively. No rutting data was available for section M4.

For all sections, the IWP rut depth was typically greater than that in the OWP. The progression of rut depth was less obvious for the Mitchell Freeway sections with values varying throughout the service life. Confirmed resurfacing works in 2006 and 2010 along M1 and M2 respectively can be inferred from the rut data; however, the rut data suggests other resurfacing works which were not identified by IRIS or by aerial image review, suggesting incorrect rut values.

Figure 4.20: Mitchell Freeway – rutting data (2 m straight edge, left lane, IWP)

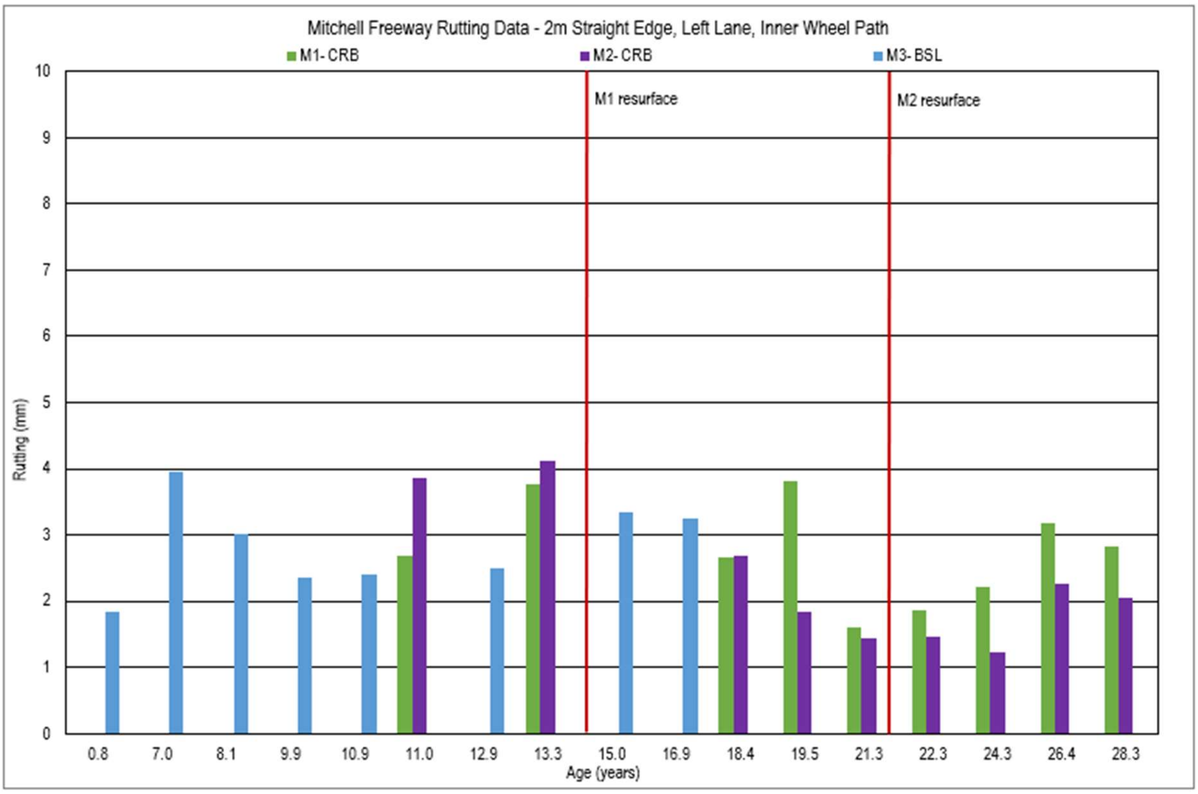
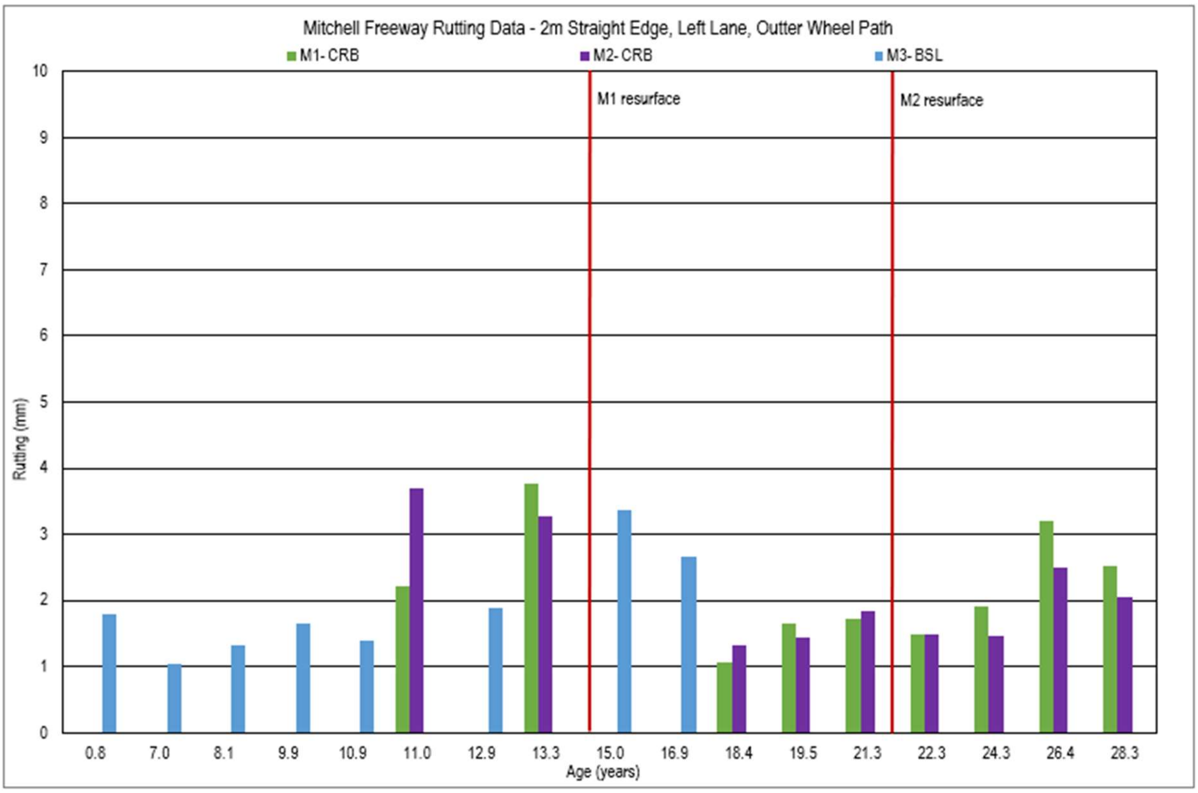


Figure 4.21: Mitchell Freeway – rutting data (2 m straight edge, left lane, OWP)



4.5 Reid Highway

4.5.1 Overview

MRWA Contract 118/92 covers the Reid Highway investigation sections RH1 and RH2, which were both opened to traffic in November 1994. The pavement configuration of both sections comprises a Perth sand subgrade, CL subbase and a CRB basecourse surfaced with a thin layer of DGA. The posted speed limit is 90 km/h.

As-constructed and design thicknesses were available for these sections of interest, in addition to roughness, rutting data and FWD data at the network level. Density and dryback conformance information could not be sourced for the sections of interest. However, it may be assumed that non-conformances would have been identified and addressed prior to practical completion.

4.5.2 Sections of Interest

Based on the availability of performance data, two sections along the Reid Highway were selected for analysis comprising a total of 2.42 km. The lanes of interest are L1 in the eastbound direction and R1 in the westbound direction. Table 4.43 summarises the details of the Reid Highway investigations sections accessed through IRIS on 11 October 2018.

Table 4.43: Details of Reid Highway investigation sections

Section ID	Direction	Start chainage (km)	End chainage (km)	Length (m)	Constructed material type			
					Surfacing	Basecourse	Subbase	Subgrade
RH1	EB	6.45	7.71	1260	DGA	CRB	Crushed limestone	Sand
RH2	WB	6.45	7.61	1160				

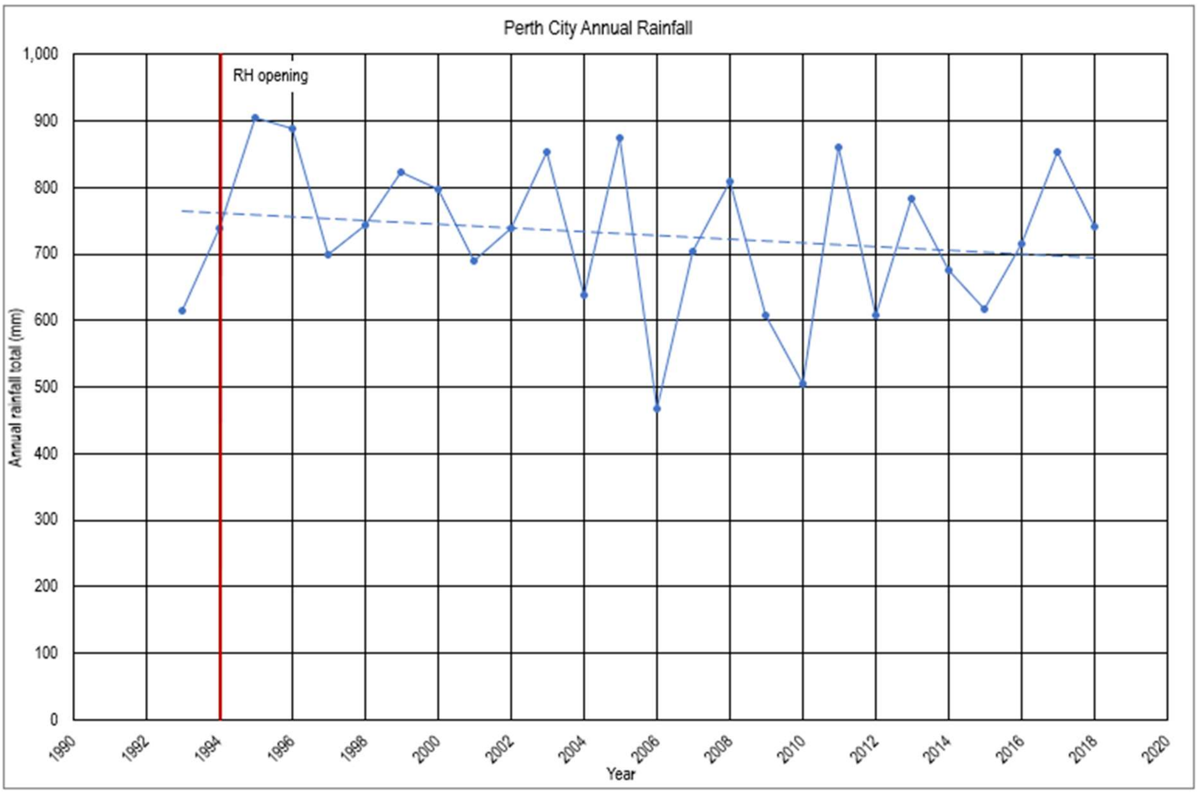
Subsurface Conditions

The 1:250 000 Environmental Geological Map series produced by the Geological Survey of Western Australia Perth sheet (Geological Survey of Western Australia 1980a) indicates that the natural subsurface material underlying the investigation sections comprises quartz sands derived from coastal Tamala limestone.

Climate Data

The annual average rainfall from the Perth City weather station, which is the closest weather station to the Reid Highway investigation sections, was collated to demonstrate the climatic history during the in-service life of the pavements. Data from one year prior to construction through to 2017 obtained from the Bureau of Meteorology (BOM 2013) is presented in Figure 4.22. The plot demonstrates a clear downward trend of annual rainfall for this location.

Figure 4.22: Reid Highway – historic annual rainfall data



Source: BOM (2013).

4.5.3 Design and Construction Data

Design data for the Reid Highway investigation sections was sourced from various documents relating to MRWA Contract 118/92. Specific pavement dipping depths were not available and specified construction target thicknesses were taken from as constructed contract drawings. This data is summarised in Table 4.44.

It is postulated that, due to the age of the pavement, granular layer thicknesses were determined using design charts and as such, design moduli for ME design is not available.

Table 4.44: Reid Highway – design and specified target thicknesses

Identification number	Layer	Material	Design thickness (mm)	Specified target thickness (mm)
RH1	Asphalt	10 mm DGA	30	30
	Basecourse	CRB	175	170
	Subbase	Crushed limestone	200	200
	Subgrade	White sand	–	–
RH2	Asphalt	10 mm DGA	30	30
	Basecourse	CRB	175	170
	Subbase	Crushed limestone	200	200
	Subgrade	White sand	–	–

The subgrade design CBR for the Reid Highway sections of interest was documented as 12%.

4.5.4 Traffic Loading

The design traffic loading (ESAs) and the design traffic growth rate for each of the Reid Highway investigation sections is summarised in Table 4.45.

Table 4.45: Reid Highway – traffic design data

Section ID	Design traffic (ESAs)	Design period (years)	Design traffic growth rate (%)	Percentage of heavy vehicles (%)
RH1	1.3×10^8	40	2.1	11
RH2				

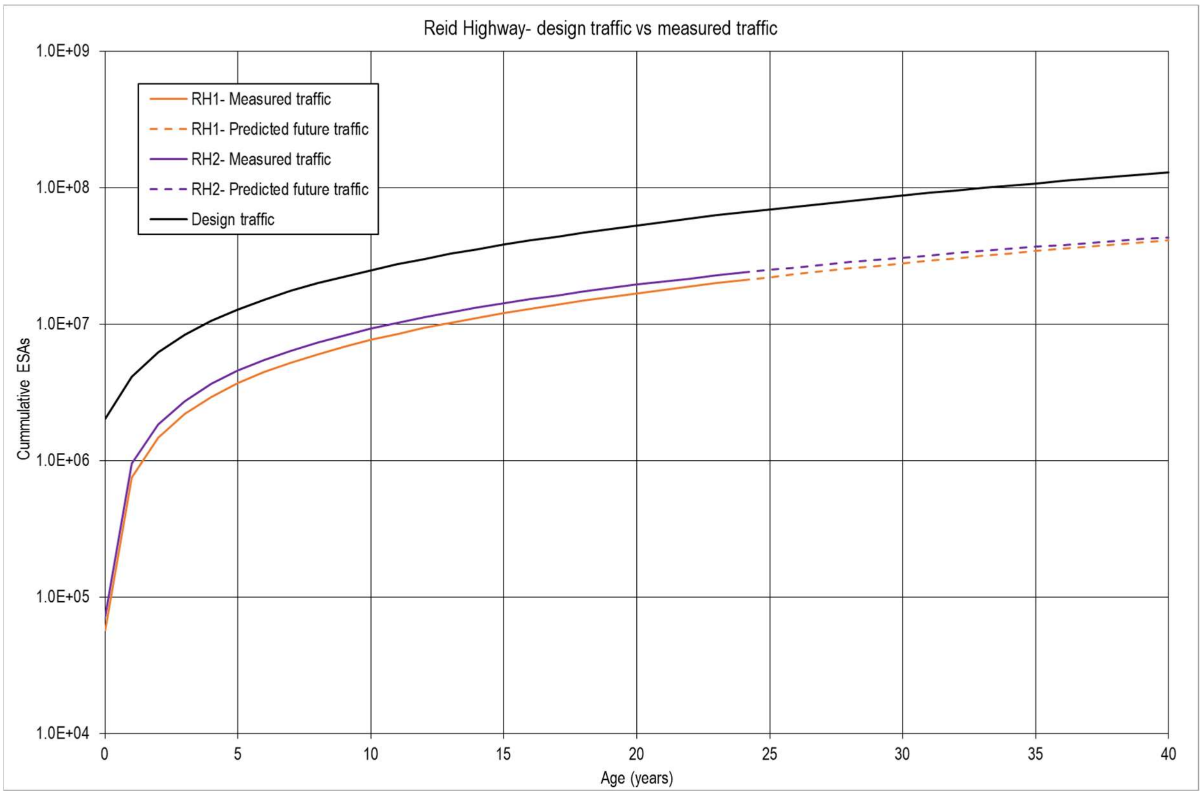
The Reid Highway traffic data was extracted from the IRIS database in April 2019. Using the available measured traffic data, the average annual traffic growth rate was calculated for each investigation section. The first year of traffic loading and the cumulative number of ESAs from opening until June 2019 were subsequently estimated using the back-calculated average growth rate and the available traffic data sets. This calculated traffic data is presented in Table 4.46.

Table 4.46: Reid Highway – calculated traffic data

Section ID	First year of traffic data available	Most recent year of traffic data available	Average growth rate (%)	First year traffic (ESAs)	Cumulative traffic from opening to Jun-19 (ESAs)	Pavement age in Jun-19 (years)
RH1	2003	2018	1.9	7.5×10^5	2.2×10^7	24.6
RH2	2004	2018	1.0	9.5×10^5	2.4×10^7	24.6

The cumulative ESAs for each of the two sections RH1 and RH2 are shown in Figure 4.23, in addition to the future predicted traffic and the original design traffic.

Figure 4.23: Reid Highway – traffic data



4.5.5 *Pavement Maintenance*

Pavement maintenance and resurfacing detail data for the Reid Highway investigation sections extracted from the IRIS database in April 2019 indicated that section RH2 had been resurfaced in early 2019. IRIS data extracted in August 2018 indicated no resurfacings since construction, indicating that the 2019 resurfacing was the first resurfacing works since construction.

The aerial image review also did not identify resurfacing works along RH1 or RH2 prior to 2019. This was supported by the roughness and rutting timelines.

Table 4.47 presents the rutting and roughness data measured before the resurfacing dates along RH2.

Considering the data in Table 4.47, the rutting data had no measurements above 10 mm, suggesting that rutting was not the reason for resurfacing. The most recent roughness data available was 2010 which was also well below the typical intervention level of 110 counts/km. Furthermore, roughness progression along RH2 was historically very low, suggesting that high roughness was also not the reason for resurfacing.

Table 4.47: Reid Highway – roughness and rutting before resurfacing

Section ID	Resurface date	Chainage	Average roughness and survey date (counts/km)	Average rutting and survey date (mm)	
				IWP	OWP
RH2	Mar-19	6.45 – 7.61	31.9 (Nov-10)	3.6 (Nov-16)	2.6

Using the traffic data calculated from the measured data sets, the approximate cumulative traffic levels applied to each resurfacing are shown in Table 4.48.

According to this data, asphalt fatigue was observed along RH2, approximately 24 years after the surfacing was first constructed.

Table 4.48: Reid Highway – resurfacing dates

Section ID	Date of 1st resurfacing and surfacing age (years)	Cumulative traffic at resurfacing (ESAs)	Layer replaced	Subsequent resurfacing dates and age	Cumulative traffic at subsequent resurfacings (ESAs)
RH1	N/A			N/A	N/A
RH2	Mar-19 (24.3)	2.4×10^7	DGA		

4.5.6 *Performance Monitoring*

Deflection and curvature

Only network level FWD data was available for the Reid Highway sections as detailed in Table 4.49. Full FWD data sets are presented in Appendix A.

Table 4.49: Reid Highway – available FWD data

Section ID	Years of network level FWD	Age at network level FWD	Years of Project level FWD	Age at network level FWD
RH1	2002 – 2010 (inclusive)	8 – 16 years	N/A	
RH2				

Deflection results

A summary of the Reid Highway mean deflection data from the network level FWD surveys is presented in Table 4.50. This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth) using the method described in Austroads (2011). This data is also presented in Figure 4.24. Deflection trend statistics are presented in Table 4.51.

Table 4.50: Reid Highway – network level maximum deflection

Section ID	2002	2003	2004	2005	2006	2007	2008	2009	2010
RH1	0.35	0.36	0.29	0.32	0.38	0.28	0.35	0.42	0.41
RH2	0.41	0.44	0.34	0.37	0.39	0.43	0.40	0.40	0.31

Figure 4.24: Reid Highway – Mean maximum deflection

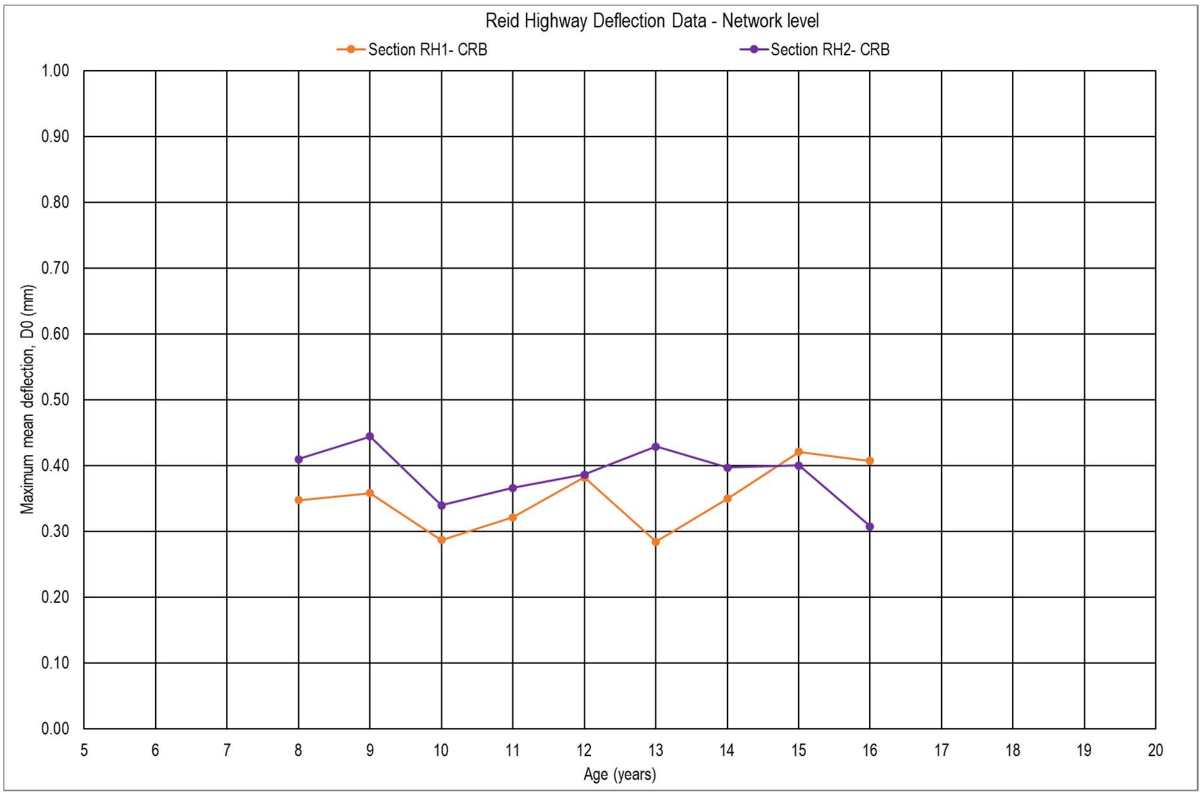


Table 4.51: Reid Highway investigation sections – network level deflection trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 3 (5th year to 15th year)	RH1	0.2	0.01	0.2
	RH2	0.1	-0.01	0.3

Curvature results

A summary of the Reid Highway mean curvature data from the network level FWD surveys is presented in Table 4.52. This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth) using the method described in Austroads (2011). This data is also presented in Figure 4.25. Curvature trend statistics are presented in Table 4.53.

Table 4.52: Reid Highway – network level curvature

Section ID	2002	2003	2004	2005	2006	2007	2008	2009	2010
RH1	0.12	0.13	0.10	0.10	0.13	0.11	0.13	0.13	0.13
RH2	0.14	0.15	0.11	0.11	0.13	0.15	0.12	0.12	0.11

Figure 4.25: Reid Highway – curvature data

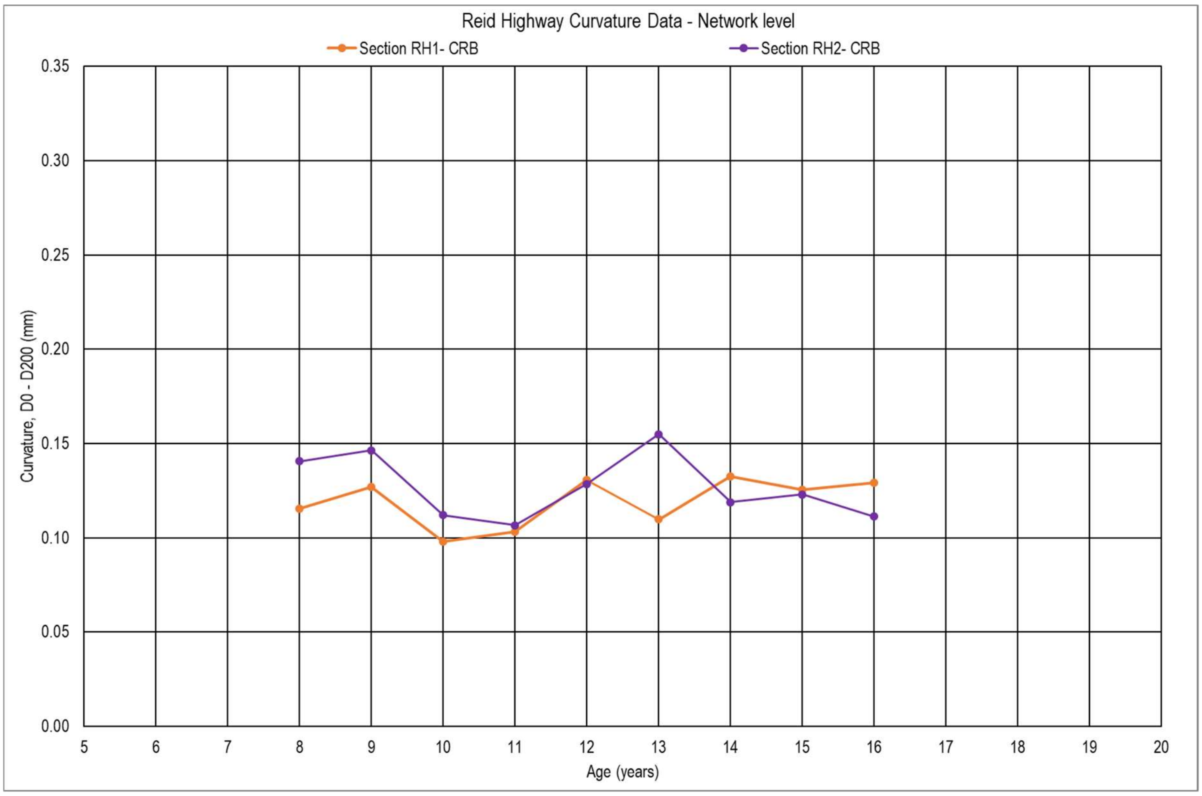


Table 4.53: Reid Highway – network level curvature trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 3 (5th year to 15th year)	RH1	0.2	0.01	0.2
	RH2	0.1	–0.01	0.3

Trends in Deflection and Curvature

Both the deflection and curvature data for the Reid Highway investigation sections demonstrated an even trend over the nine-year monitoring period.

As discussed in Section 2, four similar phases of performance were identified from the deflection and curvature data of the trial sections investigated previously in Stage 2. The deflection and curvature data for the Reid Highway investigation sections covers the proposed third phase of these previously identified performance trends.

From Figure 4.24 and Figure 4.25 and Table 4.51 and Table 4.53, the following trends within this previously identified phase can be inferred as the following:

- Phase 3: end of 5th year to end of 15th year

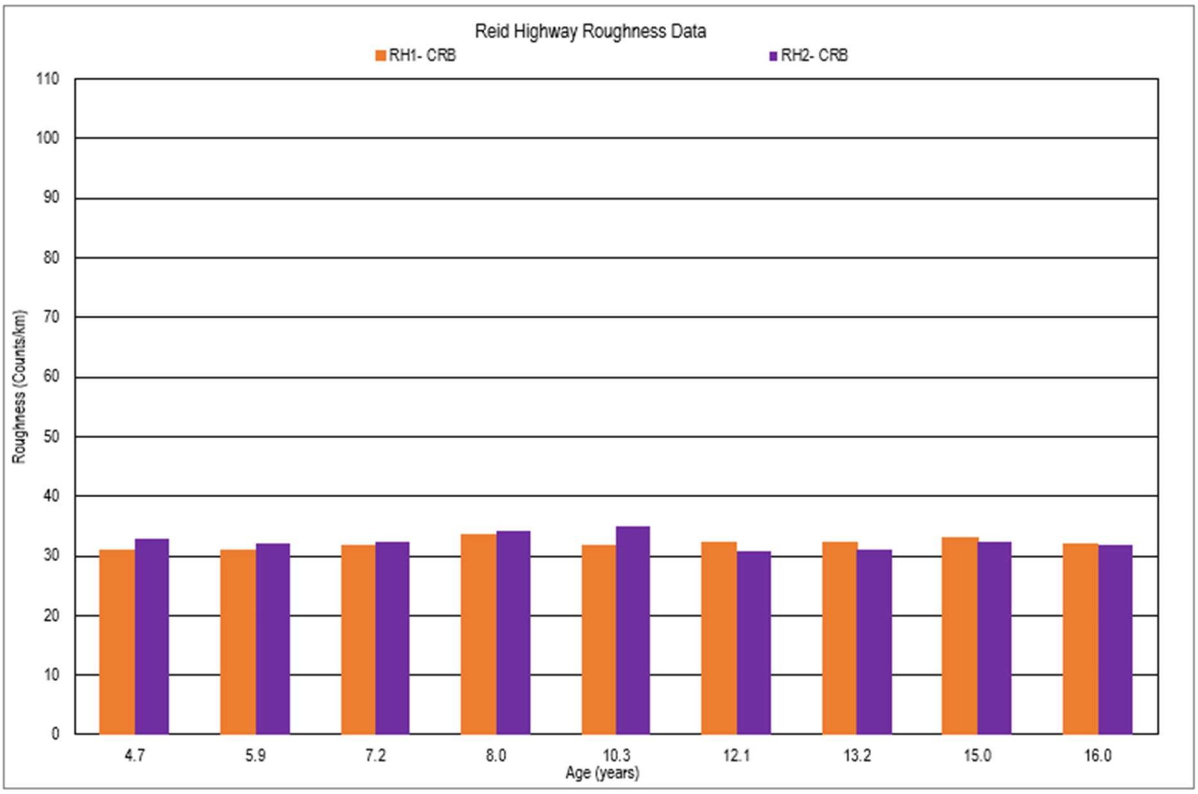
— Deflection and curvature show no strong increasing or decreasing trend over this period.

This observation is consistent with the trends identified from the Mitchell Freeway investigation sections. However, Stage 2 trial data for this phase demonstrated an increase in deflection and curvature for this phase.

Roughness data

Roughness data over the first 16 years of service life on the Reid Highway investigation sections was extracted from the IRIS database. Figure 4.26 presents the roughness progression of each section over time. Both investigation sections demonstrated a consistent roughness level over the first 16 years with both sections having a similar roughness level.

Figure 4.26: Reid Highway – roughness data



Rutting data

Rutting data for the Reid Highway sections was extracted from IRIS. This included a mean value of rutting using the 2 m straight edge for both the IWP and OWP which are shown in Figure 4.27 and Figure 4.28 respectively.

For all sections, the IWP rut depth was typically greater than that in the OWP. Rut depths were not as consistent as the roughness measurements and varied throughout the data set.

Figure 4.27: Reid Highway – rutting data (2 m straight edge, left lane, IWP)

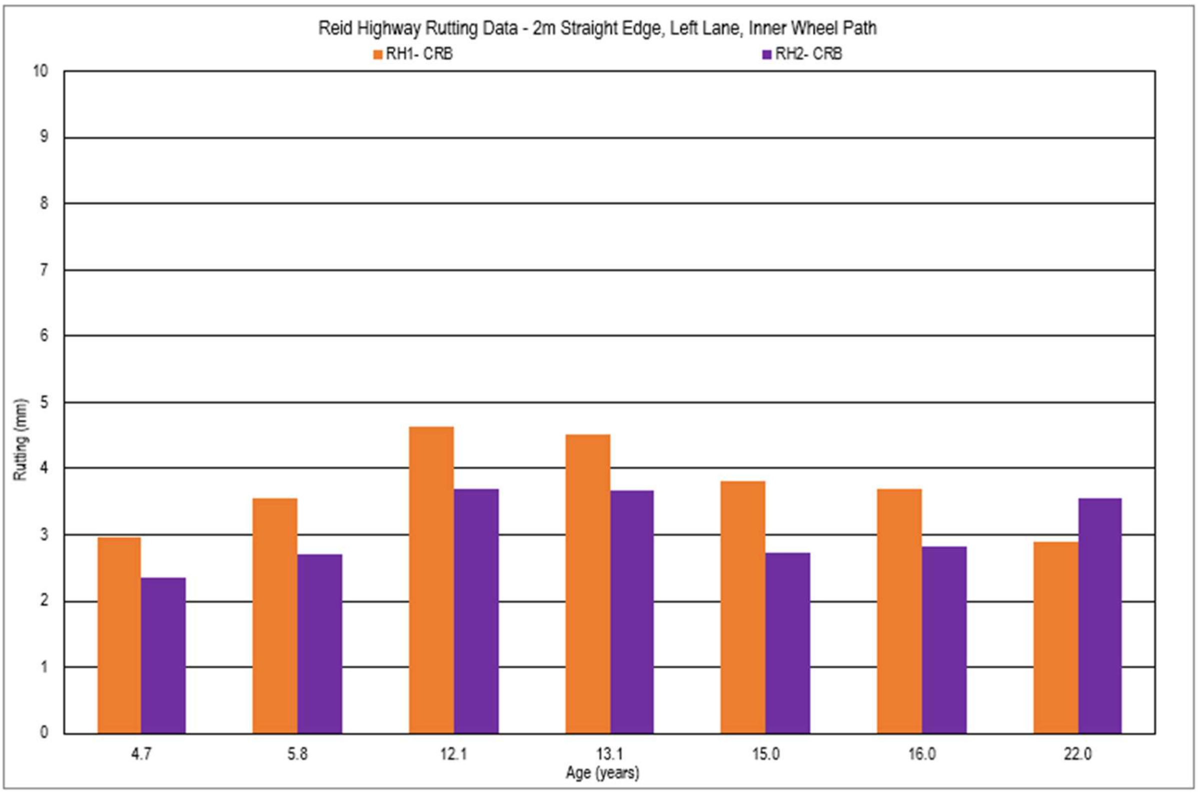
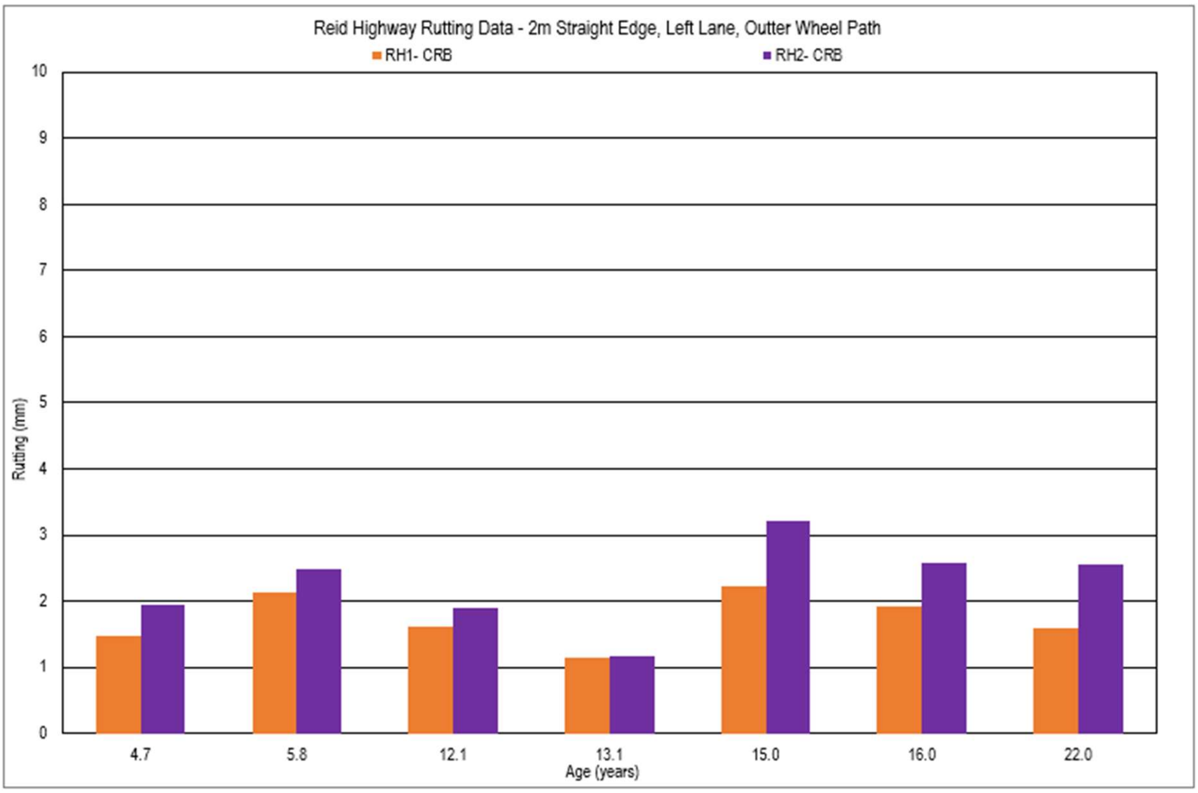


Figure 4.28: Reid Highway – rutting data (2 m straight edge, left lane, OWP)



4.6 Roe Highway

4.6.1 Overview

MRWA Contract 161/82 covers the single Roe Highway investigation section, which was opened to traffic in December 1984. The pavement structure comprises a Perth sand subgrade, CL subbase and a CRB basecourse with a thin DGA surfacing. Collected data includes the design thicknesses, design traffic in addition to roughness and rutting data. The posted speed limit is 100 km/h.

Only design thicknesses were available for this section of interest. Design traffic, measured traffic, roughness, rutting and FWD data at the network and project levels were also sourced. Density and dryback conformance information could not be sourced. However, it was assumed that non-conformances would have been identified and addressed prior to practical completion.

4.6.2 Sections of Interest

Based on the availability of performance data and the criteria outlined in Section 3.1, one section of the Roe Highway was selected for analysis comprising a total of 0.22 km. The lane of interest is L1 in the northbound direction. Table 4.54 summarises the details of the Roe Highway investigation section.

Table 4.54: Details of Roe Highway investigation section

Section ID	Direction	Start chainage (km)	End chainage (km)	Length (m)	Constructed material type			
					Surfacing	Basecourse	Subbase	Subgrade
RO1	NB	38.18	38.40	220	DGA	CRB	Crushed limestone	Sand

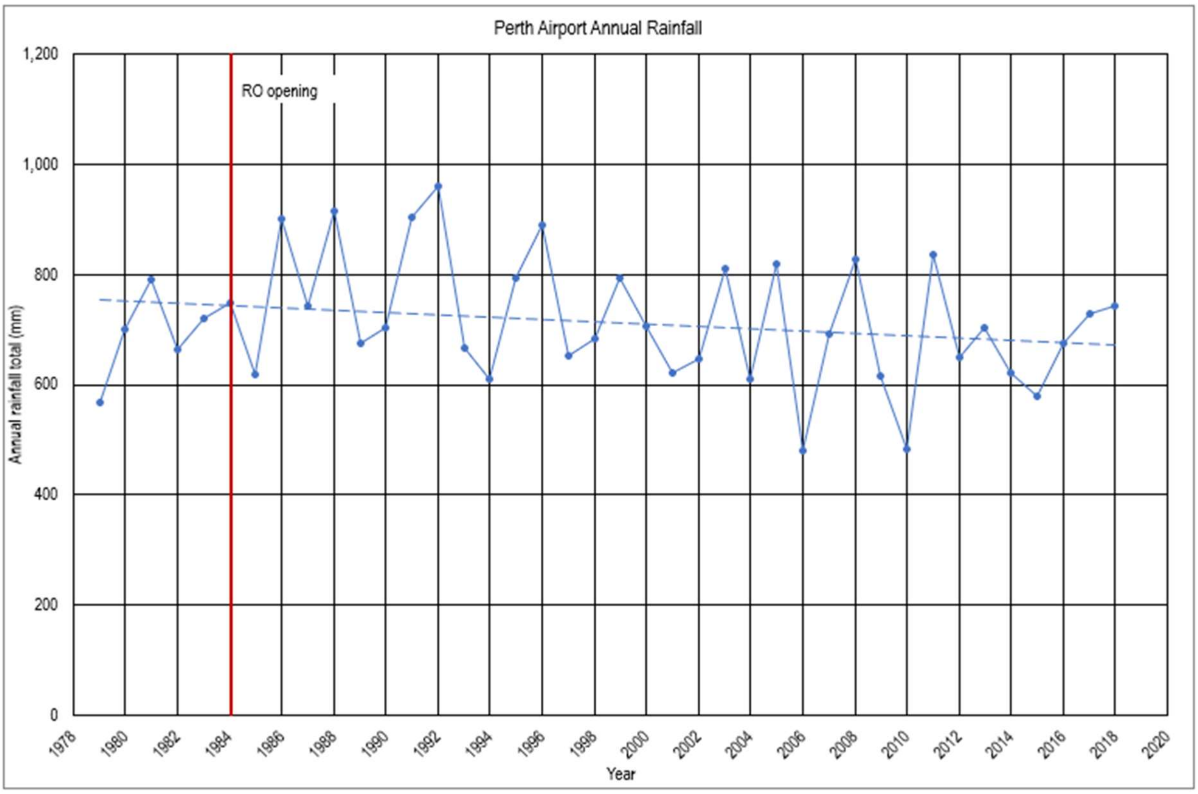
Subsurface Conditions

The 1:250 000 Environmental Geological Map series produced by the Geological Survey of Western Australia Perth sheet (Geological Survey of Western Australia 1980a) indicates that the natural subsurface material underlying the investigation section comprises Bassendean dune quartz sands.

Climate Data

Annual rainfall averages obtained from the Perth Airport weather station, located approximately 4 km from the Roe Highway investigation section, were used to demonstrate the climatic history of the area. Data from five years prior to construction through to 2018 obtained from the Bureau of Meteorology (BOM 2013) is presented in Figure 4.29. The plot demonstrates a clear downward trend of annual rainfall for this location.

Figure 4.29: Roe Highway – historic annual rainfall data



Source: BOM (2013).

4.6.3 Design and Construction Data

The Roe Highway investigation section design data was obtained from a number of sources related to MRWA Contract 161/82. Specific pavement dipping depths were not available and specified construction target thicknesses were taken from as-constructed contract drawings. This data is summarised in Table 4.55.

It is postulated that, due to the age of the pavement, granular layer thicknesses were determined using design charts and as such, the design moduli for ME design were not available.

Table 4.55: Roe Highway – design and specified target thicknesses

Identification number	Layer	Material	Design thickness (mm)	Specified target thickness (mm)
RO1	Asphalt	10 mm DGA	30	–
	Basecourse	CRB	75	85
	Subbase	Crushed limestone	150	160
	Subgrade	White sand	–	–

4.6.4 Traffic Loading

The design traffic loading (ESAs) and the design traffic growth rate for the Roe Highway investigation section is summarised in Table 4.56.

Table 4.56: Roe Highway – traffic design data

Section ID	Design traffic (ESAs)	Design period (years)	Design traffic growth rate (%)
RO1	7.6×10^6	20	3.5

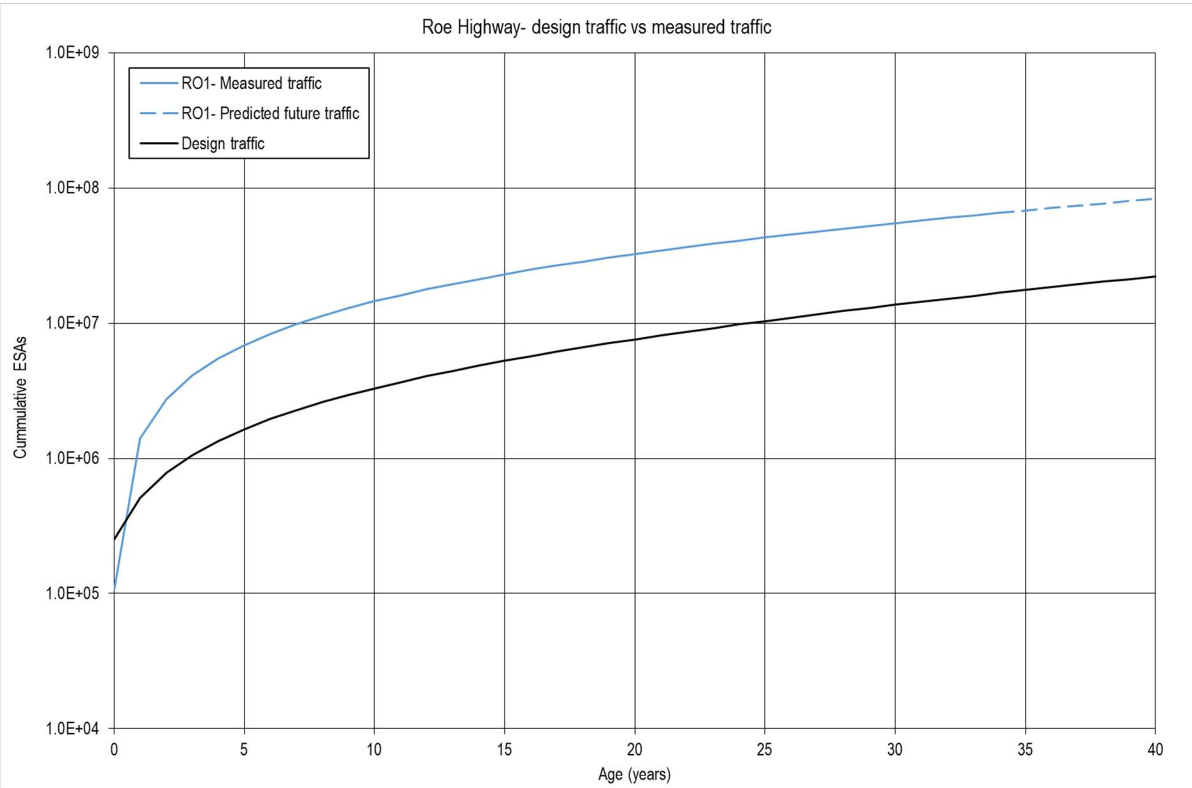
Traffic data was extracted from IRIS in April 2019. Using the available measured traffic data, the average annual traffic growth rate was calculated for each investigation section. The first year of traffic loading in addition to the cumulative number of ESAs from opening up until June 2019 were subsequently estimated using the back-calculated average growth rate and the available traffic data sets. This calculated traffic data is presented in Table 4.57.

Table 4.57: Roe Highway investigation section – calculated traffic data

Section ID	First year of traffic data available	Most recent year of traffic data available	Average growth rate (%)	First year traffic (ESAs)	Cumulative traffic from opening to Jun-19 (ESAs)	Pavement age in Jun-019 (years)
RO1	1994	2018	2.3	1.4×10^6	6.7×10^7	34.5

The cumulative ESAs for the Roe Highway investigation section RO1 is shown in Figure 4.30, in addition to the future predicted traffic and the original design traffic.

Figure 4.30: Roe Highway – traffic data



4.6.5 Pavement Maintenance

Pavement maintenance and resurfacing detail data for the Roe Highway investigation section was extracted from the IRIS database in April 2019 indicated that resurfacing had occurred in 2002.

The historic aerial image review identified the late 2002 resurfacing as indicated in IRIS in addition to resurfacing in late 2010 and early 2016. Roughness and rutting timelines were also used to corroborate the possible works identified by historic images, with a decrease typically demonstrated after works are undertaken.

Table 4.58 presents the rutting and roughness data measured before the resurfacing dates along RO1. Considering the data in Table 4.58, the rutting data in 2000, two years prior to the first resurfacing was above 10 mm in the IWP. Roughness in the same year was below typical intervention level of 110 counts/km. As cracking data was not available, and the roughness was below the intervention level, it is inferred that the high IWP rutting may have been caused by moisture ingress through surface cracking.

Table 4.58: Roe Highway – roughness and rutting before resurfacing

Section ID	Resurface date	Chainage	Average roughness and survey date (counts/km)	Average rutting and survey date (mm)	
				IWP	OWP
RH2	Dec-02	38.18 – 38.40	56.8 (Sep-00)	11.7 (Sep-00)	2.6
	Dec-10		51.0 (Nov-10)	4.8 (Nov-10)	1.3
	Feb-16		N/A[1]	1.8 (Nov-12)	0.9

Notes:
1 Roughness data only available up to November 2010

Using the traffic data calculated from measured data sets, the approximate cumulative traffic levels applied to each resurfacing was calculated. These are shown in Table 4.59.

According to this data, asphalt fatigue was observed along these sections approximately 18 years after the surfacing was first constructed.

Table 4.59: Roe Highway – resurfacing dates

Section ID	Date of 1st resurfacing and surfacing age (years)	Cumulative traffic at resurfacing (ESAs)	Layer replaced[1]	Subsequent resurfacing dates and age	Cumulative traffic at subsequent resurfacings (ESAs)	Layers replaced[1]
RO1	Dec-02 (18)	2.9×10^7	DGA	Dec-10 (8), Feb-16 (5.2)	1.7×10^7, 1.3×10^7	DGA

Notes:
1 Layer replaced at resurfacing advised by MRWA

4.6.6 Performance Monitoring

Deflection and curvature

Network and project level FWD data was available for the Roe Highway section as detailed in Table 4.60. Full FWD data sets are presented in Appendix A.

Table 4.60: Roe Highway – available FWD data

Section ID	Years of network level FWD	Age at network level FWD	Years of Project level FWD	Age at network level FWD
RO1	2002 – 2005 (inclusive)	17 – 20 years	2013	28 years

Deflection results

A summary of the Roe Highway mean deflection data from the network and project level FWD surveys is presented in Table 4.61 and Table 4.62 respectively This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth). This data is also presented in Figure 4.31. Deflection trend statistics are presented in Table 4.63.

Table 4.61: Roe Highway – network level maximum deflection

Section ID	2002	2003	2004	2005
RO1	0.52	0.44	0.38	0.33

Table 4.62: Roe Highway – project level deflection

Section ID	2013
RO1	0.44

Figure 4.31: Roe Highway – maximum mean deflection

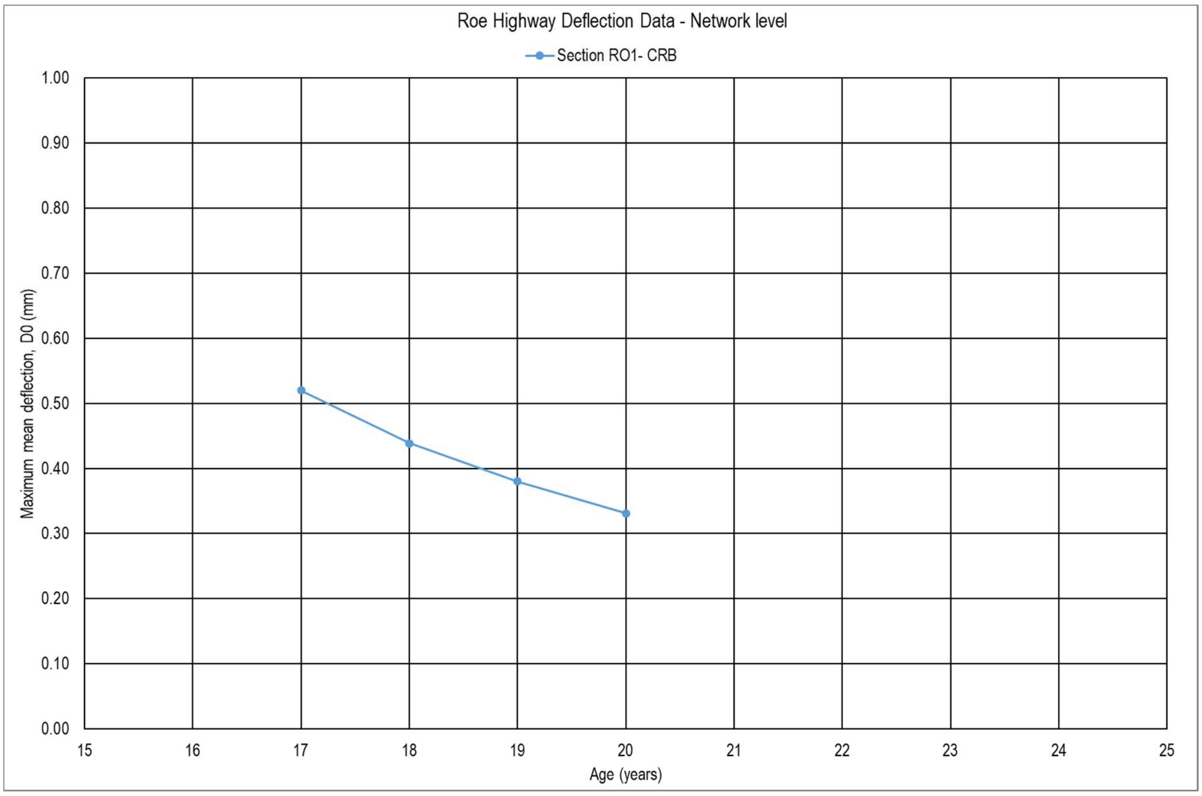


Table 4.63: Roe Highway – network level deflection trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 4 (15th year onwards)	RO1	0.02	-0.002	0.8

Curvature results

A summary of the Roe Highway mean curvature data from the network and project level FWD surveys is presented in Table 4.64 and Table 4.65 respectively . This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth). This data is also presented in Figure 4.32. Curvature trend statistics are presented in Table 4.66.

Table 4.64: Roe Highway – network level curvature

Section ID	2002	2003	2004	2005
RO1	0.17	0.13	0.12	0.13

Table 4.65: Roe Highway – project level curvature

Section ID	2013
RO1	0.15

Figure 4.32: Roe Highway – curvature data

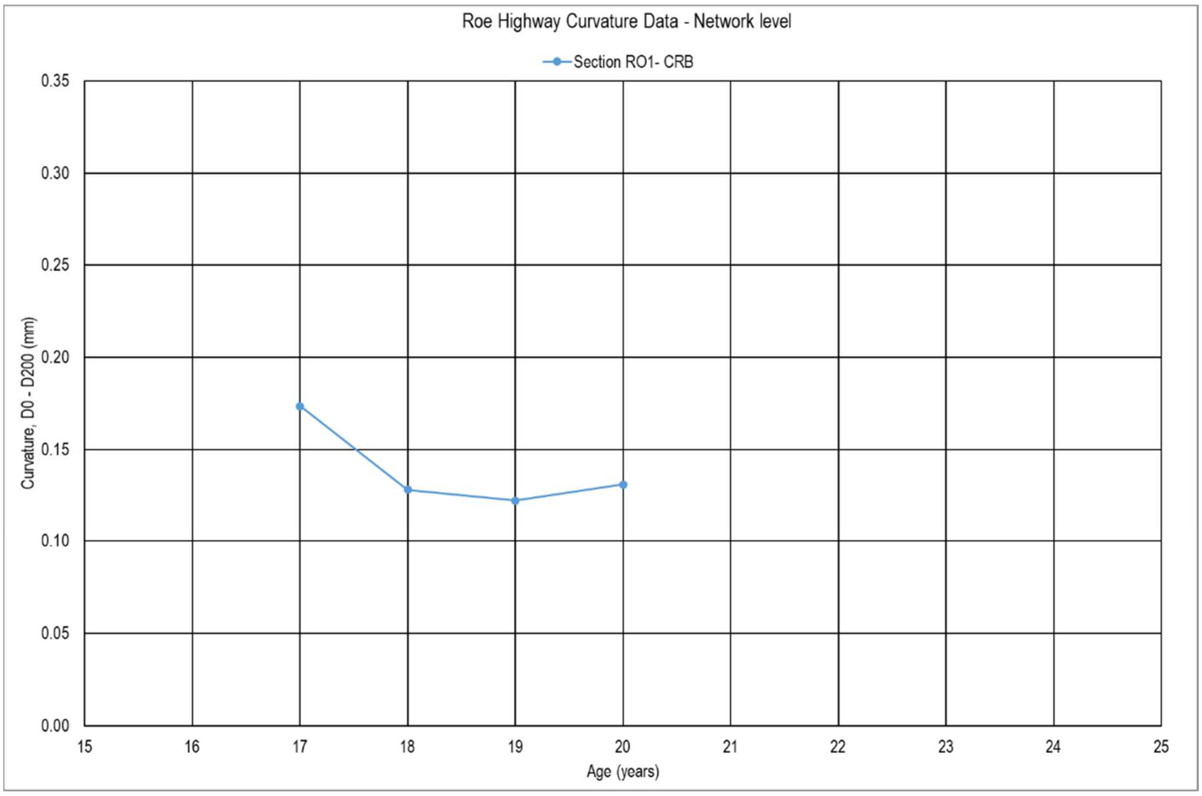


Table 4.66: Roe Highway – network level curvature trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 4 (15th year onwards)	RO1	0.001	-0.0002	0.96

Trends in deflection and curvature

As discussed in Section 2, four similar phases of performance were identified from the deflection and curvature data of the trial sections investigated previously in Stage 2. The deflection and curvature data for the Roe Highway investigation section covers the proposed fourth phase of these previously identified performance trends.

From Figure 4.31 and Figure 4.32 and Table 4.63 and Table 4.66, the following trends within this previously-identified phase can be inferred as the following:

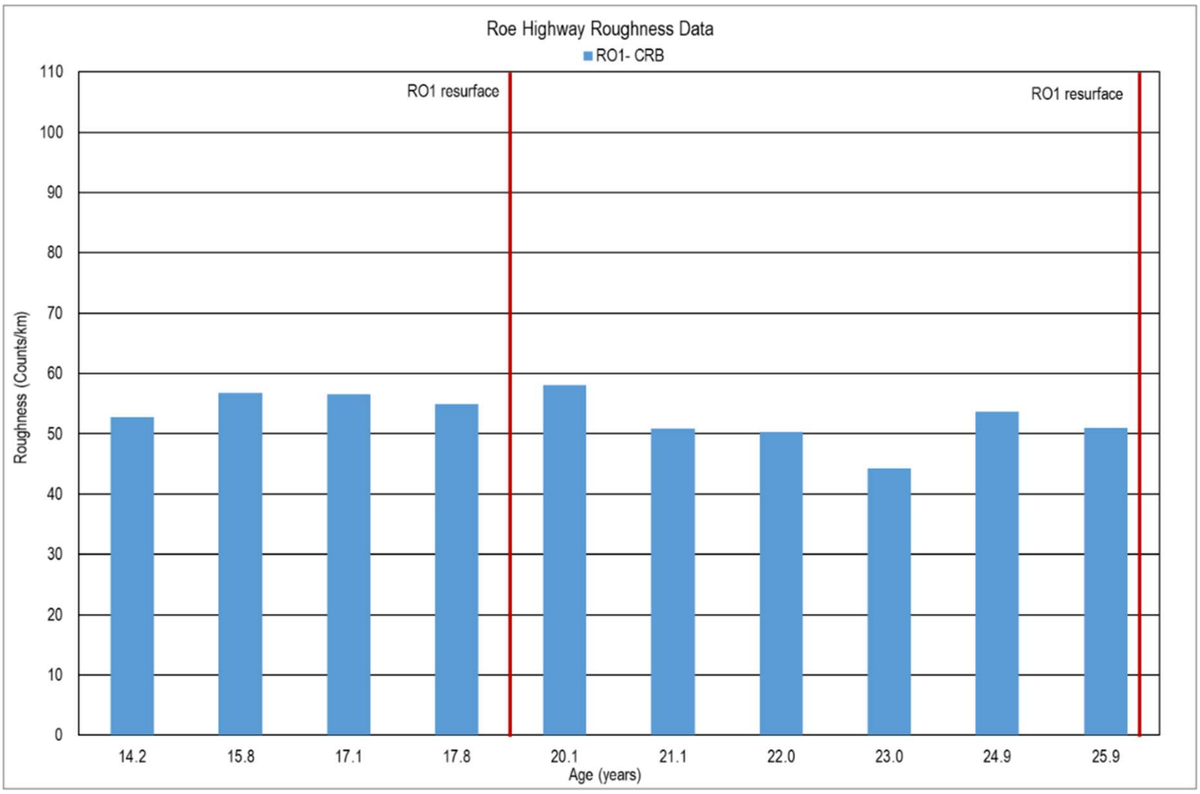
- Phase 4: end of 15th year onwards:
  - Deflection and curvature do not show and increasing or decreasing trend during this phase.

These trends are not consistent with the trends identified from the Stage 2 trial data for this phase. It would be expected that both deflection and curvature would show an increase throughout the fourth phase of performance.

Roughness data

The roughness data for the Roe Highway investigation section extracted from the IRIS is presented in Figure 4.33. The roughness measurements were typically consistent over the data set.

Figure 4.33: Roe Highway – roughness data



Rutting data

Rutting data for the Roe Highway section was extracted from IRIS. This included a mean value of rutting using the 2 m straight edge for both the IWP and OWP which are shown in Figure 4.34 and Figure 4.35 respectively.

As with the other investigation sections, the IWP rut depth was typically greater than that in the OWP. Similarly to the roughness, rut depths were consistent throughout the data set, with obvious decreases after resurfacing in 2002 and 2010. The 2016 resurfacing could not be inferred from the rutting data.

Figure 4.34: Roe Highway – rutting data (2 m straight edge, left lane, IWP)

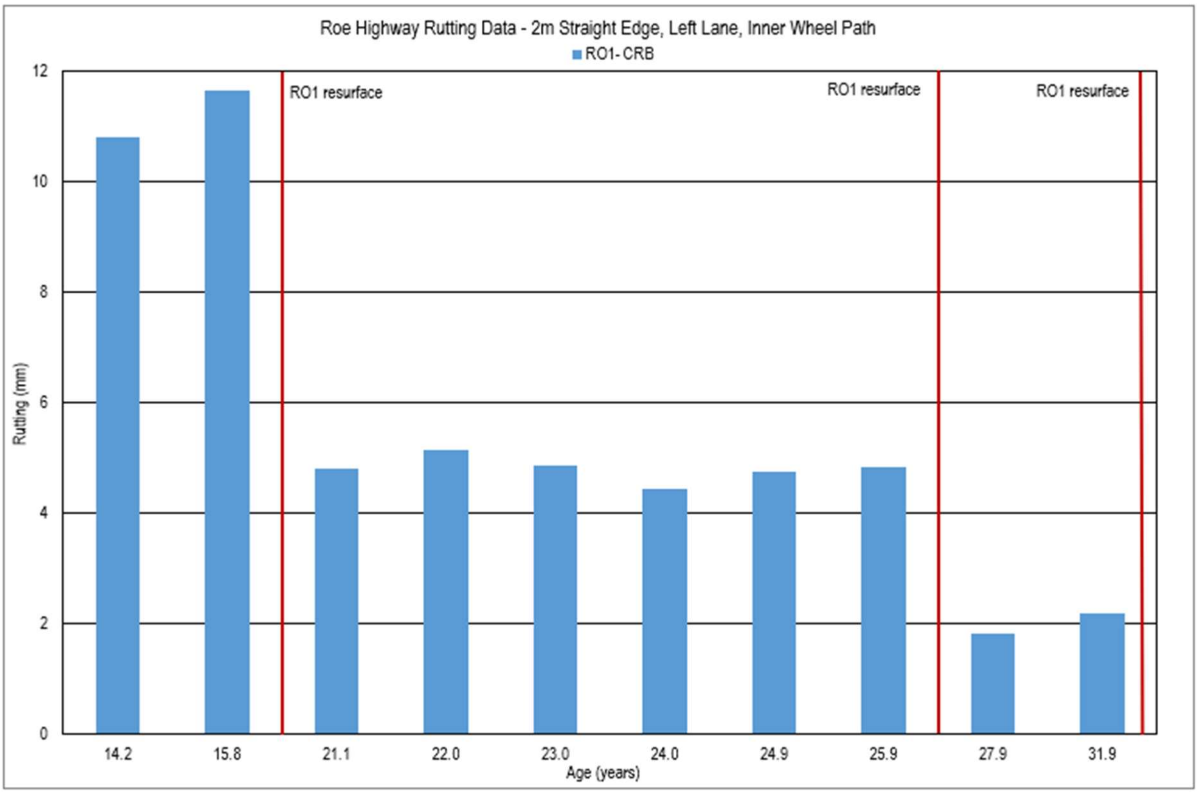
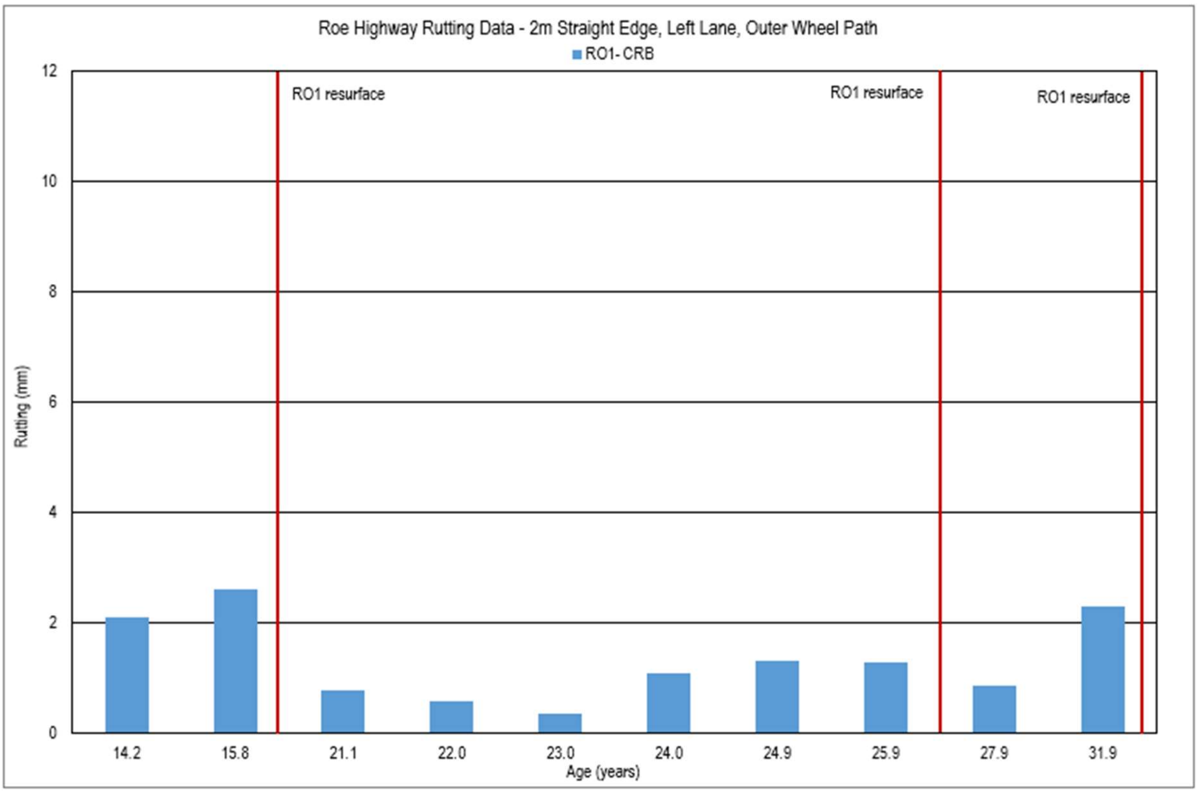


Figure 4.35: Roe Highway – rutting data (2 m straight edge, left lane, OWP)



4.7 Tonkin Highway

4.7.1 Overview

The Tonkin Highway investigation section is located between Morley and Embleton on the northbound side of the carriageway. This section was covered under the MRWA Contract 136/82. It was opened to traffic in July 1984. It is comprised of a Perth sand subgrade, CL subbase, CRB basecourse overlaid with a DGA surfacing. The posted speed limit is 100 km/h.

As of late 2016 this section of the Tonkin Highway was reconstructed as part of the Northlink project. Data after September 2016 has therefore not been considered.

As specified and design thicknesses were available for this section of interest. Design traffic, measured traffic, roughness, rutting and FWD data at the network level were also sourced. Density and dryback conformance information could not be sourced. However, it was assumed that non-conformances would have been identified and addressed prior to practical completion.

4.7.2 Sections of Interest

Based on the availability of performance data and compliance with the criteria presented in Section 3.1, one section of Tonkin Highway was selected for analysis comprising a total of 1.18 km. The lane of interest is R1 in the northbound direction. Table 4.67 summarises the details of the Tonkin Highway investigation section.

Table 4.67: Details of Tonkin Highway investigation sections

Section ID	Direction	Start chainage (km)	End chainage (km)	Length (m)	Constructed material type			
					Surfacing	Basecourse	Subbase	Subgrade
TH1	NB	2.57	3.75	1180	DGA	CRB	Crushed limestone	Sand

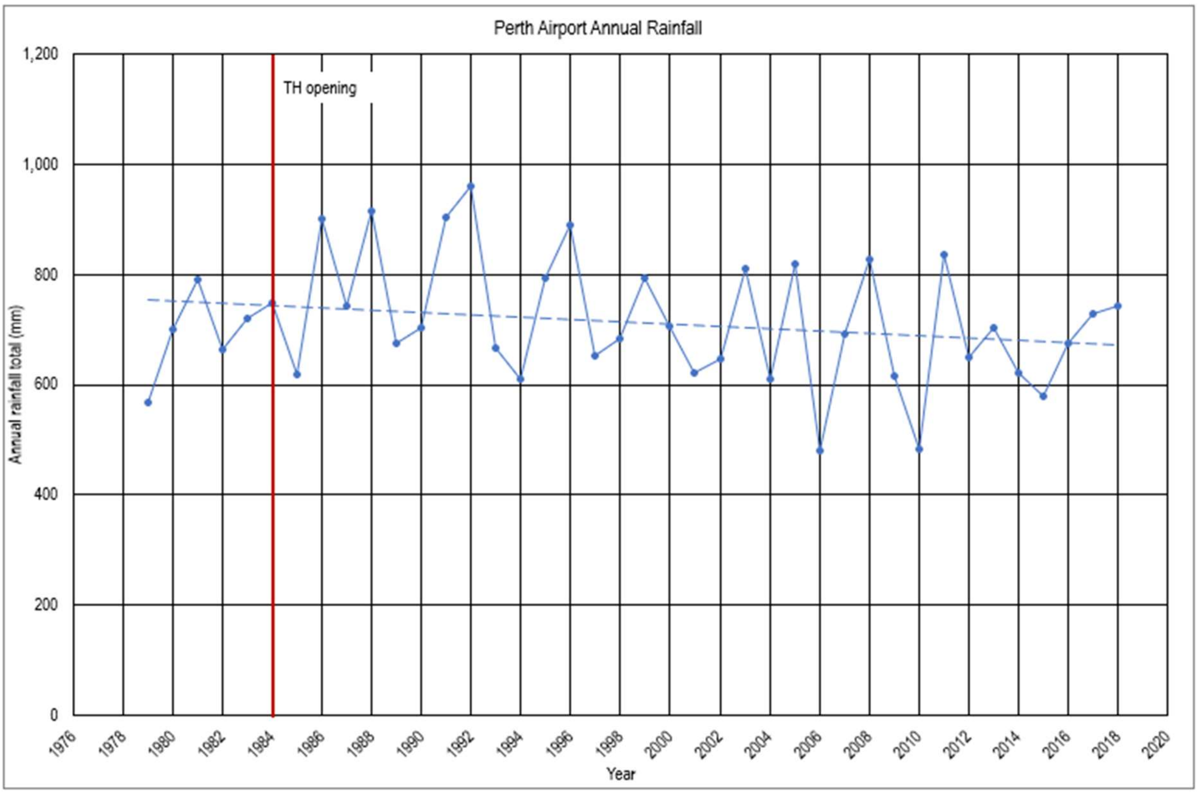
Subsurface Conditions

The 1:250 000 Environmental Geological Map series produced by the Geological Survey of Western Australia Perth sheet (Geological Survey of Western Australia 1980a) indicates that the natural subsurface material underlying the investigation section comprises Bassendean dune quartz sands.

Climate Data

Annual rainfall averages obtained from the Perth Airport weather station, located approximately 5 km from the Tonkin Highway investigation section, were used to demonstrate the climatic history of the area. Data from five years prior to construction through to 2017 obtained from the Bureau of Meteorology (BOM 2013) is presented in Figure 4.36. The plot demonstrates a clear downward trend of annual rainfall for this location.

Figure 4.36: Tonkin Hwy – historic annual rainfall data



Source: BOM (2013).

4.7.3 *Design and Construction Data*

The Tonkin Highway investigation section design and construction data was sourced through a number of documents related to MRWA Contract 136/82. Specific pavement dipping depths were not available and specified construction target thicknesses were taken from as-constructed contract drawings. This data is summarised in Table 4.68.

It is postulated that, due to the age of the pavement, granular layer thicknesses were determined using design charts and, as such, design moduli for ME design was not available.

Table 4.68: Tonkin Highway – design and specified target thicknesses

Section ID	Layer	Material	Design thickness (mm)	Specified target thickness (mm)
TH1	Asphalt	DGA	30	55
	Basecourse	CRB	75	65
	Subbase	Crushed limestone	255	255
	Subgrade	White sand	–	–

4.7.4 *Traffic Loading*

The design traffic loading (ESAs and the design traffic growth rate is summarised in Table 4.69. Design traffic growth rate could not be sourced, so a value of 3% was assumed for analysis purposes.

Table 4.69: Tonkin Highway – traffic design data

Section ID	Design traffic (ESAs)	Design period (years)
TH1	1.0×10^8	40

Tonkin Highway traffic data was extracted from IRIS in April 2019. Using the available measured traffic data, the average annual traffic growth rate was calculated for each investigation section. The first year of traffic loading in addition to the cumulative number of ESAs from opening up until December 2016 were subsequently estimated using the back-calculated average growth rate and the available traffic data sets. This calculated traffic data is presented in Table 4.70.

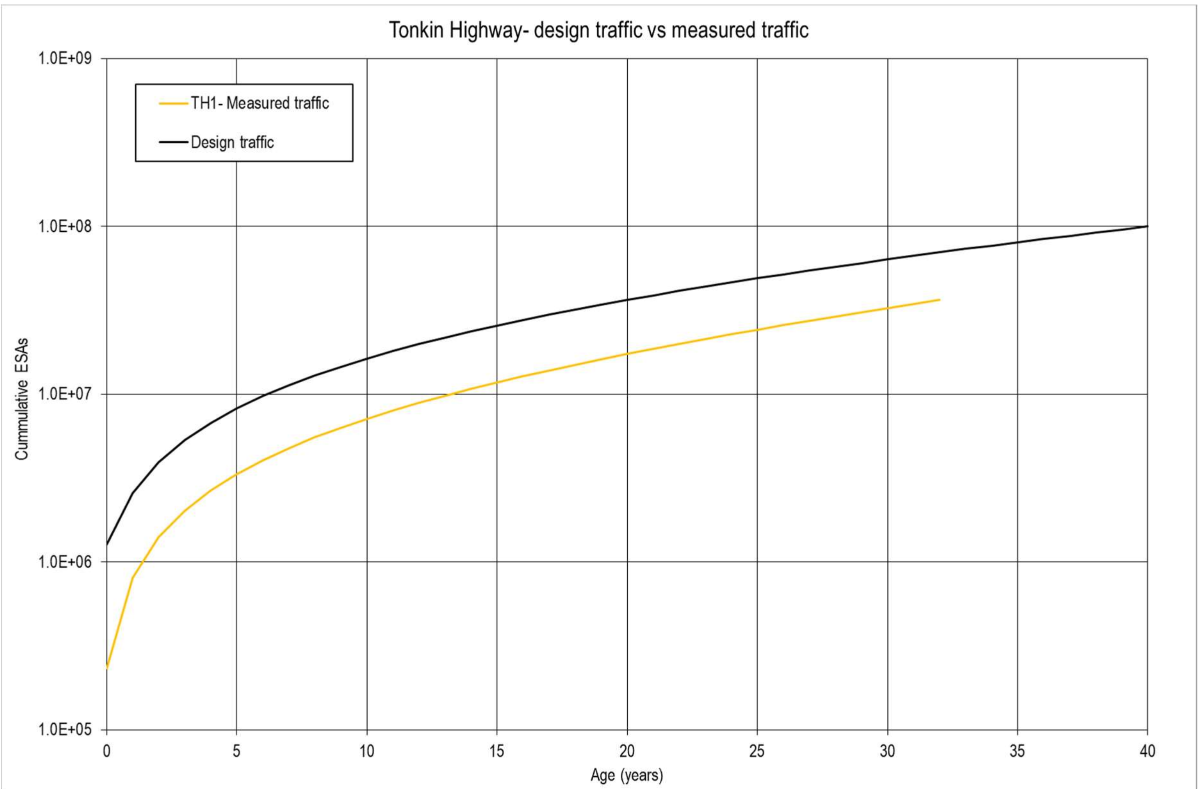
Table 4.70: Tonkin Highway – calculated traffic data

Section ID	First year of traffic data available	Most recent year of traffic data available	Average growth rate (%)	First year traffic (ESAs)	Cumulative traffic up to Dec-16* (ESAs)	Pavement age in Dec-16 (years)
TH1	1994	2014	4.0	8.1×10^5	3.6×10^7	32.4

Notes: Investigation section reconstructed after Dec-16.

The cumulative ESAs for the Tonkin Highway investigation section TH1 is shown in Figure 4.37, in addition to the original design traffic. A design growth rate of 3.0% has been assumed for the design traffic as this data could not be sourced.

Figure 4.37: Tonkin Highway – traffic data



4.7.5 *Pavement Maintenance*

Pavement maintenance and resurfacing detail data for the Tonkin Highway investigation section was extracted from the IRIS database in April 2019 and indicated a resurfacing in 2009. The historic aerial image review identified the late 2009 resurfacing as indicated in IRIS. No other resurfacing works could be identified.

The roughness and rutting data, which is presented and discussed in Section 4.7.6 and Section 4.7.6 respectively, also corroborated the resurfacing works in late 2009. The rutting data measured in the OWP suggests another resurfacing at some point between September 2000 and January 2006. However, the historic aerial image study revealed no works were conducted during this period, suggesting this data to be outlying. Furthermore, as the September 2000 (pavement age 16.2 years) data is outside the typical data collection date range of November to January, seasonal variations may have increased the rut depth at this data point.

Table 4.71 presents the rutting and roughness data measured before the resurfacing of TH1. Considering the data in Table 4.71, the rutting data had no measurements above 10 mm and the roughness data was also well below typical intervention level of 110 counts/km. This suggests that high roughness or rutting was not the reason for resurfacing.

Table 4.71: Tonkin Highway – roughness and rutting before resurfacing

Section ID	Resurface date	Chainage	Average roughness and survey date (counts/km)	Average rutting and survey date (mm)	
				IWP	OWP
TH1	Dec-09	2.57 – 3.75	39.1 (Nov-09)	5.0 (Nov-09)	4.0

Using the traffic data calculated from measured data sets, the approximate cumulative traffic levels applied to each resurfacing were calculated. These are shown in Table 4.72. According to this data, asphalt fatigue cracking was observed along these sections approximately 25 years after the surfacing was first constructed.

Table 4.72: Tonkin Highway – resurfacing dates

Section ID	Date of 1st resurfacing and surfacing age (years)	Cumulative traffic at resurfacing (ESAs)	Layer replaced[1]	Subsequent resurfacing dates and age	Cumulative traffic at subsequent resurfacings (ESAs)
TH1	Dec-09 (25.4)	2.5×10^7	DGA	N/A	

Notes:

1 Layer replaced at resurfacing advised by MRWA

4.7.6 Performance Monitoring

Deflection and curvature

Only network level FWD data was available for the Tonkin Highway section as detailed in Table 4.73. Full FWD data sets are presented in Appendix A.

Table 4.73: Tonkin Highway – available FWD data

Section ID	Years of network level FWD	Age at network level FWD	Years of Project level FWD	Age at network level FWD
TH1	2002 – 2010 (inclusive)	18 – 26 years	N/A	

Deflection results

A summary of the Tonkin Highway mean deflection data from the network level FWD survey is presented in Table 4.74. This data has been corrected from the measurement temperature to the

WMAPT (29°C for Perth). This data is also presented in Figure 4.38. Deflection trend statistics are presented in Table 4.75

Table 4.74: Tonkin Highway – network level maximum deflection

Section ID	2002	2003	2004	2005	2006	2007	2008	2009	2010
TH1	0.45	0.50	0.39	0.46	0.44	0.39	0.5	0.47	0.43

Figure 4.38: Tonkin Highway – mean maximum deflection

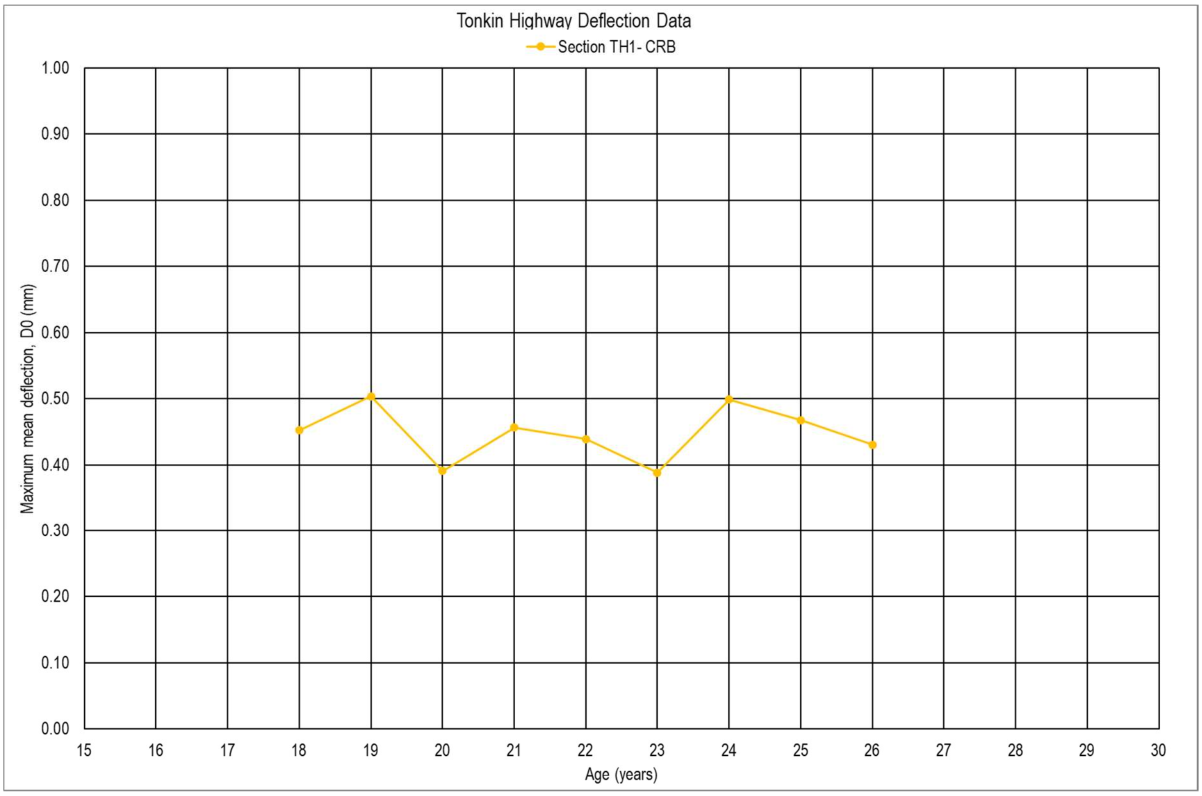


Table 4.75: Tonkin Highway – network level deflection trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 4 (15th year onwards)	TH1	0.003	-0.0008	0.90

Curvature results

A summary of the Tonkin Highway mean curvature data from the network level FWD survey is presented in Table 4.76. This data has been corrected from the measurement temperature to the WMAPT (29°C for Perth). This data is also presented in Figure 4.39. Curvature trend statistics are presented in Table 4.77.

Table 4.76: Tonkin Highway – network level curvature

Section ID	2002	2003	2004	2005	2006	2007	2008	2009	2010
TH1	0.14	0.18	0.13	0.15	0.14	0.16	0.15	0.16	0.14

Figure 4.39: Tonkin Highway – curvature

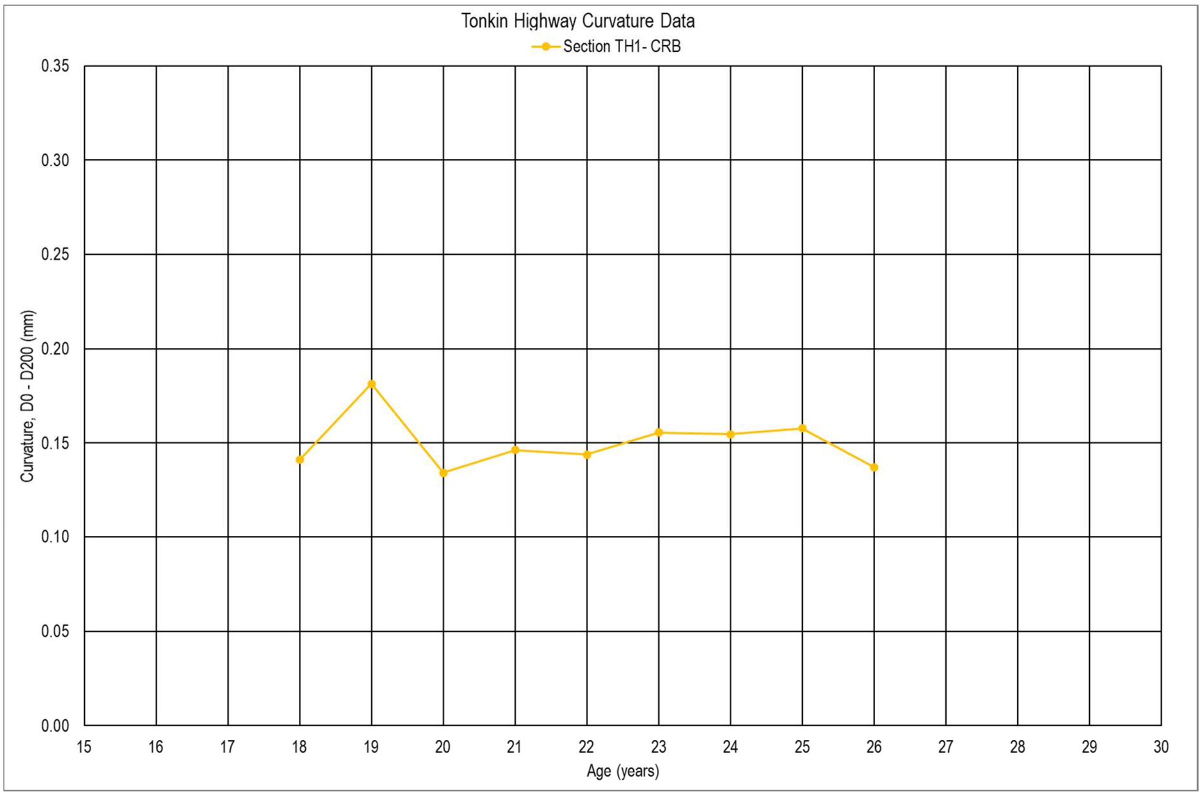


Table 4.77: Tonkin Highway – network level curvature trend statistics

Performance phase	Section ID	R2	Slope	P-value
Phase 4 (15th year onwards)	TH1	0.014	–0.0006	0.76

Trends in deflection and curvature

As discussed in Section 2, four similar phases of performance were identified from the deflection and curvature data of the trial sections investigated previously in Stage 2. The deflection and curvature data for the Tonkin Highway investigation section covers the proposed fourth phase of these previously identified performance trends.

From Figure 4.38 and Figure 4.39 and Table 4.75 and Table 4.77, the following trends within this previously-identified phase can be inferred as the following:

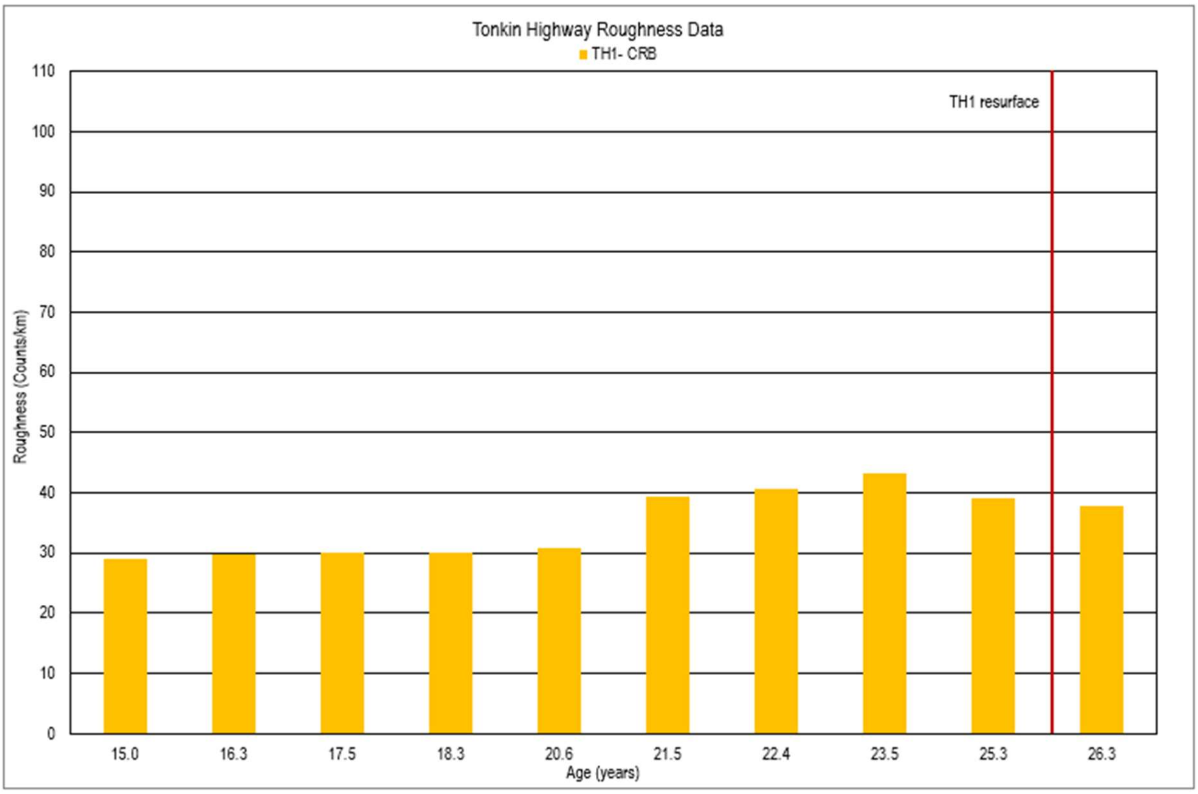
- Phase 4: end of 15th year onwards:
  - — Deflection and curvature both show neither an increasing nor decreasing trend.

This observation is not consistent with the trends identified from the Stage 2 trial data for this phase. However, a similar trend was identified along the Roe Highway investigation section. It would be expected that both deflection and curvature would show an increase throughout the fourth phase of performance.

Roughness data

The roughness data for the Tonkin Highway investigation section TH1 was extracted from the IRIS and is presented in Figure 4.40. The roughness measurements were typically consistent over the data set increasing with age.

Figure 4.40: Tonkin Highway – roughness data



Rutting data

Rutting data for the Tonkin Highway section was extracted from IRIS. This included a mean value of rutting using the 2 m straight edge for both the IWP and OWP which are shown in Figure 4.41 and Figure 4.42 respectively.

As with the other investigation sections, the IWP rut depth was typically greater than that in the OWP. Similarly to the roughness, rut depths were consistent throughout the data set, increasing with age and with an obvious decreases after resurfacing in late 2009. The data collected in September 2000 (pavement age 16.2) has been inferred as an outlying value as this data was collected outside the typical collection months (November – January) and may be influenced by seasonal variations such as high moisture levels.

Figure 4.41: Tonkin Highway – rutting data (2 m straight edge, left lane, IWP)

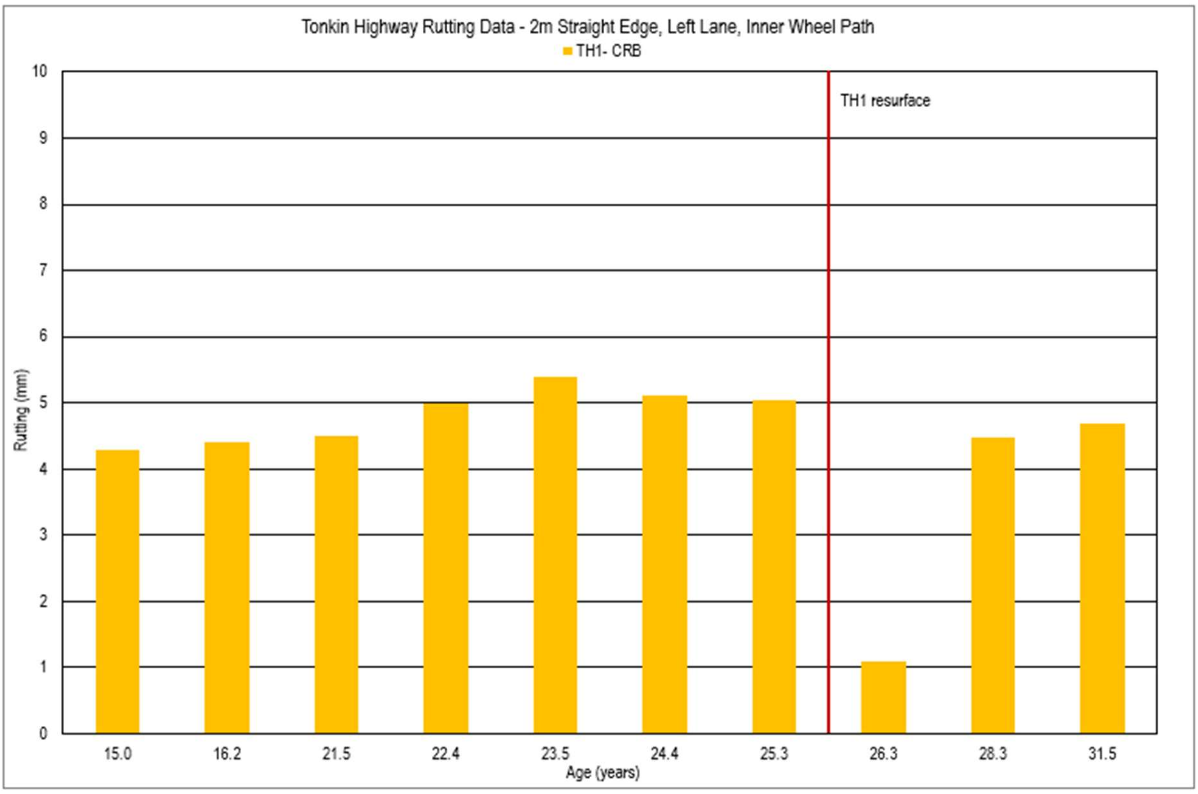
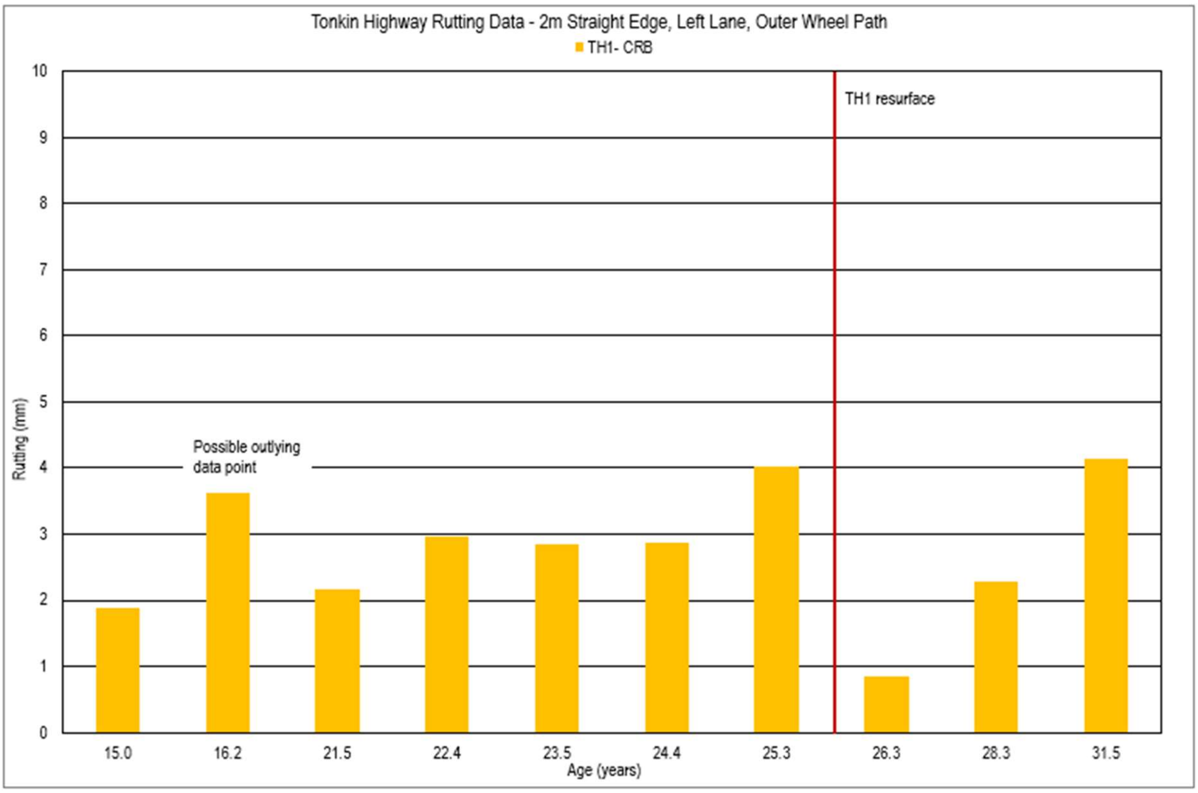


Figure 4.42: Tonkin Highway – rutting data (2 m straight edge, left lane, OWP)



5 COMPARISON OF IN-SERVICE AND PREDICTED PERFORMANCE

5.1 Introduction

The following sections investigate how well the current design method (MRWA 2013) replicates the observed performance of granular pavements with thin asphalt surfacings by comparing the predicted allowable traffic loading to asphalt fatigue failure with the cumulative traffic loading if, or when, asphalt resurfacings were undertaken, the assumption being that such resurfacings were needed due to asphalt fatigue cracking.

The design-predicted performance was calculated using the as-constructed thicknesses of the granular layer (where available) in conjunction with the presumptive moduli values and ME process as per ERN9 (MRWA 2013). As per ERN9, a 10 mm construction tolerance was added to the DGA layer for modelling purposes.

5.2 Granular Thickness Requirements

5.2.1 Stage 3 Findings

The as-specified thicknesses of the investigation sections were compared to the required granular thickness calculated using Figure 6 of ERN9 (MRWA 2013) in conjunction with both the 40-year design traffic and measured 40-year traffic. This ensures that the as-constructed pavement is compliant to the empirical design method both from a design and in-service traffic perspective.

Sections which have as-specified thicknesses lower than the required thicknesses (i.e. non-compliant) would be expected to show premature fatigue or deformation.

5.2.2 Previous Stage 2 Findings

The same thickness comparison was undertaken to the trial sections investigated in Stage 2. The results are presented in Table 5.2.

Table 5.1: Required granular thickness based on measured traffic vs as-specified granular thickness

Section ID	Design CBR (%)	As-specified granular thickness (mm)	40- year design traffic (ESA)	Required thickness of granular material[1] – design traffic (mm)	Mean 40 year measured traffic (ESA)[2]	Required thickness of granular material[1] – measured traffic (mm)
G1 G2 G3 G4	12	*320 (non-compliant)*	1.0×10^8	350	4.1×10^7	330
KF1 KF2 KF3	12	400	1.0×10^8	350	1.1×10^8	360
M1[4] M2[4]	15	275	6.6×10^6	250	7.5×10^6	250
M3	15	*280 (non-compliant)*	3.0×10^7	280	1.1×10^8	310
M4	15	430	3.0×10^7	280	N/A[3]	
RH1 RH2	12	370	1.3×10^8	360	4.2×10^7	330
RO1[4]	12	*245 (non-compliant)*	7.6×10^6	290	3.2×10^7	320
TH1	15	320	1.0×10^8	310	5.5×10^7	290

Notes:

1 Required thickness of granular material calculated using Figure 6 of MRWA ERN9 (MRWA 2013).

2 Mean 40-year measured traffic predicted using calculated traffic data as presented in Section 4.

3 M4 only open for 1.5 years.

4 20 year design traffic and 20 year measured traffic used as originally designed.

Table 5.2: Stage 2 trial sections – required granular thickness based on measured traffic vs as-specified granular thickness

Trial section ID	Design CBR (%)	As-specified granular thickness (mm)	40 year design traffic (ESA)	Required thickness of granular material[1] – design traffic (mm)	Mean 40 year measured traffic (ESA)[2]	Required thickness of granular material[1] – measured traffic (mm)
T2	12	*297 (non-compliant)*	1.0×10^7	290	3.8×10^7	330
T4		*298 (non-compliant)*				
T6		*298 (non-compliant)*				
RH2	12	384	3.5×10^7	330	3.9×10^7	330
RH3		349				
K2	12	410	2.2×10^8	370	2.5×10^7	320
K3		415				
K12		430				

Notes:

1 Required thickness of granular material calculated using Figure 6 of MRWA ERN9 (MRWA 2013).

2 Mean 40 year measured traffic predicted using calculated traffic data.

5.3 Predicted Allowable Traffic Loading to Asphalt Fatigue

The allowable traffic loadings, in terms of asphalt fatigue, were predicted in accordance with ERN9 except that the as-constructed granular layer thicknesses were used rather than the design thicknesses.

As per the design requirements of ERN9, a 95% reliability is required for design using the ME procedure. This level of reliability represents the assumption that 19 out of 20 pavements will exceed the design traffic loading.

The mean allowable traffic loading (MATL) is a more accurate way of comparing the fatigue design predictions with the actual observed pavement fatigue performance. The MATL, in terms of asphalt fatigue, can be calculated by multiplying the 95% reliability allowable traffic loading by a shift factor of 6 (Austroads 2018). Both the 95% reliability allowable traffic and the MATL are shown in Table 5.3.

The measured cumulative traffic when DGA resurfacing was undertaken is assumed to be the observed traffic loading to fatigue cracking. It is designated the mean observed traffic loading (MOTL) in Table 5.3. For the investigation sections where the DGA was not replaced (i.e. the fatigue life assumed not been reached) the current cumulative traffic level up until June 2019 has been presented as the MOTL. The ratio of MOTL to MATL has also been included to demonstrate how close the design predicted performance is to the observed performance.

Table 5.3: Predicted allowable traffic loading vs measured fatigue traffic

Section ID	Nominal surfacing thickness design (mm)[1]	Design granular modulus (MPa)[2]	Required granular thickness compliance	Allowable traffic loading at 95% reliability (ESA)	Mean allowable traffic loading, MAT (ESA)	Cumulative traffic on asphalt up to first resurfacing (ESA)[3,4]	Mean observed fatigue traffic, MOF (ESA)[5]	Ratio of MOF/ MAT
G1	60	500 (BSL)	Non-compliant	1.3 x 10^6	7.9 x 10^6	> 1.4 x 10^7	> 1.4 x 10^7	> 1.7
G2						1.2 x 10^7	1.2 x 10^7	**1.5**
G3						> 1.2 x 10^7	> 1.2 x 10^7	> 1.5
G4						> 1.5 x 10^7	> 1.5 x 10^7	> 1.9
KF1	60	600 (CRB)	Compliant	2.1 x 10^6	1.3 x 10^7	4.2 x 10^7 [6]	3.6 x 10^7	**2.8**
KF2						3.0 x 10^7 [6]		
KF3						> 3.1 x 10^7 [6]	> 3.1 x 10^7	> 2.4
M1	60	600	Compliant	1.6 x 10^6	9.5 x 10^6	> 2.4 x 10^7	> 2.1 x 10^7	> 2.2
M2						> 1.8 x 10^7		
M3		500	Non-compliant	9.6 x 10^5	5.8 x 10^6	> 1.7 x 10^7	> 1.7 x 10^7	> 3.0
M4		600	Compliant	2.4 x 10^6	1.5 x 10^7	N/A		
RH1	30	600	Compliant	7.2 x 10^6	4.3 x 10^7	> 2.2 x 10^7	> 2.2 x 10^7	> 0.5
RH2						2.4 x 10^7	2.4 x 10^7	**0.6**
RO1	30	600	Non-compliant	9.3 x 10^5	5.6 x 10^6	2.9 x 10^7	2.9 x 10^7	**5.1**
TH1	30	600	Compliant	5.4 x 10^6	3.3 x 10^7	2.5 x 10^7	2.5 x 10^7	**0.8**

Notes:

1 Mechanistic analysis uses as-constructed granular thicknesses and includes additional 10 mm asphalt construction tolerance as per ERN9 Clause 1.8.

2 Design granular modulus as per ERN9. 3 Measured cumulative traffic at first DGA resurfacing works assumed to correspond to fatigue life.

4 Where no resurfacing works have taken place, fatigue life is assumed to be greater than (.>) current cumulative traffic level.

5 Mean measured fatigue corresponds to a mean value of cumulative traffic on sections of the same composition.

6 Taken as the cumulative traffic once spray seal was replaced by OGA/DGA.

Of the investigation sections where the DGA was replaced (bold text in Table 5.3), the Tonkin Highway design prediction traffic loading was the closest to the observed loading. The observed loadings for both the Kwinana Freeway and Roe Highway sections were significantly greater than the predictions, while the reverse was the case for the Reid Highway sections.

5.3.1 Previous Stage 2 Observations

The same comparison was undertaken in Stage 2 for the trial sections. Updated values of this comparison are presented in Table 5.4.

As of 2018, the Reid Highway and Kwinana Freeway trial sections had not undergone any resurfacing works and do not show signs of asphalt fatigue. The Tonkin Highway trial sections were altered during an intersection upgrade since the conclusion of the Stage 2 investigation but had no prior fatigue observed.

Table 5.4: Stage 2 trial sections – predicted allowable traffic loading vs measured fatigue traffic

Trial section ID	Nominal surfacing thickness design (mm)[1]	Design granular modulus (MPa)[2]	Required granular thickness compliance	Allowable traffic loading at 95% reliability (ESA)	Mean allowable traffic loading, MAT (ESA)	Cumulative traffic on asphalt up to first resurfacing, MMF (ESA)[3]	Ratio of MMF/MAT
T2	60	500 (BSL)	Non-compliant	1.5×10^6	9.1×10^6	$> 3.3 \times 10^7$	> 3.7
T4	30	500		4.7×10^6	2.8×10^7		> 1.2
T6		600 (CRB)		1.8×10^6	1.1×10^7		> 3.1
RH2	30	500	Compliant	2.6×10^6	1.6×10^7	$> 1.3 \times 10^7$	> 0.8
RH3		600		6.4×10^6	3.8×10^7		> 0.3
K2	60	600	Compliant	2.8×10^6	1.7×10^7	$> 2.8 \times 10^6$	> 0.2
K3		600		2.9×10^6	1.7×10^7		> 0.2
K12		500		1.6×10^6	9.6×10^6		> 0.3

Notes:

1 Design granular modulus as per ERN9.

2 Mechanistic analysis uses as constructed granular thicknesses and includes additional 10 mm asphalt construction tolerance as per ERN9 Clause 1.8.

3 Where no resurfacing works have taken place, fatigue life is assumed to be greater than (.>) current cumulative traffic level

Fatigue cracking has not been observed in any of the Stage 2 trial pavements. With the exception of the Tonkin Highway trial sections, the other sections have cumulative traffic loadings to date in excess of the predicted loading.

5.4 Fatigue Life Design Period

5.4.1 Predicted Fatigue Life

Table 5.5 lists the allowable traffic loadings to fatigue cracking for a design reliability of 95% using the ME described in ERN9. From these predicted loadings and the cumulative traffic loadings, the predicted fatigue lives, in years, were calculated. The minimum required fatigue life as per ERN9 is also presented for each of the investigation sections to demonstrate what would be required when undertaking the ME design of the thin asphalt surfacing for the investigation sections.

Table 5.5: Design predicted fatigue life

Section ID	Required granular thickness compliance	Allowable traffic loading at 95% reliability (ESA)[1,2]	Fatigue life based on design traffic (years)	Minimum ERN9 required fatigue design life (years)
G1	Non-compliant	1.3×10^6	***0.5***	15
G2				
G3				
G4				
KF1	Compliant	2.1×10^6	***3***	15
KF2				
KF3				
M1	Non-compliant	1.6×10^6	5	5
M2				
M3	Non-compliant	9.6×10^5	***1***	15
M4	Compliant	2.4×10^6	5	5
RH1	Compliant	7.2×10^6	***2***	15
RH2				
RO1	Non-compliant	9.3×10^5	***2***	5
TH1	Compliant	5.4×10^6	***3***	15

Notes:

1 Mechanistic analysis uses as-constructed granular thicknesses and includes additional 10 mm asphalt construction tolerance as per ERN9 Clause 1.8.

2 See Table 5.3 for granular modulus values.

All investigation sections except M1, M2 and M4 do not meet the required asphalt fatigue design life requirements as per ERN9. This corresponds to a non-compliant ME design of the thin asphalt surfacing.

Sections M1, M2 and M4 are compliant to the required fatigue life as per ERN9 only because of the reduction of required fatigue life to five years as per Clause 1.2 (c). This clause was incorporated into ERN9 to aid designers in meeting the ME design requirements. However, this is not always the case as section RO1 allows the reduction, but still does not meet the fatigue requirements.

Previous Stage 2 Observations

The same comparison of design fatigue life with required fatigue life was undertaken for the Stage 2 trial sections. The results are shown in Table 5.6.

The Reid Highway and Kwinana Freeway trial sections do not meet the required asphalt fatigue design life requirements as per ERN9. This corresponds to non-compliant ME designs of the thin asphalt surfacings for these sections.

5.4.2 Observed Fatigue Life

Table 5.7 compares the observed fatigue lives as reported in Section 4 with the minimum lives specified in ERN9. M4 was excluded from this comparison due to this section only being 1.5 years old.

Table 5.6: Stage 2 trial sections – design predicted fatigue life

Trial section ID	Required granular thickness compliance	Allowable traffic loading at 95% reliability (ESA)[1,2]	Fatigue life based on design traffic (years)	Minimum ERN9 required fatigue design life (years)
T2	Non-compliant	1.0×10^6	6	
T4	Non-compliant	1.7×10^6	10	5
T6	Non-compliant	4.4×10^6	23	
RH2	Compliant	2.6×10^6	***4***	15
RH3		6.4×10^6	***11***	
K2		6.3×10^6	***1***	
K3	Compliant	2.5×10^6	***0.5***	15
K12		2.5×10^6	***0.5***	

Notes:

1 Mechanistic analysis uses as constructed granular thicknesses and includes additional 10 mm asphalt construction tolerance as per ERN9 Clause 1.8.

2 See Table 5.4 for granular modulus values

Table 5.7: Observed fatigue life

Section ID	Required granular thickness compliance	Observed fatigue life (years)[1]	Minimum ERN9 required fatigue design life (years)
G1		> 18	
G2	Non-compliant	≈ 15	15
G3		> 18	
G4			
KF1		≈ 15	
KF2	Compliant		15
KF3		> 24	
M1	Non-compliant	> 30	5
M2			
M3	Non-compliant	> 19	15
RH1	Compliant	≈ 24	15
RH2			
RO1	Non-compliant	≈ 18	5
TH1	Compliant	≈ 26	15

Notes:

1 Where no resurfacing works have taken place, fatigue life is assumed to be greater than (.>) current cumulative traffic level

All sections have an observed fatigue life above the minimum required by ERN9 (MRWA 2013). Furthermore, all investigation sections indicate a fatigue life of 15 years or above. This includes sections which have both compliant and non-compliant granular thicknesses.

This comparison demonstrates that for granular pavements with thin asphalt surfacings which have:

- crushed limestone subbase
- sand subgrade
- granular thickness compliant with the empirical design process

last more than 15 years before the DGA needs to be replaced due to fatigue cracking. It also suggests that the allowance to reduce the design fatigue life from 15 to 5 years through

Clause 1.2 (c) of ERN9 is based on achieving a compliant ME design rather than mimicking actual observed performance.

It is recommended that the design period for asphalt fatigue of 15 years be adopted with the proposed changes to elastic characterisation used in ERN9 design predictions.

Previous Stage 2 Observations

The same comparison of design fatigue life with required fatigue life was undertaken for the Stage 2 trial sections. The results are shown in Table 5.8.

Table 5.8: Stage 2 trial sections– observed fatigue life

Trial section ID	Required granular thickness compliance	Cumulative traffic on asphalt up to first resurfacing, MMF (ESA)	Observed fatigue life (years)[1]	Minimum ERN9 required fatigue design life (years)
T2	Non-compliant	2.4×10^7	> 37	5
T4	Non-compliant	2.6×10^7		
T6	Non-compliant	2.4×10^7		
RH2	Compliant	> 1.3×10^7	> 22	15
RH3		> 1.3×10^7		
K2	Compliant	> 2.8×10^6	> 9[2]	15
K3		> 2.8×10^6		
K12		> 2.8×10^6		

Notes:

1 Where no resurfacing works have taken place, fatigue life is assumed to be greater than (.>) current cumulative traffic level

2 K2, K3, K12 currently 9 years old, with no resurfacing to date

All trial sections which have been in-service for more than 15 years have demonstrated a fatigue life greater than 15 years. This includes sections which have both compliant and non-compliant granular thicknesses.

5.5 Review of 40-year Design Traffic Limit

Currently for thin asphalt pavements, Clause 1.2 (c) of ERN9 allows a reduction of the required fatigue life to five years from 15 years if the 40-year design traffic is less than 3.0×10^7 ESAs. This design traffic limit was based on the performance of various sections of the Leach Highway. At that time, the Leach Highway was one of the few heavily-trafficked metropolitan roads which had sections approaching 40 years of in-service life and were comprised of well-performing thin asphalt surfacing over either CRB or untreated limestone and sand subgrades. An analysis of the traffic data demonstrated that most of these older sections had reached a level of traffic loading close to or just below 3×10^7 ESAs over the 40 years in-service life.

The observed fatigue life, in years, and the predicted 40 year measured traffic loadings are listed in Table 5.9, in addition to the assumed traffic growth rate. The Stage 3 performance data demonstrates that fatigue lives in excess of 15 years are possible for roads with 40 year design traffic values up to 1.1×10^8 ESAs.

Table 5.9: Fatigue life and 40-year cumulative traffic level

Section ID	Required granular thickness compliance	Observed fatigue life (years)[1]	40 year measured traffic (ESA)[2]	Traffic growth rate (%)
G1	Non-compliant	> 18	4.1×10^7	2.7
G2		≈ 15		
G3		> 18		
G4				
KF1	Compliant	≈ 15	1.1×10^8	3.6
KF2				
KF3		> 24		
M1	Non-compliant	> 30	7.5×10^6	8.6
M2				
M3	Non-compliant	> 19	1.1×10^8	8.9
RH1	Compliant	> 24	4.2×10^7	1.5
RH2		≈ 24		
RO1	Non-compliant	≈ 18	3.2×10^7	2.3
TH1	Compliant	≈ 26	5.5×10^7	4.0

Notes:

1 Where no resurfacing works have taken place, fatigue life is assumed to be greater than (.>) current service life.

2 Mean 40 year measured traffic predicted using calculated traffic data.

5.6 Predicted and Measured Deflection Bowls

A comparison of the measured and predicted deflection bowl data was undertaken to demonstrate the difference between the moduli used in design and the in situ moduli. The measured deflection bowls used were the bowls associated with the characteristic maximum deflections (95 percentile values).

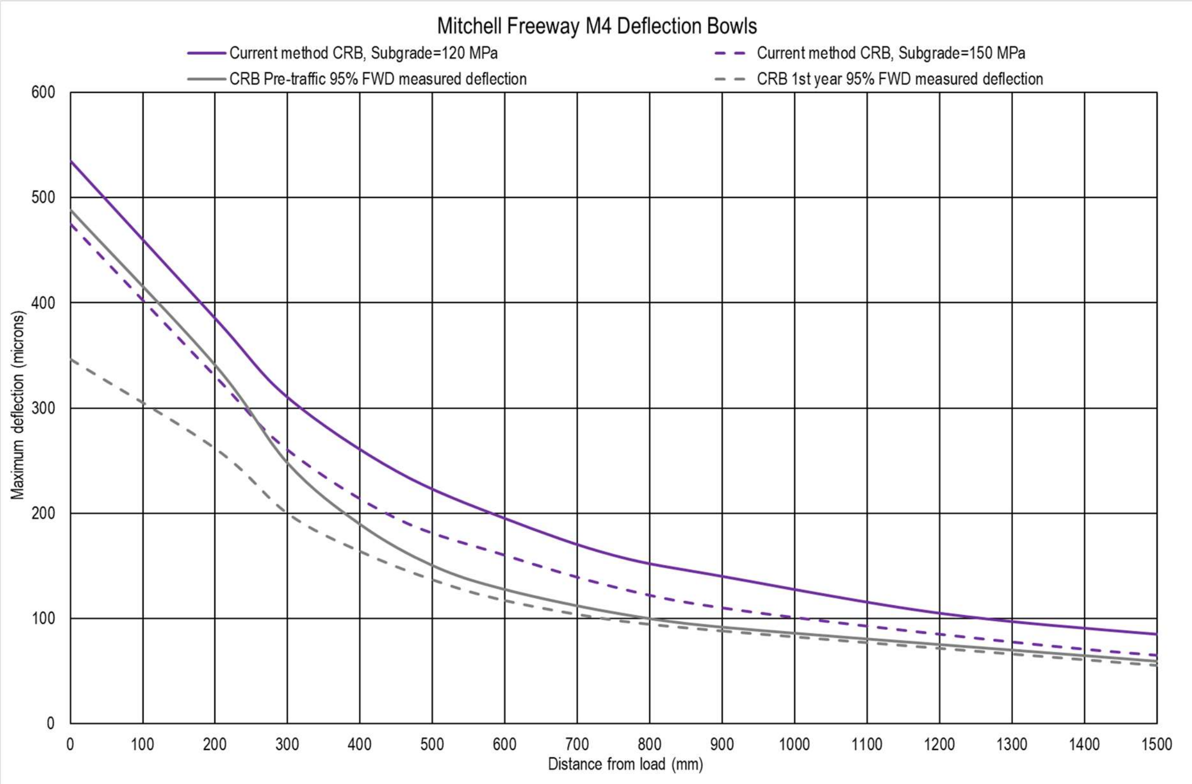
5.6.1 Early Life

Figure 5.1 shows the measured deflection bowls a month before the Mitchell Freeway investigation Section M4 was opened to traffic and eight months after. Also shown are the predicted bowls using the linear elastic model CIRCLY with the ERN9 design moduli and as-constructed layer thicknesses.

The results demonstrate that the predicted deflections are higher than the measured deflections. If a subgrade design modulus of 150 MPa is assumed instead of the ERN9 value of 120 MPa, the resulting deflections are closer to the FWD measured values, specially pre-traffic. The deflections measured eight months after opening to traffic, especially closer to the load, are smaller, indicating stiffening of the pavement with loading/age.

The same observations regarding early-life deflection bowls were also made in Stage 2.

Figure 5.1: Comparison of predicted and measured deflection bowls – Mitchell Freeway CRB



5.6.2 Long-Term Life

Figure 5.2 to Figure 5.6 show the long-term measured deflection bowls for the Graham Farmer Freeway, Kwinana Freeway and Roe Highway investigation sections. Also shown are the predicted bowls using CIRCLY with presumptive moduli and as-constructed layer thicknesses.

The results demonstrate that the predicted deflections are higher than the measured deflections when the ERN9 subgrade modulus of 120 MPa is used. This is observed even when a higher subgrade design modulus of 150 MPa is used. This demonstrates the conservative nature of the current design method and the underprediction of pavement stiffness and strength.

The same observations regarding the difference in long-term measured and predicted deflection bowls were also made in Stage 2.

Figure 5.2: Comparison of predicted and measured deflection bowls – Graham Farmer Freeway G1 BSL

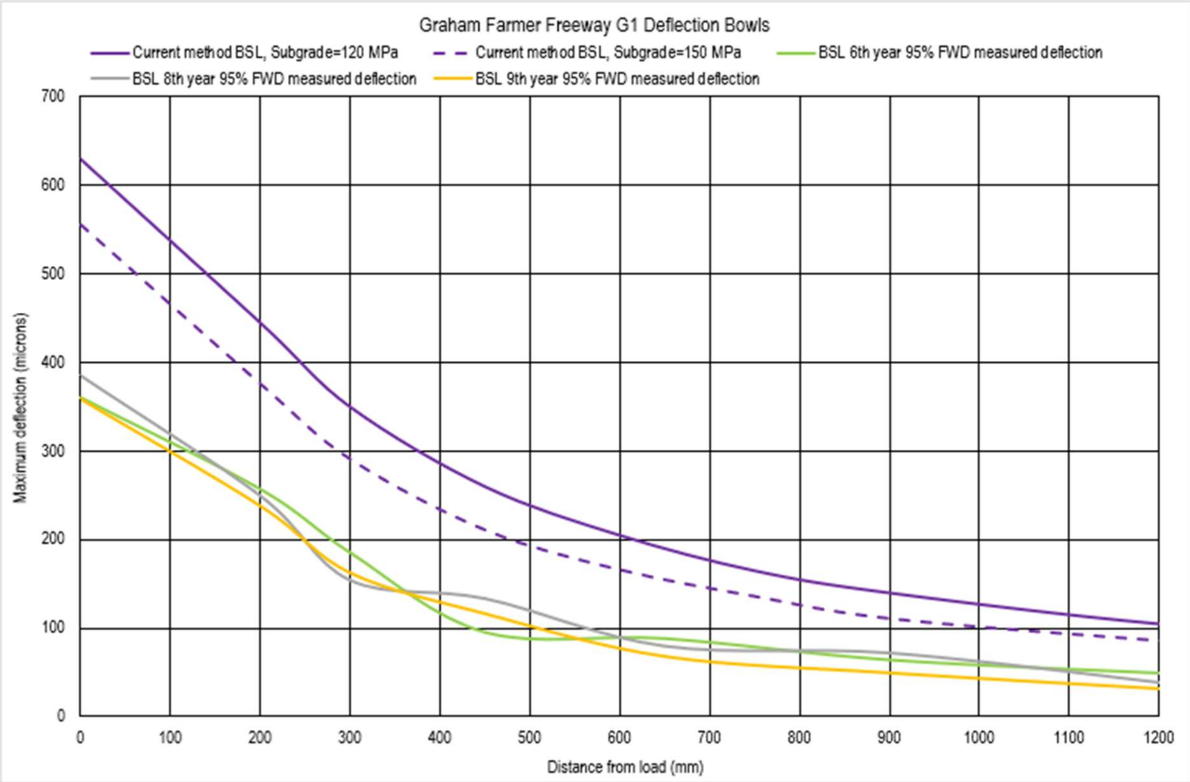


Figure 5.3: Comparison of predicted and measured deflection bowls – Graham Farmer Freeway G3 BSL

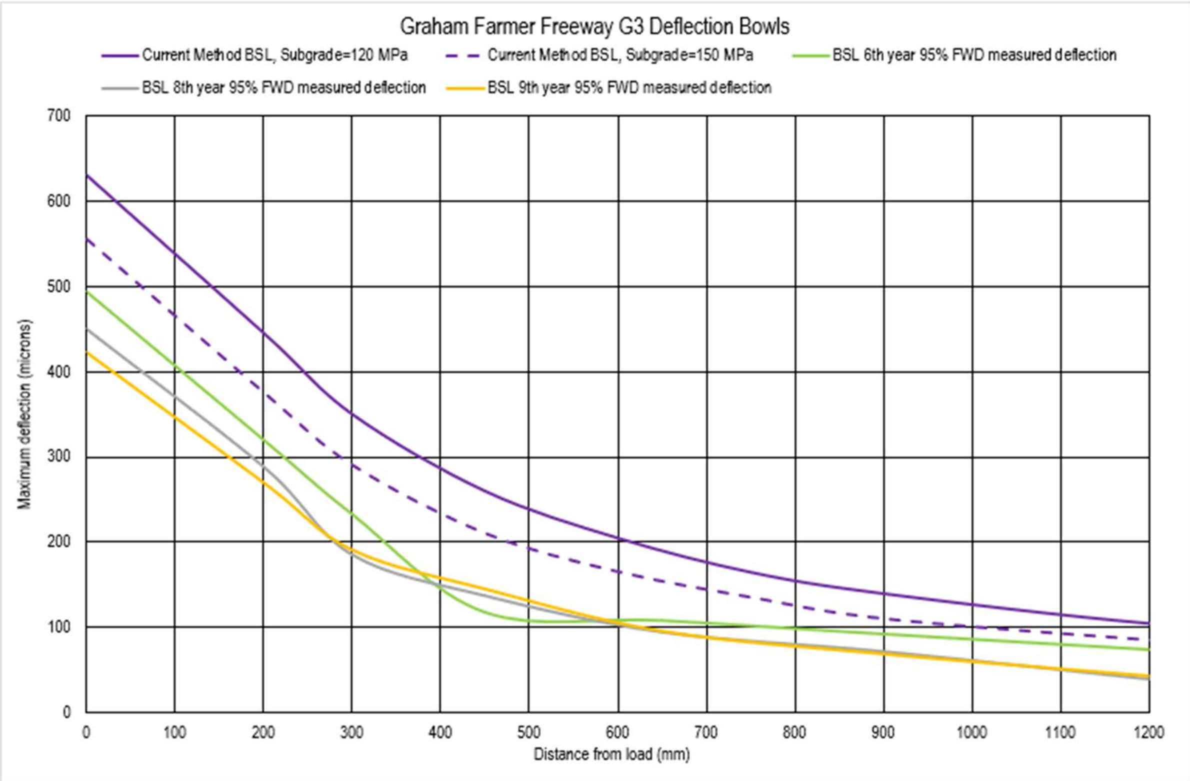


Figure 5.4: Comparison of predicted and measured deflection bowls – Kwinana Freeway KF1 CRB

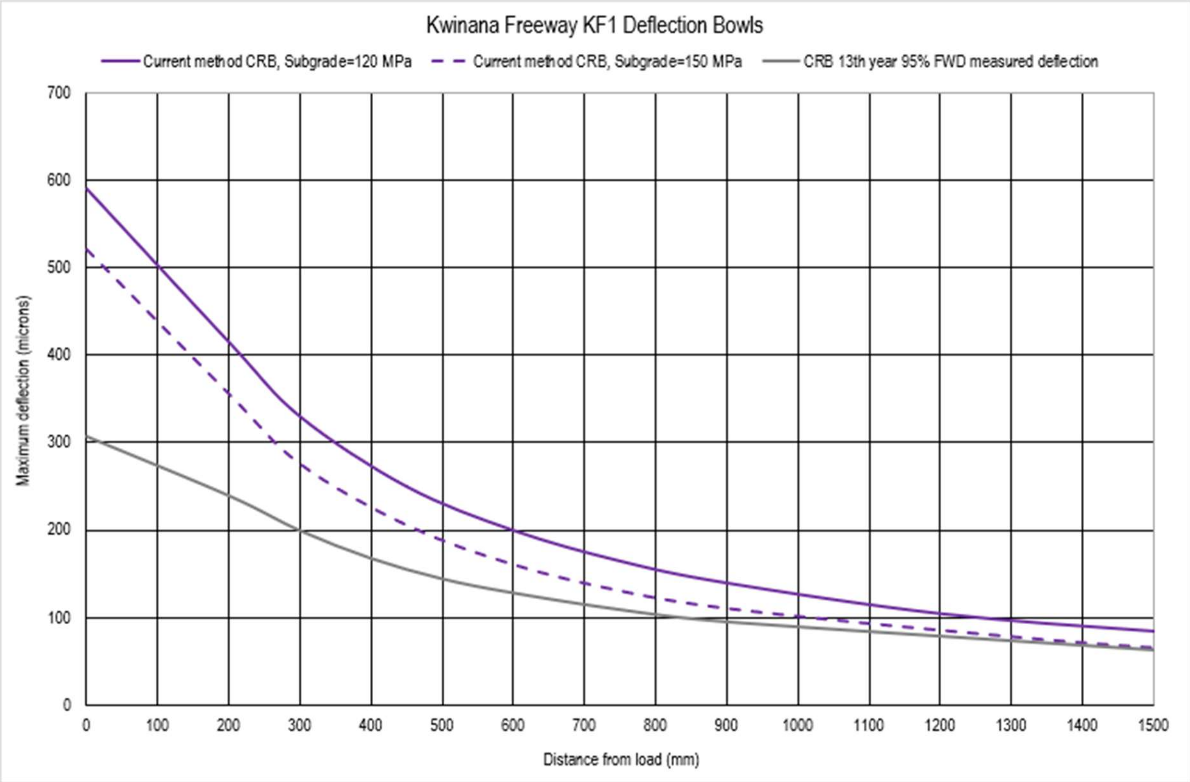


Figure 5.5: Comparison of predicted and measured deflection bowls – Kwinana Freeway KF3 CRB

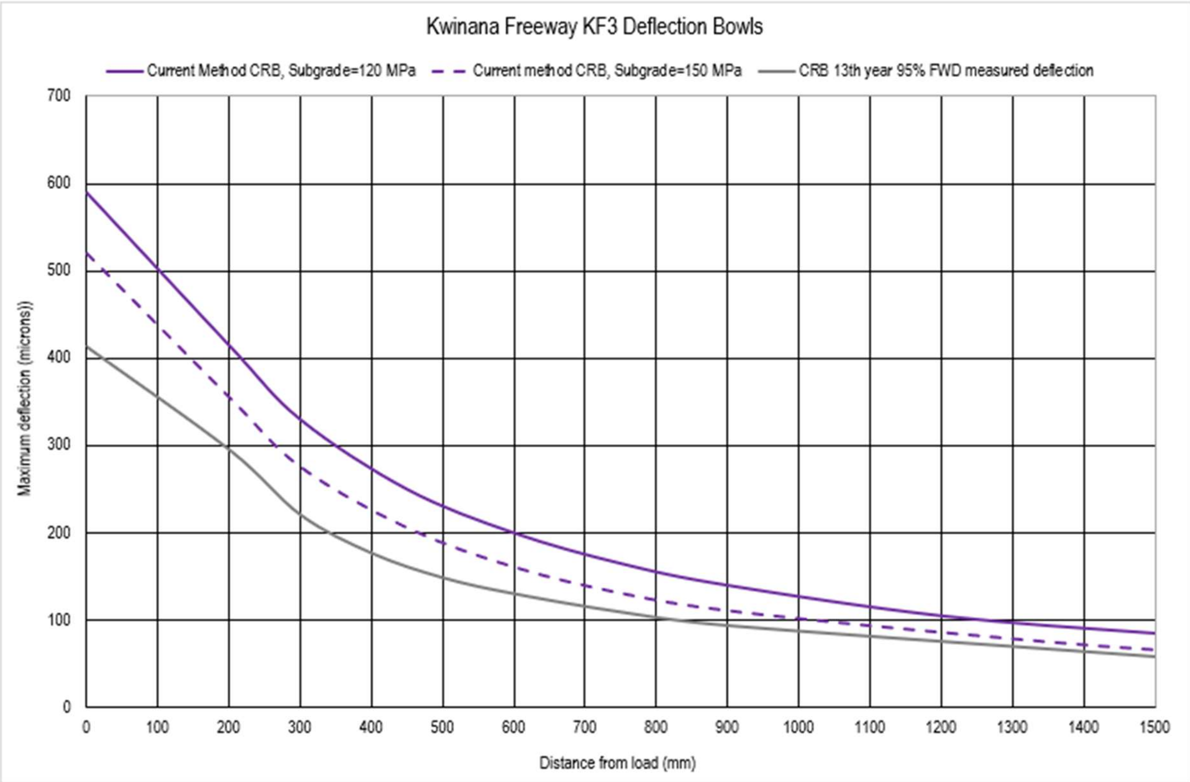
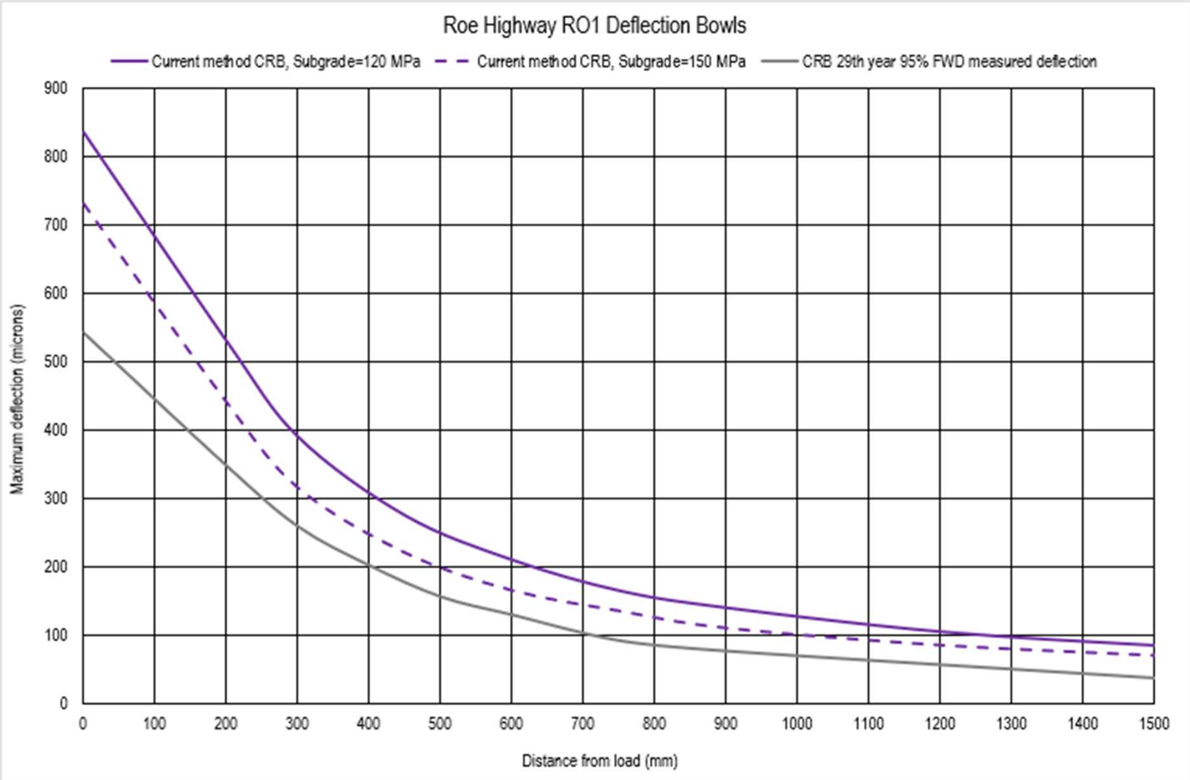


Figure 5.6: Comparison of predicted and measured deflection bowls – Roe Highway RO1 CRB



5.7 Back-Calculated Modulus

5.7.1 Method

A linear elastic analysis program which back calculates layer moduli from deflection measurements (EFROMD3) was used in conjunction with the results of the available FWD data. Network level FWD data was insufficient to provide meaningful back-calculation results; therefore only project level FWD data was used. Project level data for GFF collected in 2006 has not been presented due to high back-calculation errors. These high errors are anticipated to have been caused by incorrect documentation of geophone locations during testing.

To undertake the back-calculation, the as-constructed thicknesses of each pavement layer were used. Due to the lack of measured asphalt thicknesses for the Stage 3 investigation sections, two asphalt thicknesses were modelled in EFROMD3 and the representative layer moduli averaged for both scenarios. This included 60 and 80 mm for OGA/DGA pavements, and 30 and 40 mm for DGA pavements.

Furthermore, due to the thin asphalt thickness, the asphalt modulus was fixed during the back-calculation. The fixed modulus was based on that used for design and adjusted to reflect the surfacing age of each back-calculated section.

Layer data used for the back-calculation is presented in Appendix B in addition to the back-calculation output for each of the surfacing thicknesses modelled.

5.7.2 Results

Table 5.10 presents a summary of the back-calculated representative moduli from the project level FWD data. The values chosen to calculate these representative moduli were those at test chainages with deflections close to the characteristic deflection a method which is detailed in Appendix E of Austroads (2011).

Table 5.10: Representative back-calculated modulus – project level data

Section	Base	Pavement age	Back-calculated modulus (MPa)					
			Asphalt	Base	Limestone	Subgrade		
						0 - 300 mm	300 - 500 mm	Semi-infinite
Mitchell Fwy	CRB	Pre-traffic	2,000	309	268	192	266	404
		1 year	2,300	494	405	183	191	207
Kwinana Fwy	CRB	13 years	5,000	794	360	191	192	193
Roe Highway	CRB	28 years	2,500	1,000	375	119	168	260
Graham Farmer Fwy	BSL	8 years	3,000	394	372	279	293	314
		9 years	3,000	458	392	278	279	282

The back-calculated moduli represent the modulus value for the third sublayer of the sub-layered base material. To calculate the modulus at the top of these layers (Ev_{top}), Equation 1 can be used. The calculation of Ev_{top} is presented in Table 5.11.

$$Ev_{top} = Ev_{back-calculated}\left(\left[\frac{Ev_{back-calculated}}{Ev_{underlying\ material}}\right]^{\frac{1}{3}}\right)^{2} \qquad 1$$

where

Ev_{top} = Top layer modulus

$Ev_{back\text{-}calculated}$ = Back-calculated modulus (third sublayer)

$Ev_{underlying\ material}$ = Underlying material modulus

Table 5.11: Back-calculated Ev_{top}, base layer

Section	Base	Pavement age	Ev_{top} (MPa)
Mitchell Fwy	CRB	Pre-traffic	340
		1 year	578
Kwinana Fwy		13 years	1015
Roe Hwy		28 years	2,010
Graham Farmer Fwy	BSL	8 years	409
		9 years	507

6 DISCUSSION

6.1 Investigation Sections

From the analysis and review of the investigation sections, the following observations regarding the current use of the ME procedure for thin asphalt-surfaced pavements can be made:

- In terms of the in-service performance of sections which were considered to have fatigue cracking based on maintenance treatments applied (Table 5.3):
  - Kwinana Freeway CRB sections K1 and K2 with OGA/DGA surfacing had a mean observed fatigue life of 15 years corresponding to 3.5 x 10^7 ESAs: the predicted mean fatigue life was an underestimate of the observed performance.
  - Reid Highway CRB section RH2 with DGA surfacing had a mean observed fatigue life of 24 years corresponding to 2.3 x 10^7 ESAs: the predicted mean fatigue life was consistent with the observed performance.
  - Roe Highway CRB section RO1 with DGA surfacing had a mean observed fatigue life of 18 years corresponding to 2.9 x 10^7 ESAs: the predicted mean fatigue life was an underestimate of the observed performance. This section also had a non-compliant granular thickness.
  - Tonkin Highway CRB section TH1 with DGA surfacing had a mean observed fatigue life of 26 years corresponding to 2.6 x 10^7 ESAs: the predicted mean fatigue life was consistent with the observed performance.
- In terms of the in-service performance of sections which have not fatigue cracked (Table 5.3):
  - The design predicted mean fatigue life was an underestimate of the observed performance. This includes sections with both compliant and non-compliant granular thicknesses.
- Considering the deflection and curvature trends:
  - Within the standard thin asphalt fatigue design period of 15 years, there are two similar phases of performance behaviour. These are typically consistent with those inferred from the Stage 2 investigation, with the previously-identified 2nd and 3rd stage demonstrating similar behaviour:
    - Phase 1 (pre-traffic to end of 1st year): – pavement system strengthens with the application of the first year of traffic
    - Phase 2 (end of 1st year to around end of 15th year) – pavement strength typically remains consistent and may slightly increase.
  - Beyond the standard 15 year fatigue design period, the pavements show a stabilisation of strength and may slightly decrease.
- Considering the comparison of deflection bowls:
  - Increasing the subgrade design modulus to 150 MPa provides a closer predicted to the early-life measured bowls. This is consistent with Stage 2 observations.
  - The measured pre-traffic bowls have higher deflections than the measured bowls after traffic has been applied. This is consistent with Stage 2 observations.

  - The predicted deflection bowls after traffic is applied are consistently underestimates of the measured deflection bowls, even when the subgrade design modulus is increased to 150 MPa. This is consistent with Stage 2 observations.
- Considering the back-calculated moduli results:
  - The CRB modulus is typically around 450 MPa after the first year of traffic and was also calculated to reach 1,000 MPa after 13 years of in-service life. By comparison, in the design model a modulus of only 500 MPa is used for the top sublayer.
  - The BSL base modulus was calculated to reach a level of 450 MPa after nine years. It could be inferred using linear extrapolation that it may increase to approximately 900 MPa after 15 years; however, this is based on limited data. This is consistent with Stage 2 observations.
  - The limestone subbase modulus is typically around 340 MPa pre-traffic. During the long-term phase the modulus is on average approximately 550 MPa and typically did not increase like the base material. By comparison, in the current design model an average modulus of 250 MPa is effectively used.
  - The subgrade modulus is consistently above 150 MPa with the majority over 190 MPa.

All these observations suggest that, for thin asphalt-surfaced granular pavements in Perth which include crushed limestone subbases and sand subgrades:

- The current design method does not replicate observed performance and is typically an underestimate of fatigue life. This is consistent with the Stage 2 observations.
- The typical fatigue life of granular pavements with compliant granular thickness as per the empirical design method is at least 15 years for the roads investigated.
- Granular base materials increase in modulus and may be modelled using a short-term and long-term modulus. This is consistent with the Stage 2 observations.
- The short-term granular moduli are similar to those used by the current ME method and the long-term moduli for limestone subbase and BSL can be increased. This is consistent with the Stage 2 observations.
- The limestone subbase has higher modulus than currently assumed and has a consistent modulus throughout in-service life. This is consistent with the Stage 2 observations.
- The Perth sand subgrades are higher in modulus than currently assumed in design. This is consistent with the Stage 2 observations.

6.2 Limitations of Current Fatigue Prediction Method

When predicting the asphalt fatigue life of thin asphalt-surfaced granular pavements that operate in high-trafficked conditions, the design is highly dependent on the assumed basecourse and asphalt moduli in addition to the thickness of the asphalt layer. Small changes in these parameters affect the predicted life. An increase in the total granular pavement thickness does not significantly increase the asphalt fatigue life of the pavement but does have a slight effect, with thicker pavements providing slightly higher fatigue life. The current design methodology results in designers sometimes recommending very thick granular pavement options (thicknesses of up to about 600 mm) on a sand subgrade (design CBR 12%) in order to ensure that the predicted allowable loading exceeds the design traffic.

A review of the literature (presented in Appendix C) demonstrated that the current Austroads (2018) methodology, in conjunction to the requirements in ERN9 (MRWA 2013), underestimates the support to the thin asphalt layer for pavements in Perth with crushed limestone subbases and sand subgrades. This results in higher calculated horizontal strains at the bottom of the asphalt layer and hence a reduced allowable traffic load repetitions and asphalt design life.

In Stage 2 it was suggested that an improved characterisation of the granular pavement materials, as well as the subgrade, can substantially improve the accuracy of the design outcomes. Two proposed modifications, based on the reviewed literature include:

- the allowance of higher design subgrade modulus values
- modification of the sub-layering rules where the subbase material is capable of developing similar or higher moduli than the basecourse material at the same stress (confinement) levels.

These two modifications identified through the literature review are also supported by the findings of this investigation as summarised in Section 6.1.

Other areas of further investigation which also influence the behaviour of thin asphalt surfacings over granular pavements include:

- Asphalt fatigue relationship for thin asphalt surfacings:
  - The Austroads (2018) asphalt fatigue equation, adopted by MRWA, was originally developed more than 35 years ago using test results from European mixes. Although the laboratory-to-field shift factor and reliability factor were adjusted to fit Australian experience, these factors do not vary with asphalt thicknesses. The South African pavement design method uses different coefficients within the asphalt fatigue function when considering asphalt layers less than 50 mm thick(Appendix C). For the same level of strain, the fatigue equation used for thinner asphalt layers (i.e. < 50 mm) results in a greater number of allowable load repetitions.
- The influence of the use of polymer modified binders:
  - Similarly to asphalt thickness, the asphalt fatigue equation does not take into consideration current Australian mixes and the effect of polymer modified bitumen in extending asphalt fatigue life.
- Asphalt modulus variation with speed and temperature:
  - Austroads (2018) provides adjustment factors to asphalt design moduli to account for vehicle speed and pavement temperature. These adjustment factors are a simplified way of considering the viscoelastic nature of asphalt. The current equations provided in Austroads (2018) to calculate these adjustment factors are a function of vehicle speed and WMAPT. Differences in load time and temperature with depth are not considered. According to Jameson (2013), the adjustment factors are calculated for a 100 mm thick asphalt layer.
- Asphalt self-healing phenomenon:
  - Austroads (2018) does not take into consideration the effect of healing in asphalt. Studies by several researchers, such as Bazin and Saunier (1967) and Van Dijk and Visser (1977), have identified that rest periods between loading allow the recovery of tensile strength by the asphalt, thus extending the fatigue life of the pavement.

7 POSSIBLE DESIGN REVISIONS

Based on the above discussion and on the findings presented in Section 0, the following design revisions to predict allowable traffic loading to fatigue relevant to granular pavements with thin asphalt surfacings are suggested.

7.1 Option 1: Short-term and Long-term Fatigue

7.1.1 Short-term Life

The short-term life (≤ 1 year) represents the weakest stage of a granular pavement. The modulus values during this phase are similar to those already implemented. In particular, there is no need to increase the current design modulus of the limestone subbase in this phase and therefore no change is required to the current Austroads sub-layering process. The subgrade support in the short-term phase has been kept at 120 MPa as this phase represents the lowest modulus of the pavement system.

Table 7.1 presents the proposed short-term life modelling assumptions.

Table 7.1: .Proposed short-term presumptive granular moduli and elastic modelling

Material	Layer	Modulus (MPa)[1]	Poisson ratio, ν	Isotropy	Sub-layering
CRB	Base	600	0.35	Anisotropic	Base and subbase total thickness divided into five sub-layers
BSL	Base	500			
Limestone	Subbase	250			
Sand	Subgrade	120			N/A

Notes:
1 Maximum allowed top sublayer vertical modulus.

7.1.2 Long-term Life

During the long-term life (≥ 1 year), a different design method is proposed which reflects the higher modulus of base, subbase and subgrade due to loss of compaction moisture, trafficking and curing. Furthermore, to allow the revised design method to align with the observed lives and to allow the greater contribution of the subbase, the long-term elastic characterisation includes sub-layering of the base into five sublayers, whilst the subbase is modelled separately and not sub layered.

Subgrade

The back-calculation data demonstrated that, in the long-term, Perth sand subgrades have much higher moduli than assumed currently. To reflect this higher support, the subgrade modulus during the long-term phase has been increased from 120 MPa to 150 MPa. Back-calculation on the subgrade indicated a higher subgrade modulus and finite element modelling (Jameson et al. 2017). However, a vertical modulus of 150 MPa is proposed consistent with the maximum subgrade modulus recommended by Austroads (2018).

Base and Subbase

As it is proposed to limit the subgrade vertical design modulus to 150 MPa, it was of interest to repeat the modulus back-calculation with this top subgrade layer (0-330 mm) constrained to this maximum. The revised back-calculated moduli are presented in Table 7.2.

Table 7.2: Representative back-calculated modulus with fixed subgrade

Section	Base	Pavement age	Back-calculated modulus (MPa)					
			Asphalt	Base	Subbase	Subgrade		
						0 – 300 mm	300 – 500 mm	Semi-infinite
Mitchell Fwy	CRB	1 year	2,300	530	390	150	2300	530
Kwinana Fwy		13 years	5,000	740	476	150	5000	740
Graham Farmer Fwy	BSL	8 years	3,000	515	547	150	3000	515
		9 years	3,000	524	534	150	3000	524

Note:

- Mitchell Freeway pre-traffic and Roe Highway data excluded as outside of the long-term phase (1-15 years) and high back-calculation errors obtained when fixing the subgrade modulus.

As the base will be sub-layered for the long-term fatigue life assessment, the modulus of the base material is based on the Ev_{top} value of this layer. The calculated Ev_{top} for the base layers using the previously presented Equation 1 and the moduli values are presented in Table 7.3.

Table 7.3: Calculated base Ev_{top} values

Section	Base	Ev_{top} (MPa)
		Base
Mitchell Fwy	CRB	650
Kwinana Fwy	CRB	1000
Graham Farmer Fwy	BSL	500
		520

As the subbase will not be sub-layered during long-term life modelling, the design modulus for the subbase is therefore based on the back-calculated modulus (Table 7.2).

Long-term life elastic characterisation

Table 7.4 presents the proposed elastic characterisation for the long-term life phase.

Table 7.4: Proposed long-term presumptive elastic characterisation of base, subbase and sand subgrade

Material	Layer	Modulus (MPa)	Poisson's ratio, ν	Isotropy	Sublayering
CRB	Base	850 (E_{vtop})	0.35	Anisotropic	Total thickness of each base material is divided into 5 sublayers
BSL	Base	550 (E_{vtop})			
Limestone	Subbase	500			Subbase not sub-layered
Sand	Subgrade	150			N/A

7.1.3 Calculating Total Fatigue Damage and Allowable Traffic Loading

Equation 2 through to Equation 5 detail the method of calculating the fatigue damage in the short-term and long-term phases, the overall fatigue damage and the total allowable traffic loading to fatigue.

$$STD = \frac{1st\ year\ design\ traffic}{short - term\ allowable\ loading} \quad (2)$$

where

STD	=	Short-term fatigue damage
1st year design traffic	=	Cumulative traffic loading at the end of the 1st year in service (ESAs)
short-term allowable loading	=	Allowable traffic loading in terms of fatigue calculated using short-term elastic characterisation

$$LTD = \frac{15\ year\ design\ traffic -\ 1st\ year\ design\ traffic}{long - term\ allowable\ loading} \quad (3)$$

where

LTD	=	Long-term fatigue damage
1st year design traffic	=	Cumulative traffic loading at the end of the 1st year in service (ESAs)
15 year design traffic	=	Cumulative traffic loading over the 15 year design period (ESAs)
long-term allowable loading	=	Allowable traffic loading in terms of fatigue calculated using long-term elastic characterisation (ESAs)

$$CFD = STD + LTD\ \leq 1.0 \quad (4)$$

where

CFD	=	Cumulative fatigue damage
STD	=	Short-term damage factor
LTD	=	Long-term damage factor

$$Total\ allowable\ traffic\ loading\ to\ fatigue = \frac{15\ year\ design\ traffic}{CFD} \quad (5)$$

An example of the proposed revision incorporated in to of ERN9 Clause 1.2 (c) is presented in Appendix D.

7.1.4 Revised Method and Observations

To check that the revised elastic characterisation reflects the observation that a fatigue life of at least 15 years is typical of granular pavements with thin asphalt surfacings, revised designs were undertaken for the investigation sections which were concluded to have fatigue cracked. The allowable fatigue traffic was subsequently compared to the measured cumulative traffic after 15 years. This comparison is presented in Table 7.5.

Table 7.5: Revised method using back-calculated moduli compared to observations

Section	Base	1st year traffic (ESA)	15 year traffic (ESA)	ST allowable loading at 95% reliability	STD	LT allowable loading 95% reliability	LTD	Total damage
Kwinana Fwy	CRB	1.6×10^6	2.5×10^7	2.1×10^6	0.7	9.1×10^6	2.6	3.3
Reid Hwy		9.5×10^5	1.4×10^7	7.2×10^6	0.1	5.8×10^7	0.2	0.3
Roe Hwy		1.4×10^6	2.3×10^7	9.3×10^5	1.5	2.1×10^7	1.0	2.5
Tonkin Hwy		8.1×10^5	1.2×10^7	5.4×10^6	0.1	1.1×10^7	1.0	1.1
Graham Farmer Fwy	BSL	1.2×10^6	1.2×10^7	1.3×10^6	0.9	3.3×10^6	3.3	4.2

The Roe Highway investigation section has a short-term damage factor above 1. This is anticipated to be due to the pavement thickness of this section being below that required by the empirical thickness design presented in Figure 8.4 of Austroads (2018).

The revised method is an improvement and typically matches the observation of a 15-year fatigue life better than the current design method (see Section 7.2). For the four pavements with CRB the change in the elastic characterisation increases the predicted fatigue life by factors of 3.5, 6.5, 9.9 and 2.0, respectively. For the Graham Farmer Freeway which has the BSL base, the predicted fatigue life increases by a factor 2.0.

7.2 Option 2: Current Method Shift-factor

An alternative method to improve the agreement between predicted and observed fatigue performance is to increase the current laboratory-to-field shift factor of 6 (95% reliability) rather than change the elastic characterisation.

Table 7.1 lists the allowable traffic loading using the current design method and quantifies the extent to which the current shift factor of 6 underestimates the observed fatigue lives.

Table 7.6: Adjustment factors to the current shift factor of 6

Section	Base	15 year traffic (ESAs)	Allowable traffic (ESAs)	15 year traffic / allowable traffic
Kwinana Fwy	CRB	2.5×10^7	2.1×10^6	11.7
Reid Hwy		1.4×10^7	7.2×10^6	2.0
Roe Hwy		2.3×10^7	9.3×10^5	25.0
Tonkin Hwy		1.2×10^7	5.4×10^6	2.2
Graham Farmer Fwy	BSL	1.2×10^7	1.3×10^6	9.1

To determine the shift factor, various percentile values of the five calculated ratios of 15 year traffic to allowable traffic from Table 7.6 have been calculated and are presented in Table 7.8. At 95% reliability level the shift factor is calculated by multiplying the percentile values by the reliability factor of 6.0.

Table 7.7: Percentile values to determine SF/RF

Percentile	SF/RF	SF
95th	2.0	12.0
90th	2.1	12.6
80th	2.2	13.2
70th	3.6	21.6

Using the modulus and elastic characterisation data presented in Table 7.1 in conjunction with the shift-factors derived in Table 7.8, revised designs were undertaken for the investigation sections which demonstrated fatigue. The allowable fatigue traffic was subsequently compared to the measured cumulative traffic at 15 years. This comparison is presented in Table 7.8.

Table 7.8: Current method with the revised shift-factor

Section	Base	Allowable traffic loading, SF=6 (ESAs)	15 year traffic / Allowable traffic, SF=12.0	15 year traffic / Allowable traffic, SF=12.6	15 year traffic / Allowable traffic, SF=13.2	15 year traffic / Allowable traffic, SF=21.6
Kwinana Fwy	CRB	2.1×10^6	5.8	5.7	5.4	3.3
Reid Hwy		7.2×10^6	1.0	0.9	0.9	0.5
Roe Hwy		9.3×10^5	12.4	12.1	11.5	6.9
Tonkin Hwy		5.4×10^6	1.1	1.1	1.0	0.6
Graham Farmer Fwy	BSL	1.3×10^6	4.6	4.4	4.2	2.5

This demonstrates that the increased shift factor of 21.6 in conjunction with the current method is an improvement and matches the observation of a 15-year fatigue life most appropriately compared with the lower shift factor values.

7.2.1 ERN9 Investigation

As part of the update of ERN9 undertaken through a separate WARRIP project (Tseng & van Aswegen 2019), an investigation was undertaken to develop a similar shift factor which enabled similar fatigue traffic outcomes when comparing AGPT02-17 (rev. edn. Austroads 2018) using a 15 year design period with AGPT02-12 (Austroads 2012) substituting Clause 1.2 (c) (five year design period). This investigation found that the required shift factor varies with traffic growth.

The required shift factor – depending on annual traffic growth – was analysed and the results are presented in Table 7.9 and Figure 7.1.

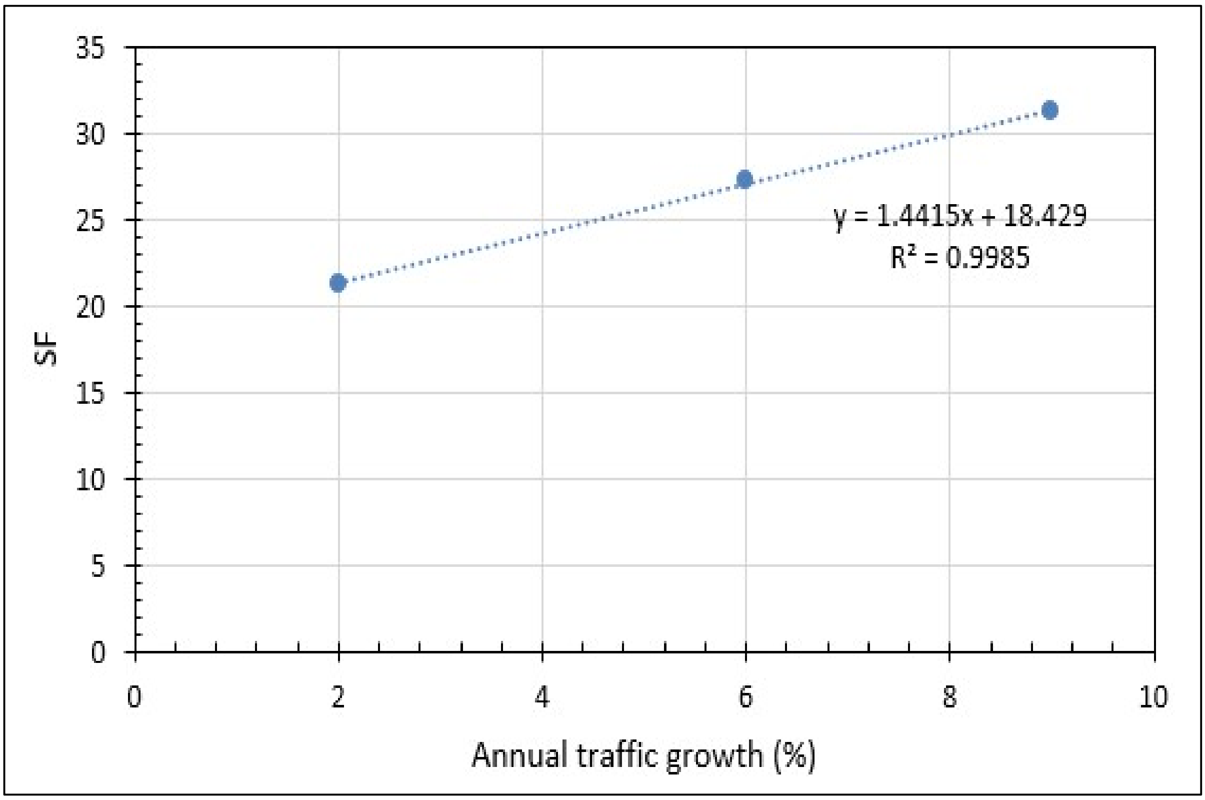
Table 7.9: Suggested SF/RF and RF for use with the AGPT2 (2017) and a 15 year design life

Annual Traffic Growth Factor (%)	SF/RF	SF (assuming RF = 6.0)
2.0	3.6	21.6
6.0	4.5	27.0
9.0	5.3	31.8

Source: Tseng & van Aswegen (2019).

The investigation recommended a single shift factor of 23.0 to be used which corresponded to the average growth rate in Perth of 3.0%.

Figure 7.1: Suggested SF/RF and RF for use with the Austroads (2017) and a 15-years design life



Source: Tseng & van Aswegen (2019).

Considering the data set used for this investigation, the measured growth rates for each of the investigation sections which did demonstrate fatigue are presented in Table 7.10. The average growth rate of this data set is 2.4%. This would correspond to a shift factor of 21.8 when using the analysis reported by Tseng and van Aswegen (2019) (Figure 7.1). This demonstrates that choosing a shift factor of 21.6 using the 70th percentile value from Table 7.8 is justified.

Table 7.10: Investigation sections which demonstrated fatigue – measured growth rate

Section and ID	Measured growth rate (%)
Kwinana Fwy (KF1, KF2)	3.3
Reid Hwy (RH2)	1.0
Roe Hwy (RO1)	2.3
Tonkin Hwy (TH1)	4.0
Graham Farmer Fwy (G2)	1.6
Average	2.4

8 SUMMARY

The aim of Stage 3 of this project was to further investigate, and ultimately validate, observations made in Stage 2 through the analysis of similar sets of observational and historic performance and monitoring data relating to various sections of the metropolitan network. The focus of Stage 3 was pavements which had been constructed either as part of Design and Construct, Construct only, or Alliance contracts, rather than as part of pavement trials. This was undertaken with the aim of providing vital insight into the applicability of the previous Stage 2 findings using pavements constructed to a more representative, standard quality level.

Asphalt fatigue was deduced to have occurred along several of the Stage 3 investigation sections based on maintenance records. For each investigation section, the cumulative traffic loading when fatigue was observed, or in the non-fatigued cases cumulative traffic to date, was compared to the predicted allowable traffic loading. The predicted lives were the mean asphalt fatigue lives rather than the lives with 95% reliability commonly used in design. The mean fatigue life is approximately six times the 95% fatigue life. The predicted mean or best estimate of the allowable loading is most suitable for comparison against the observed performance.

It was determined that, for all but two investigation sections, the cumulative traffic loadings were far larger than the mean predicted allowable traffic loadings. It was also observed that, for the pavements investigated, the typical fatigue life of thin asphalt-surfaced granular pavements was at least 15 years in Perth.

Analysis of deflection and curvature data also demonstrated similar performance phases throughout the in-service life of the pavements. These apparent phases were also observed in the Stage 2 investigation and have subsequently been used as the basis of a revised design methodology which considers both short- and long-term fatigue design phases.

The 40 year predicted traffic levels at the investigation sections which demonstrated fatigue were between 3.2×10^7 and 1.1×10^8 ESAs with a mean value of 6.8×10^7 ESAs. This is above the current specified limit of 3.0×10^7 ESAs for Clause 1.2(c) of the current ERN9. Furthermore, the fatigue life of these sections was typically above 15 years which may remove the need for a reduced fatigue life clause and subsequently remove the need for traffic criteria.

In situ base, subbase and subgrade moduli were back-calculated from measured surface deflections. The moduli of the sand subgrade exceeded 150 MPa, which was were well in excess of the ERN9 design modulus of 120 MPa. In addition, the moduli of the limestone subbase were greater than 250 MPa. These findings are supported by the outcomes of the Stage 2 investigation and a study reported by Jameson et al. (2017) in which layer moduli were estimated from the laboratory testing of samples from the Kwinana Freeway trials. As with the Stage 2 observations, Stage 3 confirmed that a new method of elastic characterisation is needed to more appropriately reflect the structural contribution of the sand subgrade/limestone subbase to the fatigue performance of thin asphalt surfacings. Such a change to the elastic characterisation may enable the restoration of a 15 year design period for thin asphalt fatigue across all traffic loadings rather than the five year period currently specified.

It is recommended that MRWA consider the findings presented in this report in relation to the revision of the ERN9 design procedures for thin asphalt-surfaced granular pavements.

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APPENDIX A FWD DATA

Graham Farmer Freeway Network Level Data

															Normalised to 700 kPa		Normalised to 700 kPa					
															No Temp Correction		Austroads 2008 Correction to 29C					
Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Deflection (mm)	Curvature (mm)	Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
G1	60	EB	3.29	2002 survey	17.2	700	492	363	276	213	170	142	116	101	62	0.49	0.13	1.08	0.53	***0.53***	1.24	0.16	***0.16***
			3.30	2003 Survey	14.3	700	477	332	249	183	145	120	97	83	53	0.48	0.15	1.11	0.53	***0.53***	1.33	0.19	***0.19***
			3.29	2004 Survey	19.5	700	463	339	258	200	162	135	109	93	57	0.46	0.12	1.06	0.49	***0.49***	1.18	0.15	***0.15***
			3.30	2005 Survey	16.0	700	528	399	297	222	177	143	115	96	60	0.53	0.13	1.09	0.58	***0.58***	1.28	0.16	***0.16***
			3.31	2007 Survey	15.9	700	481	356	288	225	182	153	125	105	61	0.48	0.13	1.09	0.52	***0.52***	1.28	0.16	***0.16***
			3.16	2008 Survey	16.7	700	390	284	233	182	148	122	101	85	54	0.39	0.11	1.08	0.42	***0.42***	1.26	0.13	***0.13***
			3.07	2010 Survey	24.9	700	447	273	189	129	96	76	62	56	38	0.45	0.17	1.02	0.46	***0.46***	1.07	0.19	***0.19***
G2	60	EB	5.60	2002 Survey	17.2	700	380	263	195	156	125	100	86	71	44	0.38	0.12	1.08	0.41	***0.41***	1.24	0.15	***0.15***
			5.60	2003 Survey	14.3	700	350	245	187	143	115	97	79	66	39	0.35	0.11	1.11	0.39	***0.39***	1.33	0.14	***0.14***
			5.60	2004 survey	19.5	700	324	219	158	113	88	73	58	50	30	0.32	0.11	1.06	0.34	***0.34***	1.18	0.12	***0.12***
			5.60	2005 survey	17.0	700	327	231	167	124	95	78	63	52	31	0.33	0.10	1.08	0.35	***0.35***	1.25	0.12	***0.12***
			5.60	2006 Survey	22.7	700	308	210	153	114	90	74	63	53	33	0.31	0.10	1.04	0.32	***0.32***	1.11	0.11	***0.11***
			5.60	2007 Survey	15.9	700	293	205	153	114	94	77	64	52	28	0.29	0.09	1.09	0.32	***0.32***	1.28	0.11	***0.11***
			5.60	2008 Survey	16.7	700	375	261	210	170	148	127	112	96	62	0.38	0.11	1.08	0.41	***0.41***	1.26	0.14	***0.14***
			5.60	2009 Survey	23.8	700	422	317	237	185	151	126	104	90	58	0.42	0.11	1.03	0.43	***0.43***	1.09	0.11	***0.11***
G3	60	WB	2.90	2002 Survey	17.2	700	484	351	268	209	171	147	124	111	73	0.48	0.13	1.08	0.52	***0.52***	1.24	0.17	***0.17***
			2.90	2003 Survey	14.3	700	504	366	287	221	179	155	130	114	68	0.50	0.14	1.11	0.56	***0.56***	1.33	0.18	***0.18***
			2.90	2004 survey	19.5	700	466	332	247	192	156	135	114	103	67	0.47	0.13	1.06	0.49	***0.49***	1.18	0.16	***0.16***
			2.91	2005 survey	17.0	700	469	349	264	203	165	139	117	102	67	0.47	0.12	1.08	0.51	***0.51***	1.25	0.15	***0.15***
			2.94	2006 Survey	23.4	700	449	288	203	143	106	84	67	58	35	0.45	0.16	1.03	0.46	***0.46***	1.10	0.18	***0.18***
			2.95	2007 Survey	15.9	700	481	327	237	169	133	103	79	67	41	0.48	0.15	1.09	0.52	***0.52***	1.28	0.20	***0.20***
			2.79	2008 Survey	16.7	700	497	355	286	217	175	144	120	102	62	0.50	0.14	1.08	0.54	***0.54***	1.26	0.18	***0.18***
			2.70	2010 Survey	23.9	700	518	374	276	185	152	123	100	88	62	0.52	0.14	1.03	0.53	***0.53***	1.09	0.16	***0.16***
G4	60	WB	5.20	2002 Survey	17.2	700	336	240	182	144	113	93	78	65	42	0.34	0.10	1.08	0.36	***0.40***	1.24	0.12	***0.14***
			6.00	2002 Survey	17.2	700	410	287	206	155	118	91	80	66	40	0.41	0.12	1.08	0.44		1.24	0.15	
			5.20	2003 Survey	14.3	700	396	284	213	182	140	121	99	86	56	0.40	0.11	1.11	0.44	***0.41***	1.33	0.15	***0.14***
			6.00	2003 Survey	14.3	700	352	252	197	148	119	101	83	72	43	0.35	0.10	1.11	0.39		1.33	0.13	
			5.20	2004 survey	19.5	700	317	222	161	126	100	86	72	62	38	0.32	0.10	1.06	0.34	***0.33***	1.18	0.11	***0.11***
			6.00	2004 survey	19.5	700	297	202	143	105	85	72	59	50	30	0.30	0.10	1.06	0.32		1.18	0.11	
			5.20	2005 survey	17.0	700	315	220	160	119	95	79	64	54	33	0.32	0.10	1.08	0.34	***0.38***	1.25	0.12	***0.12***
			6.00	2005 survey	17.0	700	383	280	210	159	128	105	86	74	44	0.38	0.10	1.08	0.41		1.25	0.13	
			5.20	2006 Survey	23.4	700	320	233	181	145	122	106	93	82	55	0.32	0.09	1.03	0.33	***0.35***	1.10	0.10	***0.11***
			6.00	2006 Survey	23.4	700	363	247	180	133	102	83	68	57	34	0.36	0.12	1.03	0.37		1.10	0.13	
			5.20	2007 Survey	15.9	700	333	241	187	146	119	100	86	71	41	0.33	0.09	1.09	0.36	***0.35***	1.28	0.12	***0.11***
			6.00	2007 Survey	15.9	700	307	220	175	135	107	88	74	60	36	0.31	0.09	1.09	0.33		1.28	0.11	
			5.20	2008 Survey	16.7	700	287	198	156	121	100	82	70	60	37	0.29	0.09	1.08	0.31	***0.33***	1.26	0.11	***0.12***
			6.00	2008 Survey	16.7	700	315	215	169	134	115	98	83	64	40	0.32	0.10	1.08	0.34		1.26	0.13	
			5.20	2009 Survey	23.9	700	333	245	183	139	113	94	77	65	43	0.33	0.09	1.03	0.34	***0.38***	1.09	0.10	***0.10***
			6.00	2009 Survey	23.9	700	403	305	235	186	152	128	102	85	48	0.40	0.10	1.03	0.41		1.09	0.11	

Graham Farmer Freeway Project Level Data

																Normalised to 700 kPa No Temp Correction		Normalised to 700 kPa Austroads 2008 Correction to 29C					
Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 450mm	Defl. 5 (Micron) 650mm	Defl. 6 (Micron) 900mm	Defl. 7 (Micron) 1200mm			Deflection (mm)	Curvature (mm)	Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
G1	60	EB	2.90	26/10/2006	15.0	570	283	173	128	69	69	52	40			0.35	0.14	1.10	0.38		1.31	0.18	
			2.95		15.0	568	287	183	127	65	59	42	34			0.35	0.13	1.10	0.39		1.31	0.17	
			3.00		15.0	568	293	186	134	69	64	46	37			0.36	0.13	1.10	0.40		1.31	0.17	
			3.05		15.0	569	287	181	129	68	63	45	36			0.35	0.13	1.10	0.39		1.31	0.17	
			3.10		15.0	564	284	212	153	79	72	52	40			0.35	0.09	1.10	0.39	***0.39***	1.31	0.12	***0.16***
G3	60	WB	3.11		26.0	572	300	185	129	65	62	47	39			0.37	0.14	1.02	0.37		1.05	0.15	
			3.05		26.0	568	325	217	160	83	77	58	46			0.40	0.13	1.02	0.41		1.05	0.14	
			3.00		26.0	570	372	237	165	84	73	53	40			0.46	0.17	1.02	0.46		1.05	0.17	
			2.95		26.0	567	314	201	142	75	65	46	36			0.39	0.14	1.02	0.39		1.05	0.15	
			2.90		26.0	569	317	194	139	74	63	44	34			0.39	0.15	1.02	0.40		1.05	0.16	
			2.85		26.0	566	380	238	169	85	78	60	46			0.47	0.18	1.02	0.48		1.05	0.18	
			2.80		26.0	563	411	264	188	93	88	66	52			0.51	0.18	1.02	0.52		1.05	0.19	
			2.75		26.0	562	357	227	163	82	74	54	41			0.44	0.16	1.02	0.45		1.05	0.17	
			2.70		26.0	559	364	247	186	96	86	80	65			0.46	0.15	1.02	0.46	***0.44***	1.05	0.15	***0.16***
G1	60	EB	2.90	26/10/2008	20.0	702	320	191	141	95	51	50	25			0.32	0.13	1.06	0.34		1.17	0.15	
			2.95		20.0	701	360	243	155	120	78	72	23			0.36	0.12	1.06	0.38		1.17	0.14	
			3.00		20.0	701	393	226	151	118	69	63	28			0.39	0.17	1.06	0.41		1.17	0.20	
			3.05		20.0	702	339	252	141	137	80	71	41			0.34	0.09	1.06	0.36		1.17	0.10	
			3.10		20.0	703	352	232	143	110	70	47	14			0.35	0.12	1.06	0.37	***0.37***	1.17	0.14	***0.14***
G3	60	WB	3.11		21.0	701	287	183	119	92	64	46	28			0.29	0.10	1.05	0.30		1.15	0.12	
			3.05		21.0	703	368	243	173	134	91	68	41			0.37	0.12	1.05	0.38		1.15	0.14	
			3.00		21.0	701	408	261	164	131	73	59	33			0.41	0.15	1.05	0.43		1.15	0.17	
			2.95		21.0	701	417	277	181	131	81	57	31			0.42	0.14	1.05	0.44		1.15	0.16	
			2.90		21.0	701	348	231	152	120	71	56	26			0.35	0.12	1.05	0.36		1.15	0.13	
			2.85		21.0	698	362	237	156	121	71	59	34			0.36	0.13	1.05	0.38		1.15	0.14	
			2.80		21.0	704	476	298	189	140	97	74	36			0.47	0.18	1.05	0.50		1.15	0.20	
			2.75		21.0	701	376	245	158	120	76	62	28			0.38	0.13	1.05	0.39		1.15	0.15	
			2.70		21.0	703	346	229	156	126	76	67	32			0.34	0.12	1.05	0.36	***0.39***	1.15	0.13	***0.15***
G1	60	EB	2.90	14/12/2009	27.3	552	215	140	99	67	44	34	17			0.27	0.10	1.01	0.28		1.03	0.10	
			2.95		27.4	552	259	166	111	77	50	34	21			0.33	0.12	1.01	0.33		1.02	0.12	
			3.00		27.7	549	229	152	105	78	51	35	24			0.29	0.10	1.01	0.29		1.02	0.10	
			3.05		27.3	552	240	161	110	82	54	38	25			0.30	0.10	1.01	0.31		1.03	0.10	
			3.10		27.0	565	295	197	135	96	49	40	20			0.37	0.12	1.01	0.37	***0.32***	1.03	0.13	***0.11***
G3	60	WB	3.11		28.5	550	340	205	142	98	61	49	28			0.43	0.17	1.00	0.43		1.01	0.17	
			3.05		28.0	563	266	179	117	81	50	37	21			0.33	0.11	1.01	0.33		1.02	0.11	
			3.00		27.6	569	234	158	108	78	55	41	16			0.29	0.09	1.01	0.29		1.02	0.10	
			2.95		26.9	568	272	183	124	86	56	40	26			0.34	0.11	1.01	0.34		1.03	0.11	
			2.90		26.8	562	269	178	128	93	63	40	26			0.34	0.11	1.01	0.34		1.03	0.12	
			2.85		26.5	563	241	159	112	79	54	40	20			0.30	0.10	1.01	0.30		1.04	0.11	
			2.80		26.7	569	330	224	161	124	80	60	30			0.41	0.13	1.01	0.41		1.04	0.14	
			2.75		26.2	566	229	144	100	77	44	33	21			0.28	0.11	1.02	0.29		1.04	0.11	
			2.70		26.2	566	246	176	132	109	75	51	39			0.30	0.09	1.02	0.31	***0.34***	1.04	0.09	***0.12***

Kwinana Freeway Network Level Data

Trial Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Normalised to 700 kPa No Temp Correction: Deflection (mm)	Normalised to 700 kPa No Temp Correction: Curvature (mm)	Normalised to 700 kPa Austroads 2008 Correction to 29C: Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
K1	60	SB	24.96	2007 Survey	18.6	700	371	291	250	216	192	168	147	130	85	0.37	0.08	1.07	0.40	***0.40***	1.21	0.10	***0.10***
			24.97	2006 Survey	19.7	700	386	303	256	217	187	167	146	128	87	0.39	0.08	1.06	0.41	***0.41***	1.18	0.10	***0.10***
			25.19	2008 Survey	23.2	700	453	265	183	126	96	76	61	50	31	0.45	0.19	1.03	0.47	***0.47***	1.10	0.21	***0.21***
			25.24	2009 Survey	11.1	700	460	319	219	161	125	101	81	69	46	0.46	0.14	1.13	0.52	***0.52***	1.42	0.20	***0.20***
K2	60	SB	26.33	2002 survey	21	700	380	304	252	215	184	163	137	118	76	0.38	0.08	1.05	0.40	***0.40***	1.15	0.09	***0.09***
			26.42	2003 Survey	16.5	700	310	225	175	134	107	88	71	60	37	0.31	0.09	1.09	0.34	***0.34***	1.26	0.11	***0.11***
			26.44	2004 Survey	22.4	700	336	220	155	110	84	66	50	43	33	0.34	0.12	1.04	0.35	***0.35***	1.12	0.13	***0.13***
			26.48	2005 Survey	18.0	700	322	242	187	142	111	87	64	50	32	0.32	0.08	1.07	0.35	***0.35***	1.22	0.10	***0.10***
			25.77	2006 Survey	19.4	700	333	243	196	156	127	104	83	68	41	0.33	0.09	1.06	0.35	***0.36***	1.19	0.11	***0.12***
			26.57	2006 Survey	19.1	700	351	246	189	139	107	82	62	48	28	0.35	0.11	1.06	0.37		1.19	0.13	
			25.76	2007 Survey	20.9	700	356	259	206	163	132	109	85	72	42	0.36	0.10	1.05	0.37	***0.37***	1.15	0.11	***0.12***
			26.56	2007 Survey	21.3	700	354	240	185	135	105	82	60	48	27	0.35	0.11	1.05	0.37		1.14	0.13	
			25.98	2008 Survey	23.2	700	399	273	211	149	111	91	69	57	35	0.40	0.13	1.03	0.41	***0.37***	1.10	0.14	***0.13***
			26.78	2008 Survey	23.2	700	314	206	160	126	95	74	57	47	30	0.31	0.11	1.03	0.32		1.10	0.12	
			26.04	2009 Survey	10.5	700	425	306	231	178	141	111	84	68	40	0.43	0.12	1.13	0.48	***0.42***	1.43	0.17	***0.14***
			26.84	2009 Survey	10.4	700	329	250	196	153	124	101	79	65	39	0.33	0.08	1.13	0.37		1.43	0.11	
K3	60	SB	28.01	2003 Survey	16.5	700	395	322	271	223	185	155	125	102	59	0.40	0.07	1.09	0.43	***0.43***	1.26	0.09	***0.09***
			28.06	2004 Survey	22.4	700	371	276	218	172	143	120	101	85	52	0.37	0.10	1.04	0.39	***0.39***	1.12	0.11	***0.11***
			28.07	2005 Survey	18.0	700	447	328	250	190	152	122	96	80	47	0.45	0.12	1.07	0.48	***0.48***	1.22	0.15	***0.15***
			28.15	2006 Survey	19.7	700	417	301	238	186	149	121	95	76	41	0.42	0.12	1.06	0.44	***0.44***	1.18	0.14	***0.14***
			28.16	2007 Survey	21	700	484	353	276	213	176	138	107	85	45	0.48	0.13	1.05	0.51	***0.51***	1.15	0.15	***0.15***
			28.36	2008 Survey	23.2	700	436	293	222	169	134	106	87	70	36	0.44	0.14	1.03	0.45	***0.45***	1.10	0.16	***0.16***
			28.41	2009 Survey	10.5	700	477	349	261	199	155	124	95	77	42	0.48	0.13	1.13	0.54	***0.54***	1.43	0.18	***0.18***

Kwinana Freeway Project Level Data

Trial Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Normalised to 700 kPa, No Temp Correction: Deflection (mm)	Normalised to 700 kPa, No Temp Correction: Curvature (mm)	Normalised to 700 kPa, Austroads 2008 Correction to 29C: Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
K1	60	SB	24.95	21/01/2007	33.3	698	242	193	169	150	132	116	98	83	49	0.24	0.05	0.97	0.24		0.93	0.05	
			24.97		33.3	706	204	172	158	142	126	113	94	80	47	0.20	0.03	0.97	0.20		0.93	0.03	
			24.99		33.3	703	234	192	168	143	122	107	89	78	44	0.23	0.04	0.97	0.23		0.93	0.04	
			25.01		33.3	710	257	193	160	135	117	102	85	73	42	0.25	0.06	0.97	0.25		0.93	0.06	
			25.03		37.0	708	273	220	188	153	128	113	94	79	46	0.27	0.05	0.95	0.26		0.88	0.05	
			25.05		37.0	706	214	170	148	130	114	102	84	72	42	0.21	0.04	0.95	0.20		0.88	0.04	
			25.07		37.0	701	242	187	160	138	118	104	85	72	42	0.24	0.06	0.95	0.23		0.88	0.05	
			25.09		37.0	717	198	166	148	131	116	102	85	74	43	0.19	0.03	0.95	0.18		0.88	0.03	
			25.11		37.0	708	273	202	163	137	113	97	79	68	42	0.27	0.07	0.95	0.26		0.88	0.06	
			25.13		37.0	710	279	230	178	140	115	100	81	69	42	0.28	0.05	0.95	0.26		0.88	0.04	
			25.15		37.0	709	287	218	177	146	121	102	84	72	44	0.28	0.07	0.95	0.27		0.88	0.06	
			25.17		37.0	719	229	190	168	146	126	109	87	76	47	0.22	0.04	0.95	0.21		0.88	0.03	
			25.19		37.0	708	334	235	189	155	131	115	98	87	58	0.33	0.10	0.95	0.31		0.88	0.09	
			25.21		37.0	707	297	253	224	195	168	150	129	113	74	0.29	0.04	0.95	0.28	***0.24***	0.88	0.04	***0.05***
K3	60	SB	28.00	21/01/2007	39.9	703	381	252	185	139	112	96	80	69	43	0.38	0.13	0.94	0.36		0.85	0.11	
			28.02		39.9	708	336	208	144	101	75	59	46	38	25	0.33	0.13	0.94	0.31		0.85	0.11	
			28.16		39.3	693	347	252	210	177	151	130	108	92	53	0.35	0.10	0.94	0.33		0.85	0.08	
			28.18		39.3	698	281	227	199	171	145	129	108	92	52	0.28	0.05	0.94	0.27		0.85	0.05	
			28.20		39.3	696	320	237	195	161	134	116	95	81	46	0.32	0.08	0.94	0.30		0.85	0.07	
			28.22		39.3	699	302	220	188	159	135	117	98	82	49	0.30	0.08	0.94	0.28		0.85	0.07	
			28.24		39.3	699	291	225	195	167	143	126	104	88	54	0.29	0.07	0.94	0.27		0.85	0.06	
			28.26		39.3	706	246	199	179	158	141	126	106	91	56	0.24	0.05	0.94	0.23		0.85	0.04	
			28.28		39.3	705	340	231	189	160	138	122	103	91	53	0.34	0.11	0.94	0.32		0.85	0.09	
			28.30		39.3	711	297	236	200	173	148	131	109	94	55	0.29	0.06	0.94	0.28		0.85	0.05	
			28.32		39.3	701	305	234	200	171	147	130	110	94	56	0.30	0.07	0.94	0.29		0.85	0.06	
			28.34		39.3	708	257	214	185	161	140	125	105	90	54	0.25	0.04	0.94	0.24		0.85	0.04	
			28.36		42.5	711	284	232	194	166	144	127	107	93	57	0.28	0.05	0.93	0.26		0.82	0.04	
			28.38		42.5	707	336	252	202	165	139	121	101	87	53	0.33	0.08	0.93	0.31		0.82	0.07	
			28.40		42.5	707	351	241	193	156	130	112	93	80	50	0.35	0.11	0.93	0.32		0.82	0.09	
			28.42		42.5	703	344	251	202	162	136	118	98	85	53	0.34	0.09	0.93	0.32		0.82	0.08	
			28.44		42.5	706	392	299	223	168	139	119	99	84	52	0.39	0.09	0.93	0.36		0.82	0.08	
			28.46		42.5	706	435	292	221	171	138	116	95	82	53	0.43	0.14	0.93	0.40		0.82	0.12	
			28.48		42.5	711	428	307	228	177	144	123	103	89	54	0.42	0.12	0.93	0.39		0.82	0.10	
			28.50		42.5	710	365	274	220	180	151	130	106	90	55	0.36	0.09	0.93	0.33		0.82	0.07	
			28.52		42.5	713	312	236	194	160	135	116	96	81	48	0.31	0.08	0.93	0.28		0.82	0.06	
			28.54		42.5	715	287	220	183	154	133	118	98	86	55	0.28	0.07	0.93	0.26		0.82	0.05	
			28.56		43.6	715	295	215	177	153	132	117	98	87	56	0.29	0.08	0.92	0.27		0.81	0.06	
			28.58		43.6	709	325	257	198	158	133	117	100	88	56	0.32	0.07	0.92	0.30		0.81	0.05	
			28.60		43.6	710	349	253	204	168	143	125	105	92	59	0.34	0.09	0.92	0.32		0.81	0.08	
			28.62		43.6	710	326	243	202	173	149	130	110	96	62	0.32	0.08	0.92	0.30	***0.30***	0.81	0.07	***0.07***

Mitchell Freeway Network Level Data

															Normalised to 700 kPa No Temp Correction		Normalised to 700 kPa Austroads 2004 Correction to 29C					
Trial Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Deflection (mm)	Curvature (mm)	Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
M1	60	SB	19.17	2004 survey	21.6	700	228	152	110	82	66	56	45	38	23	0.23	0.08	1.04	0.24	***0.24***	1.13	0.09	***0.09***
			19.53	2007 Survey	19.9	700	260	172	124	90	72	60	48	40	23	0.26	0.09	1.06	0.27	***0.27***	1.17	0.10	***0.10***
			19.10	2008 Survey	20.5	700	274	192	152	117	93	75	61	51	31	0.27	0.08	1.05	0.29	***0.29***	1.16	0.10	***0.10***
			19.16	2009 Survey	22.4	700	290	213	157	118	94	76	59	49	28	0.29	0.08	1.04	0.30	***0.30***	1.12	0.09	***0.09***
M2	60	SB	21.25	2002 Survey	17.2	700	245	179	137	110	91	78	65	56	38	0.25	0.07	1.08	0.26		1.24	0.08	
			22.05	2002 Survey	17.2	700	126	80	56	45	38	34	30	26	19	0.13	0.05	1.08	0.14		1.24	0.06	
			22.84	2002 Survey	17.2	700	127	70	42	27	18	13	8	5	3	0.13	0.06	1.08	0.14	***0.18***	1.24	0.07	***0.07***
			21.22	2003 Survey	16.4	700	138	88	63	49	40	35	33	30	21	0.14	0.05	1.09	0.15		1.27	0.06	
			22.01	2003 Survey	16.4	700	260	184	142	108	89	76	60	50	26	0.26	0.08	1.09	0.28		1.27	0.10	
			22.81	2003 Survey	16.4	700	183	114	80	54	38	26	15	8	2	0.18	0.07	1.09	0.20	***0.21***	1.27	0.09	***0.08***
			21.56	2004 survey	21.6	700	101	65	50	40	34	30	26	24	18	0.10	0.04	1.04	0.11		1.13	0.04	
			22.36	2004 survey	21.6	700	359	257	190	140	109	88	66	54	32	0.36	0.10	1.04	0.38	***0.24***	1.13	0.12	***0.08***
			21.29	2005 survey	18	700	350	266	205	163	136	114	93	75	43	0.35	0.08	1.07	0.38		1.22	0.10	
			22.09	2005 survey	18	700	272	202	155	122	100	85	70	59	35	0.27	0.07	1.07	0.29	***0.33***	1.22	0.09	***0.09***
			21.33	2006 Survey	22.2	700	225	158	117	87	65	47	33	26	10	0.23	0.07	1.04	0.23		1.12	0.08	
			22.13	2006 Survey	21.9	700	162	104	76	60	50	44	40	34	25	0.16	0.06	1.04	0.17	***0.20***	1.13	0.07	***0.07***
			21.12	2007 Survey	19.5	700	203	139	111	83	78	65	54	42	26	0.20	0.06	1.06	0.22		1.18	0.08	
			21.91	2007 Survey	20.1	700	325	231	178	135	111	91	71	60	35	0.33	0.09	1.06	0.34		1.17	0.11	
			22.71	2007 Survey	19.9	700	283	186	139	102	78	61	47	38	22	0.28	0.10	1.06	0.30	***0.29***	1.17	0.11	***0.10***
			22.30	2008 Survey	20.5	700	331	231	188	148	119	98	73	67	34	0.33	0.10	1.05	0.35	***0.35***	1.16	0.12	***0.12***
			22.35	2009 Survey	22.6	700	427	308	223	168	132	104	80	64	36	0.43	0.12	1.04	0.44	***0.44***	1.11	0.13	***0.13***
			22.39	2010 Survey	14.9	700	336	237	180	137	110	89	70	57	32	0.34	0.10	1.10	0.37	***0.37***	1.31	0.13	***0.13***
M3	60	SB	23.64	2002 Survey	17.2	700	343	246	189	149	120	99	78	65	32	0.34	0.10	1.08	0.37		1.24	0.12	
			24.44	2002 Survey	17.2	700	374	253	188	143	115	96	77	65	40	0.37	0.12	1.08	0.40	***0.39***	1.24	0.15	***0.14***
			23.61	2003 Survey	16.4	700	331	246	198	160	134	116	94	78	40	0.33	0.09	1.09	0.36		1.27	0.11	
			24.41	2003 Survey	16.4	700	330	245	198	157	131	113	92	77	39	0.33	0.09	1.09	0.36	***0.36***	1.27	0.11	***0.11***
			23.95	2004 survey	21.6	700	261	175	127	98	80	69	57	48	28	0.26	0.09	1.04	0.27	***0.27***	1.13	0.10	***0.10***
			24.49	2005 survey	18	700	291	213	164	130	110	94	75	61	35	0.29	0.08	1.07	0.31	***0.31***	1.22	0.10	***0.10***
			24.53	2006 Survey	22.3	700	330	239	190	153	127	109	90	76	45	0.33	0.09	1.04	0.34	***0.34***	1.12	0.10	***0.10***
			23.50	2007 Survey	19.5	700	320	221	170	134	108	87	70	58	30	0.32	0.10	1.06	0.34		1.18	0.12	
			24.30	2007 Survey	18.7	700	231	168	142	118	103	91	76	65	38	0.23	0.06	1.07	0.25	***0.29***	1.20	0.08	***0.10***
			23.89	2008 Survey	20.5	700	266	185	149	122	103	93	72	60	35	0.27	0.08	1.05	0.28	***0.28***	1.16	0.09	***0.09***
			23.94	2009 Survey	22.7	700	329	250	200	165	142	120	98	81	39	0.33	0.08	1.04	0.34	***0.34***	1.11	0.09	***0.09***

Mitchell Freeway Project Level Data

Trial Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Normalised to 700 kPa No Temp Correction: Deflection (mm)	Normalised to 700 kPa No Temp Correction: Curvature (mm)	Normalised to 700 kPa Austroads 2004 Correction to 29C: Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
M4	60	NB	33.00	2/08/2017	29.7	699	469	333	242	184	144	121	99	85	55	0.47	0.14	1.00	0.47		0.99	0.13	
			33.02		30.1	700	426	311	235	181	143	123	98	84	54	0.43	0.12	0.99	0.42		0.98	0.11	
			33.04		29.8	694	442	320	239	186	147	124	101	86	57	0.45	0.12	0.99	0.44		0.99	0.12	
			33.06		30.4	705	426	309	234	183	145	123	100	86	55	0.42	0.12	0.99	0.42		0.97	0.11	
			33.08		30	697	443	315	235	182	145	123	101	87	56	0.44	0.13	0.99	0.44		0.98	0.13	
			33.10		30.5	705	436	301	219	170	137	117	97	83	53	0.43	0.13	0.99	0.43		0.97	0.13	
			33.12		29.8	691	443	313	227	174	140	120	101	88	59	0.45	0.13	0.99	0.45		0.99	0.13	
			33.14		30.7	693	466	325	238	185	149	129	108	94	62	0.47	0.14	0.99	0.47		0.97	0.14	
			33.16		31	688	413	290	213	168	137	120	101	89	58	0.42	0.12	0.99	0.41		0.96	0.12	
			33.18		31.4	698	489	342	249	190	152	129	108	94	60	0.49	0.15	0.98	0.48		0.96	0.14	
			33.20		31.5	692	492	345	249	189	150	128	107	92	58	0.50	0.15	0.98	0.49		0.96	0.14	
			33.22		31.9	702	398	253	168	115	80	61	44	35	16	0.40	0.14	0.98	0.39		0.95	0.14	
			33.24		31.9	702	409	258	168	110	72	50	32	23	8	0.41	0.15	0.98	0.40		0.95	0.14	
			33.26		31.4	707	415	270	178	121	82	59	40	29	11	0.41	0.14	0.98	0.40		0.96	0.14	
			33.28		31.3	698	490	341	250	193	150	125	102	87	53	0.49	0.15	0.98	0.48		0.96	0.14	
			33.30		32	697	485	338	245	187	147	124	101	87	49	0.49	0.15	0.98	0.48		0.95	0.14	
			33.32		31	702	476	312	213	154	117	92	68	53	22	0.47	0.16	0.99	0.47		0.96	0.16	
			33.34		31.2	691	400	264	180	133	101	83	65	53	29	0.41	0.14	0.99	0.40		0.96	0.13	
			33.36		31	700	417	283	203	153	123	105	86	75	47	0.42	0.13	0.99	0.41		0.96	0.13	
			33.38		30.5	701	415	283	199	149	113	97	75	62	36	0.41	0.13	0.99	0.41		0.97	0.13	
			33.40		30.5	699	442	300	209	158	124	105	85	71	40	0.44	0.14	0.99	0.44		0.97	0.14	
			33.42		30.5	703	370	253	186	140	105	83	63	50	25	0.37	0.12	0.99	0.37		0.97	0.11	
			33.44		30.5	710	297	189	123	86	55	41	29	16	7	0.29	0.11	0.99	0.29		0.97	0.10	
			33.46		31.5	705	327	202	127	86	59	44	31	24	10	0.33	0.12	0.98	0.32		0.96	0.12	
			33.48		31.6	700	388	253	169	119	86	68	51	42	23	0.39	0.13	0.98	0.38		0.95	0.13	
			33.50		30.6	701	326	190	109	64	37	24	13	9	3	0.33	0.14	0.99	0.32		0.97	0.13	
			33.52		30.1	690	311	171	96	52	29	18	9	5	1	0.32	0.14	0.99	0.31		0.98	0.14	
			33.54		29.8	687	325	188	105	68	43	31	21	16	7	0.33	0.14	0.99	0.33		0.99	0.14	
			33.56		29.7	698	298	169	95	55	31	20	11	6	2	0.30	0.13	1.00	0.30		0.99	0.13	
			33.58		29.1	695	292	174	98	57	34	22	13	8	3	0.29	0.12	1.00	0.29		1.00	0.12	
			33.60		29	699	283	166	96	57	34	22	14	9	3	0.28	0.12	1.00	0.28		1.00	0.12	
			33.62		28.4	695	355	238	157	109	76	60	43	34	17	0.36	0.12	1.00	0.36		1.01	0.12	
			33.64		28.1	686	399	266	181	130	95	77	61	52	34	0.41	0.14	1.00	0.41		1.01	0.14	
			33.66		29	691	410	287	202	148	105	85	62	51	32	0.42	0.13	1.00	0.42		1.00	0.13	
			33.68		29.7	694	395	258	166	118	87	68	52	43	25	0.40	0.14	1.00	0.40		0.99	0.14	
			33.70		29.3	698	368	246	162	114	81	63	45	30	17	0.37	0.12	1.00	0.37		0.99	0.12	
			33.72		27.7	693	302	180	105	63	37	25	14	9	2	0.30	0.12	1.01	0.31		1.02	0.13	
			33.74		28.4	701	320	191	110	67	38	30	18	7	2	0.32	0.13	1.00	0.32		1.01	0.13	
			33.76		28.1	679	335	200	124	73	45	28	16	8	3	0.35	0.14	1.00	0.35		1.01	0.14	
			33.78		28.7	734	342	206	125	78	47	29	16	9	3	0.33	0.13	1.00	0.33		1.00	0.13	
			33.80		27.7	713	343	212	131	84	51	32	18	10	2	0.34	0.13	1.01	0.34		1.02	0.13	
			33.82		26.9	694	419	267	173	114	76	52	32	22	8	0.42	0.15	1.01	0.43		1.03	0.16	
			33.84		27	690	334	192	110	63	36	22	11	6	2	0.34	0.14	1.01	0.34		1.03	0.15	
			33.86		26.2	699	380	250	166	113	77	60	39	28	16	0.38	0.13	1.02	0.39		1.04	0.14	
			33.88		25	681	417	272	180	127	91	69	51	41	19	0.43	0.15	1.02	0.44		1.07	0.16	
			33.90		26.7	698	416	275	187	132	97	78	61	52	32	0.42	0.14	1.01	0.42		1.04	0.15	
			33.92		26	701	412	263	173	121	85	64	48	39	20	0.41	0.15	1.02	0.42		1.05	0.16	
			33.94		25.7	706	286	157	89	51	29	19	11	8	3	0.28	0.13	1.02	0.29		1.05	0.13	
			33.96		26.5	701	285	159	88	50	30	19	12	8	3	0.28	0.13	1.01	0.29		1.04	0.13	
			33.98		26.8	690	272	153	85	48	26	15	7	4	1	0.28	0.12	1.01	0.28	***0.39***	1.03	0.13	***0.13***

Mitchell Freeway Project Level Data

																Normalised to 700 kPa		Normalised to 700 kPa					
																No Temp Correction		Austroads 2004 Correction to 29C					
Trial Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Deflection (mm)	Curvature (mm)	Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
			33.00		12	706	358	265	201	161	134	113	95	86	54	0.35	0.09	1.12	0.40		1.39	0.13	
			33.02		12	699	315	233	180	142	120	105	87	80	55	0.32	0.08	1.12	0.35		1.39	0.11	
			33.04		11.9	695	324	253	189	153	124	105	90	81	53	0.33	0.07	1.12	0.37		1.40	0.10	
			33.06		11.9	703	292	223	171	137	119	102	90	82	51	0.29	0.07	1.12	0.33		1.40	0.10	
			33.08		11.8	699	332	249	197	155	136	110	100	82	53	0.33	0.08	1.12	0.37		1.40	0.12	
			33.10		11.8	708	337	262	195	156	132	113	96	85	53	0.33	0.07	1.12	0.37		1.40	0.10	
			33.12		11.9	687	329	247	187	153	128	111	95	84	52	0.34	0.08	1.12	0.38		1.40	0.12	
			33.14		11.7	698	284	214	165	138	120	108	96	87	60	0.28	0.07	1.12	0.32		1.40	0.10	
			33.16		11.7	699	295	226	173	144	121	108	96	88	57	0.30	0.07	1.12	0.33		1.40	0.10	
			33.18		11.7	703	315	237	180	147	126	111	98	90	54	0.31	0.08	1.12	0.35		1.40	0.11	
			33.20		11.7	699	300	225	173	141	119	105	87	81	50	0.30	0.08	1.12	0.34		1.40	0.11	
			33.22		11.7	698	268	189	140	105	83	66	57	43	25	0.27	0.08	1.12	0.30		1.40	0.11	
			33.24		11.7	694	208	140	100	71	53	38	30	25	12	0.21	0.07	1.12	0.24		1.40	0.10	
			33.26		11.8	703	291	219	165	130	107	88	74	63	35	0.29	0.07	1.12	0.33		1.40	0.10	
			33.28		11.8	700	236	168	118	90	67	48	34	28	13	0.24	0.07	1.12	0.27		1.40	0.10	
			33.30		11.9	703	329	253	200	167	137	120	99	86	55	0.33	0.08	1.12	0.37		1.40	0.11	
			33.32		12	695	381	292	228	181	144	119	94	78	40	0.38	0.09	1.12	0.43		1.39	0.13	
			33.34		12.1	702	265	189	136	105	82	68	54	44	26	0.26	0.08	1.12	0.30		1.39	0.11	
			33.36		12	698	289	213	159	123	104	82	70	62	38	0.29	0.08	1.12	0.33		1.39	0.11	
			33.38		12.1	703	370	273	211	173	141	125	108	93	54	0.37	0.10	1.12	0.41		1.39	0.13	
			33.40		12.1	703	333	252	192	151	124	102	87	80	47	0.33	0.08	1.12	0.37		1.39	0.11	
			33.42		12.2	698	285	205	144	110	85	66	51	42	19	0.29	0.08	1.12	0.32		1.39	0.11	
			33.44		12.2	706	165	101	60	39	26	19	13	10	5	0.16	0.06	1.12	0.18		1.39	0.09	
			33.46		12.3	697	181	118	79	57	42	31	19	17	6	0.18	0.06	1.12	0.20		1.39	0.09	
M4	60	NB	33.48	23/05/2018	12.4	698	249	174	121	87	66	49	39	29	16	0.25	0.08	1.12	0.28		1.38	0.10	
			33.50		12.5	700	254	180	125	89	71	53	40	40	17	0.25	0.07	1.12	0.28		1.38	0.10	
			33.52		12.5	703	203	128	77	49	31	20	11	8	2	0.20	0.07	1.12	0.23		1.38	0.10	
			33.54		12.6	695	164	105	66	43	28	20	14	10	4	0.17	0.06	1.12	0.18		1.38	0.08	
			33.56		12.7	704	181	115	70	47	31	22	14	13	5	0.18	0.07	1.12	0.20		1.38	0.09	
			33.58		12.7	702	172	110	65	43	32	22	16	10	6	0.17	0.06	1.12	0.19		1.38	0.09	
			33.60		12.8	700	215	140	85	56	35	23	14	9	3	0.22	0.08	1.12	0.24		1.37	0.10	
			33.62		12.9	694	198	130	86	58	41	30	19	15	5	0.20	0.07	1.12	0.22		1.37	0.09	
			33.64		13	710	274	201	142	104	79	60	46	41	29	0.27	0.07	1.12	0.30		1.37	0.10	
			33.66		13.1	701	264	191	132	97	82	60	50	46	30	0.26	0.07	1.11	0.29		1.36	0.10	
			33.68		13.2	699	230	157	99	68	50	38	31	27	14	0.23	0.07	1.11	0.26		1.36	0.10	
			33.70		13.2	696	230	157	95	67	53	37	25	25	11	0.23	0.07	1.11	0.26		1.36	0.10	
			33.72		13.4	698	165	105	68	46	30	22	15	10	4	0.17	0.06	1.11	0.18		1.36	0.08	
			33.74		13.5	703	172	108	67	42	27	20	11	8	2	0.17	0.06	1.11	0.19		1.35	0.09	
			33.76		13.6	700	183	121	67	46	32	20	12	10	2	0.18	0.06	1.11	0.20		1.35	0.08	
			33.78		13.7	699	202	126	80	55	36	26	16	12	4	0.20	0.08	1.11	0.22		1.35	0.10	
			33.80		13.8	700	218	150	101	65	48	26	16	15	4	0.22	0.07	1.11	0.24		1.34	0.09	
			33.82		13.9	717	285	188	119	77	52	35	21	16	6	0.28	0.09	1.11	0.31		1.34	0.13	
			33.84		14	695	306	206	139	94	65	48	31	24	9	0.31	0.10	1.11	0.34		1.34	0.13	
			33.86		14.1	701	221	139	90	57	36	22	15	13	4	0.22	0.08	1.11	0.24		1.33	0.11	
			33.88		14.1	713	303	210	145	107	82	65	48	43	24	0.30	0.09	1.11	0.33		1.33	0.12	
			33.90		14.1	693	280	198	132	98	73	56	43	38	24	0.28	0.08	1.11	0.31		1.33	0.11	
			33.92		14	708	287	194	136	99	73	58	45	37	25	0.28	0.09	1.11	0.31		1.34	0.12	
			33.94		14	694	257	172	118	84	57	43	30	24	16	0.26	0.09	1.11	0.29		1.34	0.11	
			33.96		14	697	233	159	108	78	56	39	28	21	10	0.23	0.07	1.11	0.26		1.34	0.10	
			33.98		13.9	703	259	175	118	80	54	39	26	20	10	0.26	0.08	1.11	0.29	***0.29***	1.34	0.11	***0.10***

Reid Highway Network Level Data

																Normalised to 700 kPa No Temp Correction		Normalised to 700 kPa Austroads 2008 Correction to 29C					
Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Deflection (mm)	Curvature (mm)	Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
RH1	30	EB	6.47	2002 survey	20.8	700	373	255	177	125	101	89	68	59	36	0.37	0.12	1.03	0.38		1.05	0.12	
			7.26	2002 survey	20.8	700	306	205	146	113	93	80	67	57	35	0.31	0.10	1.03	0.31	***0.35***	1.05	0.11	***0.12***
			6.46	2003 Survey	18.8	700	385	251	180	130	104	89	73	62	33	0.39	0.13	1.03	0.40		1.07	0.14	
			7.25	2003 Survey	18.8	700	309	206	155	119	98	84	69	58	34	0.31	0.10	1.03	0.32	***0.36***	1.07	0.11	***0.13***
			6.46	2004 Survey	22.4	700	297	198	137	106	88	76	63	54	32	0.30	0.10	1.02	0.30		1.04	0.10	
			7.26	2004 Survey	22.4	700	266	177	127	99	85	72	59	50	32	0.27	0.09	1.02	0.27	***0.29***	1.04	0.09	***0.10***
			6.48	2005 Survey	21.0	700	316	217	149	109	88	73	60	50	31	0.32	0.10	1.02	0.32		1.05	0.10	
			7.28	2005 Survey	21.0	700	312	215	152	114	93	79	65	54	33	0.31	0.10	1.02	0.32	***0.32***	1.05	0.10	***0.10***
			6.61	2006 Survey	24.3	700	361	233	158	109	85	70	58	50	32	0.36	0.13	1.01	0.37		1.03	0.13	
			7.41	2006 Survey	24.4	700	394	268	199	149	120	100	83	68	38	0.39	0.13	1.01	0.40	***0.38***	1.03	0.13	***0.13***
			6.52	2007 Survey	22.7	700	271	170	117	87	73	60	49	42	25	0.27	0.10	1.02	0.28		1.04	0.10	
			7.32	2007 Survey	22.6	700	288	178	123	88	74	60	54	46	32	0.29	0.11	1.02	0.29	***0.28***	1.04	0.11	***0.11***
			6.29	2008 Survey	16.2	700	344	223	171	130	105	86	71	59	33	0.34	0.12	1.04	0.36		1.10	0.13	
			7.09	2008 Survey	16.2	700	327	206	149	108	85	71	58	49	27	0.33	0.12	1.04	0.34	***0.35***	1.10	0.13	***0.13***
			6.86	2009 Survey	15.5	700	375	273	201	153	126	106	87	73	39	0.38	0.10	1.05	0.39		1.10	0.11	
			7.65	2009 Survey	15.2	700	430	305	218	160	128	103	81	66	36	0.43	0.13	1.05	0.45	***0.42***	1.11	0.14	***0.13***
			6.88	2010 Survey	21.0	700	430	301	225	179	151	129	107	90	51	0.43	0.13	1.02	0.44		1.05	0.14	
			7.67	2010 Survey	21.3	700	366	249	180	141	115	93	75	61	33	0.37	0.12	1.02	0.37	***0.41***	1.05	0.12	***0.13***
RH2	30	WB	6.11	2002 Survey	20.8	700	429	271	199	155	129	112	92	78	47	0.43	0.16	1.03	0.44		1.05	0.17	
			6.90	2002 Survey	20.8	700	371	262	199	158	131	114	97	84	51	0.37	0.11	1.03	0.38	***0.41***	1.05	0.11	***0.14***
			6.10	2003 Survey	19.6	700	419	287	216	159	130	112	93	79	43	0.42	0.13	1.03	0.43		1.06	0.14	
			6.90	2003 Survey	19.6	700	445	302	224	171	140	123	103	90	49	0.45	0.14	1.03	0.46	***0.44***	1.06	0.15	***0.15***
			6.10	2004 survey	22.4	700	320	222	162	128	110	96	80	68	40	0.32	0.10	1.02	0.33		1.04	0.10	
			6.89	2004 survey	22.4	700	347	230	167	132	115	99	84	72	45	0.35	0.12	1.02	0.35	***0.34***	1.04	0.12	***0.11***
			6.11	2005 survey	20.5	700	364	258	185	141	117	101	85	71	42	0.36	0.11	1.03	0.37		1.06	0.11	
			6.87	2005 survey	20.5	700	350	254	190	152	130	113	95	79	47	0.35	0.10	1.03	0.36	***0.37***	1.06	0.10	***0.11***
			6.20	2006 Survey	23.5	700	365	242	167	118	94	80	66	56	31	0.37	0.12	1.02	0.37		1.03	0.13	
			6.99	2006 Survey	23.8	700	396	270	192	142	116	95	79	66	37	0.40	0.13	1.01	0.40	***0.39***	1.03	0.13	***0.13***
			6.16	2007 Survey	22.8	700	441	292	206	153	123	103	86	72	41	0.44	0.15	1.02	0.45		1.04	0.15	
			6.92	2007 Survey	23.5	700	403	253	186	129	105	83	69	62	34	0.40	0.15	1.02	0.41	***0.43***	1.03	0.15	***0.15***
			7.49	2008 Survey	16.2	700	357	255	195	146	116	95	75	63	31	0.36	0.10	1.04	0.37		1.10	0.11	
			6.70	2008 Survey	16.2	700	405	290	225	177	151	128	109	94	56	0.41	0.12	1.04	0.42	***0.40***	1.10	0.13	***0.12***
			6.44	2009 Survey	15.0	700	433	307	215	157	124	101	82	69	40	0.43	0.13	1.05	0.45		1.11	0.14	
			7.26	2009 Survey	15.1	700	331	235	174	137	113	95	75	60	30	0.33	0.10	1.05	0.35	***0.40***	1.11	0.11	***0.12***
			6.48	2010 Survey	18.7	700	369	241	158	111	86	70	58	50	31	0.37	0.13	1.03	0.38		1.07	0.14	
			7.31	2010 Survey	19.3	700	228	148	97	66	55	45	37	31	18	0.23	0.08	1.03	0.24	***0.31***	1.07	0.09	***0.11***

Roe Highway Network and Project Level Data

																Normalised to 700 kPa No Temp Correction		Normalised to 700 kPa Austroads 2008 Correction to 29C					
Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Deflection (mm)	Curvature (mm)	Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
RO1	30	NB	38.40	2002 survey	29.5	700	521	347	257	184	165	151	113	100	74	0.52	0.17	1.00	0.52	***0.52***	1.00	0.17	***0.17***
			38.41	2003 Survey	22.3	700	431	308	237	191	158	136	115	99	57	0.43	0.12	1.02	0.44	***0.44***	1.04	0.13	***0.13***
			38.41	2004 Survey	19.7	700	370	255	192	150	127	105	87	75	53	0.37	0.12	1.03	0.38	***0.38***	1.06	0.12	***0.12***
			38.43	2005 Survey	30.5	700	333	201	125	87	65	52	41	37	25	0.33	0.13	1.00	0.33	***0.33***	0.99	0.13	***0.13***
RO1	30	NB	38.17	2013	15.0	560	443	251	171	118	80	68	34	20	4	0.55	0.24	1.05	0.58		1.11	0.27	
			38.22		15.0	566	296	195	140	97	72	46	39	22	8	0.37	0.12	1.05	0.38		1.11	0.14	
			38.27		15.0	565	302	227	178	139	109	90	65	43	21	0.37	0.09	1.05	0.39		1.11	0.10	
			38.32		15.0	558	336	235	173	127	91	67	43	28	11	0.42	0.13	1.05	0.44		1.11	0.14	
			38.37		15.0	564	405	289	219	171	132	110	78	69	32	0.50	0.14	1.05	0.53		1.11	0.16	
			38.42		15.0	562	236	173	128	98	76	60	51	36	27	0.29	0.08	1.05	0.31	***0.44***	1.11	0.09	***0.15***

Tonkin Highway Network Level Data

																Normalised to 700 kPa No Temp Correction		Normalised to 700 kPa Austroads 2008 Correction to 29C					
Section ID	Designed Asphalt Thickness (mm)	Direction	Station (km)	Date	Surface Temp	Load (kPa)	Defl. 1 (Micron) 0 mm	Defl. 2 (Micron) 200mm	Defl. 3 (Micron) 300mm	Defl. 4 (Micron) 400mm	Defl. 5 (Micron) 500mm	Defl. 6 (Micron) 600mm	Defl. 7 (Micron) 750mm	Defl. 8 (Micron) 900mm	Defl. 9 (Micron) 1500mm	Deflection (mm)	Curvature (mm)	Deflection Factor	Corrected Deflection	Section Mean	Curvature Factor	Corrected Curvature	Section Mean
TH2	30	NB	2.86	2002 Survey	23.3	700	500	368	252	196	162	138	115	93	56	0.50	0.13	1.02	0.51		1.03	0.14	
			3.66	2002 Survey	23.3	700	390	249	170	121	94	78	64	53	37	0.39	0.14	1.02	0.40	***0.45***	1.03	0.15	***0.14***
			2.86	2003 Survey	16.8	700	535	352	263	198	159	138	115	99	55	0.54	0.18	1.04	0.56		1.09	0.20	
			3.66	2003 Survey	16.8	700	433	283	205	141	108	91	75	65	40	0.43	0.15	1.04	0.45	***0.50***	1.09	0.16	***0.18***
			2.86	2004 survey	22.4	700	453	306	211	164	140	115	95	82	50	0.45	0.15	1.02	0.46		1.04	0.15	
			3.66	2004 survey	22.4	700	314	203	133	97	77	66	56	48	32	0.31	0.11	1.02	0.32	***0.39***	1.04	0.12	***0.13***
			2.96	2005 survey	20.5	700	453	309	220	171	145	128	113	101	70	0.45	0.14	1.03	0.46		1.06	0.15	
			3.76	2005 survey	20.5	700	437	304	223	171	141	122	103	89	55	0.44	0.13	1.03	0.45	***0.46***	1.06	0.14	***0.15***
			2.65	2006 Survey	24.5	700	420	266	188	138	112	95	83	74	50	0.42	0.15	1.01	0.43		1.03	0.16	
			3.45	2006 Survey	23.8	700	446	320	244	187	154	128	107	90	56	0.45	0.13	1.01	0.45	***0.44***	1.03	0.13	***0.14***
			2.69	2007 Survey	20.4	700	421	264	186	131	103	86	75	67	44	0.42	0.16	1.03	0.43		1.06	0.17	
			3.48	2007 Survey	20.0	700	335	198	140	104	91	82	77	73	51	0.34	0.14	1.03	0.34	***0.39***	1.06	0.15	***0.16***
			3.17	2008 Survey	16.1	700	478	337	264	203	167	140	119	97	56	0.48	0.14	1.04	0.50	***0.50***	1.10	0.15	***0.15***
			3.26	2009 Survey	26.8	700	465	309	207	143	106	81	60	45	25	0.47	0.16	1.01	0.47	***0.47***	1.01	0.16	***0.16***
			3.16	2010 Survey	22.6	700	423	291	211	157	128	106	85	71	36	0.42	0.13	1.02	0.43	***0.43***	1.04	0.14	***0.14***

APPENDIX B STAGE 3 BACK CALCULATION DATA

B.1 Material Profile

The input pavement profile for the back-calculation is detailed in Table B 1. The basecourse and subbase thicknesses were taken from the as constructed drawings and monitoring reports.

Table B 1: Input into EFROMD3: pavement profiles

ID	Material purpose	Material	Thickness (mm)
Mitchell Freeway	Asphalt	OGA	60 – 80
		DGA	
	Basecourse	CRB	250
	Subbase	Crushed limestone	180
	Subgrade	Sand	300, 500, semi-infinite
Kwinana Fwy	Asphalt	OGA	60 – 80
		DGA	
	Basecourse	CRB	190
	Subbase	Crushed limestone	210
	Subgrade	Sand	300, 500, semi-infinite
Roe Highway	Asphalt	OGA	30 – 40
	Basecourse	DGA	85
	Subbase	Crushed limestone	160
	Subgrade	Sand	300, 500, semi-infinite
Graham Farmer Freeway		OGA	60 – 80
		DGA	
		CRB	150
		Crushed limestone	170
		Sand	300, 500, semi-infinite

Notes:

- Multiple values indicates consecutive layers of the same material.

B.2 Results of Back-calculation

Table B 2: Representative back-calculated modulus, project level data

Section	Base	Date & Age	Thickness of surfacing (mm)	Back-calculated modulus (MPa)					
				Asphalt	Base	Subbase	Subgrade 1	Subgrade 2	Subgrade 3
Mitchell Fwy	CRB	2017 pre-traffic	60	2,000	314	254	160	266	464
			80	2,000	303	282	224	266	344
				Average	309	268	192	266	404
		2018 1 year	60	2,300	464	418	223	211	189
			80	2,300	515	345	170	189	226
				Average	489	381	196	200	207
Kwinana Fwy	CRB	2007 13 years	60	5,000	884	357	195	194	193
			80	5,000	704	363	187	189	193
				Average	794	360	191	192	193
Roe Highway	CRB	2013 28 years	30	2,500	1081	395	119	168	260
			40	2,500	1019	354	119	168	260
				Average	1050	375	119	168	260
Graham Farmer Fwy	BSL	2008 8 years	60	3,000	434	402	271	291	321
			80	3,000	355	343	288	296	307
				Average	394	372	279	293	214
		2009 9 years	60	3,000	437	402	207	219	238
			80	3,000	478	382	350	340	325
				Average	458	392	278	279	282

APPENDIX C LITERATURE REVIEW

C.1 Non-linear Finite Element Modelling

Jameson et al. (2017) conducted a study on the mechanic modelling of the granular pavements with thin asphalt surfacing trial sections at Kwinana Freeway. The study compared measured FWD deflection bowls with deflection bowls obtained from modelling of the pavement. The analysis included non-linear finite element modelling (FEM), and linear elastic modelling as per the Austroads (2012) pavement design method. In the FEM, the moduli of the granular material at a given location in relation to the load were defined as a function of the stress condition at this location. This relationship was derived from laboratory repeated loading triaxial (RLT) testing results. The samples were compacted and moisture conditioned to simulate conditions similar to those encountered in the field.

Figure C 1 shows the modulus contour plot obtained using the finite element model (APADS software) under the FWD load for one of the sections analysed. The pavement comprised 160 mm of crushed rock basecourse, 250 mm of crushed limestone subbase and a sand subgrade. The analysis restricted the modulus of the sand subgrade to a maximum of 150 MPa. It is noted that APADS assumes that the behaviour of the granular material is isotropic, whereas the current Austroads method assumes it is anisotropic with a degree of anisotropy of two (the vertical modulus is twice the horizontal modulus).

Figure C 1: Modulus from non-linear finite element analysis – Kwinana Freeway

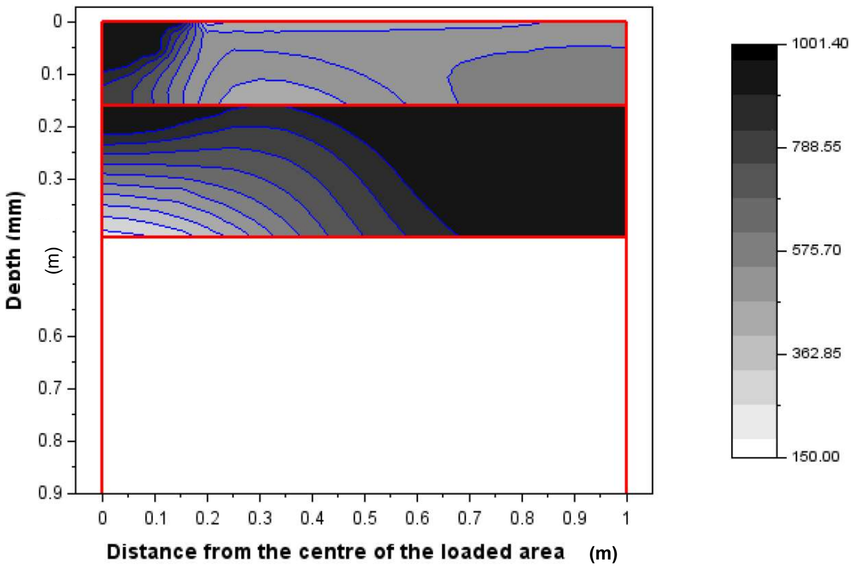


Figure C 1 also shows that the modulus developed in the subgrade was 150 MPa. If no maximum value were assigned, the calculated modulus would be around 200 MPa. The moduli developed in the basecourse and subbase layers throughout the depth of the pavement, as shown in the Figure, are not consistent with the characterisation following the Austroads (2018) pavement design procedure. The finite element analysis indicates that the modulus at the top of the crushed limestone subbase is similar to that at the top of the crushed rock basecourse layer. Using the Austroads (2018) granular pavement sub-layering procedure, the entire granular pavement thickness is divided in five sublayers, with the highest modulus value assigned to the top sublayer and the modulus decreasing with depth. Therefore, the sub-layering method in Austroads (2018) results in an underestimation of the support provided by the crushed limestone sub-base.

The main findings of the study reported by Jameson et al. (2017) were as follows:

- Modelling of the sand subgrade with a modulus higher than that currently used by MRWA (120 MPa) resulted in an improved agreement between the predicted and measured deflection bowls. The finite element modelling indicated a value of 200 MPa.
- The finite element modelling indicates that the Austroads granular characterisation method underestimates the support provided by crushed limestone subbase in the long-term.
- If the characterisation of materials in the linear elastic model is improved, then the linear elastic model can predict deflection bowls with comparable accuracy to the finite element model.

Other authors have also investigated the use of non-linear finite element analysis to model granular pavements. They concluded that they provide a better representation of what happens in the field (Steven, Alabaster & de Pont 2007; Masad, Little & Masad 2006; Erlingson & Ingason 2004; Adu-Osei 2000). Erlingson & Ingason (2004) found linear analysis to overestimate stresses closer to the surface in non-stabilised granular pavements.

C.2 Degree of Anisotropy in the Characterisation of Granular Materials

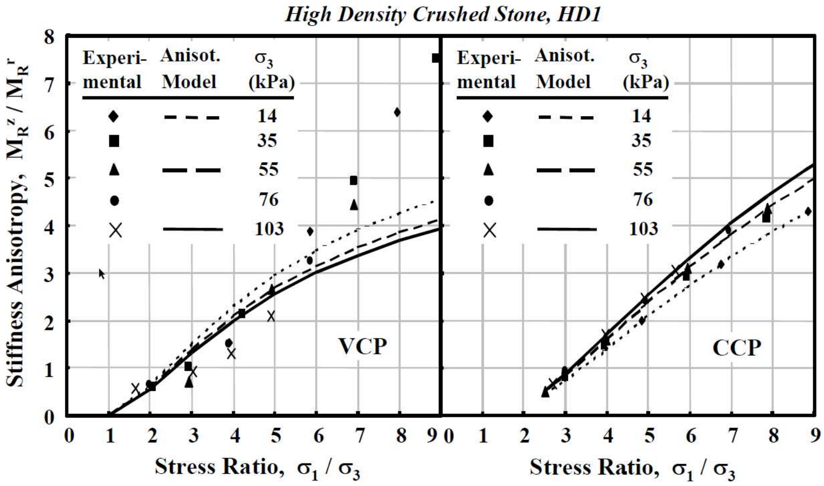
The current Austroads pavement design method (Austroads 2018) assumes granular materials to be anisotropic, with a degree of anisotropy (ratio of vertical modulus and horizontal modulus: E_v/E_h) of 2. The vertical modulus used in design is determined from triaxial test results assuming the material is isotropic.

According to Jameson (2013), measured deflection bowls were found to be narrower than bowls estimated from elastic layer analysis using isotropic characterisation. A literature review conducted by Jameson identified degree of anisotropy values ranging from 1 to 4 for granular materials. Steven et al. (2007) cite values varying from 1.25 to 10.

Masad et al. (2006) found a better correlation between predicted deflections and deflections measured at the AASHO road test when assuming a degree of anisotropy of 3.3.

The degree of anisotropy depends on aggregate gradation, shape, form and textural properties. More elongated particles are associated with higher levels of anisotropy. On the other hand, more angular aggregates and those with higher texture, as well as a coarser and well-graded gradation, result in lower levels of anisotropy (Kim 2004). Tutumluer, Seyhan and Garg (1998) showed that the degree of anisotropy also increases with higher principal stress ratios (σ_1/σ_3), as illustrated in Figure C 2.

Figure C 2: Variation of stiffness anisotropy with stress ratio for variable confining pressure (VCP) and constant confining pressure (CCP) tests

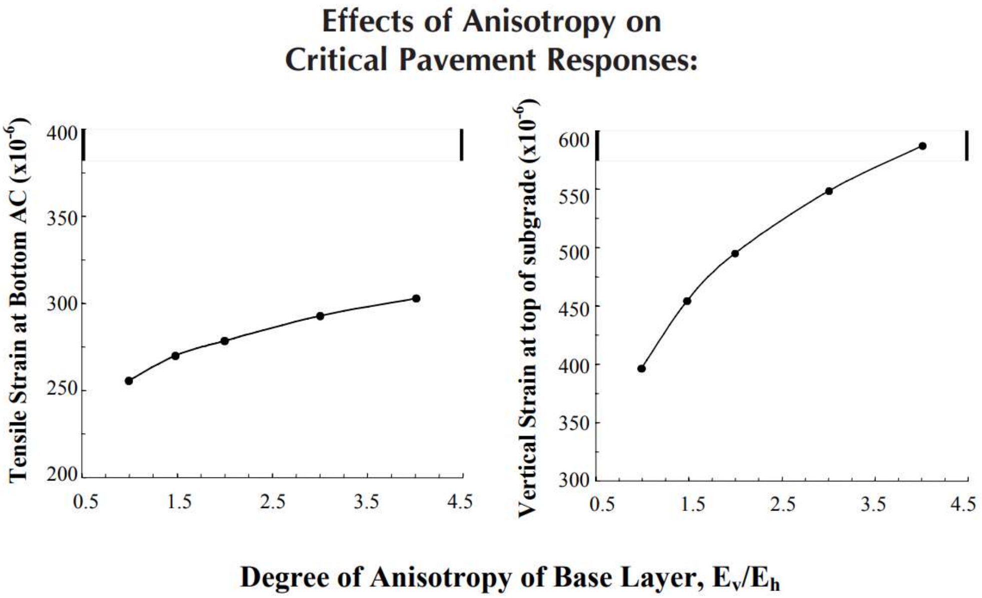


Source: Tutumluer et al. (1998).

Anisotropic modelling was found to result in a more accurate stress distribution (Tutumler et al. 2001; Adu-Osei, Little & Lytton 2001; Masad et al. 2006). Al-Qadi, Wang and Tutumluer (2010) obtained higher pavement strains and reduced fatigue life when considering the effects of anisotropy and stress-dependency. The effects of stress dependency and cross-anisotropy were found to become more significant as the thickness of the asphalt layer decreased and the ratio of vertical modulus to horizontal modulus increased.

Figure C 3 illustrates how critical strains can vary with the degree of anisotropy assumed for the granular basecourse. The tensile strain at the bottom of the asphalt layer varies from about 250 microstrain (for an isotropic assumption) to 300 microstrain (for a degree of anisotropy of 4). Based on the values presented in this Figure and assuming a 5th power law correlating strain at the bottom of the asphalt layer to asphalt fatigue life, the fatigue life assuming isotropic behaviour is about 2.5 times greater than if the granular basecourse is assumed to have a degree of anisotropy of 4. If the design method is changed from a degree of anisotropy of 2 (as assumed in Austroads 2018) to 1 (isotropy), then the fatigue life increases by about 1.6 times.

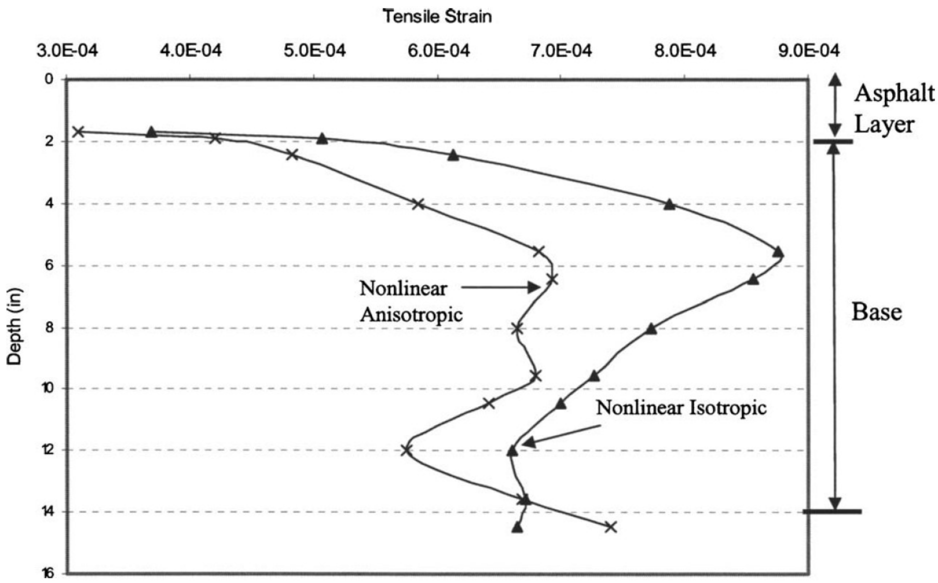
Figure C 3: Effect of degree of anisotropy on the development of critical pavement responses



Source: International Center for Aggregates Research (ICAR) (2001).

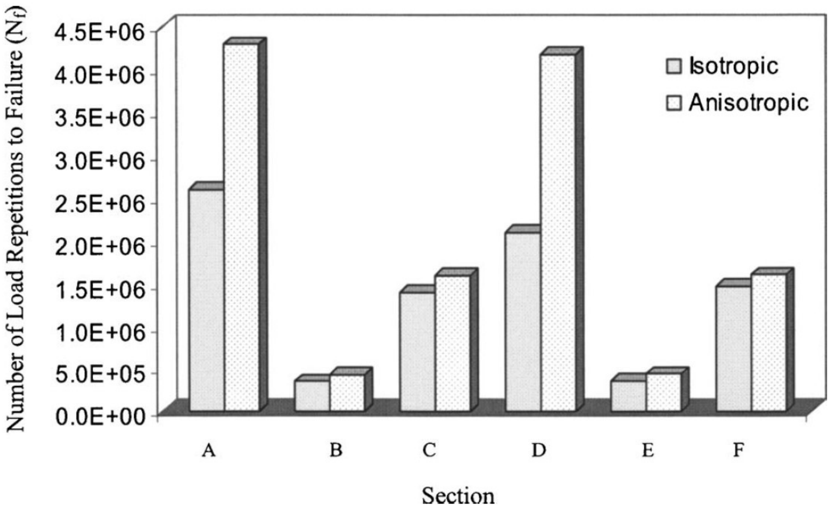
Masad et al. (2006) encountered the opposite behaviour, with calculated tensile strains generally lower when using a non-linear anisotropic model. Their results, for a granular pavement with a 50 mm asphalt surfacing layer, are illustrated in Figure C 4 whilst Figure C 5 includes a comparison of calculated asphalt fatigue life using the non-linear isotropic and anisotropic models for various pavement sections. (The authors mention the use of degree of anisotropy values varying from 2 to 3.3; it is not clear which value was assumed when preparing these Figures.)

Figure C 4: Horizontal tensile strain profiles in asphalt and base layers using non-linear isotropic and anisotropic properties



Source: Masad et al. (2006).

Figure C 5: Comparison between non-linear isotropic and anisotropic models of allowable number of load repetitions



Source: Masad et al. (2006).

C.3 Load Contact Area and Pressure

The Austroads pavement design guide (Austroads 2018) considers a simplified circular contact area with a constant pressure. The net contact area (area between tyre rubber and pavement excluding grooves) is assumed to be 69% of the gross contact area. Load radii are assumed based on data collected during a COST project (COST 2001), which represent average values for several manufacturers' representative of the European truck fleet in 2000. It is noted in COST (2001) that the values recorded can vary for the same type of tyre with different manufacturers, over the years, with different measurement methods and measurement variability.

Although pavement design methods often consider a simplified circular load with constant pressure, the tyre contact area shape can be circular, elliptical or rectangular depending on the tyre type and inflation; and the pressure applied by the tyre varies over the contact area. In general, overloaded and underinflated tyres result in the highest contact stresses at the edge of the contact area, whereas high inflated tyres concentrate the loads in the centre of the contact area (De Beer et al. 2012). Tyre contact area was also found to increase with temperature and decrease with vehicle speed (Neaylon, Harrow & van den Kerkhof 2017).

De Beer et al. (2018) studied the load distribution under dynamic tyre loadings on thin asphalt over granular pavements using a Stress-in-Motion (SIM) system. The SIM uses a 3D tyre/pavement contact sensor pad that detects the load distribution and stress. Tyre loads on thin asphalt over granular pavements were found to be more rectangular shaped than circular, with the rectangular load producing a pavement response approximately 14% lower compared to the circular tyre model. Sharp, Sweatman and Potter (1986) also found tyre prints to be more rectangular shaped than circular when taken from two sites: a sealed granular pavement and a granular pavement with 75 mm of asphalt. De Beer et al. (2012) concluded that consideration of actual distribution of stresses under the tyre can result in up to a 94% reduction in predicted asphalt life. The authors stressed the importance of refining the modelling of tyre-road interaction to represent more realistic contact shapes, especially when considering the design of flexible pavements with thin asphalt surfacings.

C.4 Current Studies on Granular Pavements with Thin Asphalt Surfacings

There are currently two other projects studying the performance of granular pavements with thin asphalt surfacings:

- Austroads project APT6158 'Improving the cost effectiveness of asphalt-surfaced gravel roads': The objective of the project is to improve laboratory-to-field performance correlations for granular pavements with thin asphalt surfacings. The project commenced in August 2018 and has an expected end date of June 2022.
- National Asset Centre of Excellence (NACOE) P69 'Selection and use of unbound granular pavements with thin asphalt surfacing': The objective of this project is to update the TMR procedure for pavement thickness design of granular pavements with thin asphalt surfacings. The project commenced in 2017 and has an expected end date in 2022. The scope of work includes the following tasks:
  - literature review of current design methods
  - monitoring of sites with thin asphalt surfacings
  - analysis using different methodologies to predict pavement performance
  - field testing to assess the condition of the pavement (including Falling Weight Deflectometer)
  - development of a guide on the selection and use of unbound granular pavements with thin asphalt surfacings
  - accelerated pavement testing
  - development of an improved pavement design model.

C.5 Design Methodologies and Other Recommendations

Table C 1 summarises design methods and recommendations regarding the modelling of granular pavements with thin asphalt surfacings.

Table C 1: Summary of design methods and recommendations on modelling of granular pavements with thin asphalt surfacings

Source	Asphalt thickness	Design procedure and/or recommendations
AAPA *Implementation Guide No. 6: Selection and Design of Flexible Pavements* AAPA (2002)	< 40 mm	▪ Design procedure is the same as for pavements designed for sprayed seals (limiting vertical compressive strain in the subgrade). ▪ It is not possible to predict fatigue life, but fatigue life can be improved by using PMB in DGA and stone mastic asphalt (SMA) (recommended for traffic of more than about 5×10^6 ESAS). ▪ Not generally recommended for urban Class 6 applications (design ESAs greater than 10^7). ▪ Can be used on all classes of rural road pavements.
	> 40 mm ≤ 75 mm	▪ Design considers asphalt fatigue mechanism (in addition to permanent deformation). ▪ The balance of asphalt depth vs depth of supporting base is difficult as the asphalt thickness affects the stress state in the granular pavement, which affects its strength. ▪ At high stress areas (i.e. intersections and roundabouts) there are high shear/torsion stresses and vertical loads, which are not accounted for in current design methods – a minimum asphalt wearing course thickness of about 70 mm is recommended in such circumstances. ▪ Catalogue pavement designs provided. ▪ Main application for medium to high traffic urban roads (design ESAs range 10^5–10^7). ▪ May also be suitable for rural Classes 1 and 2 (design ESAs greater than 10^5) depending on traffic loads. .
New Zealand *Guide to Pavement Structural Design* (Gribble 2018)	< 200 mm	▪ ME pavement design should not rely on the stiffness of a thin asphalt surfacing being greater than the underlying layer. ▪ OGA modulus limited to a maximum of 500 MPa (no need to adjust for speed or temperature). ▪ SMA modulus limited to a maximum of 1250 MPa (no need to adjust for speed or temperature). ▪ Asphalt thickness greater than 40 mm over foamed bitumen-stabilised pavement shall be modelled for fatigue performance.

Source	Asphalt thickness	Design procedure and/or recommendations
South African *Pavement Engineering Manual Chapter 10, Pavement Design* South African National Roads Agency (2014)		▪ South Africa generally uses asphalt layers less than 50 mm thick on granular pavements and does not assume that failure of this layer necessarily represents a terminal condition, as the pavement can still carry traffic with the application of crack sealants, a seal to waterproof the layer or patches to correct particularly weak areas. Although the designers check the estimated asphalt fatigue life of thin asphalt over granular pavements, the design is not limited by the thin asphalt fatigue life. Even if the predicted asphalt fatigue life, in terms of repetitions of axle loads, is less than the required design life, it is assumed that the thin asphalt layer lasts for about 8 to 12 years and maintenance is required after that. ▪ The transfer functions (which relates the number of allowable load repetitions to the tensile strain at the bottom of the asphalt layer) have different coefficients for asphalt layers less than 50 mm thick (continuously-graded and gap-graded mixes) and more than 75 mm thick (for asphalt moduli varying from 1000 MPa to 8000 MPa). Figure C 6 shows the allowable number of equivalent standard axle repetitions for a range of horizontal strains at the bottom of the asphalt layer for different asphalt thicknesses, mixes and moduli. It can be observed that, for the same level of strain, the transfer functions used for thinner asphalt layers (i.e. <50 mm) result in a greater number of allowable load repetitions. ▪ It is noted, however, that, for asphalt layer thickness greater than 25 mm, the South African method includes a shift factor to account for the propagation of cracks from the bottom of the layer to the surface. The shift factor increases with the thickness of the asphalt layer. Figure C 7 shows the effect of the shift factor for thin asphalt layers with 45 mm and 80 mm thickness. Austroads (2018) does not include a shift factor to account for the time it takes for a fatigue crack to travel to the surface. ▪ Figure C 8 shows the comparison between the South African method and the Austroads (2018) method assuming asphalt moduli of 1000 MPa and 8000 MPa and bitumen contents of 11.8% and 10.3%. It can be observed that, for the same levels of strain, the Austroads method generally predicts longer fatigue life than the South African method. ▪ The Poisson's ratio of asphalt is typically assumed to be 0.44 (compared to 0.40 in the Austroads 2018 method); the Poisson's ratio for granular pavement materials is assumed to be 0.35 (similar to Austroads 2018). ▪ The granular layers are modelled as isotropic linear elastic layers. Each material corresponds to one layer in the model, which is not sub-layered (i.e. a basecourse material is modelled as a single layer and the subbase material modelled as another single layer). This is different from Austroads (2018), which requires the entire granular pavement to be sub-layered into five layers with the same thickness and decreasing modulus with depth (to reflect the stress dependency of the material). ▪ The basecourse is usually limited to a maximum thickness of 150 mm (whereas Main Roads usually allow an upper limit of 250 mm). ▪ Where a high volume of traffic is expected, a cement-stabilised layer is used below the granular basecourse (inverted pavement). This is not a common pavement type in WA. ▪ Suggested range of elastic moduli for granular materials and expected values are included in Table C 2. Expected values for a good-quality crushed rock over a granular layer vary from 200 to 300 MPa, whereas the presumptive typical vertical modulus provided in Austroads (2018) is 500 MPa (applied to the top sub-layer); this is frequently used in the design of granular pavements with thin asphalt surfacings in Perth's metropolitan area. ▪ South Africa allows the use of different design software (all use linear elastic analysis assuming isotropic materials): – Cyrano: based on ELSYM 5 – considers changes in stiffness of the surface, base and subbase with increased load cycles or time through a recursive simulation scheme (not yet incorporated in the South African Mechanistic-Empirical Method) – Me-PADS: based on GAMES (Multi-layered Elastic Systems) considers non-uniform contact stress distribution – Rubicon Toolbox: based on WESLEA (multi-layer linear elastic program developed by the U.S. Army Corps of Engineers) - includes a finite element tool.

Source	Asphalt thickness	Design procedure and/or recommendations
Leischner et al. (2016)	< 50 mm	▪ Develops a mechanistic framework for estimating realistically the performance of thin asphalt pavements over unbound granular materials in Germany. ▪ Performed repeated load triaxial (RLT) tests on four granular materials at constant density and varying moisture contents. ▪ Considers non-linear stress-dependent behaviour of the unbound basecourse. ▪ Concludes possible to have pavements with asphalt layers less than 50 mm thick for traffic volume less than 1×10^5 10 t-standard axles.
Paul (2012)		▪ The asphalt modulus value should be the same for all traffic speeds when modelling the fatigue performance of thin asphalt surfacings (it is unlikely that the life of the asphalt is greater in a roundabout or signalised intersection compared to a mid-block location). ▪ OGA on a thin DGA over granular pavement should be modelled, as it results in higher strains compared to if the OGA is not modelled. ▪ Further investigation recommended in regards to adopting a factor increasing fatigue life when PMT (A10E) is used: laboratory testing indicated that asphalt with A10E can be up to 10 times that of conventional asphalt C320 asphalt mixes. AGPT05 suggests the fatigue life should be increased by a factor of 3 with A10E PMB, which is not included in AGPT02. ▪ Recommends multi-layered surfacing systems rather than single layer (allows placing SAMI on asphalt rather than on granular base; permits a lower standard of granular pavement preparation; provides a stronger and durable system for high stress situations; permits the top wearing course to be delayed). ▪ Suggests that a multi-layered thin asphalt surfacing is modelled assuming the layer below the SAMI to be a cracked layer. ▪ SAMI between layers appear to permit some slippage, alleviating horizontal strain and resisting the propagation of flexural fatigue cracking.

Figure C 6: South African asphalt fatigue transfer functions for a reliability level of 95% with no shift factor

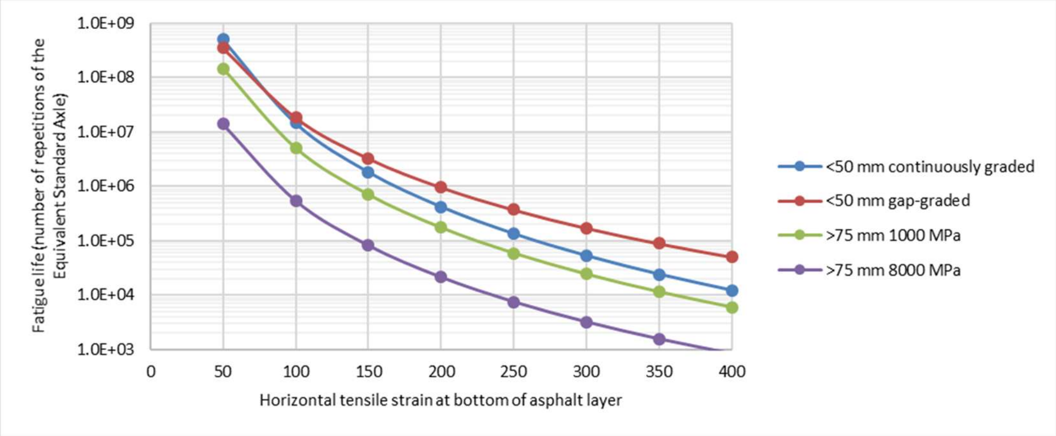


Figure C 7: South African asphalt fatigue transfer functions for a reliability level of 95% with shift factor

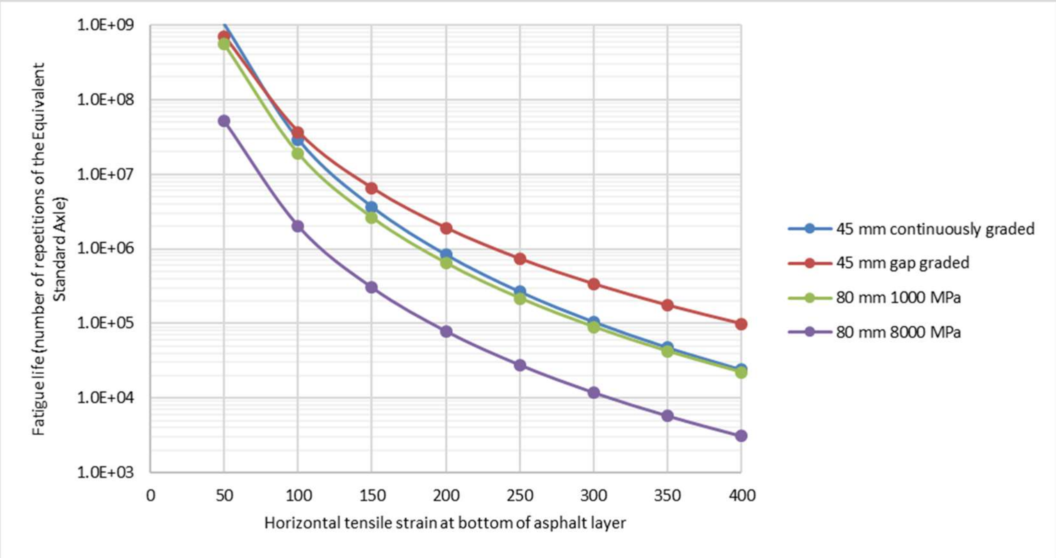


Figure C 8: South African asphalt fatigue transfer functions for a reliability level of 95% with shift factor compared to Austroads (2018)

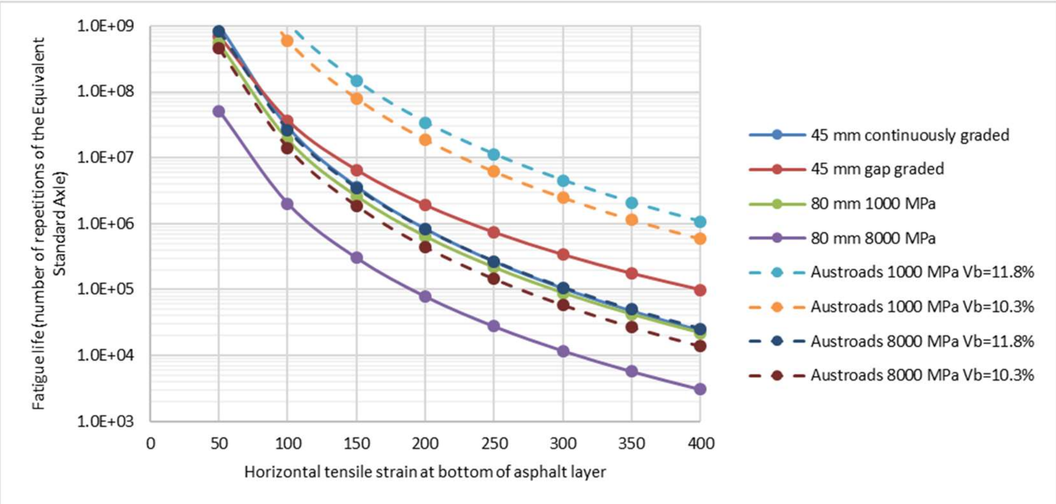


Table C 2: Suggested ranges of elastic moduli for granular materials (MPa) with expected values indicated in brackets

Material Code	Material Description	Over cemented layer in slab state	Over granular layer or equivalent	Wet condition (good support)	Wet condition (poor support)
G1	High quality crushed stone	250–1000 (450)	150–600 (300)	50–250 (250)	40–200 (200)
G2	Crushed stone	200–800 (400)	100–400 (250)	50–250 (250)	40–200 (200)
G3	Crushed stone	200–800 (350)	100–350 (230)	50–200 (200)	40–150 (150)
G4	Natural gravel (base quality)	100–600 (300)	75–350 (225)	50–200 (200)	30–150 (150)
G5	Natural gravel	50–400 (250)	40–300 (200)	30–150 (150)	20–120 (120)
G6	Natural gravel (sub-base quality)	50–200 (200)	30–200 (150)	20–150 (150)	20–120 (120)

Source: Theyse, De Beer & Rust (1996).

C.6 References

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APPENDIX D EXAMPLE OF PROPOSED REVISION OF ERN9 CLAUSE 1.2(C)

Figure D 1: Example of proposed revision of ERN9 Clause 1.2(c)

1.2 Minimum Design Life

Unless specified otherwise by the Principal:

a) the permanent deformation of flexible pavements must have a minimum design life of 40 years;

b) concrete pavement must have a minimum design life of 40 years for fatigue and erosion damage; and

c) the asphalt fatigue design life must greater than or equal to the values in Table 1. The 15 year fatigue design life may be checked considering both the short and long term design period if the following is true:

- asphalt nominal total thickness is 60 mm or less
- pavement conforms to Clause 1.4
- the pavement is well drained, the subgrade is Perth sand, the subbase is crushed limestone and the basecourse material is either crushed rock base or bitumen-stabilised limestone.

1. Calculate the short term fatigue damage (STFD) using the current mechanistic procedure and Equation 1.

$$STFD = \frac{1^{st}\text{ year design traffic}}{95\%\text{ allowable short term fatigue life}} \leq 1.0 \qquad \text{Equation (1)}$$

2. Calculate the long-term fatigue damage (LTFD) using the long term design modulus, sub-layering method and Equation 2 below.
   - — CRB long term design modulus 850 MPa
   - — BSL long term design modulus 550 MPa
   - — Subbase long term design modulus 500 MPa
   - — The base material is sub-layered and the subbase layer is not sub-layered.

$$LTFD = \frac{15\text{ year design traffic} - 1^{st}\text{ year design traffic}}{95\%\text{ allowable long term fatigue life}} \leq 1.0 \qquad \text{Equation (2)}$$

3. Calculate the cumulative fatigue damage using Equation 3 below:

$$CFD\ (\%) = STDF + LTDF \leq 1.0 \qquad \text{Equation (3)}$$

Table 1. Minimum Asphalt Design Fatigue Life

Asphalt Nominal Total Thickness	
60 mm or less	Greater Than 60 mm
15 Years	40 Years