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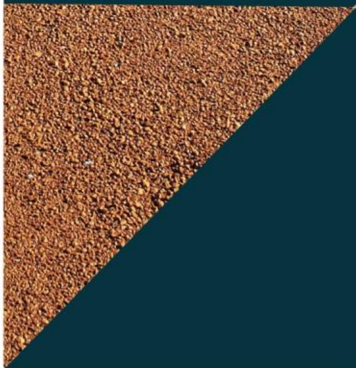
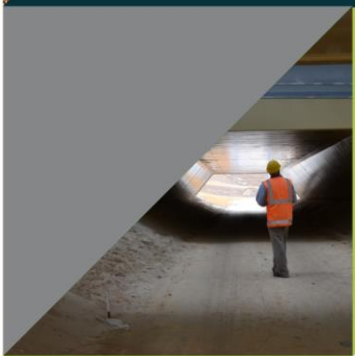


Investigation of MRWA Hydrated Cement Treated Crushed Rocks Base Trial Sections – Stage 1



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Investigation of MRWA Hydrated Cement Treated Crushed Rocks Base Trial Sections – Stage 1

for Main Roads Western Australia

Reviewed

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FINAL STAGE 1

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SUMMARY

This report is stage 1 of the Western Australian Road Research and Innovation Program (WARRIP) project. The main purpose of this stage is to investigate the performance of Hydrated Cement Treated Crushed Rock Base (HCTCRB) trial sections in the Perth metropolitan area. This includes performance monitoring, HCTCRB specification comparisons, review of the current design procedures and providing a possible project plan for further research.

HCTCRB has been used in numerous projects in Western Australia, with continuing performance tests being done on some major sections of road. This report specifically focusses on HCTCRB sections of Kwinana Freeway, Mitchell Freeway and Reid Highway. Performance monitoring for this project includes deflection measurements, rutting and roughness related to traffic counts as well as visual assessments. In this report, visual assessments are not discussed due to limited inspection data, which is only available for Kwinana Freeway and only assessed for rutting.

Overall performance for the roads and sections in this report is acceptable, which include base thicknesses ranging between 123 mm and 200 mm and cement contents between 1% and 2%. The Reid Highway sections are almost 21 years old while the other two roads are between 8 and 9 years old. No apparent deterioration relationships can be predicted with the available data for roughness and rutting. All maximum deflection results are still below 0.5 mm, which indicates good subgrade conditions. The curvature values are also below the design values and indicates adequate upper layer stiffness according to MRWA criteria.

As concluded in this report, continuous performance monitoring particularly in relation to cracking is required on these roads. The curvature results of Reid Highway Section 5 show that a 1% cement content can perform just as well as 2%, and Section 1 shows that a 123 mm base thickness could be considered as a design thickness with similar performance to a 200 mm base thickness, given the overall pavement thicknes remains constant and for lower trafficked roads. If other HCTCRB sections have cracked in the past, these sections should be investigated to identify their exact cracking mechanism. Currently no such sections are reported on in this stage.

Stage 2 of this investigation could include studies into more recent HCTCRB sections constructed with current specifications, if these exist. Furthermore, a study into failed HCTCRB sections could possibly identify other problems in construction, if these sections exist. Mechanistic pavement design standards could be reviewed to possibly identify other design concerns and improve on current design procedures.

The carbonation effect on low cement content in modified granular material could also be investigated.

This research indicates that the pavements mentioned are behaving similar to bound layers, thus highlighting the need for alternative or modified tests specifications in order to improve the confidence of this material to behave as an unbound material.

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FIGURES

1 INTRODUCTION

Cement stabilised materials have been used in Australian pavement design since the early 1950s. Hydrated cemented treated crushed rock base (HCTCRB) was first considered as basecourse material due to the development of high curvature and subsequent premature fatigue cracking of the thin asphalt layers along several heavily trafficked sections of the Kwinana Freeway. Investigations of these sections identified that in most instances the untreated crushed rock base (CRB) was of either poor quality or weakened in service due to moisture ingress.

As a preliminary response to the poor performance of the untreated CRB, bitumen stabilised limestone (BSL) was used and performed well with minimal problems. Subsequently, MRWA developed HCTCRB as a cost-effective alternative to BSL and as a higher modulus, less moisture-sensitive alternative to untreated CRB.

The use of HCTCRB is currently limited to heavily trafficked applications where an asphalt wearing course is commonly required. However, HCTCRB pavements have not been constructed since 2009 due to perceived industry risks of premature distress associated with the technology. Construction considerations such as curing of the HCTCRB prior to compaction are thought to have a significant impact on performance (i.e. fatigue cracking). Additionally, structural design procedures for HCTCRB need further development or validation. These compounding risks have led MRWA to limit the use of HCTCRB to Construct Only contracts, and in these instances, with an asphalt fatigue life of 15 years.

The intention of this project is to improve confidence in the use of HCTCRB by reviewing the following:

- Review and compare the differences in the past and present MRWA specification 501, in order to identify key changes and possible construction and manufacturing procedural changes, that can significantly influence the performance of HCTCRB.
- Review current design procedures of modified granular material and cemented granular material as specified in the Austroads *Guide to Pavement Technology Part 2*.
- Identify and compare differences between current specifications for the design modulus and minimum thickness requirements of modified granular material pavement layers.
- Develop specific HCTCRB design guidance through review and investigation of the design, construction and performance of existing HCTCRB trial sections. Performance measurements will include traffic data, roughness measurements, rutting measurements, maintenenace treatments and falling weight deflectometer (FWD) test results.
- Confirm pavement as-built thicknesses on trial site by excavation of test pits.

Trial sections investigated were:

- Sections 1, 5 and 6 of the Reid Highway trials located between West Swan Road and Bennett Brook Bridge in West Swan, constructed between 1995 and 1996.
- Selected sections of the Kwinana Freeway trials located on the southbound side of the freeway between Paganoni Road and Lymon Road, constructed in 2009.
- Sections of Mitchell Freeway between Hodges Drive and Burns Beach Road, constructed in 2008.

Section 2 of this report describes the development of HCTCRB by MRWA. In Section 3, the flexible pavement design process is outlined based on the Austroads design guidelines. Section 4 covers the Queensland Department of Transport and Main Roads (TMR) guidelines for lightly-bound pavement layers, while Section 5 reviews the Western Australian pavement design supplement (ERN 9) for modified granular and cemented materials. Section 6 focuses on the amendments to cemented materials characterisation in the 2017 edition of the *Austroads Guide to Pavement Technology Part 2* (AGPT02 (2017).

MRWA have carried out numerous tests on HCTCRB samples and trials. Section 7 summarises two of these studies. Section 9 focuses on the comparison of HCTCRB specifications and how these were amended since the first trial.

Finally, Section 10 introduce the performance measures assessed in this report, while Section 11 to Section 13 describe the performance of the three above-mentioned roads and apparent trends identified. The report concludes with a summary of the current performance, as well as a project plan for further investigation into the use of HCTCRB in pavements.

2 HCTCRB – A BRIEF HISTORY

HCTCRB usually consists of a mixture of 2% of general purpose (GP) cement and water, added to crushed rock base (CRB). This mixture is allowed to partially hydrate at a moist condition, disturbed before it can stabilise, and finally compacted into a basecourse layer. The main purpose of the layer should be to act as an unbound layer with improved modulus without being susceptible to fatigue cracking, and hence the vertical resilient modulus was previously limited to 1500 MPa according to MRWA specification 501(2008).

MRWA developed HCTCRB as an alternative to bitumen stabilised limestone (BSL) base, after evidence of premature fatigue cracking of thin asphalt surfacings of some heavily trafficked pavements with CRB (Harris 2008). Fatigue cracking developed prematurely due to problems with the quality of the CRB, as well as the moisture sensitivity of the CRB layers. One of these roads was the Kwinana Freeway, in which three CRB sections all showed signs of premature surface distress. These sections are:

- Leach Highway to South Street (constructed in 1981)
- South Street to Armadale Road (completed in 1992) and
- Armadale Road to Thomas Street (completed in 1994).

Both the dense graded asphalt (DGA) and open graded asphalt (OGA) surfacing layers showed premature cracking after only six years of service.

Initially, after the premature distress of sections of the Kwinana Freeway, MRWA used BSL basecourses as an alternative to CRB. However, the drawback of using BSL basecourses in heavily trafficked freight routes was that time to set and harden was required when an asphalt layer was placed immediately after the basecourse construction and the BSL might be damaged by early-life trafficking, when this was not done. The second issue was that the specifications for a BSL basecourse needed further development, and additional testing still had to be done on certain material types and their applicability. BSL basecourses also tended to be more expensive to construct than HCTCRB.

HCTCRB has been used under both sprayed seal surfacings and thin asphalt surfacings. The sections with an asphalt wearing course include the Kwinana Freeway sections from Berrigan Drive to Thomas Road (2001), Thomas Road to Safety Bay Road (2001) and Roe Highway Welshpool Road to Kwinana Freeway (2002 to 2006). The performance of asphalt surfaced pavements has generally not met expectations under the heavily trafficked freight routes. On the sections with a sprayed seal surfacing, HCTCRB has generally performed well. These sections included the Great Eastern Highway (GEH) Mundaring to Sawyers duplication (1998), Great Northern Highway (GNH) north of Bullsbrook (2004) and Tonkin Highway Mills Road West to Thomas Road (2005).

FWD measurements were used to observe the change in HCTCRB modulus in the slow lane on Kwinana Freeway, Roe Highway and Tonkin Highway.

One of the possible reasons of premature distress of the HCTCRB under an asphalt wearing course was thought to be that the loose HCTCRB had insufficient curing time in stockpile before compaction. This may have resulted in a bound basecourse layer susceptible to fatigue cracking under heavy traffic loading. Another reason could be inadequate pavement layer thicknesses.

Due to the above performance issues, in 2007, Main Roads Western Australia Materials Engineering Branch (MEB) revised the specifications for the construction of HCTCRB. Currently, industry is unwilling to certify that they can comply with these specifications.

Due to limited understanding of the effect of variation between cement contents, hydration periods, curing periods, moisture contents, cement sources and other additional influencing factors, there have been numerous updates and continuous research on HCTCRB materials.

3 FLEXIBLE PAVEMENT DESIGN BASED ON 2012 AUSTROADS GUIDE

3.1 Introduction

This section describes the pavement design procedures in the Austroads *Guide to Pavement Technology Part 2* (AGPT02*): Pavement Structural Design* (Austroads 2012). This publication has since been superceded by the 2017 edition, which is the method now used by road agencies in Australia and New Zealand. The 2017 method is described in Section 6.

The general design procedure, which is contained in this Part of the guide, is mechanistic-empirical in nature. In addition to this, a specific empirical procedure is provided for the design of granular pavements with thin bituminous surfacings. This empirical procedure has been used extensively by Austroads member agencies, and has been found to give results consistent with the mechanistic-empirical procedure (Austroads 2012).

The design procedure is as follows:

- evaluating the input parameters (materials, traffic, environment etc.)
- selecting a trial pavement
- analysing the trial pavement to determine the allowable traffic
- comparing this with the design traffic
- finally, accepting or rejecting the trial pavement (Austroads 2012).

The two most important inputs for the mechanistic design procedure are the traffic loading over the chosen design period, in addition to the performance and elastic properties of the pavement materials.

Pavement materials are classified by their fundamental behaviour under traffic loading into five categories:

- unbound granular materials
- modified granular materials
- cemented materials
- asphalt
- concrete.

3.2 Modified Granular Material

Modified granular materials are granular materials to which small amounts of stabilising agent(s) have been added to improve modulus or to correct other deficiencies in properties (e.g. by reducing plasticity) without causing a significant increase in tensile capacity (i.e. producing a bound material). Modified materials have a maximum 28-day unconfined compressive strength (UCS) of 1 MPa, tested after moist curing, but without soaking, at 100% standard maximum dry density and optimum moisture content (Austroads 2012).

These materials are designed and modelled as traditional unbound granular flexible structures, as any gain in tensile strength is assumed negligible (Austroads 2012).

3.2.1 Elastic Characterisation

The modulus of the pavement material is preferably determined through laboratory testing, but in the absence of test data, presumptive values for unbound granular material may be used for design. Unbound materials are considered cross-anisotropic (degree of anisotropy of 2) with a Poisson's ratio of 0.35, whilst the presumptive modulus value varies depending on the quality and intended application of the material. The presumptive values for elastic characterisation of unbound and modified granular materials are shown in Table 3.1 below.

Table 3.1: Austroads presumptive values for elastic characterisation of unbound and modified granular materials under thin bituminous surfacings

Elastic property	Base quality materials			Subbase quality materials
	High standard crushed rock	Normal standard crushed rock	Base quality gravel	
Range of vertical modulus (MPa)	300–700	200–500	150–400	150–400
Typical vertical modulus (MPa)	500	350	300	250(1)
Degree of anisotropy (2)	2	2	2	2
Range of Poisson's ratio (vertical, horizontal and cross)	0.25–0.4	0.25–0.4	0.25–0.4	0.25–0.4
Typical value of Poisson's ratio	0.35	0.35	0.35	0.35

1 The values are those at typical subbase stress levels in unbound granular pavements with thick bituminous surfacings.
2 Degree of anisotropy is the ratio between vertical modulus and horizontal modulus.

Source: Austroads (2012).

3.3 Cemented Granular Material

Cemented granular materials incorporate sufficient cementitious binder to produce a bound layer with significant tensile strength. Although the structural capacity of cemented materials is significantly higher than unbound or modified materials, they are susceptible to tensile fatigue cracking.

3.3.1 Elastic Characterisation

The modulus of the cemented pavement material is ideally determined through an estimate of the in situ flexural modulus after 28 days curing in situ but may also be estimated from laboratory test data or presumptive values. The structural design model considers cemented granular materials isotropic, with a Poisson's ratio typically assumed at 0.2. The presumptive values for elastic characterisation of cemented granular materials are shown in Table 3.2 below.

Table 3.2: Austroads presumptive values for elastic characterisation of cemented granular materials

Elastic property	Lean concrete subbase	Base 4–5% cement	Subbase quality crushed rock 2–4% cement	Subbase quality crushed rock 4–5% cement
Range of vertical modulus (MPa)(1)	5 000–15 000	3 000–8 000	2 000–5 000	1 500–3 000
Typical vertical modulus (MPa)	7000 (Rolled) 10 000 (Screeded)	5 000	3 500	2 000
Degree of anisotropy (2)	1	1	1	1

Elastic property	Lean concrete subbase	Base 4–5% cement	Subbase quality crushed rock 2–4% cement	Subbase quality crushed rock 4–5% cement
Range of Poisson's ratio (vertical, horizontal and cross)	0.1–0.3	0.1–0.3	0.1–0.3	0.1–0.3
Typical value of Poisson's ratio	0.2	0.2	0.2	0.2

1 Although figures are only quoted for cement, other cementing binders such as lime, lime fly ash, cement fly ash and granulated slag may be used. The modulus of such should be determined by laboratory testing.
2 Degree of anisotropy is the ratio between vertical modulus and horizontal modulus.

Source: Austroads (2012).

3.3.2 Performance Characterisation

The principal distress mode for cemented granular materials is cracking, because of shrinkage, fatigue or over-stressing. The fatigue characteristics of cemented materials may be determined through laboratory testing, preferably in conjunction with field trials, or can be estimated through the cemented material fatigue relationship as presented in Equation 1.

$$N = RF\left(\frac{[\frac{113000}{E^{0.804}} + 191]}{\mu\varepsilon}\right)^{12} \quad (1)$$

where

N = allowable number of standard axle repetitions

με = load-induced tensile strain at base of cemented material (microstrain)

E = cemented material modulus (MPa)

RF = reliability factor for cemented materials fatigue

Source: Austroads (2012).

Cemented layers of a pavement structure that reach the allowable loading, in terms of fatigue cracking, may subsequently enter a post-cracking phase, in which other layers further the fatigue of the pavement. The post-cracking phase is only considered in the design calculations if the cracking in the cemented material does not reflect through to the surface. This is typically achieved by providing a minimum of 175 mm of asphalt or equivalent granular material over the cemented material.

Cracked cemented materials are modelled as unbound granular materials and considered cross-anisotropic materials, with a vertical modulus of 500 MPa and a Poisson's ratio of 0.35. However, it is important to note that there is no sublayering when modelling cracked cemented materials and the fatigue life of both the asphalt layer and the cemented layer are considered together.

4 TMR PAVEMENT DESIGN SUPPLEMENT

4.1 Introduction

The Queensland Department of Transport and Main Roads (TMR) has developed a *Pavement Design Supplement* (2017b) to complement the 2017 edition of AGPT02, which incorporates Queensland-specific design considerations, the details of which are elaborated on in this section.

4.2 Modified Granular Materials

The characterisation of modified granular materials in the TMR *Pavement Design Supplement* (2017b) differs from that in AGPT02 when considering the maximum 28-day unconfined compressive strength (UCS). AGPT02 states that modified materials have a maximum UCS of 1.0 MPa, while TMR states that the UCS may be between 1.0 and 2.0 MPa. Such materials are increasingly being referred to as 'lightly-bound' rather than modified materials. Although TMR acknowledges that the increase in maximum UCS may result in material more prone to fatigue or shrinkage cracking, it states that the benefits include:

- reduced moisture sensitivity
- higher strength and stiffness
- reduced permeability
- reduced erodibility
- reduced sensitivity to variations in grading and plasticity
- higher binder content is more readily available and consistently achieved.

When modified granular materials are used as base courses, TMR typically adopt controls to alleviate the risk of cracking. These include strain alleviating membranes (SAM's) or strain absorbing interlayer (SAMI)s seals, minimum layer thicknesses and/or minimum support conditions.

Pavements utilising a modified granular base course are typically between 200 mm and 300 mm in total thickness, with a construction tolerance of 20 mm added to the design thickness. It is important to note that multiple layers of modified materials are not permitted in accordance with TMR's specifications.

A maximum presumptive modulus of 500 MPa should be used for cement modified base (CMB) materials in the TMR supplement, as described in the report titled 'Considerations for the Selection, Design and Construction of CMB using ET05C' (TMR 2012b). This maximum modulus can be increased to 600 MPa, only if laboratory measurements confirmed higher modulus values.

4.3 Cemented Materials

The typical characteristics of cemented materials supplied to TMR technical standards include:

- Category 1 materials typically produce wider shrinkage cracks, which will be more prone to reflection into overlying layers, than cracks in Category 2 materials.
- Higher standard unbound materials (such as Type 1.1) in the cemented layer should produce narrower, more closely spaced shrinkage cracks and will be less prone to reflective cracking.
- Category 1 materials may be less prone to erosion and crushing than Category 2 materials. Increasingly important for pavements subject to higher traffic volumes and/or higher rainfall.

These characteristics must be considered in the design process.

Materials likely to breakdown in-service, such as decomposed fine grained igneous, metamorphic and sedimentary rocks are not typically used in cemented layers.

TMR's preferred method of determining the design modulus is by laboratory flexural beam testing, in accordance with the 2017 edition of AGPT02.

To produce cemented materials, the volume of stabiliser used, by mass, typically ranges from 3% to 6% to that of the untreated material. Presumptive design modulus values of cemented materials typically adopted for standard TMR materials are presented in Table 4.1.

Table 4.1: TMR presumptive values for elastic characterisation of standard cement materials

Category	Presumptive design modulus (MPa)	Material to be stabilised (MRTS05 Type)	Typical minimum UCS (28-day) (MPa)
Category 1	3500	1.1, 2.1	3.5–4.5
Category 2	2500	1.1, 2.1, 2.2, 3.1 or 3.2	2.5–3.5

Source: TMR (2017b).

4.4 Plant-mixed Cement Modified Base Considerations

4.4.1 Introduction

The TMR report, 'Considerations for the Selection, Design and Construction of CMB using ET05C' was prepared to provide guidance into the use of Main Roads Technical Standard (MRTS) ET05C *Plant-mixed Cement Modified Base (CMB)* (TMR 2012b). To ensure the benefits of utilising CMB are achieved, there must be a high level of control over the constituent materials and mixing process. The benefits of stabilisation are inconsequential if the material does not meet the design characterisation and may lead to pavement distress, requiring additional maintenance or rehabilitation expenditure. Furthermore, achieving in situ uniform distribution of the stabilising binder is difficult at low quantities, leading to a greater adoption of centralised plant mixing.

4.4.2 Design

The structural design process is outlined in ET05C which is a supplement to TMR (2017b) and should also be used in conjunction with AGPT02 (2017).

The two main thickness design steps are as follows:

1. Design the support for the CMB.
2. Determine the CMB thickness.

A minimum unbound granular support layer thickness of 150 mm of Type 2.4 or Type 3.4 material is specified for all support layers of CMB. The minimum modulus achieved on top of the unbound granular material should not be less than 150 MPa. Table 4.2 summarises the minimum support requirements for CMB according to ET05C.

Table 4.2: Minimum support requirements for CMB

Subgrade design CBR (%)	Unbound granular support	Unbound granular plus select fill support
3 to 4	300 mm (min Type 2.4/3.4)	150 mm (min Type 2.4/3.4) 170 mm (select fill min CBR 7%)
5 to 6	200 mm (min Type 2.4/3.4)	N/A
≥ 7	150 mm (min Type 2.4/3.4)	N/A

Source: TMR (2012b).

A capping layer is required for supporting material with a subgrade design CBR less than 3%.

The CMB thickness is determined by using the Austroads (AGPT02) mechanistic design procedure. ET05C specifies a minimum required CMB thickness of 200 mm, which includes the construction tolerance. A 20 mm construction tolerance shall be added to the design output using CIRCLY mechanistic design software.

4.4.3 Construction

The construction of stabilised layers follows a general process of mixing the constituents, delivering and paving the mixture, compaction, preparation of the edges and surfaces, curing and quality control testing.

Plant-mixed CMB layers are constructed using purpose-built mechanical spreaders capable of placing and spreading the mix on the prepared surface to the required uncompacted layer thickness, width and shape in one pass. The strength and durability properties of the CMB layer are significantly influenced by the degree of compaction, calculated using the relative dry density (RDD), typically specified as a minimum of 100% of standard Proctor compaction. Compaction must be completed within the allowable working time of the stabilising agent, especially in cementitious stabilisation. Following compaction, the CMB surface is required to be hard, uniform and have a homogenous appearance.

Effective bonding of multiple layers is essential to maximise bearing capacity and minimise the risk of delamination. A cement/water slurry between layers is used to enhance bonding.

Curing is a vital process in the construction of cementitious stabilised bases, as it is necessary to ensure the design strength is achieved through the hydration process. Cementitious stabilised bases are water-cured, ensuring the surface does not form a slurry, until the next layer of pavement is applied.

4.4.4 Quality Control

The construction quality assurance system implemented by TMR incorporates a series of hold points, witness points and milestones to ensure the treated pavement material complies with the design requirements.

The thickness of each layer needs to be within the range of 150–200 mm with an equal lift thickness throughout. Layers below the modified layer shall have a minimum design thickness and California bearing ratio (CBR) of 300 mm and 3% respectively, unless a capping layer is provided (TMR 2012a).

Trial sections are used for plant-mixed stabilised pavements. This is done to determine the suitability of the contractor's methodology for the works and to ensure all quality assurance requirements are met and any areas of non-compliance rectified.

Properties covered in the quality control system include material gradations and specifications, stabilising agent content, moisture content, mix uniformity, level of compaction, curing conditions and geometric requirements. The stabilised materials are sampled to determine the densities after final mixing but prior to the commencement of compaction. Other requirements such as moisture content should be addressed at the appropriate stages of construction. Testing of compaction and stabilising agent content is achieved by taking representative field samples from in situ conditions using test method Q140 (TMR 2017a) and Q134 (TMR 2017a) respectively. Modified materials are required to have a 7-day unconfined compressive strength (UCS) between 1.0 MPa and 2.0 MPa, with a target of 1.5 MPa in accordance with test method Q115 (TMR 2017a).

5 WESTERN AUSTRALIAN PAVEMENT DESIGN SUPPLEMENT

5.1 Introduction

Similarly, to the Queensland Department of Transport and Main Roads (TMR) *Pavement Design Supplement* (2017b), Western Australian also has a AGPT02 design supplement, namely *Engineering Road Note 9 (ERN9)* (MRWA 2013). Note that this 2013 edition of ERN9 is a supplement to the 2012 edition of AGPT02. In the near future ERN9 will be revised to align with the 2017 edition of AGPT02.

For flexible pavement design, ERN9 has a specific paragraph of compliance for HCTCRB. This states that HCTCRB pavements must have a minimum fatigue life of 40 years and should only be used in pavements where Main Roads carries the design risk. ERN9 does not provide a method to design of fatigue of HCTCRB.

5.2 Modified Granular Materials

ERN 9 specifies that no pavement should incorporate modified granular material unless the 7-day unconfined compressive strength is less than 1.0 MPa when tested at in-service density according to Western Australian test methods WA 143.1 and WA 143.2. In addition, when granular pavements are in situ stabilised, ERN9 specifies that the material should comply with the guidance notes of Specification 501.

The modulus limits for unbound or modified granular materials in the mechanistic design procedure should not exceed those presumptive values as speculated in Table 6.3 of AGPT02 (2012) if the nominal asphalt thickness is less than 60 mm, unless higher values are proven by laboratory tests. Even if so, when the nominal asphalt thickness is less than 60 mm, the vertical modulus used in the mechanistic design process for the top granular sublayer should not exceed 1000 MPa.

When the nominal asphalt thickness is more than 60 mm, the vertical modulus shall not exceed the values of Table 6.4 in AGPT02 (2012).

It is important to note that ERN9 states that no reduction in thickness requirements can be made for pavements incorporating granular material modified with cement, lime, bitumen or other similar materials.

5.3 Cemented Materials

- MRWA excludes the use of cemented materials in flexible pavements, but ERN9 indicates a few exceptions as follows: In situ cement stabilised crushed rock (2% by mass of Type LH cement) or crushed recycled concrete may be placed as a subbase under full depth asphalt pavements or under a HCTCRB. The vertical modulus used in the mechanistic design process for such subbase may not exceed 500 MPa.
- Cemented materials may be used as a working platform below the design subgrade surface. The CBR of the cemented material used in the pavement design must not exceed that of the unbound granular material used to manufacture the cemented material.

6 IMPROVED CHARACTERISATION OF CEMENTED MATERIALS IN 2017 AUSTROADS GUIDE

The objective of the Austroads research project TT1664 *(Cemented Materials Characterisation)* (Austroads 2014) was to develop improved methods to design flexible pavements with cemented materials, building on the research previously undertaken in Austroads research project TT1358 *(Cost-effective Structural Treatments for Rural Highways: Cemented Materials)* (Austroads 2010). The outcomes and findings from the research include:

- Development of test methods for the determination of flexural modulus, flexural strength and fatigue of cement treated crushed rocks and natural gravels.
- Strain-based fatigue laboratory relationships were a better fit to the data than stress-based relationships and it is proposed to continue the use of logN - logµε fatigue relationship.
- Strain damage exponents from 9 to 24 were calculated from the data.
- The variation in flexural modulus and strength in relation to density was quantified and procedures proposed for use in design.

The outcomes of the research project were achieved by conducting laboratory testing on a diverse range of cement stabilised materials utilising GP Portland cement, including crushed rocks, natural gravels and recycled crushed concrete. Testing was conducted by preparing specimens in accordance with four-point flexural beam methods and sealing the moist beams in cling wrap for a curing period of either 28 days, 5 months or 9 months. Flexural beam test methods were developed to find flexural modulus, flexural strength and fatigue of cement treated materials.

Based on the research, several changes were recommended for the Austroads *Guide to Pavement Technology Part 2: Pavement Structural Design* (Austroads 2012). The changes are in terms of the characterisation of cement treated crushed rocks and natural gravels.

In terms of modulus characterisation, the proposed changes are:

- change the definition of cemented materials design modulus to be the 90-day flexural modulus in situ
- a test method to manufacture laboratory test beams and measure flexural modulus
- the inclusion of a procedure to determine the design modulus from the measured flexural modulus
- the inclusion of a procedure to adjust the measured flexural modulus for differences in density between the modulus test beams and the density in situ
- amendments to presumptive moduli values.

In terms of fatigue characterisation, the proposed changes are:

- a test method to manufacture laboratory test beams and measure fatigue characteristics and hence determine a laboratory fatigue relationship
- a test method to manufacture laboratory test beams and measure flexural modulus
- a procedure to estimate the laboratory fatigue characteristics from the measured flexural modulus and flexural strength

- a procedure to determine in-service fatigue relationships from the laboratory fatigue characteristics
- a procedure to determine in-service fatigue relationships from design flexural modulus and design flexural strength
- presumptive in-service fatigue relationships based on presumptive moduli and strength for three types of cemented materials.

The revised AGPT02 incorporating these changes was published in late 2017.

In addition, in the 2017 edition of AGPT02 the method of determining the fatigue damage was changed. In the 2012 edition, the cemented materials fatigue life was determined from the tensile strains calculated under an 80 kN standard axle. This allowable traffic loading was then compared to the deisgn traffic expressed in terms of Standard Axle Repetitions (SAR12). In the 2017 edition, the strains are calculated under each axle load from which the fatigue damage due to each axle load is calculated. The overall fatigue damage is calculated by summing the damage due to each axle load. This new method is called the axle-strain method.

7 COMPARISON OF GUIDANCE RELATIVE TO HCTCRB

A summary of the requirements specified by Austroads, TMR and MRWA regarding modified and cemented material design guidance is presented in Table 7.1.

The modified and cemented granular material design guidance generally specify the design modulus and target UCS values, where cemented granular materials also include the volume of stabiliser. General observations from the comparison between MRWA requirements with those specified in Austroads and TMR documents include:

- MRWA allow a modified granular material design modulus of up to 1000 MPa if proven by laboratory tests while TMR only allow up to 600 MPa.
- MRWA use the 7-day UCS while Austroads and TMR use the 28-day UCS.
- MRWA do not allow thickness reductions for modified granular materials.
- The volume of stabiliser in cemented granular materials varies between MRWA, Austroads and TMR where MRWA allow the least (2%).
- MRWA restrict the use of cemented granular materials where the design modulus must not exceed 500 MPa, which is significantly lower than the presumptive values outlined by Austroads and TMR.
- MRWA do not specify a target UCS.
- TMR specify a greater UCS than Austroads.
- MRWA do not generally permit cemented granular materials to be incorporated in flexible pavements.
- TMR was the only agency reviewed to include provisions for plant-mixed cemented modified bases, which include minimum support for CMB layers.

Table 7.1: Comparison of design guidance

Criteria	MRWA	Austroads	TMR
Modified granular material			
Modulus (MPa)	▪ If asphalt thickness < 60 mm modulus should not exceed presumptive values in Table 6.3 of AGPT02 unless higher values are proven by laboratory tests. Design modulus shall not exceed 1000 MPa ▪ Should not exceed presumptive values in Table 6.4 of AGPT02 if asphalt thickness > 60 mm	▪ Presumptive values outlined in AGPT02	▪ Typically 500 MPa ▪ 600 MPa if proven by laboratory tests
UCS (max, MPa)	1.0 (7-day)	1.0 (28-day)	1.0-2.0 (28-day)
Implementation	▪ No reduction in thickness requirements for pavements incorporating modified granular material	▪ Designed and modelled as traditional unbound granular flexible pavement structures	▪ When used for base courses, controls to reduce the risk of cracking (e.g. SAMs or SAMI seals) are adopted
Cemented granular material			
Volume of stabiliser by mass (%)	2	2-5	3-6
Modulus (max, MPa)	500	▪ Presumptive values outlined in AGPT02	Category 1 – 3500 Category 2 – 2500
UCS (MPa)	–	2.0 (min) for moderate to heavily trafficked roads 1-2 for lightly trafficked roads	Category 1 – 3.5-4.5 (28-day) Category 2 – 2.5-3.5 (28-day)
Implementation	▪ May be used as a working platform below the design subgrade surface ▪ May be placed as a subbase under FDA pavement or HCTCRB basecourse ▪ Generally not permitted for use in flexible pavements	▪ Modelled as an isotropic material with a Poisson's ratio of 0.2	▪ Category 1 materials may be less prone to erosion and crushing than Category 2 materials
Plant-mixed cemented modified base			
Implementation	–	–	▪ A minimum unbound granular support layer thickness of 150 mm of Type 2.4 or Type 3.4 material is specified for all support layers of CMB with a minimum modulus of 150 MPa.

8 MRWA HCTCRB INVESTIGATION

8.1 Introduction

This section summarises two investigation reports compiled by MRWA. An investigation in 2008 was carried out mainly to assess HCTCRB material produced by metropolitan quarries' compliance with MRWA Specification 501 (2008). A second investigation in 2013 mainly focussed on possible additional effects affecting HCTCRB's performance, including carbonation, cement mixtures and sample preparation techniques, but also included UCS and resilient modulus test results for the Kwinana Freeway trial sections. The test results for the Kwinana Freeway sections are discussed in Section 9 and only the additional testing procedures and outcomes are discussed below.

8.2 2008: Investigation of HCTCRB Produced by Metropolitan Quarries

8.2.1 Introduction

MRWA conducted an investigation in 2008 to assess the ability of certain metropolitan quarries to meet the requirements of Specification 501 (2008) in relation to HCTCRB. Firstly, a grading analysis was done on the material obtained from these quarries to test for compliance as a CRB material. Thereafter, UCS and resilient modulus testing was done on HCTCRB samples in a controlled environment to test their compliance to the specifications.

Specification 501 (2008) required the measured resilient modulus to be more than 1000 MPa and less than 1500 MPa.

The samples were tested under the following conditions to see the effect of different hydration periods:

- different samples of loose HCTCRB mixture were left to cure for 7, 14, 30, 60 and 90 days prior to compaction
- specimens were compacted to 99% of the Modified Dry Density (MDD) and 100% of the Optimum Moisture Content (OMC)
- specimens were cured after compaction for 28 days prior to modulus testing.

In addition, the compacted samples' UCS values needed to be less than 1.5 MPa when tested after 28 days of curing in the moulds.

This report includes a full investigation on the material properties and its suitability to be used as a HCTCRB. The resilient modulus was calculated and used to obtain an estimate for the asphalt fatigue life using mechanistic design principles.

8.2.2 Quarries used to Obtain Material Samples

Material from the following quarries was assessed, as presented in MRWA report No. 2008/02M:

- Readymix Gosnells (Readymix) (Also known as Cemex or Holcim due to change of ownership)
- WA Limestone Byford Coarse (WA LC)
- WA Limestone Byford fines added (WA LF)
- BGC lakes (BGC F)

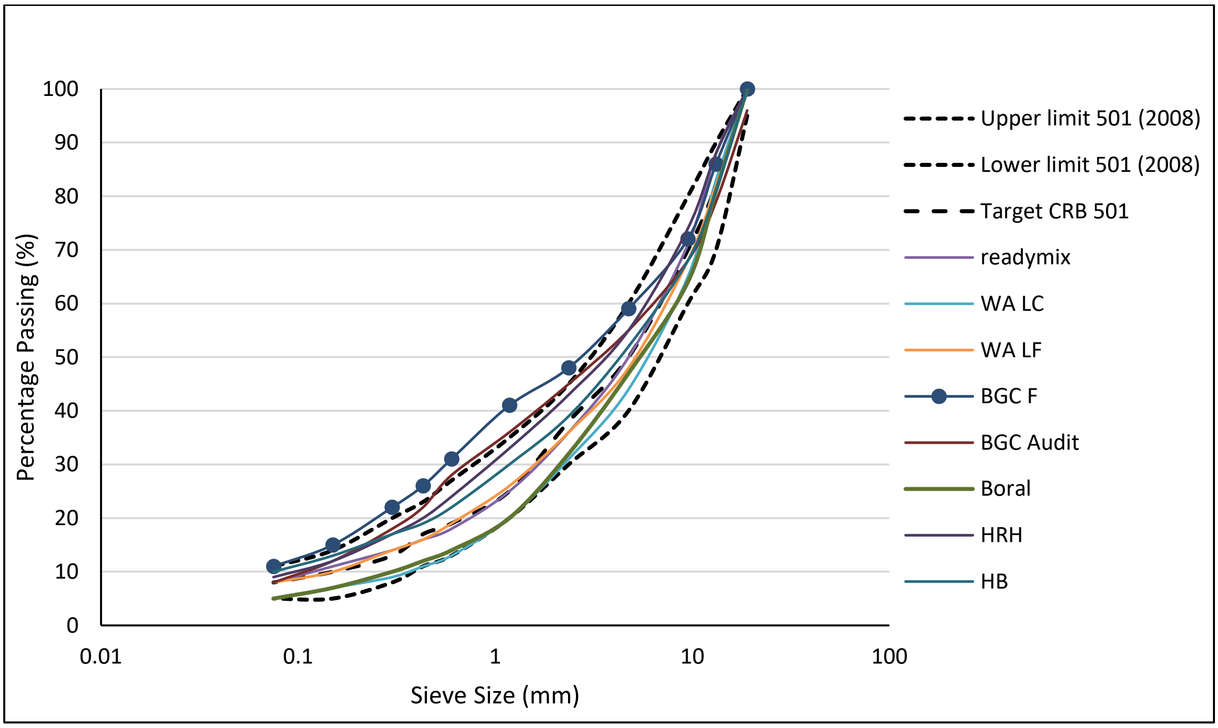
- Boral Orange Grove (Boral)
- Hanson Byford (HB)
- Hanson Red Hill (HRH).

8.2.3 Sampling and Testing

Sampling was done on 200 kg bulk samples, delivered by truck from each quarry, according to Western Australian test method WA100.1. The samples were separated into five particle size fractions and remixed, to obtain a uniform particle distribution for the UCS and modulus testing. These remixed samples were prepared to have a particle size distribution close to a target grading, as prescribed for HCTCRB in Specification 501 (2008).

Figure 8.1 summarises all the different particle size distributions for all the quarries tested. It also shows the upper and lower limits of the particle size distribution for a CRB according to MRWA specification 501(2008). Note that most samples, except the BGC fine samples, fall within the specified grading envelope. The WA Byford Limestone with fines added, as well as the Readymix sample, most closely followed the target grading.

Figure 8.1: Specified grading envelope in Specification 501 (2008) and actual grading obtained for each quarry



Source: Zaremba (2008).

The CRB classification tests conducted include the particle size distribution, Liquid Limit (LL), Linear Shrinkage (LS), the calculation of the Plasticity Index (PI), Maximum Dry Compressive Strength (MDCS), Maximum Dry Density (MDD), California Bearing Ratio (CBR), Los Angeles Abrasion (LA) and Flakiness Index (FI), all in accordance with MRWA test methods.

Table 8.1 below summarises the results. None of the samples conformed to all the specified limits as required in the table. Non-conforming values are shaded in grey.

Table 8.1: MRWA material property requirements according to Specification 501 (2008) and actual quarry results

Test	Freeway limits (HCTCRB) (2008)	Readymix	WA LC	WA LF	BGC F	Boral	HB	HRH
LL	25% Max	25.4	22.7		22.2	21.2	25.4	22.2
PI	6% Max	9.1	np	np	np	np	8.6	Np
LS	2% Max 0.4% Min	2.4	0.0	0.0	0.0	0.0	1.2	0.0
FI	30% Max	20.9	20.8	23.7	18.8	30.9	12.5	36.3
LA	35% Max	22.4	26.2	16.1	35.3	22.8	18.4	26.2
MDCS	1.7 MPa Min	3.25	0.94	0.99	6.50	2.26	3.27	3.53
CBR(S)	100% Min	140	140	140	180	180	180	120

Source: Zaremba (2008).

8.2.4 Resilient Modulus and UCS Testing

Each CRB sample was wet to optimum moisture content, cement was added and the loose mixture left to hydrate for 7 to 14 days during the initial testing. The mixture was compacted into moulds for resilient modulus and UCS testing. All quarry samples were tested with a 2% cement (by mass) content. The Readymix sample was also tested at three cement contents, namely, 1.0%, 1.5% and 2.0%, to investigate the possible differences in results for alternative cement contents.

The following test procedure was used:

- The MDD/OMC ratio at a seven days' hydration period for each sample was measured.
- The HCTCRB material was prepared for testing by calculating the mass of cement and mixing this into the CRB sample.
- The loose HCTCRB mixture was left in sealed containers to hydrate for between 7 and 14 days.
- The samples for modulus and UCS test specimens were compacted, presumably using a modified compaction hammer to a dry density of 99% MDD.
- Resilient modulus testing after curing for 28 days according to Austroads Test Method T053 using normalised stress of 240 kPa and an octahedral stress of 125 kPa.
- UCS testing according to test method WA 143.1 at 7 days and 28 days after compaction.

Table 8.2 summarises the UCS results obtained with a 2% cement content and 28-day curing in the mould at different hydration periods. Except for HCTCRB produced from the Boral quarry, the UCS results after 7 days of hydration were outside the specified range of 1.0 to 1.5 MPa.

Table 8.2: UCS results from all tested quarry samples and hydration periods with 2% cement added by mass

UCS MPa (2% cement)	Readymix	WA LC	WA LF	BGC F	Boral	HB	HRH
7 days hydration	1.53	1.55	2.31	2.20	1.36	1.56	1.68
14 days hydration	1.4	0.92	1.52	2.40	1.28	1.44	1.48
30 days hydration	1.39	0.77	1.12	1.6	1.1	1.27	1.10

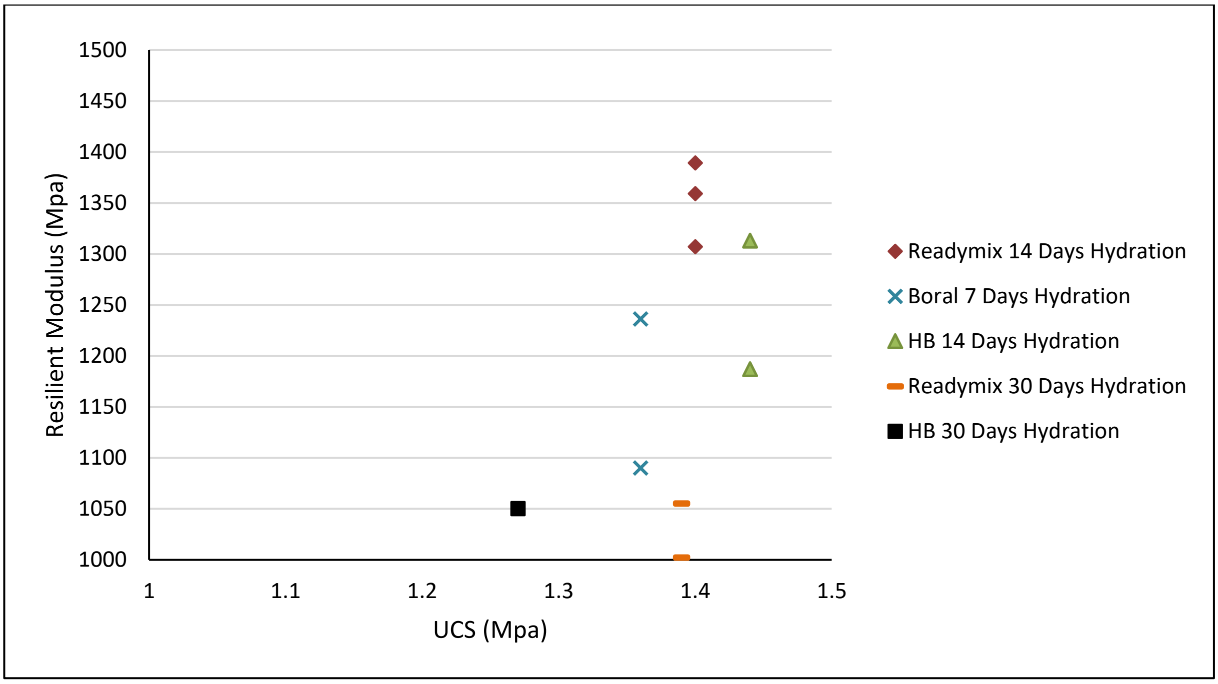
Source: Zaremba (2008).

Further results were obtained for 60 and 90 days hydration to study the influence of longer hydration periods. Generally, an increase in hydration period of more than 30 days had a decreasing effect on the UCS although this statement was not true for all quarries.

The resilient modulus tests carried out and UCS tests are tabulated in Appendix A. Only the few samples that had results within the qualifying criteria for UCS and Repeated Load Triaxial Test (RLT) are shown in Figure 8.2.

Figure 8.2 represents the HCTCRB mixes with different hydration periods plotted against RLT and UCS. All specimens had modulus values between 1000 MPa and 1500 MPa, as well as UCS values less than 1.5 MPa.

Figure 8.2: Laboratory results to Specification 501 (2008) criteria for UCS and RLT



Source: Zaremba (2008).

It was reported that the UCS did not vary significantly for different hydration periods of more than seven days. The resilient modulus, however, decreased significantly after more than 30 days hydration period. No clear relationship between the moisture ratio and resilient modulus could be established from the test data for the Readymix sample.

8.2.5 Conclusions based on Results

As seen from the results in Appendix A, of all the HCTCRB mixes made using CRB from seven quarries, only the mixes in Figure 8.2 met the modulus and UCS requirements of specification 501(2008).

There was no clear relationship between UCS and resilient modulus results. Hence, both measures need to be tested for conformance to the specifications. The required hydration period was material and source rock dependant, which requires additional research.

A recommendation of the report was that MRWA investigate and test the MDD/OMC ratio at the same hydration period as the test specimens used for UCS and resilient modulus testing. An investigation could be conducted on the impact of drying back the samples to better replicate in field samples. Another recommendation was that the test on BGC samples be repeated on materials that are compliant to the material particle size distribution.

8.3 2013: Investigation of Modulus and UCS

8.3.1 Introduction

The samples in the MRWA Report 2013 – 9M (Xiaoyan & Brookes 2008) were obtained from the stockpiles supplied by Readymix Gosnells, as used in the New Perth Bunbury Highway project.

The laboratory specimens were prepared using three different cement types.

Tests on the mixes were conducted under the following conditions:

- compacted at 50% and 100% OMC of the CRB
- the 100% OMC samples were dried back to 70%, 80% and 90% of OMC of HCTCRB prior to testing
- hydration periods ranged from 7 to 120 days
- curing periods of 28 days and 180 days.

8.3.2 Influence of Different Cement Types

The different cement types tested were as follows:

- Cement #1: Cockburn Cement Bagged Ex Swan – 100% imported Mitsubishi clinker, fineness index 370 m^2/kg
- Cement #2: Cockburn Cement Bulk – 80% Munster, 20% Mitsubishi clinker, fineness index is not available
- Cement #3: BGC Cement Bulk – 100% imported Cilicap clinker, South Java, fineness index is not available.

It was concluded that there were no significant differences between cement types based on the 28 day UCS results. However, the 28-day resilient modulus test results did indicate a difference between cement types used. It was decided to continue with the investigation using only Cockburn Bagged Cement.

8.3.3 Carbonation Effect

The effect of carbonation on the HCTCRB modulus and strength was evaluated by adding approximately 330 g of dry ice to a curing bucket to create a carbon dioxide rich environment for two days before testing. Samples from seven quarries were tested, similar to those in the 2008 investigation.

All showed visible effects of carbonation and for most samples, the 28-day UCS result increased due to carbonation. This was, however, not the case for all samples,hence this generalisation should be regarded with caution, as this may differ between material types.

8.3.4 Effect of Moisture Content and Hydration Periods

The 28-day resilient modulus of HCTCRB samples compacted to target moisture ratios without dry back showed that in general, the resilient modulus decreased with an increase in moisture ratio. However, the results showed no clear effect of different hydration periods on the resilient modulus.

Testing the impact of a lower hydration moisture content (50% of CRB OMC) on the 28-day curing samples' resilient modulus proved difficult. All the UCS samples prepared with samples hydrated at 50% of the CRB OMC deteriorated when soaked in water. Therefore, no tests on UCS values were conducted. Furthermore, no MDD/OMC ratio test was done on samples hydrated at 50% of the CRB OMC.

The resilient modulus tests at 50% OMC and 100% OMC were, however, done on 21 days hydrated samples with 28 days curing, but on 100% MDD/OMC ratios. Results showed that the 50% hydrated samples' resilient modulus values were consistently lower than those hydrated at 100% of the CRB OMC. These results could have been due to insufficient MDD/OMC data for the 50% hydrated samples and the reader should be cautioned in using these results. This could also have been due to the lower density of specimens compacted at 50% OMC.

8.3.5 Study on Different Curing Periods

Testing was carried out to determine the effect of extended curing periods on the resilient modulus of the samples. This was done by comparing the modulus of 28-day cured specimens with those cured for 180 days.

For the 28-day cured specimens, hydration periods of 14, 21 and 42 days were used. For the 180-day cured specimens, a 30-day hydration period was used.

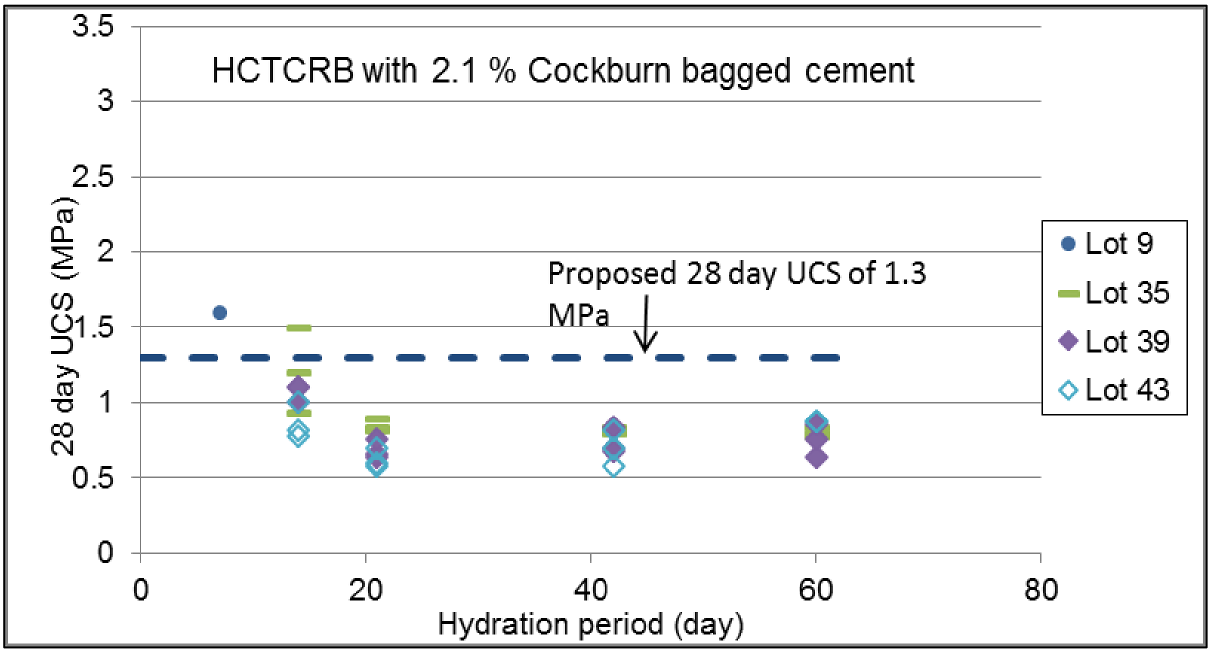
The results indicated consistently higher resilient modulus values for 180 days of curing. These results are questionable, due to a lack of information on the cement content used for the 180 days curing samples.

8.3.6 Determination of Construction Dry Back and Construction Hydration Period for HCTCRB

For the UCS tests, samples were hydrated for 7, 14, 21, 42 and 60 days. The specimens were cured for 28 days, and dried back to 70%, 80% and 90% of the OMC of HCTCRB. The MDD/OMC ratio of the HCTCRB was determined at each hydration period.

Results depicted in Figure 8.3 shows an initial decrease in UCS values with an increase in hydration period between 7 days and 21 days, and thereafter it remains constant. As reported by Xiaoyan and Brookes (2008), MRWA decided to adopt a minimum of 30 days hydration period to achieve a 28-day UCS value of less than 1.3 MPa.

Figure 8.3: Variation in UCS with hydration time



Source: Xiaoyan and Brookes (2008).

To determine the maximum hydration period, HCTCRB samples were hydrated for 14, 21, 42, 60, 90 and 120 days. Resilient modulus tests were carried out after 28 days of curing in the mould as compacted. The results showed modulus of less than 1000 MPa after hydration periods of more than 90 days. Hence, a maximum hydration period of 60 days was adopted.

The data used to derive the dry back moisture ratio was not sufficient to make a conclusion. However, a 70% dry back moisture ratio was assigned, based on past engineering experience and judgement. This should ensure that sufficient resilient modulus is achieved shortly after compaction into the pavement.

9 MRWA HCTCRB SPECIFICATIONS

9.1 Current and Historical HCTCRB Specifications for MRWA

From the initial introduction of HCTCRB basecourse, the specifications were modified numerous times in the last two decades. The latest specification from MRWA is Specification 501, issued 22 May 2017. This section highlights the major changes in the specification between 2007 and 2017.

The test sections mentioned in this report used the following specifications during construction:

- Kwinana Freeway – MRWA Specification 501 – revision 9/18/2008
- Reid Highway – MRWA HCTCRB Construction.doc file as used in Report 2004-17M
- Mitchell Freeway – MRWA Specification 501 – revision 17/01/07.

9.2 Specification Limits

The section below compares the changes in Specification 501 for HCTCRB and how these were amended over time. It compares the following:

- MRWA HCTCRB Construction.doc file as used in Report 2004-17M
- MRWA Specification 501 – revision 9/18/2008
- MRWA Specification 501 – revision 18/10/2012
- MRWA Specification 501 – revision 22/05/2017.

NOTE: Testing methods may differ from states and territories and hence it is extremely important to refer to the mentioned testing methods as stated in the latest specifications 501 document published by MRWA.

9.2.1 Particle Size Distribution

Table 9.1 below summarises the particle size distributions for CRB and for HCTCRB. Initially the specifications for HCTCRB was the same as for CRB, as used on the Reid Highway and Kwinana Freeway sections. Post 2008, the particle size distribution for HCTCRB was adjusted to contain finer particles as shown in Figure 9.1, which shows the adjusted lower limit of the grading envelope.

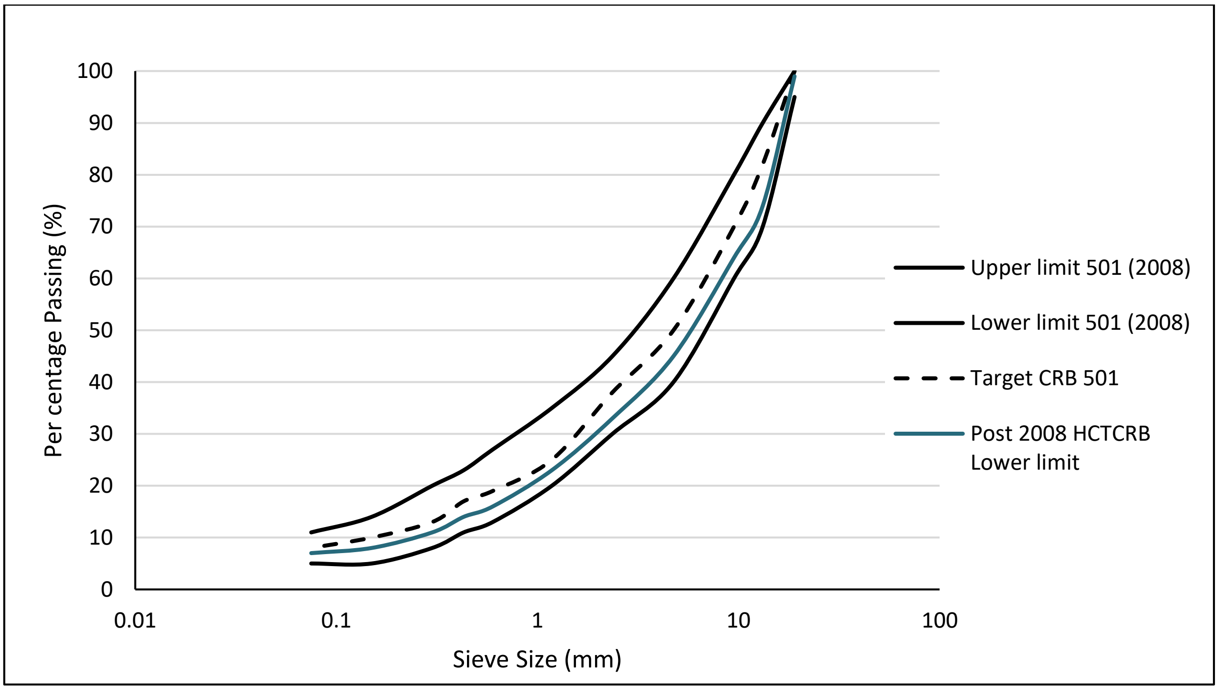
Table 9.1: Particle size distribution for CRB

AS 1152 sieve size (mm)	% Passing by mass target grading	% Passing by mass minimum and maximum limits	
		General CRB (used for Kwinana, Reid and Mitchell sections)	CRB for HCTCRB manufacturing (current specifications after 2008)
26.5		100	100
19.0	100	95–100	99–100
13.2	82	70–90	74–90
9.5	70	60–80	64–80
4.75	50	40–60	45–60
2.36	38	30–45	33–45

AS 1152 sieve size (mm)	% Passing by mass target grading	% Passing by mass minimum and maximum limits	
		General CRB (used for Kwinana, Reid and Mitchell sections)	CRB for HCTCRB manufacturing (current specifications after 2008)
1.18	25	20–35	23–35
0.600	19	13–27	16–27
0.425	17	11–23	14–23
0.300	13	8–20	11–20
0.150	10	5–14	8–14
0.075	8	5–11	7–11

Sources: Butkus (2004), MRWA specification 501(2008), MRWA specification 501 (2012), MRWA specification 501 (2017).

Figure 9.1: Grading envelopes for CRB used for HCTCRB manufacturing



Sources: Butkus (2004), MRWA specification 501(2008), MRWA specification 501 (2012), MRWA specification 501 (2017).

9.2.2 Dust Ratio

The dust ratio is defined as the percentage by mass passing the 0.075 mm sieve to the percentage by mass passing the 0.475 mm sieve. In all constructed trial sections, as well as the latest specifications, the dust ratio was required to be within the range of 0.35 to 0.60.

9.2.3 Other Limits

For the 1996 sections of the Reid Highway, all CRB material was specified to be crushed granite (Butkus 2004). This requirement was removed in the 2008 specifications and onwards.

After 2008, additional testing was required to determine the Wet/Dry Strength of the sample. Secondary mineral content, as well as Accelerated Soundness Index were introduced for basic igneous rocks only. In 2008, a maximum limit for PI was added and later again removed from the specifications. Through additional testing and improvements to testing methods, the CRB testing procedure and compaction were amended as indicated in Table 9.2.

Table 9.2: Other material property requirements for CRB used for HCTCRB

Test	Limits 2017 & 2012	Limits 2008	Limits 1996	Test method
Liquid Limit (Cone Penetrometer)	25.0% Maximum	Similar to latest	Similar to latest	WA 120.2
Plasticity Index	Not specified	6% Maximum	Not specified	WA 122.1
Linear Shrinkage	2.0% Maximum 0.4% Minimum	Similar to latest	Similar to latest	WA 123.1
Flakiness Index	30% Maximum	Similar to latest	Similar to latest	WA 216.1
Los Angeles Abrasion Value	35% Maximum	Similar to latest	Similar to latest	WA 220.1
Maximum Dry Compressive Strength	1.7 MPa Minimum	Similar to latest	Similar to latest	WA 140.1
California Bearing Ratio (soaked 4 days with 4.5 kg Surcharge) at 99% of MDD and 100% of OMC	100% Minimum (4.5 kg surcharge not included in 2012 specifications)	100% Minimum (soaked 4 days) at 98% of MDD and 100% OMC	100% Minimum (soaked 4 days)	WA 141.1
Wet/Dry Strength Variation	35% Maximum	Not specified	Not specified	AS 1141.22
Secondary Mineral Content in Basic Igneous Rock	25% Maximum	Not specified	Not specified	AS 1141.26
Accelerated Soundness Index by Reflux	94% Minimum	Not specified	Not specified	AS 1141.29

Sources: Butkus (2004), MRWA specification 501(2008), MRWA specification 501 (2012), MRWA specification 501 (2017).

9.2.4 Cement Content and Type

In the 1996 specification, the cement type was required to comply with type General Purpose (GP) cement. The 2008 specification and subsequent revisions allowed the use of approved blended cements.

In 1996 and 2008, the specified range of cement content was 2.0 ± 0.2%. The current HCTCRB specification limits the cement content to 2.0 ± 0.1%.

9.2.5 Specified Moisture Content

For the Reid Highway test sections, constructed using the 1996 specification, the moisture content (MC) of the stockpiled HCTCRB had to be within –1% and +2% of the modified Proctor Optimum Moisture Content (OMC) of the untreated CRB.

For the Kwinana Freeway trials, constructed to 2008 specifications, pre-wetting to 95–110% of the OMC for crushed rock was required. For the 2012 specifications and subsequent revisions the pre-wetting was changed to a minimum 95% of the OMC.

9.2.6 Hydration Periods

For the Reid Highway and Kwinana Freeway trial sections, the initial hydration period in stockpile was specified as not less than 7 days. The layer had to be constructed and compacted not more than 28 days after cement was added. This specification was amended after 2008 and the new minimum hydration period in stockpile was increased to 30 days, with the layer constructed and compacted not more than 60 days after the addition of cement.

The material used in the Kwinana Freeway sections had to be moved to stockpile within 24 hours after blending. This limit was adjusted in subsequent specifications to allow up to 9 days for the blended material to be delivered on site.

9.2.7 Construction Issues – Dry Back Requirements

For the Reid Highway, construction of the base was specified to commence after the subbase had dried back such that the characteristic MC was less than 85% of the OMC. The basecourse also had to be dried back to the characteristic MC of less than 85% of OMC before the wearing course could be placed.

Dry back requirements for Kwinana Freeway and current specifications are included in Table 9.5 and Table 9.6.

9.2.8 Density and Construction

During construction of the Reid Highway trial sections, the HCTCRB was spread using a grader. The specified minimum characteristic dry density ratio was 98%.

The 2008 and subsequent specifications, required HCTCRB to be paver laid. The specified minimum characteristic dry density ratio was 99%.

9.2.9 Clegg Impact Values

The 1996 specifications specified Clegg Impact Values (CIV) as an indication of pavement stiffness. This value had to be greater than or equal to 50 CIV before surfacing could commence.

In the 2008 specification, the Clegg Impact Value was 50 CIV for a sprayed final seal and 55 CIV for an asphalt surfacing.

In the latest specifications the Clegg Impact Value must be 55 CIV or greater for an asphalt surfacing.

The Clegg impact hammer test consists of a 4.5 kg hammer, which determines compaction strength and consolidation. The test measures deceleration in units of Impact Values and can be related to CBR.

9.2.10 Surfacing Requirements

For the 1996 specifications, as used on the Reid Highway sections, it was not a prerequisite to surface the road with asphalt although the sections were overlaid with asphalt to correct pavement shape.

The 2012 and 2017 specifications require a geotextile reinforced seal with a double seal of 14/7 mm Class 170 binder to maintain a waterproof seal on the basecourse followed by an asphalt surfacing.

9.2.11 UCS and Resilient Modulus Test Requirements

The 2008 specification required the modulus and 28-day UCS results to be measured after hydration periods of 7, 14 and 30 days. A hold point was introduced, with a minimum hydration period of seven days and no more than 28 days. Subsequent specifications' hold point requires the modulus and UCS specimens to be compacted after a minimum 30 days and maximum 60 days after the cement and water is added.

Table 9.3 summarises the limits for the compressive and resilient modulus values per edition of the specification.

Table 9.3: UCS and resilient modulus requirements for all specifications

Test	Limits 2017	Limits 2012	Limits 2008	Limits 1996	Test method
Unconfined Compressive Strength at 7-days. At the in-service density condition	0.8 MPa Maximum	1		Not defined	WA 143.2
Unconfined Compressive Strength at 28-days. At the in-service density condition	1.0 MPa Maximum	1.3	1.5 (WA 143.1)	Not defined	WA 143.2
Resilient modulus at the in-service conditions, including in-service stress, construction hydration period, moisture and density conditions	1000 MPa Minimum 1500 MPa Maximum	1000 1500	1000 1500 (at in-service hydration period, moisture and density conditions	Not defined	Laboratory Repeated Load Triaxial Test AG:PT/T053 using internal displacement measuring device

Source: Butkus (2004), MRWA specification 501(2008), MRWA specification 501 (2012), MRWA specification 501 (2017).

9.2.12 Additional Construction Hold Points and Tolerances

Additional construction hold points and tolerances included in the latest Specification 501 include:

- Cement used should be less than three months old.
- HCTCRB layer should be worked and compacted in maximum 250 mm and minimum 150 mm layer thickness.
- The compaction and trimming of the layer should be completed within 12 hours after construction starts.

Table 9.4, Table 9.5 and Table 9.6 give additional requirements for HCTCRB layers and are extracts from the 2017 specification 501.

Table 9.4: Compaction values for all pavement layers (Table 501A1 MRWA Specification 501, 2017)

Pavement layer	Minimum characteristic dry density ratio % (Rc)
Drainage layer	94%
All subbase	94%
Cement stabilised basecourse	96%
Lime stabilised basecourse	96%
Bitumen stabilised limestone basecourse	98%
Crushed rock base	99%
Hydrated Cement Treated Crushed Rock Base	99% (99% only if asphalt surfaced 2008)
Other basecourse materials (final surfacing – sprayed seal)	96% or 98%
Other basecourse materials (final surfacing – asphalt)	98%

Source: MRWA specification 501 (2017).

Table 9.5: Dry back requirements – subgrade and subbase (Table 501A2 MRWA Specification 501, 2017)

Subgrade or pavement layers	Maximum dry back characteristic moisture content (DMc) as a proportion of optimum moisture content
Layer 150 mm below subgrade surface (except for Perth sand)	85% (not required in 2008)
Drainage layer	Not required (85% required in 2008)
Subbase	85%

Source: MRWA specification 501 (2017).

Table 9.6: Dry back requirements – basecourse (Table 501A3 MRWA Specification 501, 2017)

Basecourse material type	Maximum dry back characteristic moisture content (DMc) as a proportion of optimum moisture content
Basecourse (final surfacing – sprayed seal)	85%
Basecourse (final surfacing – asphalt)	70%
Crushed rock base (all surfacing types)	60%
Hydrated Cement Treated Crushed Rock Base (all surfacing types)	70% (2008 asphalt only)

Source: MRWA specification 501 (2017).

9.3 Conclusion on Specification Changes

Major changes have been made to the HCTCRB specifications over time. These changes were due to previous construction sections showing signs of fatigue or shrinkage cracking and bound basecourse behaviour. The reasons for each amendment is undocumented, but some recommendations were made by Harris and Lockwood (2009) regarding an increase in the minimum hydration period, and this change has been applied to later specifications.

The major changes included the reduction of the maximum UCS values after compaction at 7 days from 1 MPa maximum to 0.8 MPa maximum, and 28 days UCS results reduced from 1.5 MPa to 1 MPa maximum.

Furthermore, the minimum and maximum hydration periods required before compaction were increased from 7 days to 30 days minimum, and 28 days to 60 days maximum hydration period. Both these changes were made to decrease the chances that the material will behave as a bound layer with associated risk of cracking.

Due to an improvement of construction techniques and more accurate quality control techniques, construction tolerances can be reached easier and hence some of these tolerances have been adjusted to smaller variations (e.g. cement content tolerances).

Moisture content and density have a major influence on HCTCRB sections and hence changes to the hydration period, dry back requirements and density requirements have been made.

10 PERFORMANCE DATA

10.1 Introduction

A review of all historic deflection and curvature data, rutting data, and roughness data was done to enable a preliminary performance analysis of the sections of interest. This included collation of all available data of the trial sections since opening to traffic. This was used to identify unusual or unexpected trends, which may relate to events such as resurfacings or profile alterations, and any out-of-the-ordinary performance trends.

Network-level performance data such as rutting, roughness, deflection and skid resistance can be used to identify a road in need of rehabilitation and assist in making treatment decisions. These tests are all classified as non-destructive testing techniques and other destructive testing could be required to do a detailed project level investigation, such as trial pits.

It is important to carry out network-level testing consistently, i.e. at the same test point intervals and in the same season. Consistency in testing facilitates accurate comparison of the performance trends of results. However, through continuous improvements in testing methods and an increase in testing speeds, it can be difficult to compare data sets with historical data. Hence, proper care and caution should be used on these data sets.

10.2 Deflection and Curvature Data

To identify sections of a road that may require remedial actions, for example resurfacing, rejuvenation, patching or reconstruction, the maximum deflection values (D_0 (measurements taken at the load centre point)), as well as curvature data ($D_0 – D_{200}$ (D_0 less deflection values 200 mm away from the load centre)) are used to identify weak spots and poor performing sections. These measurements exhibit seasonal and pavement surface temperature variations and hence the raw data needs adjustment to allow a more accurate comparison between different survey dates and surface temperatures.

It is generally accepted that the critical deflection measurements are those of the outer lane’s outer wheel path, as the outer lanes are normally more heavily trafficked. Thus, for this investigation, if complete data sets are not available for both lanes, only the most heavily trafficked lane was investigated.

All FWD deflection and curvature data presented in this report are values obtained from a 50 kN load and normalised to a contact stress of 700 kPa and at a pavement temperature of 29 °C. It should be noted that no seasonal variation factors have been applied to the deflection results, because all measurements were taken between May to December in winter and spring rain areas with a correction factor of one according to Austroads (2011).

To compare Benkelman Beam (BB) data for Reid Highway with the post 2005 50 kN FWD measurements, Figure 10.2 was used to adjust the measured BB maximum deflections to estimated FWD under 50 kN load. This is done through the following process:

1. Using the thickness of the asphalt surfacing, the standardisation factor is read from Figure 10.2 to acquire the 40 kN FWD factor.
2. The 40 kN FWD maximum deflection was calculated by dividing the BB maximum deflection by the factor obtained in step 1.

3. The 50 kN FWD maximum deflection was estimated by multiplying the 40 kN FWD maximum deflection by 50/40.

Deflection measurements were reported as the average of the maximum D_0 (taken at the load centre point) deflections per section.

Curvature is simply the gradient of the deflection bowl and is calculated using Equation 2 below. In this report, the curvature results are standardised to a contact stress of 700 kPa (load 50 kN) and a pavement temperature of 29 °C.

$$Curv. = \boldsymbol{D_{0_{@700\,kPa}}} - \boldsymbol{D_{200_{@700\,kPa}}} \qquad (2)$$

where

$Curv.$	=	curvature of deflection bowl (mm)
D_0	=	deflection at centre of test corrected to a contact stress of 700 kPa and a pavement temperature of 29 °C, (mm)
D_{200}	=	deflection at 200 mm from centre of test corrected to a contact stress of 700 kPa and a pavement temperature of 29 °C, (mm)

Previous Benkelman Beam curvature data captured for the Reid Highway sections were adjusted to 50 kN FWD curvatures following the procedure below.

1. Use Figure 10.3 to obtain a curvature standardisation factor based on asphalt thickness.
2. Estimate the 40 kN FWD curvature by multiplying the measured BB curvature by the step 1 standardisation factor.
3. The 50 kN FWD curvature was obtained by multiplying the 40 kN FWD curvature by a factor of 50/40.

Figure 10.1 lists Austroads design deflections, which are based on maximum deflections measured using a Benkelman Beam (Austroads 2008). To obtain design deflections to compare to the measured FWD values under a 50 kN load, the Benkelman Beam design deflections in Figure 10.1 were adjusted as follows:

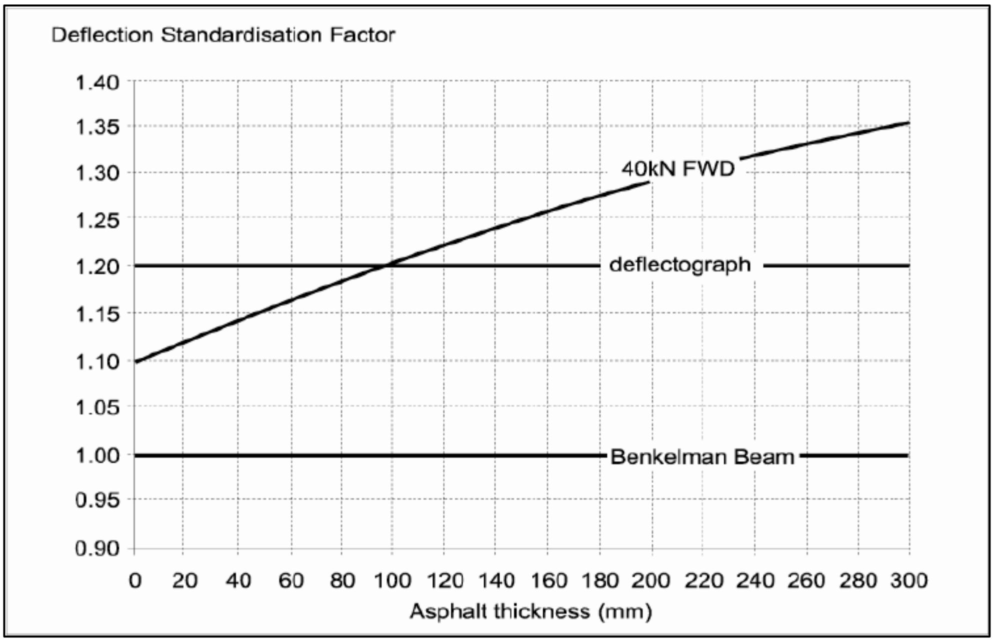
1. The maximum deflections under a 40 kN FWD load were estimated by dividing the BB maximum deflection by an adjustment factor determined from Figure 10.2, depending on the thickness of the asphalt surfacing.
2. The 50 kN FWD design deflections were then estimated by multiplying the 40 kN FWD design deflections (step 1) by 50/40.

This process only provides an estimate and should only be used where paired field measurements are not obtainable.

Figure 10.1: Benkelman Beam design deflections

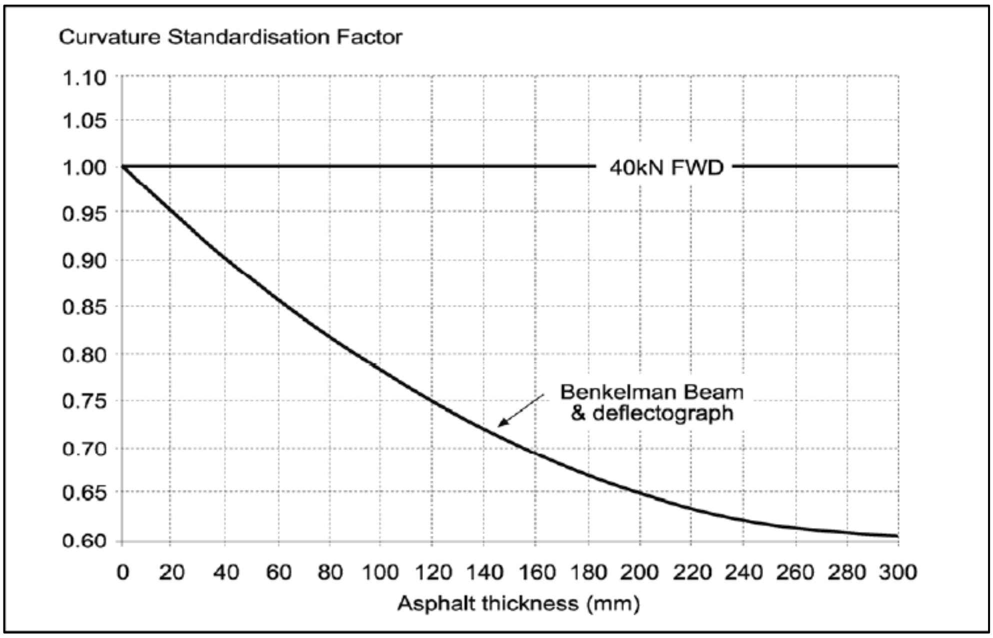
Source: Austroads (2011).

Figure 10.2: Deflection standardisation factors



Source: Austroads (2011).

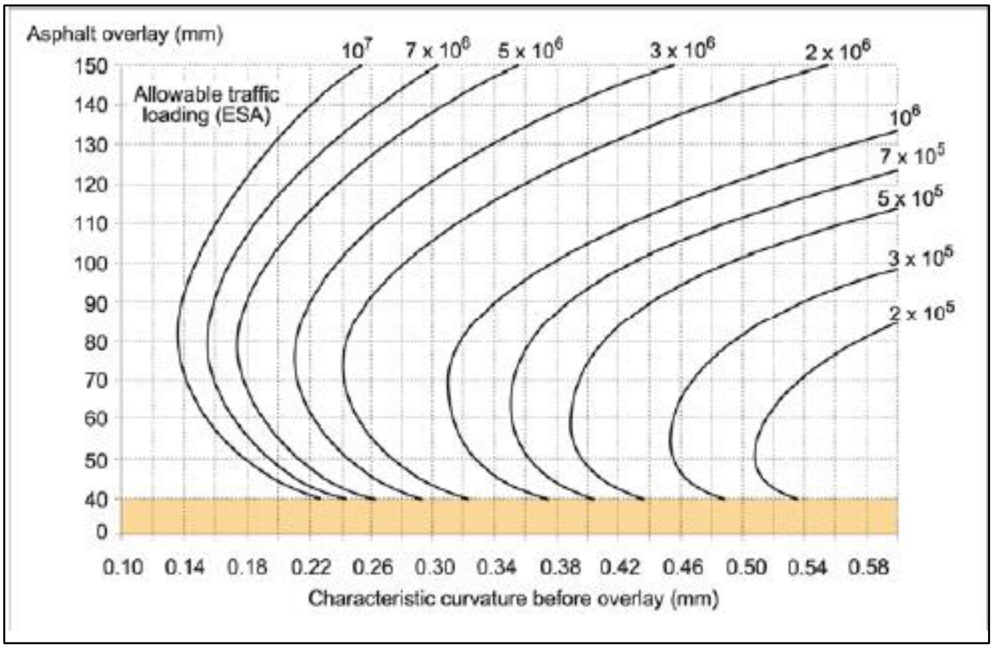
Figure 10.3: Curvature standardisation factors



Source: Austroads (2011).

Figure 10.4 is used to design the asphalt overlay thickness in terms of fatigue cracking of the asphalt overlay. This figure can be used as a guideline to assess the significance of measured curvature values.

Figure 10.4: Asphalt overlay design chart for asphalt fatigue



Source: Austroads (2011).

10.3 Roughness Data

Road roughness is a pavement condition parameter, which characterises deviations in a road surface from the intended longitudinal profile. It is widely used to rate the road condition at a network level because of its effect on vehicle dynamics and hence, vehicle operating costs, driver comfort and dynamic pavement loading (Austroads 2012).

Currently Austroads (2007a) prefers roughness being reported in Lane IRIqc (International roughness Index) values. Based on the importance of a road, as well as the travelled speed, certain roughness limits have been set to flag a pavement that requires remedial action. These values are indicated in Table 10.1 below.

Table 10.1: Levels of roughness (after Austroads 2003)

Road function	Typical maximum desirable roughness (IRI) for new construction or rehabilitation (length 500 m) (m/km)	Indicative investigation levels for roughness, IRI (m/km)	
		Isolated areas	Length > 500 m
Freeways and other high-class facilities	1.6	4.2	3.5
Highways and main roads (100 km/h)	1.9	5.3	4.2
Highways and main roads (< 80 km/h	1.9	6.1	5.3
Other local sealed roads	No limits defined	No limits defined	No limits defined

Notes:
- Lower values may be appropriate where total traffic of heavy vehicle volumes is high.
- Roughness levels depend on local conditions and traffic calming measures.

Source: Austroads (2003).

10.4 Rutting Data

Rutting is referred to as a longitudinal depression in a pavement wheel path and is measured in the transverse road profile direction. For this report and due to its international acceptance, data measured with a 2 m straight edge either manually or by an automated measurement is reported. Austroads favour the 2 m straight edge, because vehicles can wander up top 1.1 m and the rut widths are commonly wider than 1.2 m (Austroads 2011). High rut values can affect safety and cause water ponding and influence skid resistance.

Rutting can be caused by an ingress of water into the pavements through cracks and from the shoulders, through inadequate pavement thickness and inadequate material quality, as well as poor construction techniques.

The use of rutting data to predict pavement performance could be used to relate to road safety, to report on structural adequacy as well as to trigger possible remedial actions. Rutting primarily relates to repeated loading by vehicles particularly on pavement wheel paths and can be as a result of further compaction, loss of surface material or shear displacement of pavement layers.

Generally, but depending on the importance of the road, a 10 mm rut is regarded as exhibiting a potentially significant safety concern. Further rut limits are defined in Austroads (2007b). Between 20 mm and 25 mm (as measured with a 2 m straight edge), rutting becomes a major structural problem and investigation is required (Austroads 2007b).

11 REID HIGHWAY PERFORMANCE

11.1 Description of HCTCRB Trial Sections

The performance of three Reid Highway trial sections was investigated, these being located between West Swan Road and Bennett Brook Bridge in West Swan. Section 1 is located east of Lord Street, whilst Sections 5 and 6 are located west of Lord Street. The test sections include a single lane in both the eastbound and westbound directions with a posted speed limit of 90 km/h. The start and end chainages and corresponding lengths of each trial section are shown in Table 11.1. The granular pavements materials used are also shown.

Table 11.1: Reid Highway trial sections data

Trial section	Start chainage (m)	End chainage (m)	Length (m)	Granular pavement materials		
				Base	Subbase	Subgrade
1	19 890 (11 540)	19 980 (11 630)	90	2% HCTCRB	Crushed limestone	White sand
5	18 790 (10 490)	18 880 (10 580)	90	1% HCTCRB		
6	18 700 (10 400)	18 790 (10 490)	90	2% HCTCRB		

Source: Butkus (2004).

Construction of the trial sections was undertaken between October 1995 and April 1996, opening for traffic in December 1996 (Butkus 2004). All construction and laboratory details are included in a series of internal technical Main Roads reports (Butkus 2004). These reports collectively include details such as section chainage, design material profiles, as-constructed material profiles, and basic laboratory analysis of base, subbase and subgrade materials, construction quality assurance (QA) test results and after-construction monitoring data, including Benkelman Beam deflection and curvature data. The chainages shown in brackets in Table 11.1 above, were initial chainages before the referencing links changed.

Table 11.2 summarises the initial design pavement layer thicknesses and as-constructed mean thicknesses. This indicates that Section 1 has a thin HCTCRB thickness of only 100 mm and hence, has a crushed limestone subbase thickness almost twice that of Section 5 and 6.

Construction of Sections 5 and 6 was done by placing the pavement layers by grader and compacting with a 10 tonne (t) vibrating drum roller followed by a 15 t pneumatic roller. The HCTCRB was dried-back, primed, double sealed with a 10 mm and 5 mm seal and overlayed with 30 mm thickness of size 10 mm dense graded asphalt.

Asphalt thicknesses are not individually specified in the original reports, as initial surfacing was simply a two-coat primerseal. However, the test sections were subsequently surfaced with a nominal 30 mm thickness of size 10 mm dense graded asphalt (DGA) after opening to traffic in December 1996 for shape correction. The actual asphalt thickness was noted to vary between 44 mm and 65 mm, based on the findings from three test pits excavated in late 2016.

Table 11.2: Reid Highway trial sections material profiles

Trial ID	Material purpose	Material	Design thickness (mm)	As-constructed mean thickness (mm)
1	Base course	2% HCTCRB	100	123
	Subbase	Crushed limestone	230	291
	Subgrade	White sand		–
5	Base course	1% HCTCRB	200	210
	Subbase	Crushed limestone	130	150
	Subgrade	White sand	–	–
6	Base course	2% HCTCRB	200	211
	Subbase	Crushed limestone	130	143
	Subgrade	White sand	–	–

Source: Butkus (2004).

11.2 Material Properties

Material testing of the granular materials was done before construction and is presented in Table 11.3 below.

Table 11.3: Mean values of laboratory testing results – granular materials Reid Highway

Trial section	Material purpose	Material	Resilient modulus (MPa)(1)	Mean MDD (t/m^3)	Mean OMC (%)	Basecourse MC – % of OMC (2)
1	Base course	2% HCTCRB	250	2.24	6.4	70
1,5,6	Subbase	Crushed limestone	185	1.94	9.8	
1,5,6	Subgrade	White sand	–	1.81	12.5	
5	Base course	1% HCTCRB	NA	2.274	6.1	70
6	Base course	2% HCTCRB		2.24	6.4	80

1 RLT testing undertaken on specimens compacted using the static compaction method.
2 Average % basecourse in situ moisture content as tested from 1996 to 2003.

Source: Butkus (2004).

During the initial study carried out by Butkus and Lee Goh (1997), the modulus specimens were prepared using static compaction and not dynamic compaction as recently used. This resulted in an almost 50% reduction in modulus values compared to current dynamic compaction (modified Proctor drop hammer). Hence, care should be taken using these measured moduli in mechanistic design. The modulus results in Table 11.3 are those measured after approximately 3000 cycles of loading at a vertical stress of 300 kPa and a confining stress of 50 kPa. The moduli reported above should only be used to rank the materials in terms of relative modulus and not for mechanistic design.

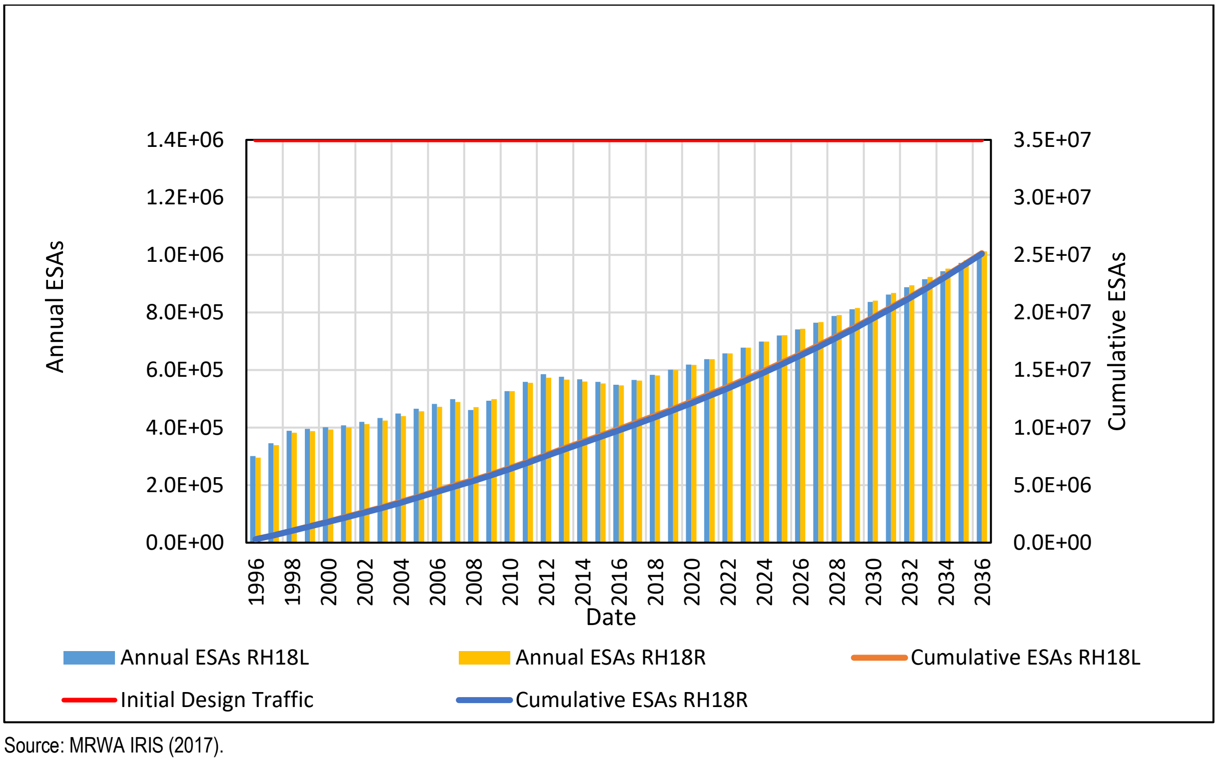
11.3 Cumulative Traffic Loading

The original design traffic loading for Reid Highway trial sections was estimated to be 3.5 x 10^7 equivalent standard axles (ESAs) over a design period of 40 years (Butkus 2004).

Reid Highway traffic data from 1991 to 2016 was extracted from the Main Roads Integrated Road Information System (IRIS) database on 4 August 2016. Trial Section 1 falls within the Reid Highway traffic section designated RH18L and RH18R. Trial Sections 5 and 6 fall within the section designated RH17L and RH17R, respectively. Actual counts were captured for 10 years for RH18L & R and 9 years for RH17 between 1996 and 2016. The average traffic growth was calculated from this data and extrapolated for the rest of the design period. The annual growth rate calculated was 3% for section 1. Sections 5 and 6 had an average growth rate of 3.7%.

The cumulative ESAs from 1996 to 2016 were estimated using the Main Roads supplied macro-enabled spreadsheet, in conjunction with the most up-to-date IRIS traffic counts data. Figure 11.1 presents both the yearly ESAs and the cumulative ESAs for both the RH18L (eastbound) and RH18R (westbound) traffic sections for a design life of 40 years up to 2036.

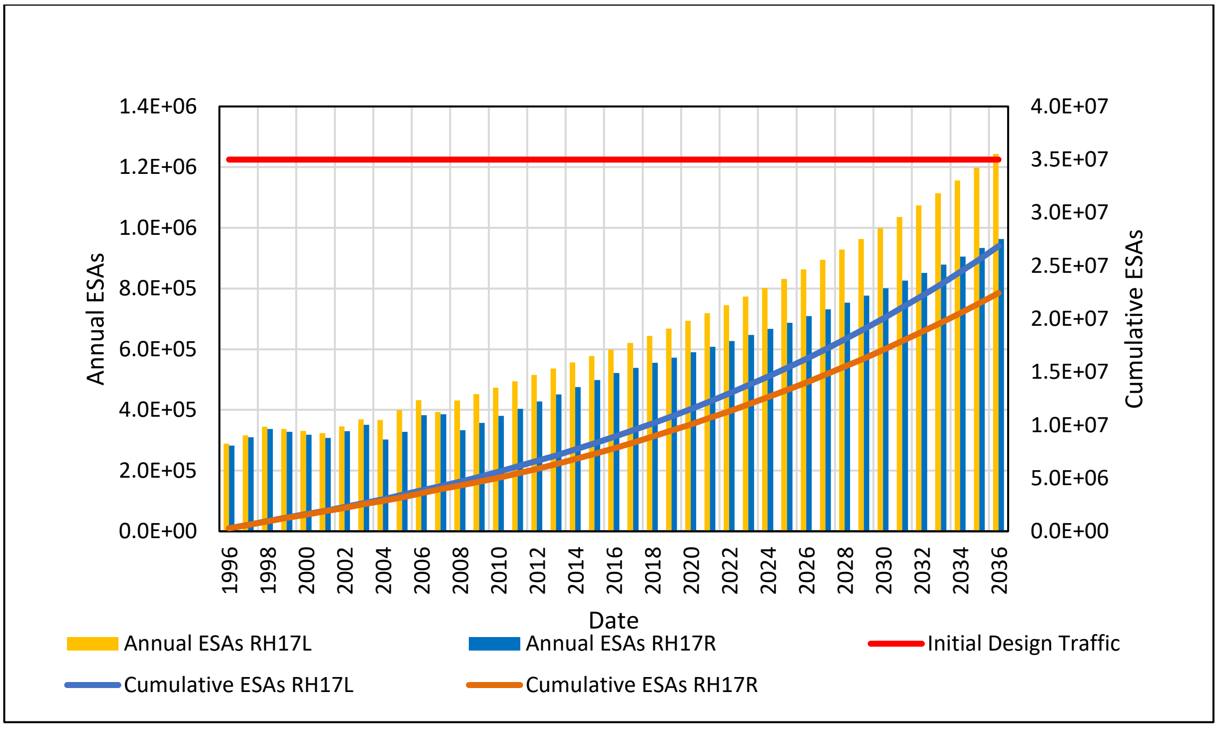
Figure 11.1: Annual ESAs and cumulative ESAs for Section 1 Reid Highway



Source: MRWA IRIS (2017).

Figure 11.2 presents both the annual ESAs and the cumulative ESAs for both the RH17L (eastbound) and RH17R (westbound) traffic sections for the same period.

Figure 11.2: Annual ESAs and cumulative ESAs for Sections 5 & 6 Reid Highway



Source: MRWA IRIS (2017).

From this traffic data, the estimated 40-year design traffic loadings were 2.5 x 10^7 ESAs for Section 1, and 2.7 x 10^7 ESAs for Sections 5 and 6.

11.4 Pavement Maintenance

Pavement data for Reid Highway was extracted from IRIS on 26 June 2016. This data indicates Sections 4 to 7 were rehabilitated in 2014 as part of the upgrade and widening of the Reid Highway and Lord Street intersection. This rehabilitation included the removal of the existing asphalt surfacing in addition to approximately 100 mm of the existing granular base course, to enable the installation of a full depth asphalt pavement. As such, Sections 4 to 7 have been strengthened and only data prior to 2014 has been considered in this study. The remaining sections have not been altered since construction.

11.5 Maximum Deflections

11.5.1 Design Deflection

The design deflection for Reid Highway based on BB data is recorded as 0.85 mm based on the design traffic of 3.5 x 10^7(Butkus 2004). To compare to the measured FWD deflections, this maximum design deflection should be converted to equivalent 50 kN FWD deflection. The conversion was calculated as follows:

- Read off the standardisation factor from Figure 10.2as 1.165 for 60 mm asphalt.
- Divide the design BB maximum deflection of 0.85 by this factor of 1.165 to calculate the equivalent 40 kN FWD value.
- Multiply by a factor of 50/40 to get the equivalent 50 kN FWD maximum deflection of 0.912 mm.

11.5.2 Results

A review of available deflection data from Benkelman beam testing and FWD testing was undertaken for the Reid Highway Sections 1, 5 and 6. Pre-2006, the only deflection data available for the Reid Highway sections was that obtained via Benkelman Beam (BB) testing. Post-2006, FWD testing became the test of choice. BB data was obtained under an 80 kN single axle with dual tyres inflated to 550 kPa. Performance data for Sections 5 and 6 of the Reid Highway collected after 2014 has not been included in the performance revision due to the rehabilitation of the pavement.

The details of the deflection tests carried out for Reid Highway are presented in Table 11.4.

Table 11.4: Reid Highway details of deflection data sources

Test data source	Test date	Test method
Reid Highway Basecourse Test Sections: Construction Details and Performance to November 2003 (Butkus 2004)	10 test data sets between June 1996 and November 2003	BB
Reid Highway Test Sections Report No. 06 FWD 46/1 and 51/1 (MRWA 2006)	November 2006 eastbound and westbound lanes	FWD
Reid Highway Trial Sections Report No. D00001A (Western Geotechnics Group 2007)	March 2007 eastbound lanes	
Reid Highway Test Sections Report No. 07 FWD 79/1 (MRWA 2007)	March 2007 westbound lanes	
Reid Highway Test Sections Report No. 08 FWD 190/1 (MRWA 2008)	December 2008 eastbound and westbound lanes	
Reid Highway Test Sections Report No. 11 FWD 337/1 (MRWA 2011)	November 2011 eastbound and westbound lanes	
Reid Highway Test Sections Report No. 16 FWD 491/3 (MRWA 2016)	June 2016 eastbound and westbound lanes	

Table 11.5 summarises the Reid Highway BB maximum deflection data for the three sections of interest. The deflections are the mean of the combined eastbound and westbound lane maximum values. The Reid Highway deflection data prior to October 2001 is quite variable, but shows a decreasing trend, which infers the pavement materials gaining modulus. The yearly traffic for this period is steady and only starts to increase significantly after 2007. The deflection after this time starts to increase, which is expected as the pavement ages and the traffic load increases.

Due to large variations between inner and outer wheel paths, as well as different survey dates, BB data was not used to estimate performance trends for Reid Highway.

Table 11.5: Reid Highway BB mean maximum deflection summary

Section ID	Base course material	Jun-96	Oct-96	Maximum deflection (mm)							
				Dec-96	Jun-97	Nov-97	Nov-98	Dec-99	Sep-00	Oct- 01	Nov-03
1	HCTCRB	0.392	0.559	0.432	0.319	0.486	0.373	0.455	0.557	0.518	0.465
1	FWD Equivalent (2)	0.421	0.600	0.464	0.342	0.521	0.400	0.488	0.598	0.556	0.499
Yearly ESAs (x 10^5) (1)				3.0	3.4		3.9	3.9	4.0	4.0	4.3
5	HCTCRB	0.509	0.563	0.394	0.353	0.444	0.468	0.384	0.457	0.428	0.462

Section ID	Base course material	Jun-96	Oct-96	Maximum deflection (mm)							
				Dec-96	Jun-97	Nov-97	Nov-98	Dec-99	Sep-00	Oct- 01	Nov-03
5	FWD Equivalent (2)	0.546	0.604	0.423	0.379	0.476	0.502	0.412	0.490	0.459	0.496
6	HCTCRB	0.393	0.415	0.323	0.242	0.286	0.290	0.222	0.325	0.277	0.259
6	FWD Equivalent (2)	0.422	0.445	0.347	0.260	0.307	0.311	0.238	0.349	0.297	0.278
Yearly ESAs (x 10^5)(1)				2.9	3.2		3.4	3.4	3.3	3.2	3.6

1 Yearly ESA values represent the average of both westbound and eastbound traffic.
2 FWD Equivalent values estimated by using Figure 10.2for 50 kN load.
Source: Butkus (2004) & MRWA IRIS (2017) for traffic.

11.5.3 *Trends and Findings*

Figure 11.3 presents all the FWD deflection data in the westbound direction. Section 1 and Section 5 have similar deflections, these values being higher than for Section 6. It should be noted that Section 5 has only 1% cement and Section 1 only has a 123 mm thickness of HCTCRB. This is most probably the cause of higher deflection values when compared to Section 6, which contains 2% cement and is 211 mm thick.

Figure 11.3: Reid Highway FWD maximum deflection data – FWD data only westbound

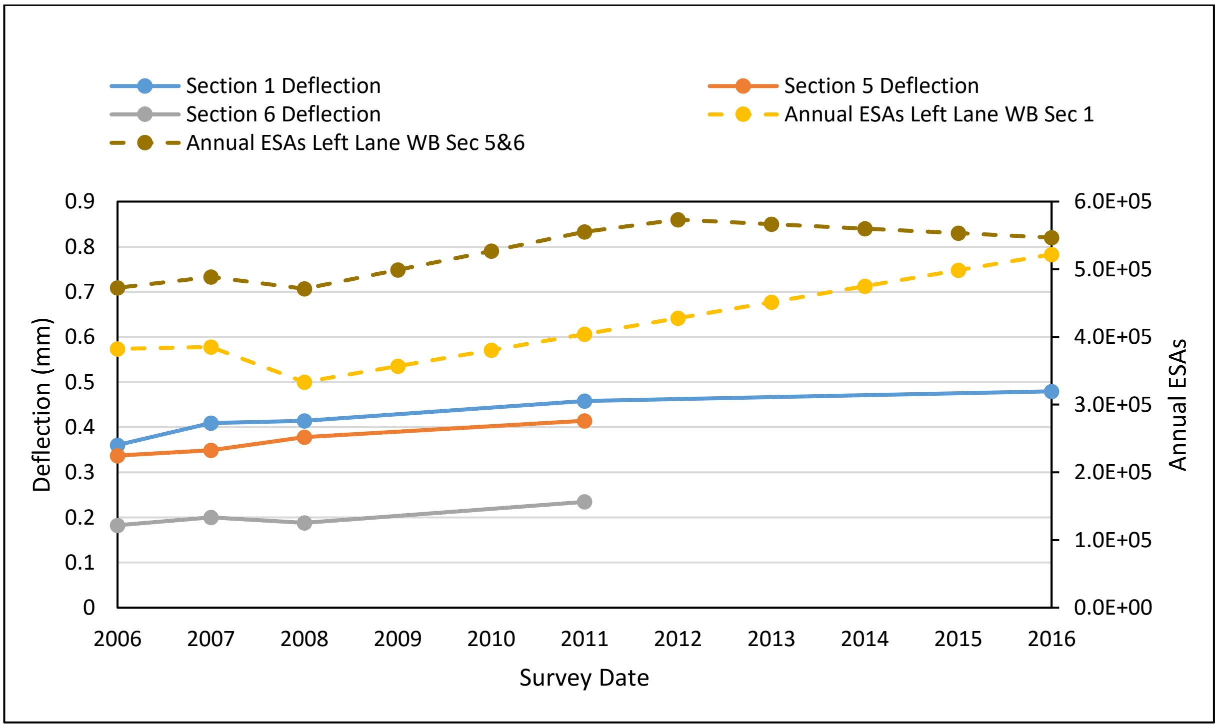
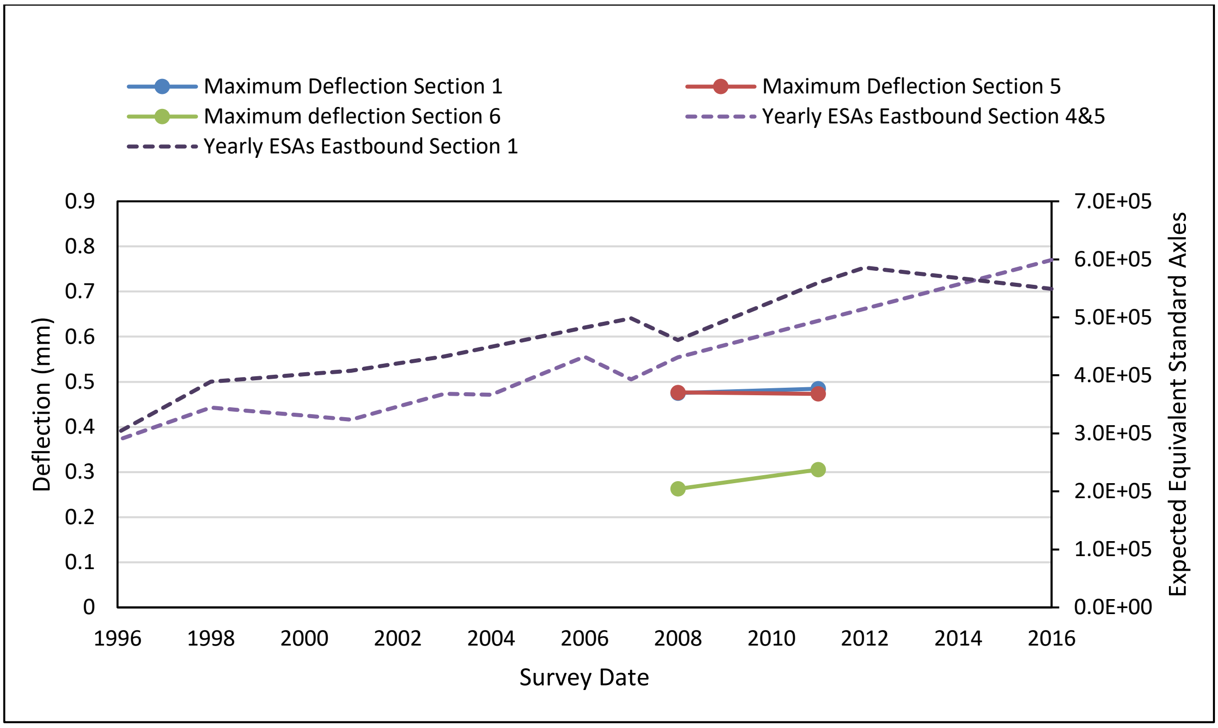


Figure 11.4 depicts the maximum deflection for the eastbound direction are similar to that of the westbound direction of Figure 11.3. Only two FWD data sets are available and hence deterioration trends cannot be estimated.

Figure 11.4: Reid Highway maximum deflection data – FWD data only eastbound



Overall, for the analysis period so far, the deflection has increased with time, although is still well below the design deflection limit of 0.912 mm. The deflection values from the most recent test (June 2016) have not exceeded the initial readings taken in December 1996 for all sections. This is evidence that the trial pavements have gained modulus over the service life.

11.6 Curvature

11.6.1 Results

Table 11.6 summarises the Reid Highway curvature data for the three sections of interest calculated from the Benkelman Beam data from 1996 to 2003. Due to inconsistent measurements, in both inner and outer wheel paths and survey dates, Benkelman Beam data will not be used to estimate performance trends for Reid Highway.

Table 11.6: Reid Highway corrected BB curvature summary

Section ID	Base course material			Curvature (mm)							
		Jun-96	Oct-96	Dec-96	Jun-97	Nov-97	Nov-98	Dec-99	Sep-00	Oct-01	Nov-03
1	HCTCRB	0.052	0.065	0.046	0.033	0.052	0.045	0.065	0.060	0.063	0.092
		0.076	0.094	0.067	0.048	0.076	0.065	0.094	0.087	0.092	0.134
Yearly ESAs (x 10^5)(1)				3.0	3.4		3.9	3.9	4.0	4.0	4.3
5	HCTCRB	0.091	0.089	0.042	0.034	0.041	0.035	0.042	0.056	0.048	0.070
		0.132	0.129	0.061	0.049	0.060	0.051	0.061	0.081	0.070	0.102

Section ID	Base course material	Jun-96	Oct-96	Curvature (mm)							
				Dec-96	Jun-97	Nov-97	Nov-98	Dec-99	Sep-00	Oct-01	Nov-03
6	HCTCRB	0.052	0.037	0.024	0.016	0.019	0.010	0.017	0.020	0.013	0.015
		0.076	0.054	0.035	0.023	0.028	0.015	0.025	0.029	0.019	0.022
Yearly ESAs (x 10^5)(1)				2.9	3.2		3.4	3.4	3.3	3.2	3.6

1 Yearly ESA values represent the average of both westbound and eastbound traffic.

Figure 11.5: Reid Highway curvature data – FWD data only westbound

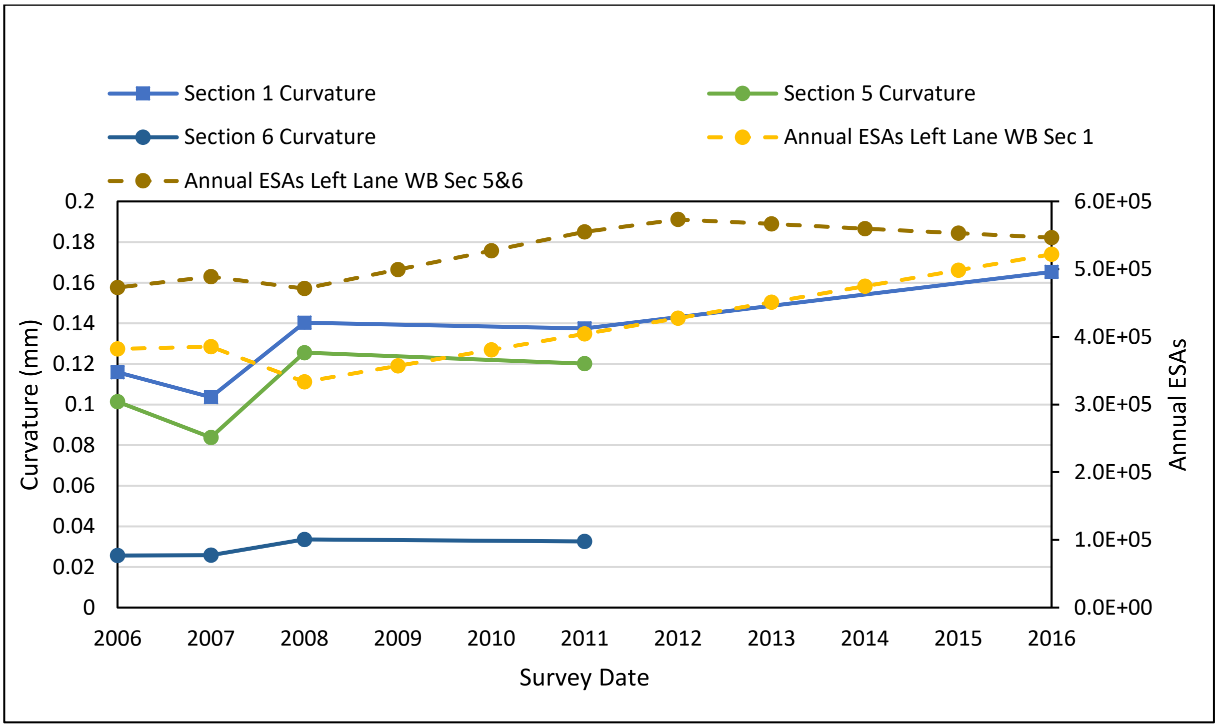
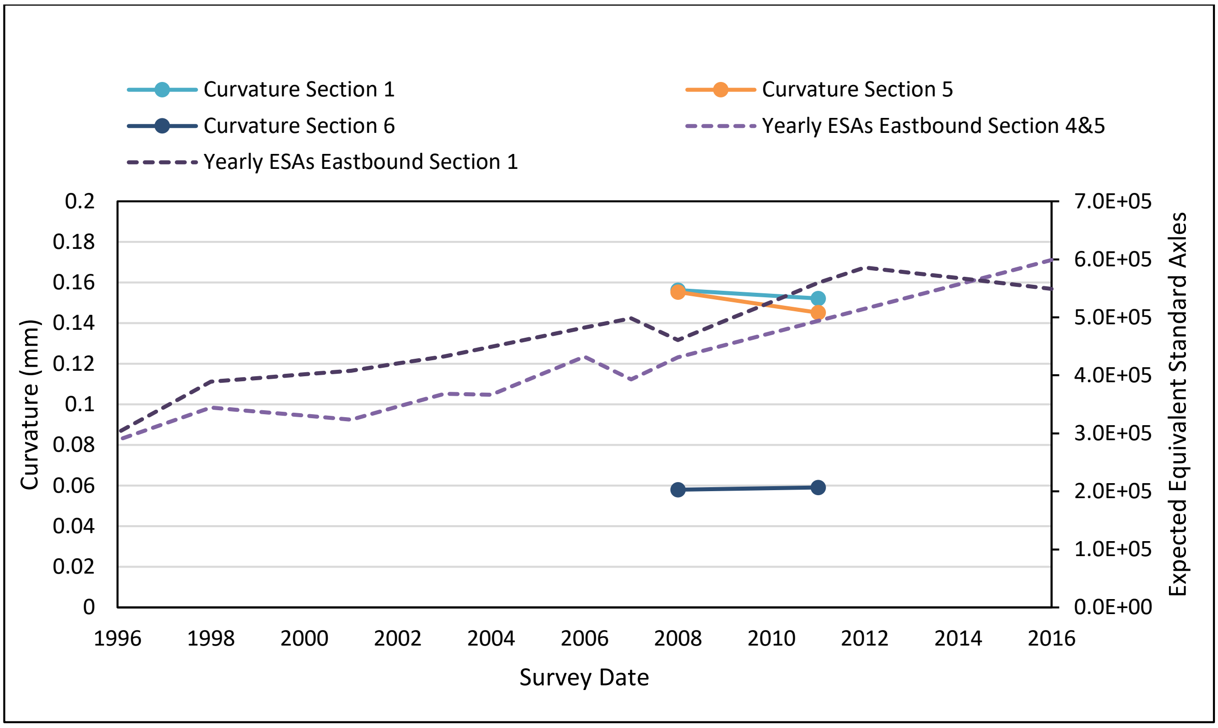


Figure 11.6: Reid Highway curvature data – FWD data only eastbound



11.6.2 Trends and Findings

The current curvature value of Section 1 in Figure 11.5 was 0.165 mm in 2016. This is the highest curvature value of all three sections. This relatively high value could mean that the HCTCRB is fatigue cracking or the asphalt layer is deteriorating. It should be noted that the section is already 20 years old and that dense graded asphalt layers are generally designed to last 15 years.

Section 1 and Section 5 in both Figure 11.5 and Figure 11.6 show higher curvature values than that of Section 6. This could be due to a thinner base thickness for Section 1 and a lower cement content for Section 5.

11.7 Roughness

11.7.1 Limits

The roughness intervention limits are defined in Table 10.1. For Reid Highway, typical new pavements should have roughness values not exceeding 1.6 m/km. The investigation limits are 4.2 m/km for isolated areas or 3.5 m/km average for sections of more than 500 m.

11.7.2 Results

The roughness progression data is presented in Figure 11.7 and Figure 11.8 below as lane IRIqc in metre per kilometre. For both the westbound and eastbound lanes, the roughness has deteriorated for Section 1, though only marginally. This is most likely due to a thin basecourse layer thickness of only 123 mm as shown in Table 11.2. The results show stable roughness values after the December 2006 survey.

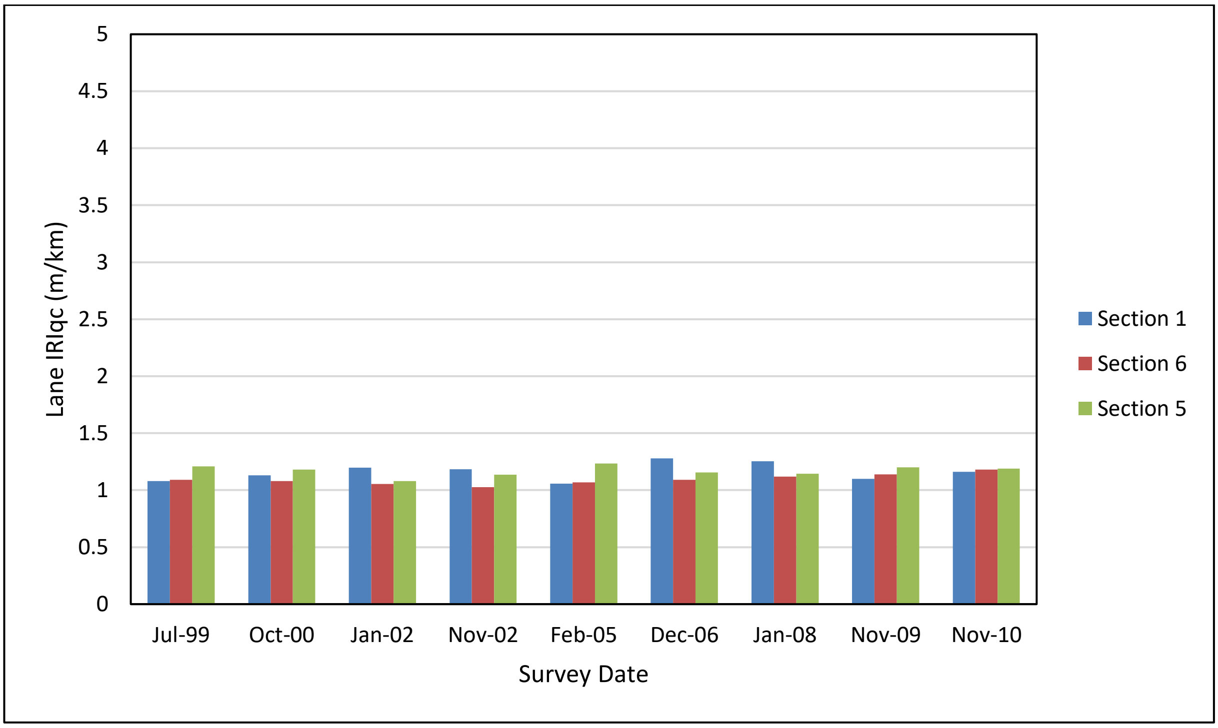
For Section 5, the roughness data also indicates an initial deterioration from December 1999 to December 2006 in the eastbound direction, but then stabilise after 2006. These values do not yet warrant any remedial action or maintenance action and still represent a pavement in a good condition.

The westbound lane roughness progression is slow with almost no deterioration throughout the 11 years testing cycle. The results show a good road condition based on the riding quality, with values below 1.2 IRIqc for all three sections in 2010. No linear relationship can be derived from this data and hence it is not possible to predict when the pavement sections will reach the investigation values.

It is important to note that these values represent average roughness values over 100 m. The test sections are only 90 m long and hence these roughness values include a 10 m section of another trial section.

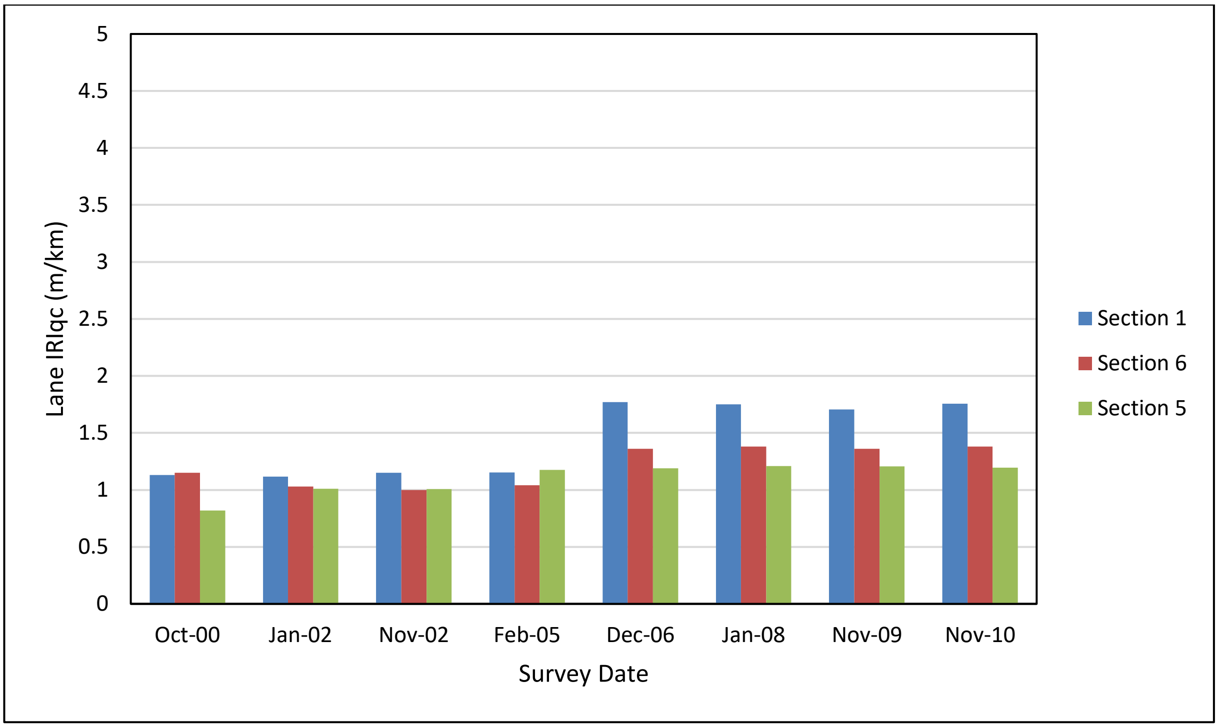
There is no indication from the roughness results that the sections will need to be corrected for poor riding quality soon.

Figure 11.7: Reid Highway roughness progression data westbound lane



Source: MRWA IRIS (2017).

Figure 11.8: Reid Highway roughness progression data eastbound lane



Source: MRWA IRIS (2017).

11.8 Rutting

11.8.1 Evaluation Limits

Generally, a 10 mm rut is regarded as exhibiting a potentially significant safety concern. Between 20 mm and 25 mm (as measured with a 2 m straight edge), rutting becomes a major structural problem and investigation is required (Austroads 2007b).

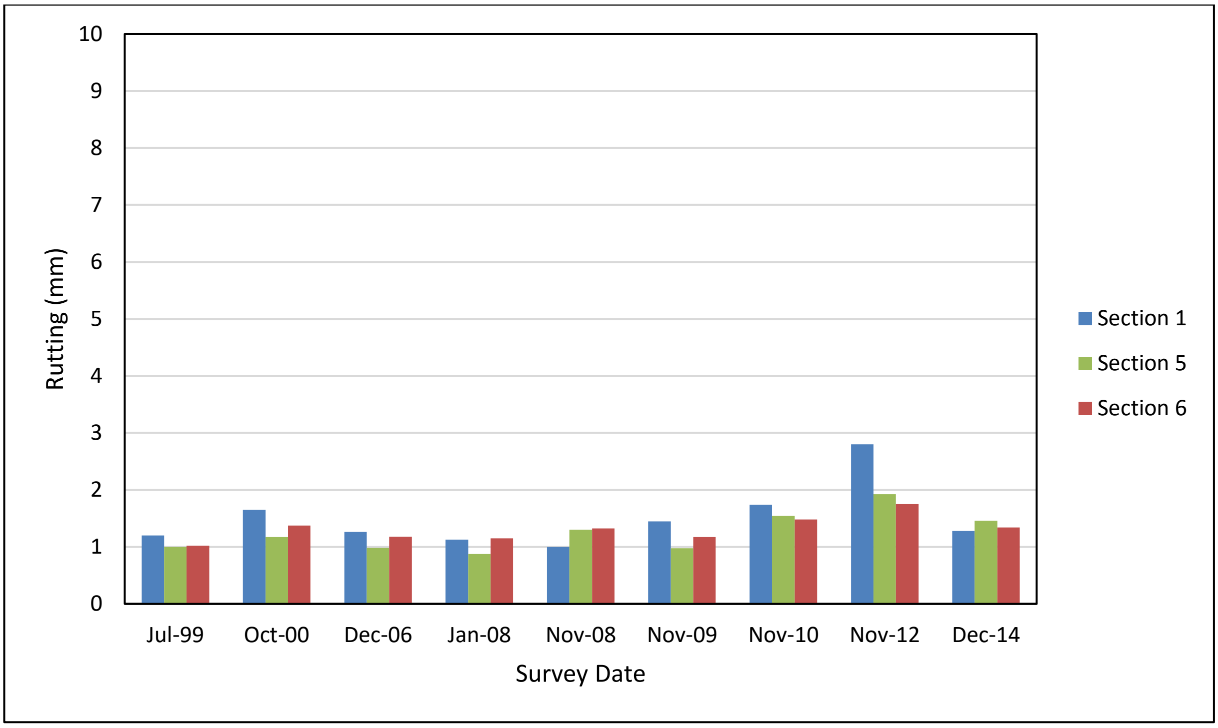
11.8.2 Results

Average rut depths are displayed in Figure 11.9, Figure 11.10, Figure 11.11 and Figure 11.12 for both wheel paths and in both directions. For all three sections of Reid Highway and for both lanes and all wheel paths, rut measurement results are still relatively low with a maximum of 4.5 mm rut in Section 5.

It should be noted that the Reid Highway sections are only 90 m long. Hence, the rut data presented in the figures below are all the 80 m average values because rutting was generally recorded every 20 m.

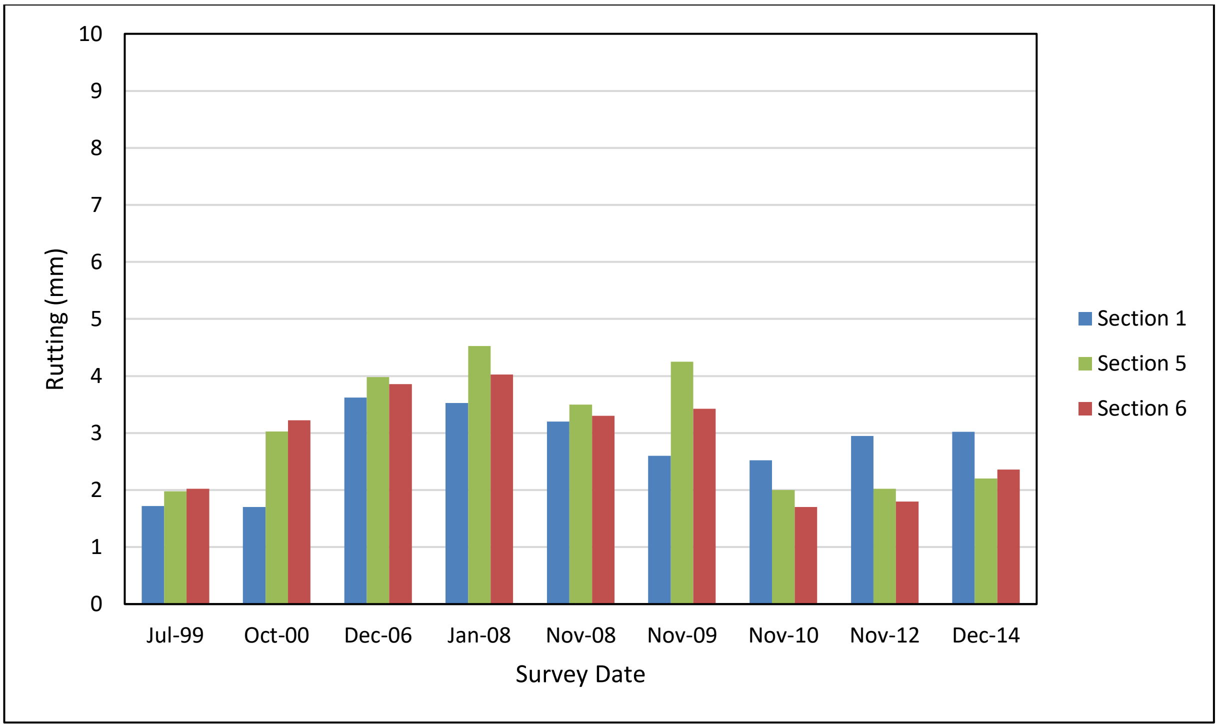
From the data in the figures below, no performance trend can be established, because there is no consistent evidence that any particular section performed worse than any other.

Figure 11.9: Rut data Reid Highway left lane outer wheel path – 2m straight edge



Source: MRWA IRIS (2017).

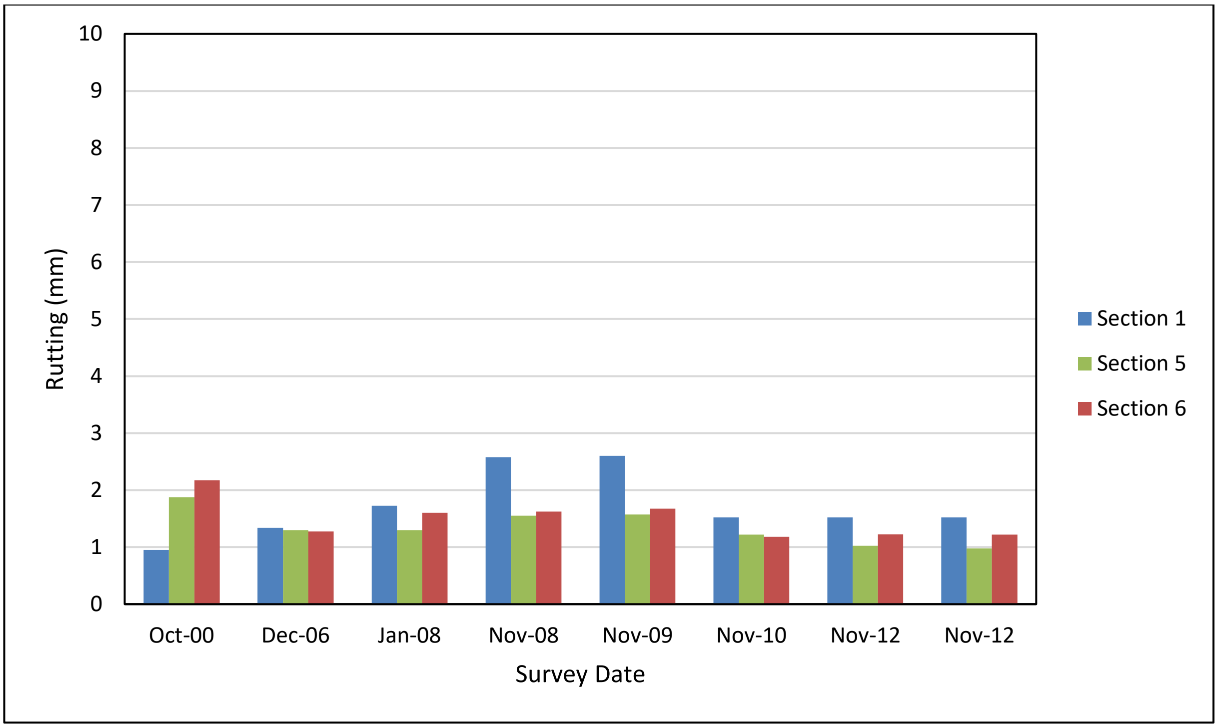
Figure 11.10: Rut data Reid Highway left lane inner wheel path – 2m straight edge



Source: MRWA IRIS (2017).

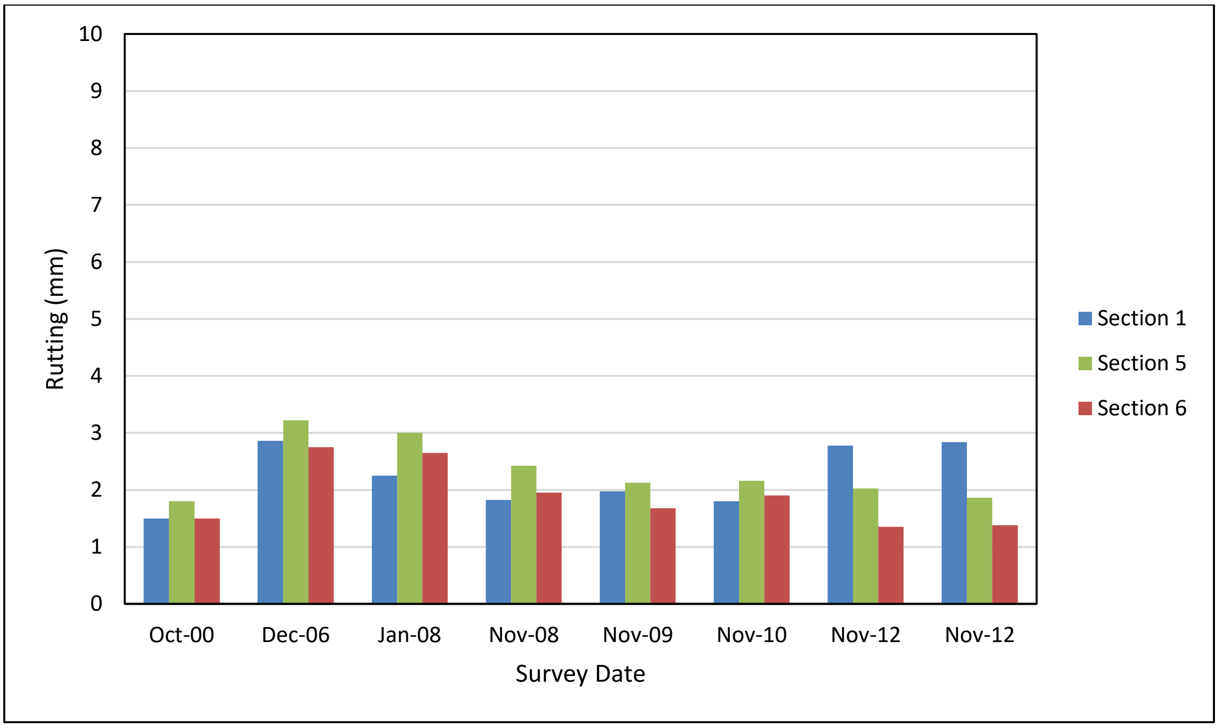
The rut data in Figure 11.10 above is the only data set with questionable data for the period of October 2000 to November 2009 with rut depth values decreasing by 1 mm or 2 mm. If the overall section performance is considered, as well as the more recent performance from 2010 to 2014 for the other lanes and wheel paths, it could be surmised that the current measurements are accurate and that some technical, operator or other inconsistency could have led to the higher rut measurements.

Figure 11.11: Rut data Reid Highway right lane outer wheel path – 2 m straight edge



Source: MRWA IRIS (2017).

Figure 11.12: Rut data Reid Highway right lane inner wheel path – 2 m straight edge



Source: MRWA IRIS (2017).

11.9 Summary

Reid Highway trial sections have been performing well since its construction in 1996. No structural or surfacing defects are evident or have been reported in the past. No accurate deterioration trends can be identified from the rutting and roughness data. For the deflection and curvature values, the results do not indicate any significant concerns based on pavement performance and strength apart from Section 1 westbound where the curvatures appear to be increasing.

12 KWINANA FREEWAY PERFORMANCE

12.1 Description of HCTCRB Sections Investigated

The Kwinana Freeway extension runs between Paganoni Road and Stock Road in the City of Rockingham. The Southern Gateway Alliance (SGA) on this section of Kwinana Freeway constructed altogether seven sections of HCTCRB trials in 2009. Their location, length, as well as constructed pavement materials are presented in Table 12.1. All sections are located on the southbound carriageway with a posted speed limit of 100 km/h.

Table 12.1: Kwinana Freeway trial sections data

Trial section	Start chainage (m)	End chainage (m)	Length (m)	Granular pavement materials			Surfacing
				Base	Subbase	Subgrade	
1	55 180	55 280	100	2% HCTCRB	Crushed limestone	White sand	Open graded asphalt over dense graded asphalt or stone mastic asphalt
6	55 680	55 780	100	2% HCTCRB	Crushed limestone	Yellow sand	
7	55 780	55 880	100				
8	55 880	55 980	100				
9	55 980	56 080	100				
10	56 080	56 180	100				
14	56 480	56 580	100				

Source: Rehman (2012).

Construction took place between May 2009 and August 2009, and the road was opened to traffic in September 2009 (Rehman 2012). MRWA report No. 2010 -13M contains construction and performance details, such as the materials acceptance testing, construction compliance testing, bituminous products testing as well as post construction monitoring and performance results until October 2011. The HCTCRB was placed and shaped using a grader as stated in the aforementioned report.

All sections, except Section 6, were sealed with a 10 mm/5 mm bitumen emulsion primerseal before overlaying with an asphalt surfacing layer. Section 6 was sealed with a 10 mm geotextile reinforced seal over the 10 mm/5 mm bitumen emulsion primerseal and overlaid with 30 mm dense graded asphalt (DGA) under a 30 mm OGA section asphalt. Section 7 was designed with a 30 mm thick, 7 mm nominal stone size stone mastic asphalt (SMA) beneath a 30 mm open graded asphalt (OGA) layer over the primerseal. All other sections were designed with 30 mm dense graded asphalt (DGA) under a 30 mm OGA section.

Actual constructed total asphalt thicknesses ranged between 60 mm and 72 mm, which includes the base seals. Table 12.2 summarise the constructed thicknesses.

Table 12.2: Kwinana Freeway trial sections material profiles

Trial ID	Material purpose	Material	Design thickness (mm)	As-constructed mean thickness (mm)	As-constructed asphalt mean thickness (mm)
1	Base course	2% HCTCRB	180	223	36 DGA 29 OGA
	Subbase	Crushed limestone	200	213	
	Subgrade	White sand	–	–	
6	Base course	2% HCTCRB	230	255	33 DGA 27 OGA
	Subbase	Crushed limestone	150	140	
	Subgrade	Yellow sand	–	–	
7	Base course	2% HCTCRB	230	255	30 SMA 27 OGA
	Subbase	Crushed limestone	150	160	
	Subgrade	Yellow sand	–	–	
8	Base course	2% HCTCRB	230	255	31 DGA 27 OGA
	Subbase	Crushed limestone	150	145	
	Subgrade	Yellow sand	–	–	
9	Base course	2% HCTCRB	230	290	31 DGA 27 OGA
	Subbase	Crushed limestone	150	125	
	Subgrade	Yellow sand	–	–	
10	Base course	2% HCTCRB	230	265	36 DGA 28 OGA
	Subbase	Crushed limestone	150	155	
	Subgrade	Yellow sand	–	–	
14	Base course	2% HCTCRB	180	220	36 DGA 28 OGA
	Subbase	Crushed limestone	200	205	
	Subgrade	Yellow sand	–	–	

Source: Rehman (2012).

12.2 Material Properties

MRWA Materials Engineering Branch (MEB) undertook material testing after 28 days of curing. Table 12.3 presents the modulus, mean MDD and mean OMC of all sections. Note that the modulus tests were done on only two batches. One batch for Sections 1, 6, 7 and 8 and the second for Sections 9,10 and 14.

Table 12.3: Laboratory testing results of Kwinana Freeway materials

Trial section	Material purpose	Material	Resilient modulus (MPa)(2)	Mean MDD (t/m^3)	Mean OMC (%)
1	Base course	2% HCTCRB (Cemex/Readymix) Gosnells	682, 926,1192	2.208(1)	6.5(1)
	Subbase	Crushed limestone (WA Quarry 9)	–	1.933(1)	11.5(1)
	Subgrade	White sand	–	1.677	14.2(1)
6,7,8	Base course	2% HCTCRB (Cemex/Readymix) Gosnells	682, 926,1192	2.161(1)	7.1(1)
	Subbase	Crushed limestone (WA Quarry 9)	–	1.88(1)	12.1(1)
	Subgrade	Yellow Sand	–	1.726	12.7(1)
9	Base course	2% HCTCRB	832, 896, 1023	2.166(1)	8.2(1)
	Subbase	Crushed limestone	–	1.88(1)	12.1(1)
	Subgrade	Yellow sand	–		14.7(1)
10	Base course	2% HCTCRB	832, 896, 1023	2.165(1)	8.2(1)
	Subbase	Crushed limestone	–	1.905(1)	11.7(1)
	Subgrade	Yellow sand	–		14.(1)
14	Base course	2% HCTCRB	832, 896, 1023	2.166(1)	7.6(1)
	Subbase	Crushed limestone	–	1.869(1)	12.1(1)
	Subgrade	Yellow sand	–	1.755	14.5(1)

1 Values obtained from testing conducted by Southern Gateway Alliance.
2 RLT testing carried out using dynamic compacted samples with a minimum octahedral shear stress of 125 kPa and a maximum mean normal stress of 240 kPa and three test samples per sample batch.

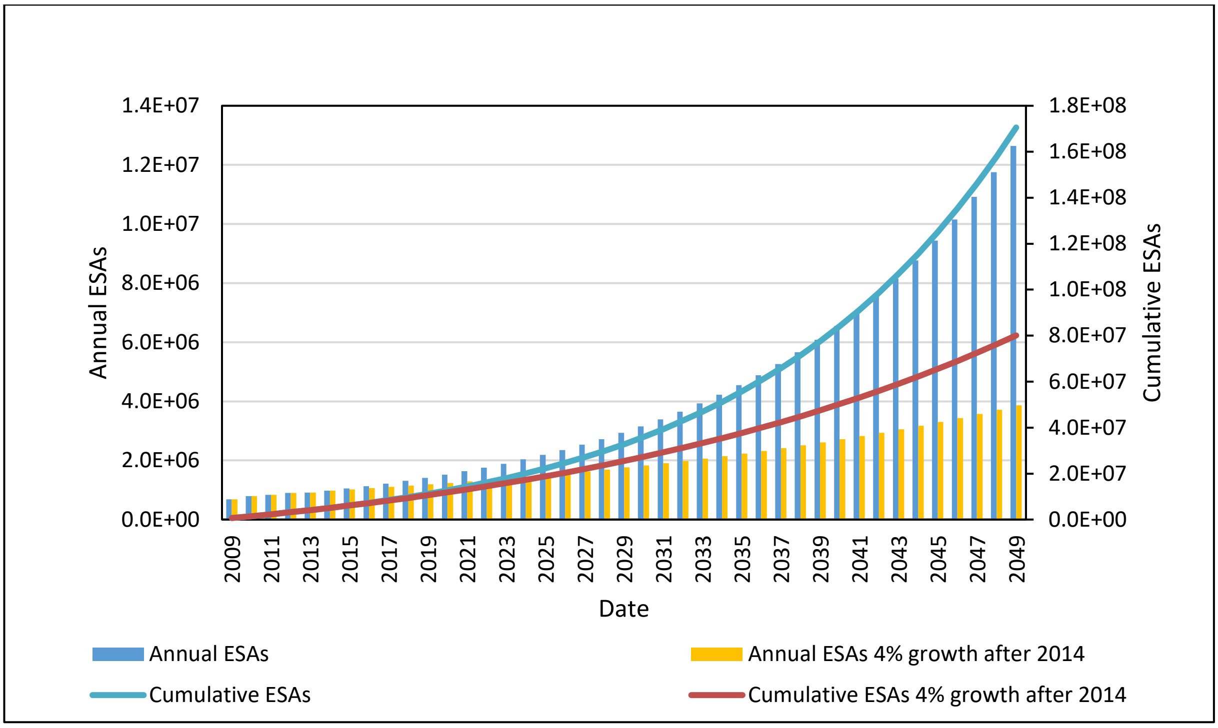
Source: Rehman (2012) & MRWA IRIS(2017).

12.3 Design Traffic Loading

Kwinana Freeway traffic data from 2009 to 2016 was extracted from IRIS database on 20 July 2017. The trial sections fall within the Kwinana Freeway traffic section designated KF36L.

The cumulative ESAs were estimated using the Main Roads supplied macro-enabled spreadsheet in conjunction with the most up-to-date IRIS data. Figure 12.1 presents both the annual ESAs and the cumulative ESAs for the KF36L traffic section for the years 2009 to 2049 inclusive. Actual traffic counts were only available from 2009 to 2014.

Figure 12.1: Annual ESAs and cumulative ESAs for Kwinana Freeway



Source: MRWA IRIS (2017).

Using the current cumulative ESAs trend, a forecast of the expected traffic at the end of a 40-year design period has been estimated. For the Kwinana Freeway trial sections, this value is estimated to be 1.7 x 10^8 ESAs in 2049 assuming a future growth rate of 7.6%. Initial traffic, from opening in 2009 to 2010, increased by 17%. The growth rate per annum stabilised in 2013 and 2014 at approximately 7.5%, which still seems high. Note that if a future growth rate of 4.0% is used, the cumulative traffic over 40 years reduces to 8 x 10^7 ESAs.

12.4 Pavement Maintenance

Pavement defect surveys carried out between 2010 and 2015 were extracted from IRIS. The surveys indicated that patching was done where asphalt cores have been drilled in all sections. No other repairs were noted.

12.5 Maximum Deflections

12.5.1 Design Deflection

Based on the 40-year traffic estimate from Section 12.3, the design deflection can be estimated from Figure 10.1. For 8 x 10^7 ESAs, the design deflection to limit permanent deformation is estimated at 0.81 mm in terms of BB rebound deflections. Using the procedure described in Section 10.2, the corresponding 50 kN FWD design deflection is 0.87 mm.

12.5.2 Results and Findings

FWD deflection data is available from IRIS and has been extracted and presented in this report. There were 11 data sets available from September 2009 until October 2016.

The FWD deflections reported have been standardised to a contact stress of 700 kPa, and adjusted to a pavement temperature of 29 °C. This is based on the assumption that the measured surface temperature is representative of the asphalt temperature. The average maximum deflection, D_0, and curvature values were calculated for 100 m sections.

Four of the data sets were measured between January and April, where seasonal variations could have a significant influence. A recommended correction factor of 1.3 according to Austroads (2011) will result in estimates that are too high for deflection and curvature. Hence, because of other available data for most of these years, data measured from January to April are omitted from this report.

The Kwinana Freeway's outer lane and outer wheel path deflection data is presented in Figure 12.2. The measured deflections reduced over the first two to three years, possibly due to an increase in modulus of the HCTCRB, as well as possible densification. In 2012 there appears to be a slight increase in maximum deflections for Sections 1 and 9, thereafter, the deflection data remains constant and no obvious deteriorating trends could have been estimated. The deflection values are highest for Section 1 and 14. These two sections, however, did not show any further deterioration and hence a detailed investigation of pavement distress was not yet warranted. Section 1 and 14 have the thinnest base layers by almost 30 mm, and Section 1 was the only section with a white sand subgrade.

Figure 12.2: Kwinana maximum deflection data – outer lane, outer wheel path

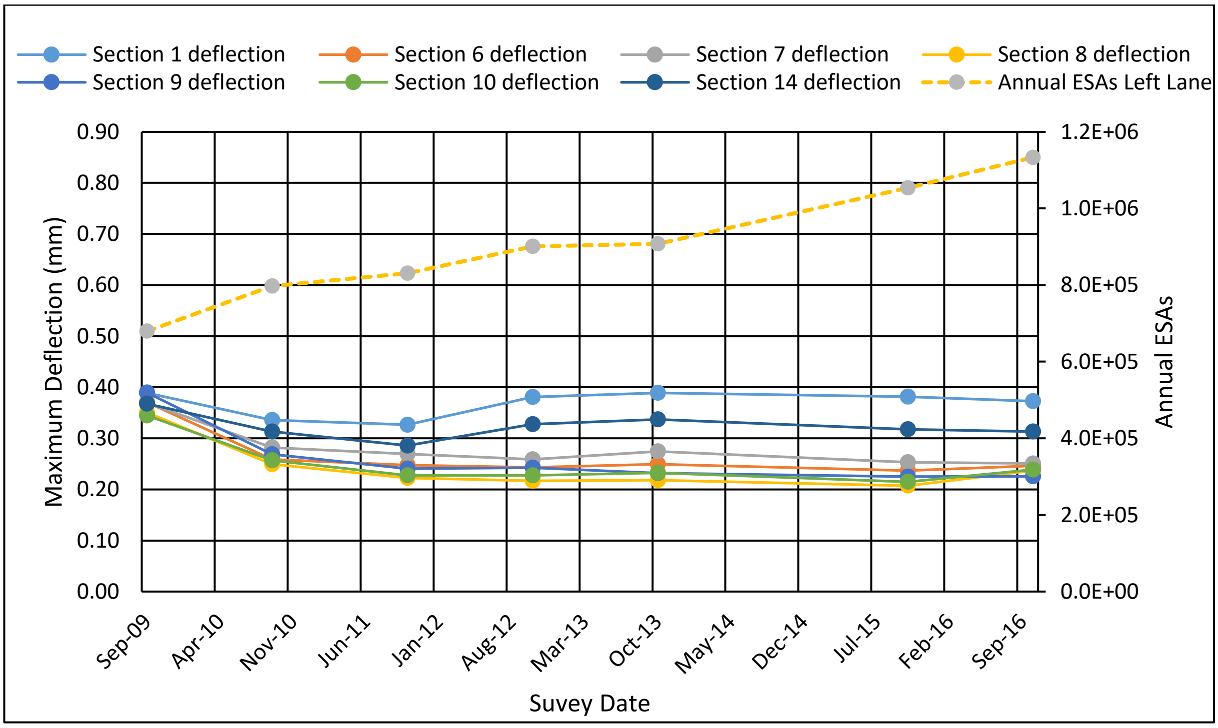
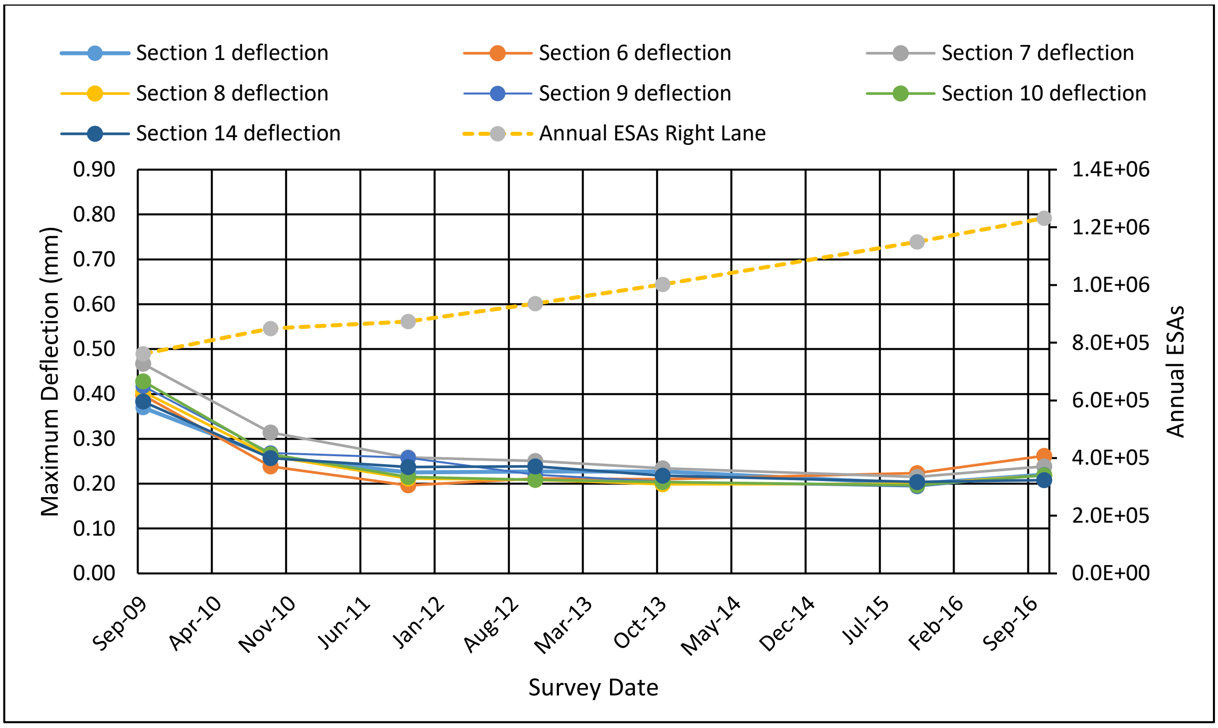


Figure 12.3 shows the inner lane data. The maximum deflection results are slightly lower than the outer lane for Sections 1 and 14. These lower values could be as a result of less damage due to less truck traffic in the inner lane, better construction compaction or, most probably in this case, reduced influence from seasonal moisture changes, as the lane is positioned further from the shoulder.

Figure 12.3: Kwinana maximum deflection data – inner lane, inner wheel path



12.6 Curvature

12.6.1 Results and Findings

Figure 12.4 and Figure 12.5 depict the FWD curvature data for both the outer lane and inner lane. The results show improvement in the upper layer modulus up until 2012. Thereafter, the curvature data for the outer lane shows initial deterioration, but does not exceed 0.12 mm in 2016. Only Section 1 and Section 14 show increasing and higher values in the outer lane, compared to the other sections, with curvature values above 0.08 mm. Note that Sections 1 and 14 have a lower HCTCRB thickness (180 mm) than other sections (230 mm). Their greater increasing in curvatures since opening may be due to the greater fatigue damage to the HCTCRB.

In the inner lane, Section 6 has a higher curvature than the other sections, although this value is still below 0.08 mm. Note for Section 1, the curvatures have not increased and are similar to those of sections with thicker HCTCRB. This is different from the trend in the more heavily trafficked slow lane and adds weight to the hypothesis that HCTCRB in Sections 1 and 14 is reducing in modulus due to traffic loading.

Figure 12.4: Kwinana curvature data – outer lane, outer wheel path

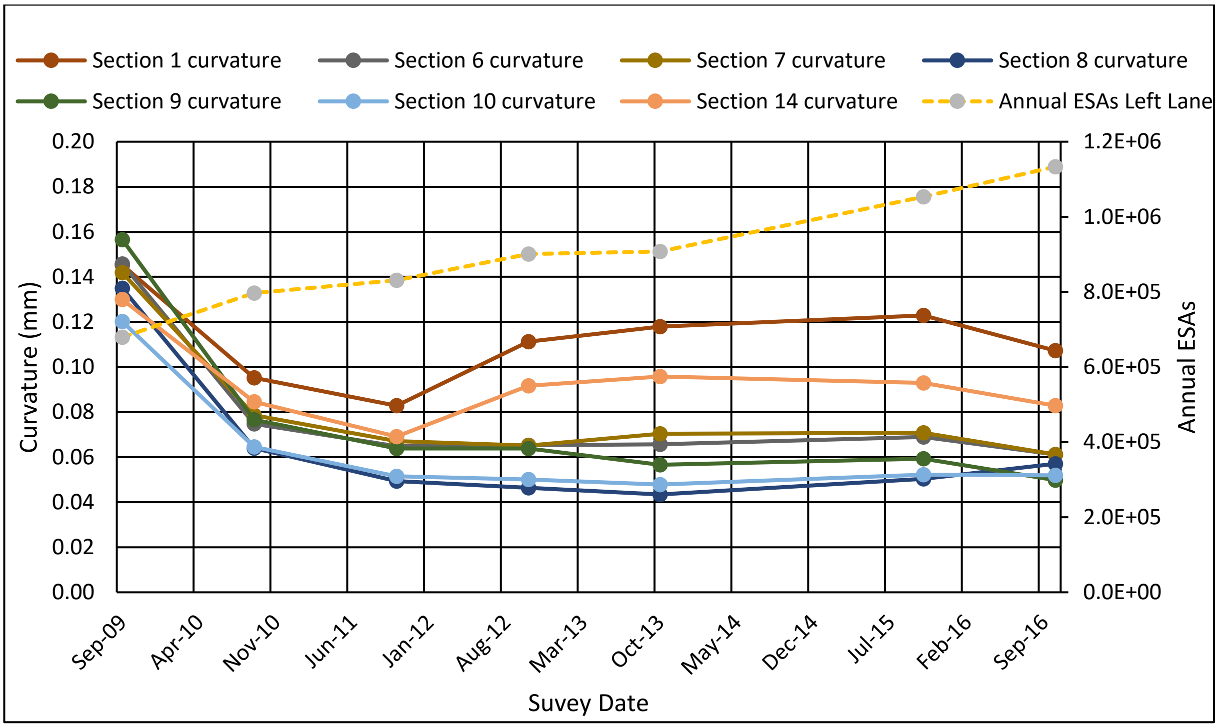
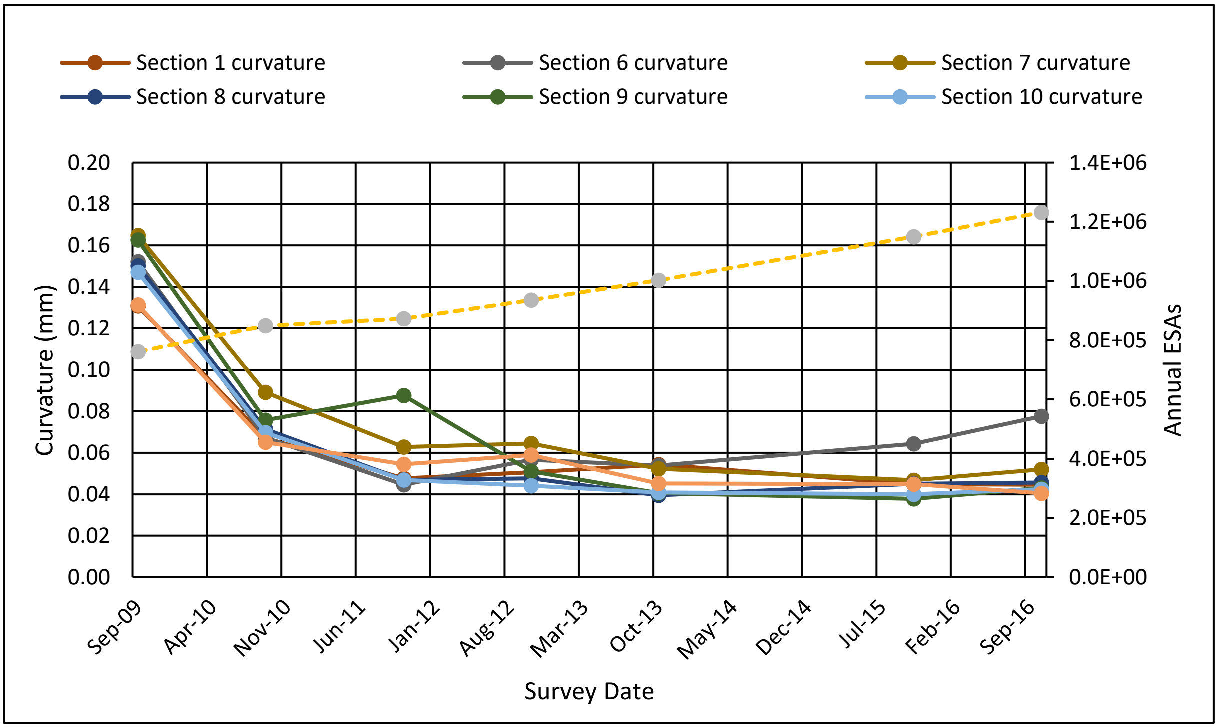


Figure 12.5: Kwinana curvature data – inner lane, inner wheel path



12.7 Roughness

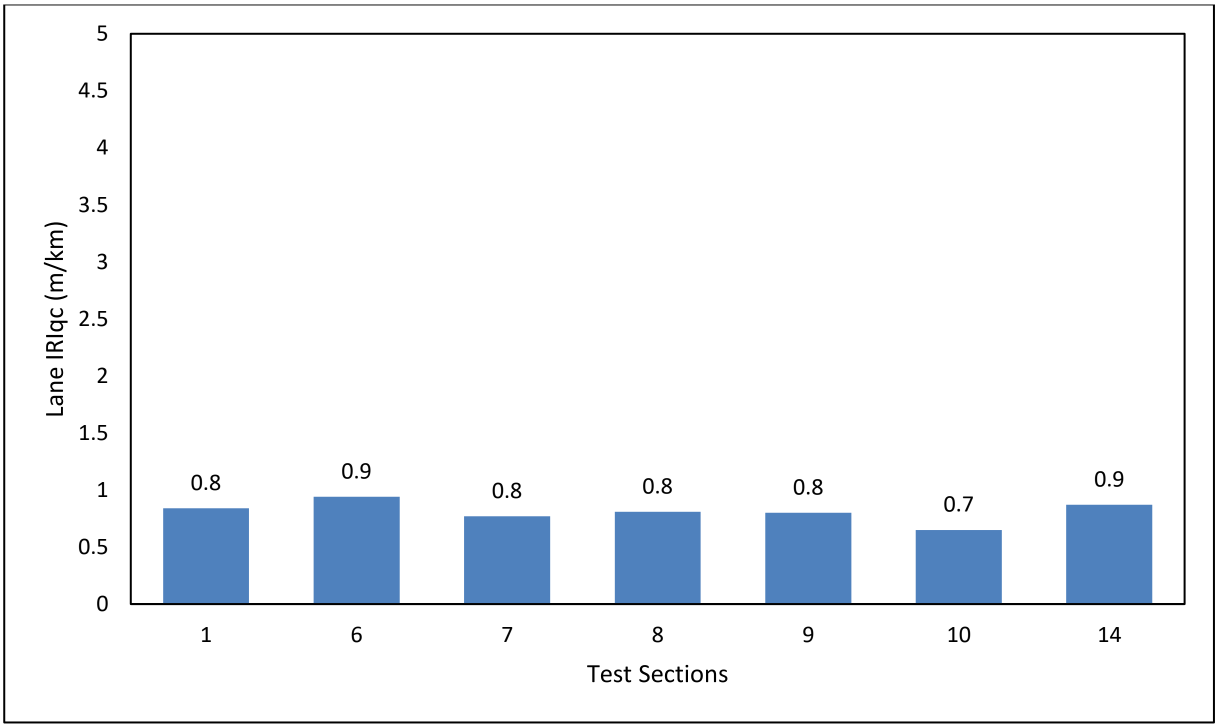
12.7.1 Results and Findings

For the Kwinana Freeway trial sections, only one data set for roughness is available. This data is illustrated in Figure 12.6 and Figure 12.7.

The outer lane's riding quality is satisfactory, with all results less than 1.0 m/km. These values typically represent a new asphalt surfaced pavement in an acceptable condition.

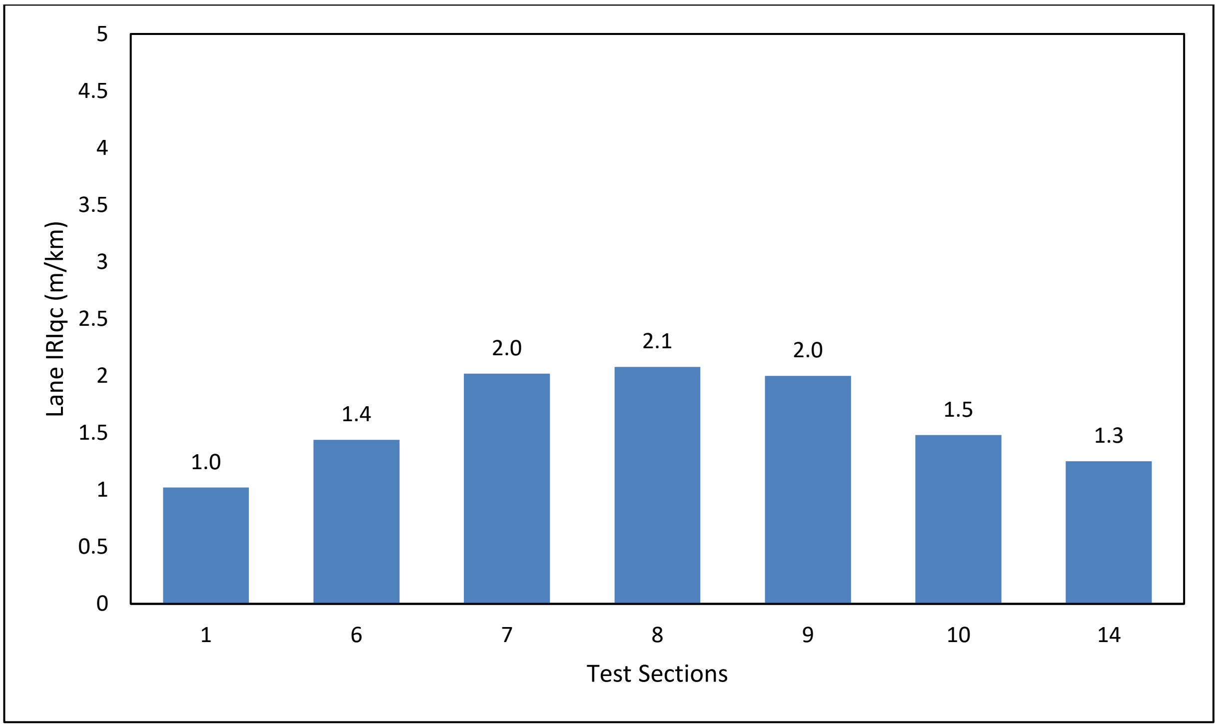
The roughness values for the inner lane also show a well-performing pavement with values between 1 and 2. No apparent roughness progression can be established from this data and it should be noted that this data is outdated.

Figure 12.6: Kwinana Freeway roughness data outer lane



Source: MRWA IRIS (2017).

Figure 12.7: Kwinana Freeway roughness data inner lane



Source: MRWA IRIS (2017).

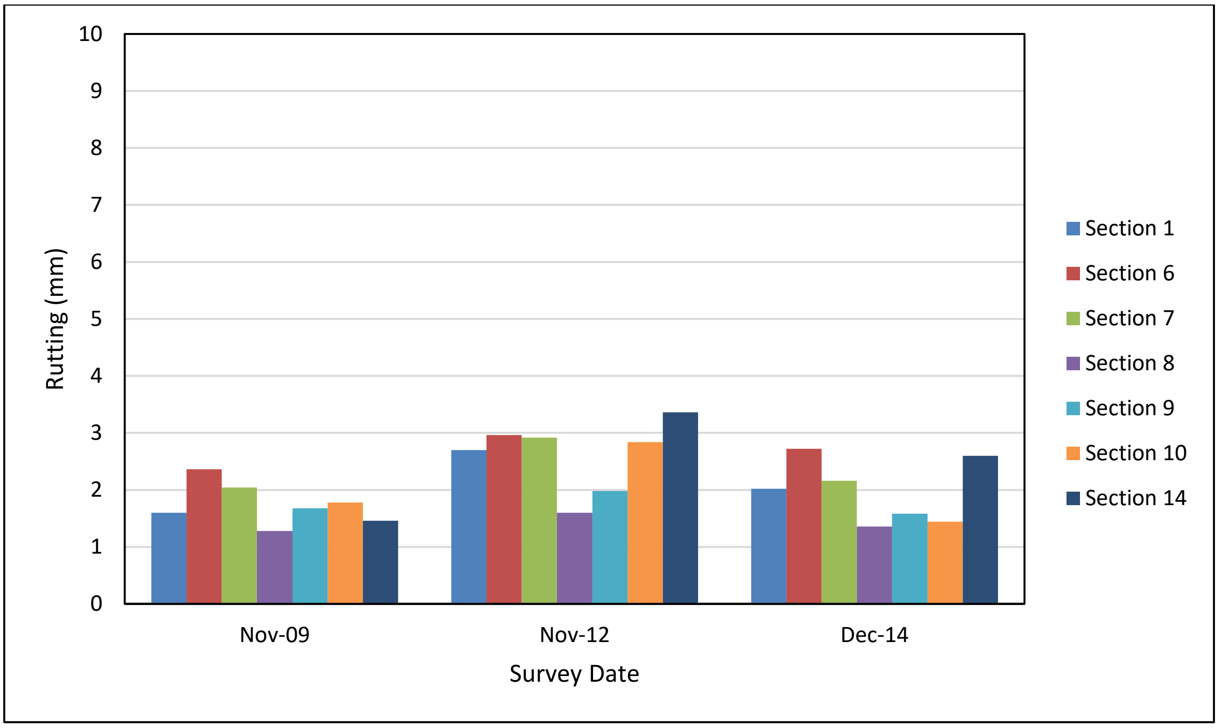
12.8 Rutting

12.8.1 Results and Findings

Rut depth data for the Kwinana Freeway test sections are reported as 100 m average values under a 2 m straight edge on data collected in 20 m intervals for the years 2009, 2012 and 2014.

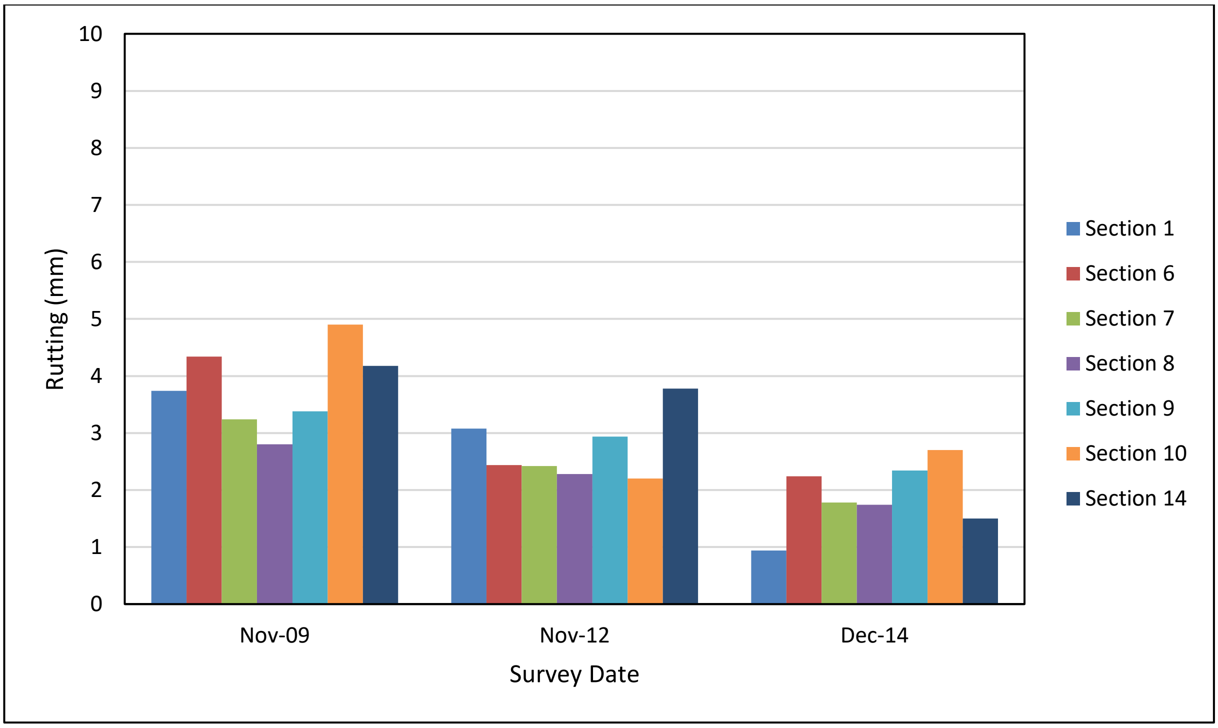
Both lanes and wheel paths data are presented in Figure 12.8, Figure 12.9, Figure 12.10 and Figure 12.11. No clear trend for deterioration can be identified from this data. All measured data was captured using automated non-contact measurement techniques. The rut values for the left lane and outer wheel path varies between 1 mm and 3 mm for all sections. Sections 6 and 7 generally have higher rutting than Sections 8 to 10. These differences are, however, so small that no conclusion can be made on pavement deterioration.

Figure 12.8: Kwinana Freeway rut data southbound outer lane outer wheel path



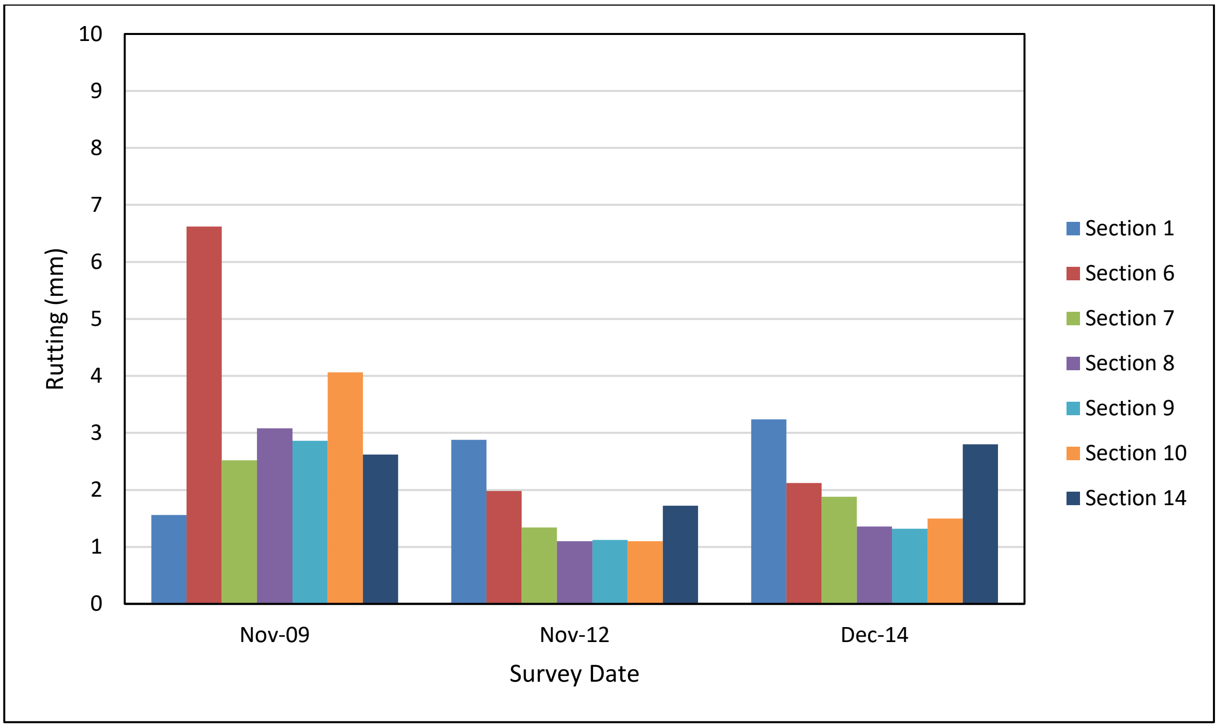
Source: MRWA IRIS (2017).

Figure 12.9: Kwinana Freeway rut data southbound outer lane inner wheel path



Source: MRWA IRIS (2017).

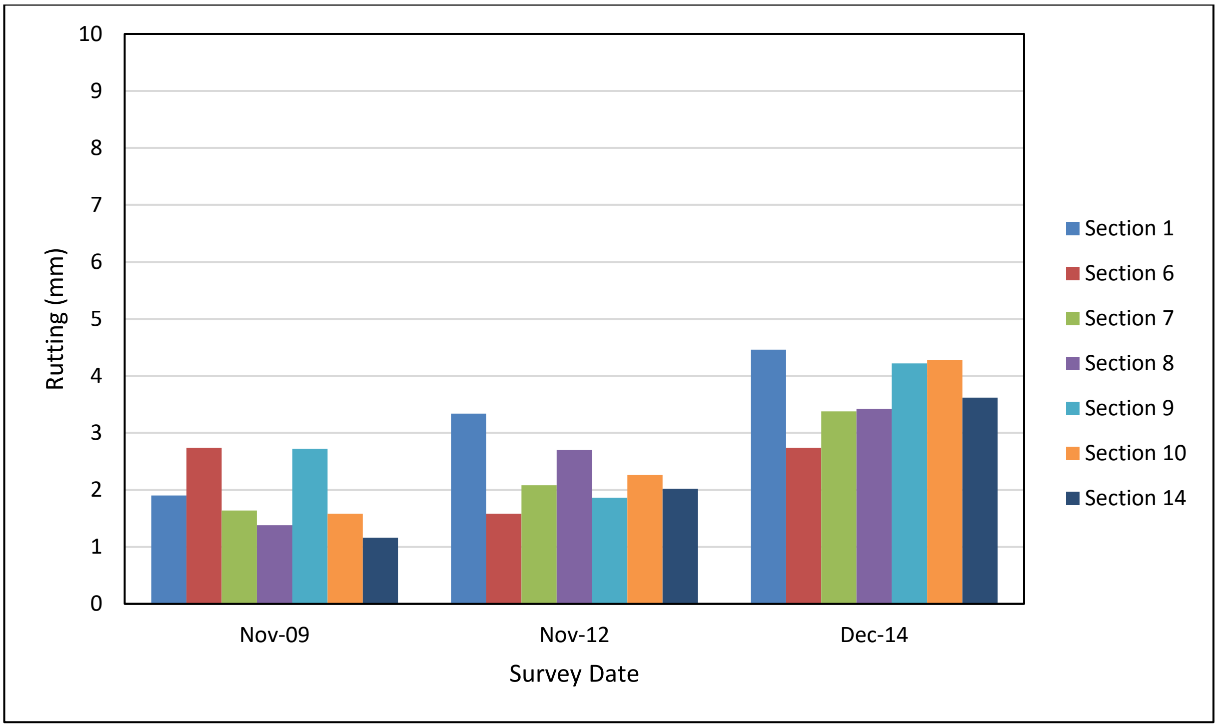
Figure 12.10: Kwinana Freeway rut data southbound inner lane outer wheel path



Source: MRWA IRIS (2017).

The inner lane, inner wheel path rut depths measured in 2014 (Figure 12.11) seem unreasonably high compared to the other wheel paths. As such, these results should be validated with new measurements.

Figure 12.11: Kwinana Freeway rut depths southbound inner lane inner wheel path



Source: MRWA IRIS (2017).

12.9 Summary

Kwinana Freeway trial sections show good performance over the 8 years of analysis for the deflection measurements, although the pavements have not been visually inspected for cracking.

In terms of the trends in curvatures, the slow lane curvatures for the thinner (180 mm) HCTCRB sections (Sections 1 and 14) have increased more since opening to traffic than the sections with thicker (230 mm) HCTCRB. This is consistent with HCTCRB reducing in modulus in the heavily trafficked slow lane. Visual inspection of all pavements is recommended to confirm whether these possible increases in HCTCRB moduli have or will result in surface cracking.

With roughness data available for 2009 only, no performance trends can be identified. The 2009 data is outdated. More recent roughness data should be collected to assess the current performance. The rut data indicates values well below 10 mm and hence does not pose a safety concern.

13 MITCHELL FREEWAY PERFORMANCE

13.1 Description of HCTCRB Sections

In 2008, about a 4 km section of Mitchell Freeway's main carriageway was constructed with HCTCRB between the chainages as indicated in Table 13.1. Some intersections between these chainages were constructed with an asphalt base or limestone base, but these intersections are not discussed in this report. The posted speed limit for this section is 100 km/h.

Table 13.1: Mitchell Freeway trial sections data

Construction section	Start chainage (m)	End chainage (m)	Length (m)	Granular pavement materials			Surfacing
				Base	Subbase	Subgrade	
1	25.12	29.29	4170	HCTCRB	Limestone	Yellow sand	30 mm DGA 30 mm OGA

Source: MRWA IRIS (2017).

Table 13.2 shows the design thickness of all the pavement layers.

Table 13.2: Mitchell Freeway trial section material profiles

Trial ID	Material purpose	Material	Design thickness (mm)
1	Surfacing	DGA & OGA	30 mm OGA on top of 30 mm DGA
	Base course	HCTCRB	230
	Subbase	Crushed limestone	175
	Subgrade		

Source: MRWA IRIS (2017).

13.2 Material Properties

MRWA measured the modified Proctor MDD and OMC during construction for only parts of the pavement sections. These results are shown in Table 13.3.

Resilient modulus testing was not undertaken.

Table 13.3: Mean values of laboratory testing results – granular materials Mitchell Freeway

Trial section	Material purpose	Material	Mean MDD (t/m^3)	Mean OMC (%)
28350–28880 NB	Base course	2% HCTCRB	2.158	7.2
28397–28846 SB	"	"	2.163	7.3
27250–27620 SB	"	"	2.174	7.0
27000–27250 NB	"	"	2.181	7.5
26340–26700 NB	"	"	2.112	8.2

Trial section	Material purpose	Material	Mean MDD (t/m^3)	Mean OMC (%)
27900–28200 SB 29140–29480 NB	Subbase	Crushed Limestone	1.915 1.926	10.4 10.4
28100–28420 SB	Subgrade	Sand	1.84	11.9

Source: MRWA IRIS(2017).

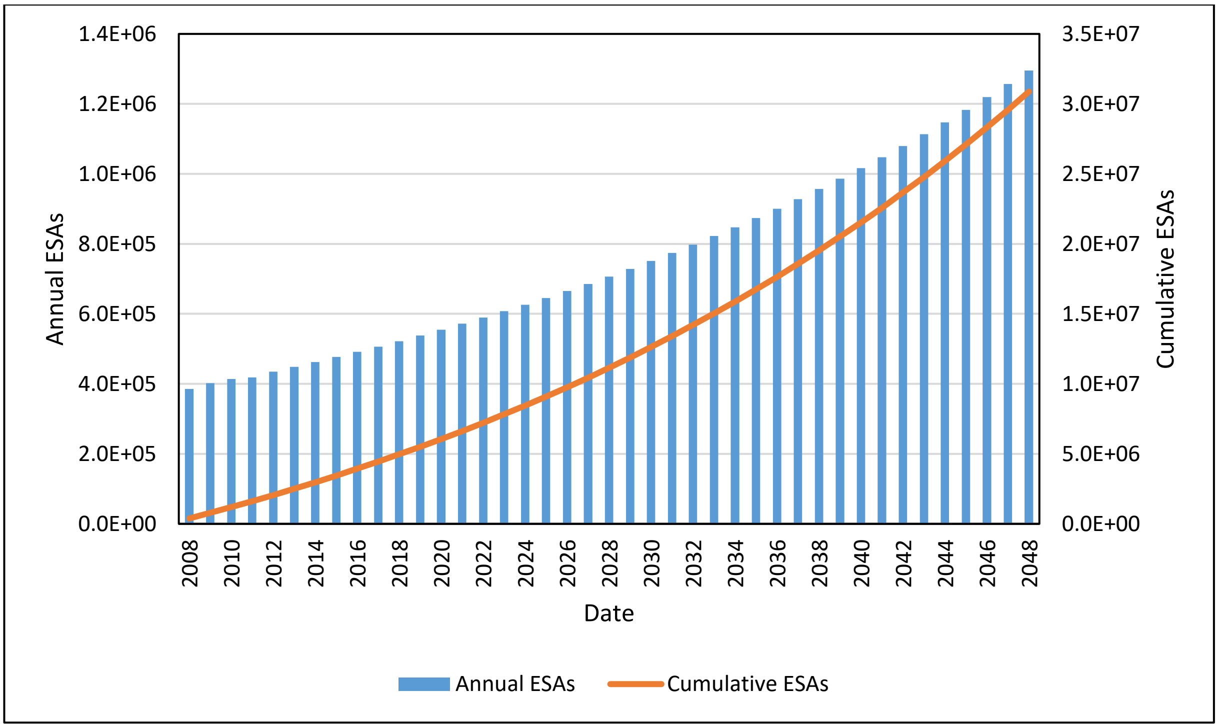
13.3 Design Traffic Loading

The 40 year design traffic was estimated at 1 x 10^8 ESAs based on the scope of works and technical criteria received from MRWA.

Traffic data available from IRIS included annual ESA's from opening the road in 2008 until 2017. The actual traffic data for this period indicated a 3% annual traffic growth for the northbound lane and 4.9% for the southbound lane based on actual traffic counts between 2008 and 2012.

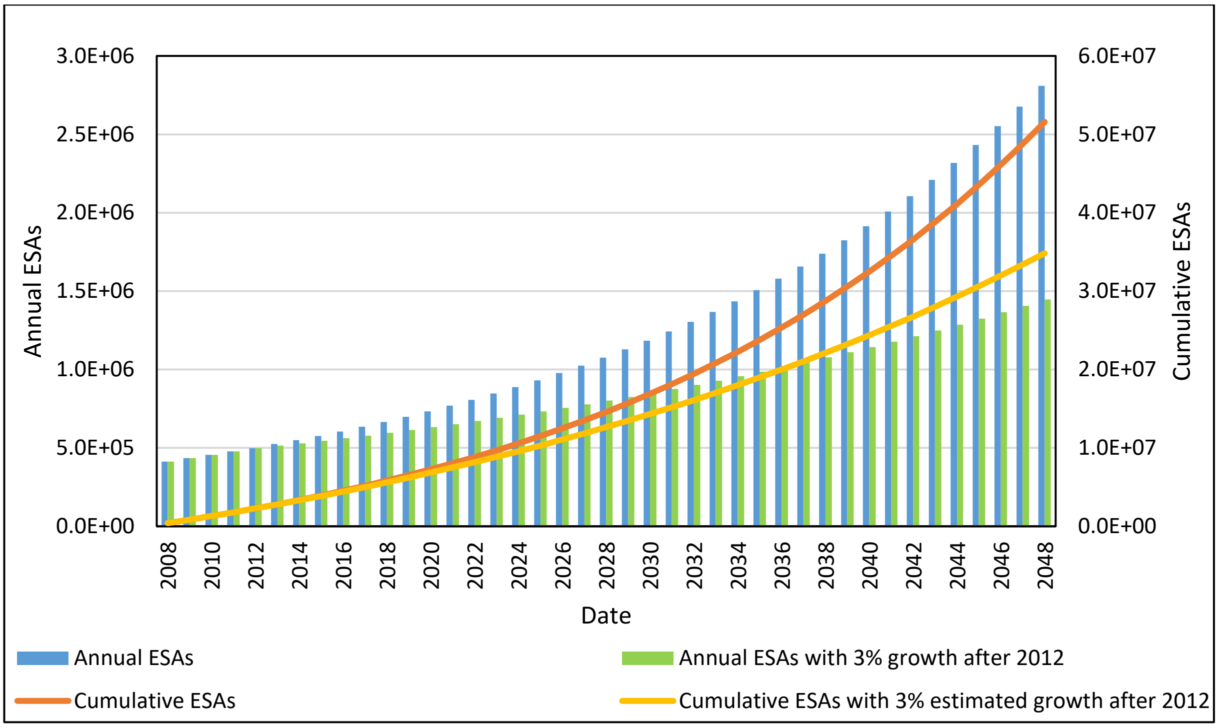
The traffic data was sourced from three traffic-counting stations. The data for the station with the highest traffic counts is shown in Figure 13.1 and Figure 13.2. Data was extrapolated after 2012 for the remainder of the design period by using the average growth rate of actual traffic counts.

Figure 13.1: Annual ESAs and cumulative ESAs for Mitchell Freeway northbound



Source: MRWA IRIS (2017).

Figure 13.2: Annual ESAs and cumulative ESAs for Mitchell Freeway southbound



Source: MRWA IRIS (2017).

Using the current cumulative ESAs trend, the 40-year design traffic loadings were predicted. For the northbound sections, this value was estimated to be 3.1 x 10^7 ESAs using a growth rate of 3%. For the southbound sections, the design traffic loading was estimated to be 5.2 x 10^7 ESAs based on a 5% growth rate after 2012.

13.4 Pavement Maintenance

Mitchell Freeway pavement maintenance detail was extracted from IRIS on 15 June 2017. This data indicates that the only intervention on these sections were from SLK chainage 25.12 to 25.39. It appears that an additional 3.5 m wide traffic lane and 1.5 m wide shoulder were constructed recently. These new pavement layers consist of an asphalt base and limestone subbase and were excluded from the investigation.

13.5 Maximum Deflections

13.5.1 Design Deflection

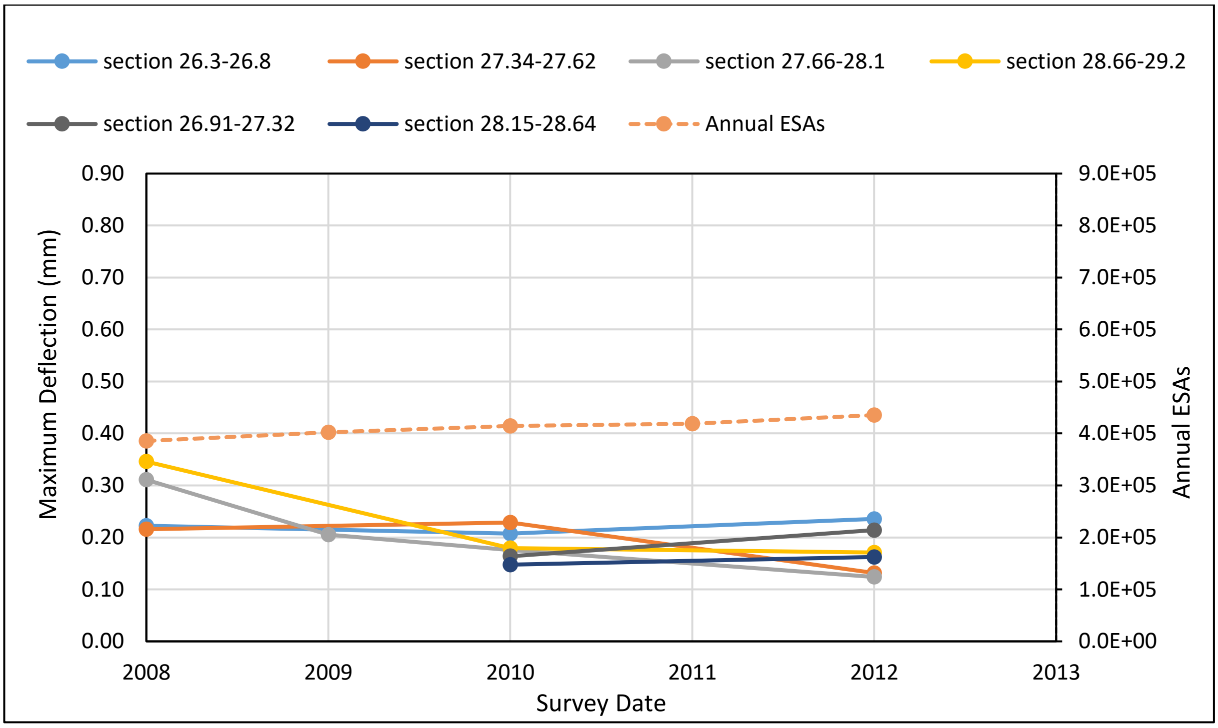
Based on the estimated design traffic from Section 13.3 above, the BB design deflection is 0.84 mm for the northbound lane and 0.82 mm for the southbound lane. Using standardisation factors, the converted 50 kN FWD design deflection is calculated to be 0.90 mm for a 60 mm asphalt surfacing for the northbound lane, and 0.88 mm for the southbound lane. By comparison, the design deflection limits during the defects correction period are tabled in the SWTC and recorded as: FWD maximum deflection limit at 700 kPa and for the 95th percentile for all segments (100 m) should be less than 1.04 mm.

13.5.2 Results and Findings

FWD deflections were measured on the HCTCRB sections from 2008 to 2012. No recent FWD measurements are available. It is recommended that FWD surveys are conducted at regular intervals to observe trends. Due to variations in measurement intervals throughout the survey years, results are reported on different section lengths and not 100 metre sections as indicated in Figure 13.3.

From this figure, there seems to be either traffic induced densification or curing for the first two years before the pavement structure stabilised or started to show slight signs of deterioration. The deflection data shows low deflections ranging from 0.16 mm to 0.24 mm.

Figure 13.3: Mitchell Freeway maximum deflection data – northbound



13.6 Curvature

13.6.1 Specified Curvature

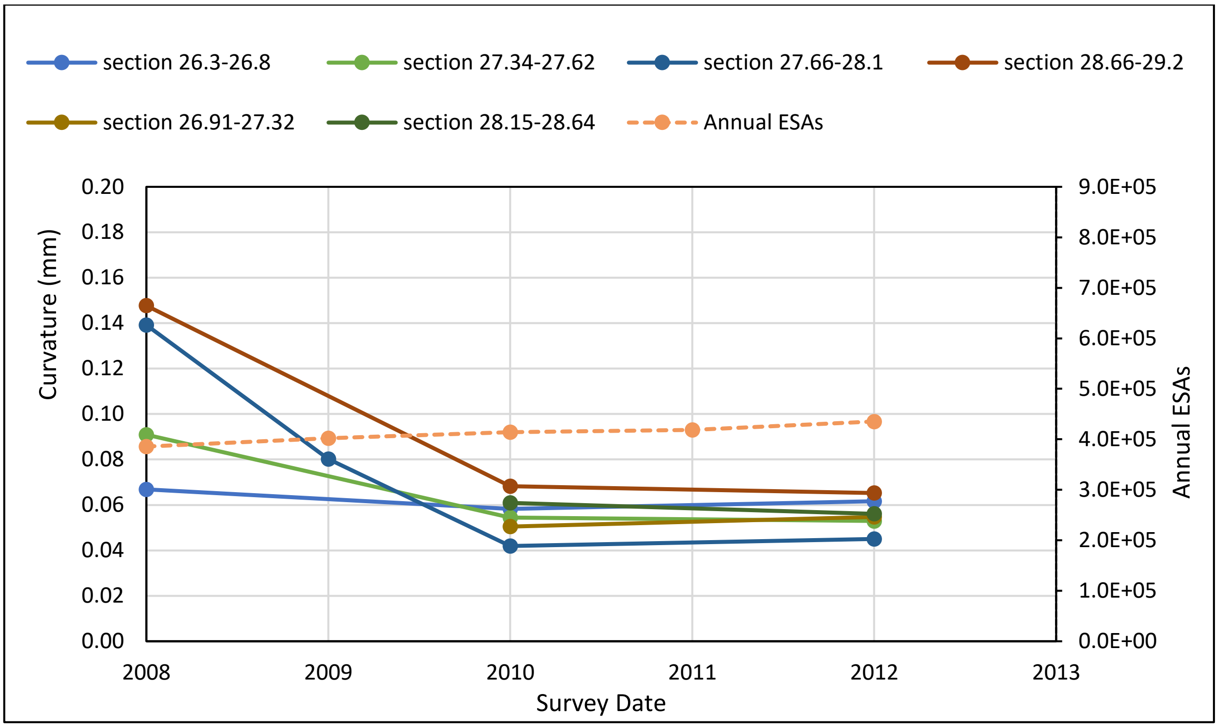
The design curvature during the defects correction period as prescribed in the Scope of Work and Technical Criteria (SWTC) are as indicated below:

Maximum Allowable Curvature: FWD 700 kPa mean should be less than 0.13 mm.

13.6.2 Results and Findings

The curvature results show no apparent deterioration. After two years in service the curvature values are low and reasonably constant (Figure 13.4).

Figure 13.4: Mitchell Freeway curvature data – northbound



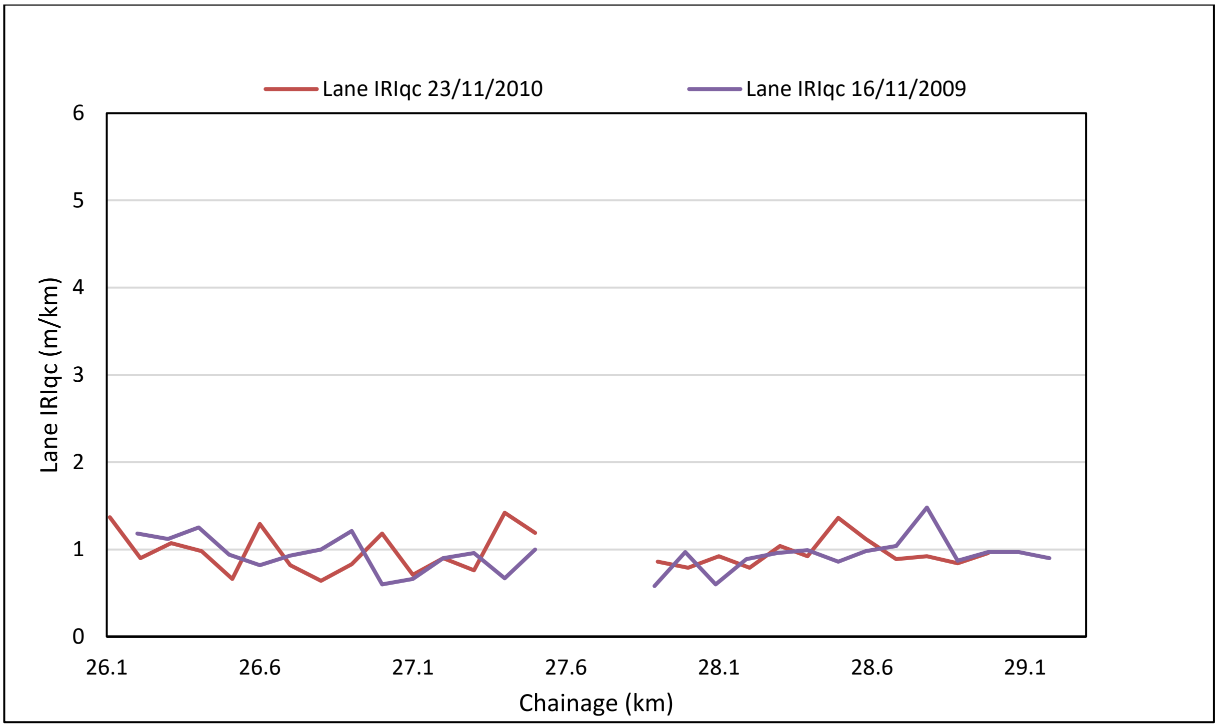
13.7 Roughness

13.7.1 Results and Findings

The Mitchell Freeway roughness data as presented in Figure 13.5 and Figure 13.6 contains only two sets of data for each direction. The roughness values are low and consistent with a newly constructed asphalt surfaced pavement.

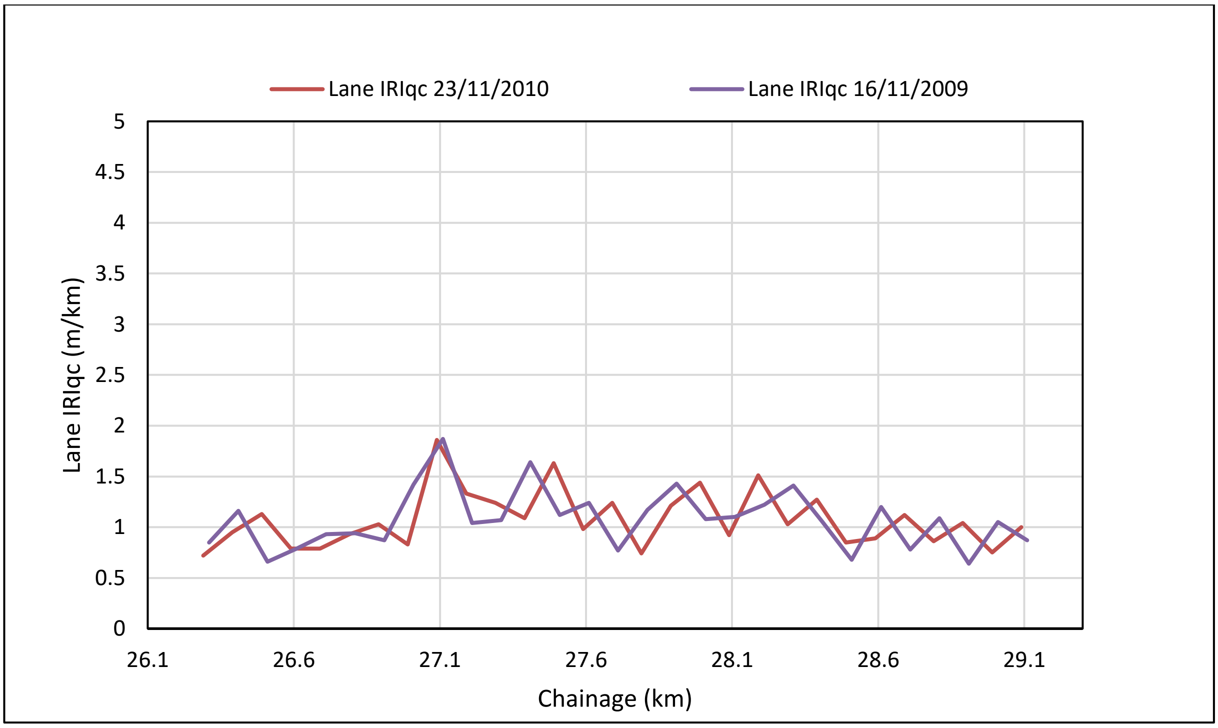
Roughness measurement chainages differed by 20 m in the northbound direction, which is evident in Figure 13.6.

Figure 13.5: Mitchell Freeway roughness southbound



Source: MRWA IRIS (2017).

Figure 13.6: Mitchell Freeway roughness northbound



Source: MRWA IRIS (2017).

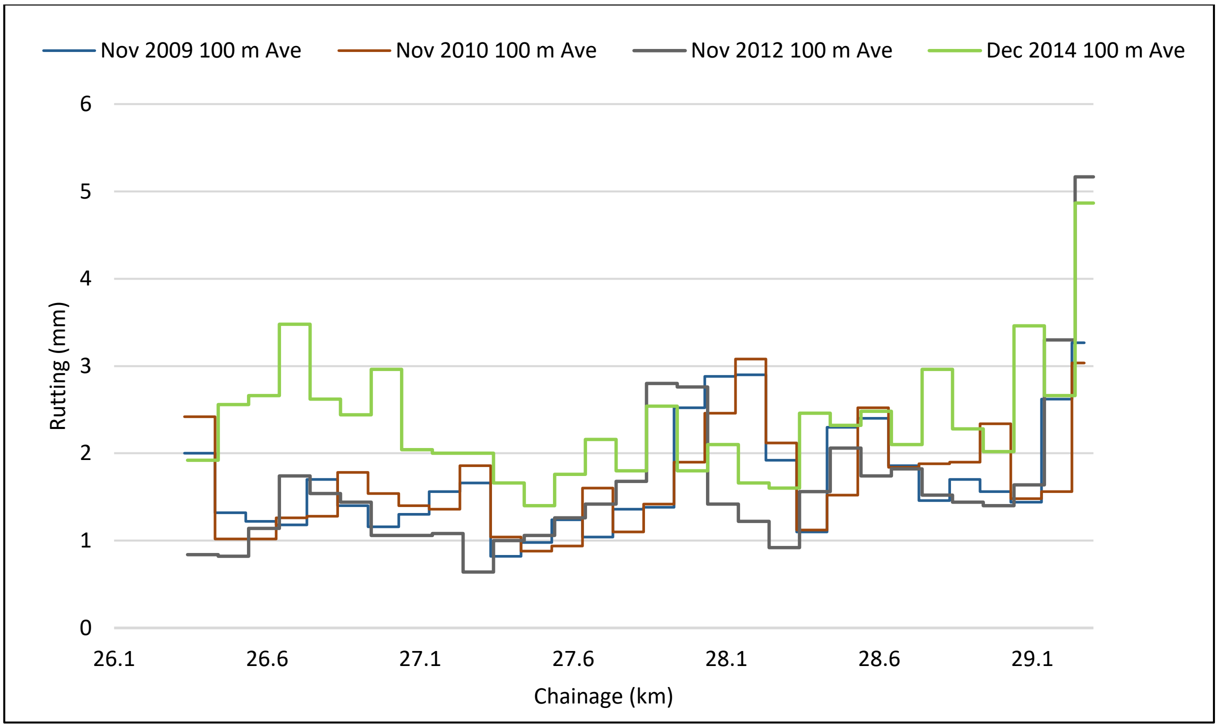
13.8 Rutting

13.8.1 Results and Findings

Four data sets are available for the Mitchell Freeway since its construction. These data sets were extracted from IRIS database and include mechanical non-impact measurements. The results are rut depths under 2 m straight edge averaged over 100 m intervals.

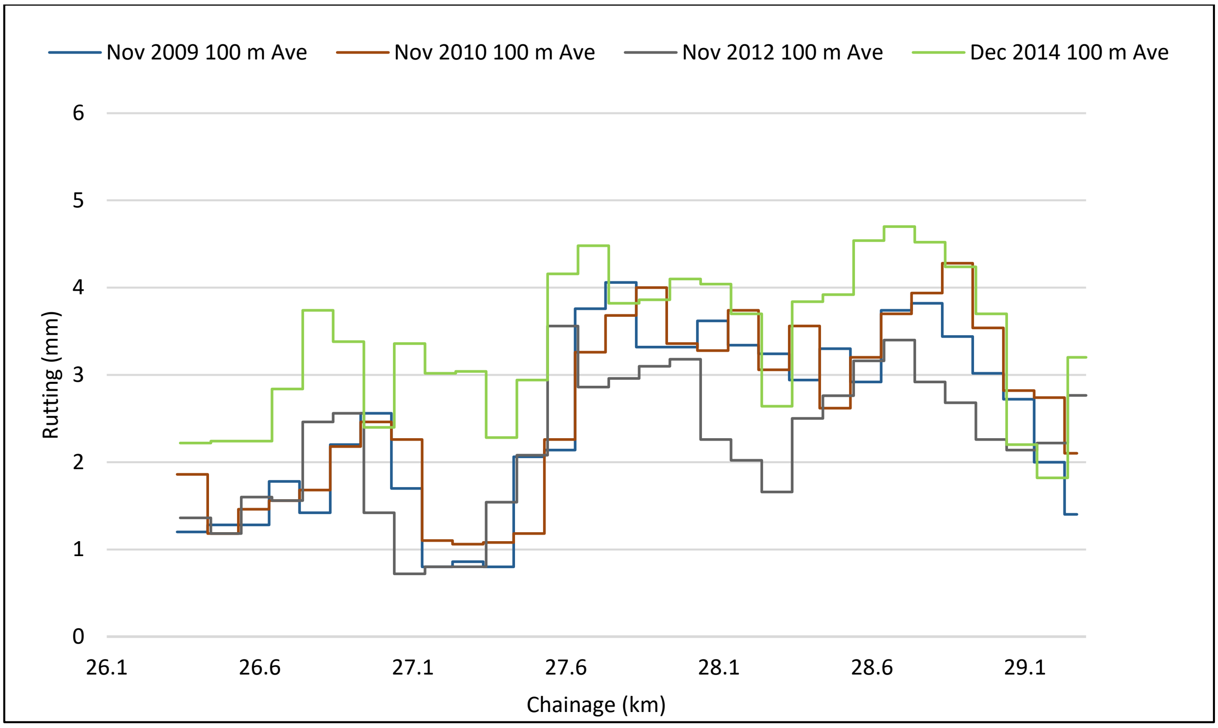
The overall performance is still acceptable with the inner wheel path exhibiting higher rut values but most are still below 4 mm.

Figure 13.7: Mitchell Freeway rut depths outer wheel path northbound



Source: MRWA IRIS (2017).

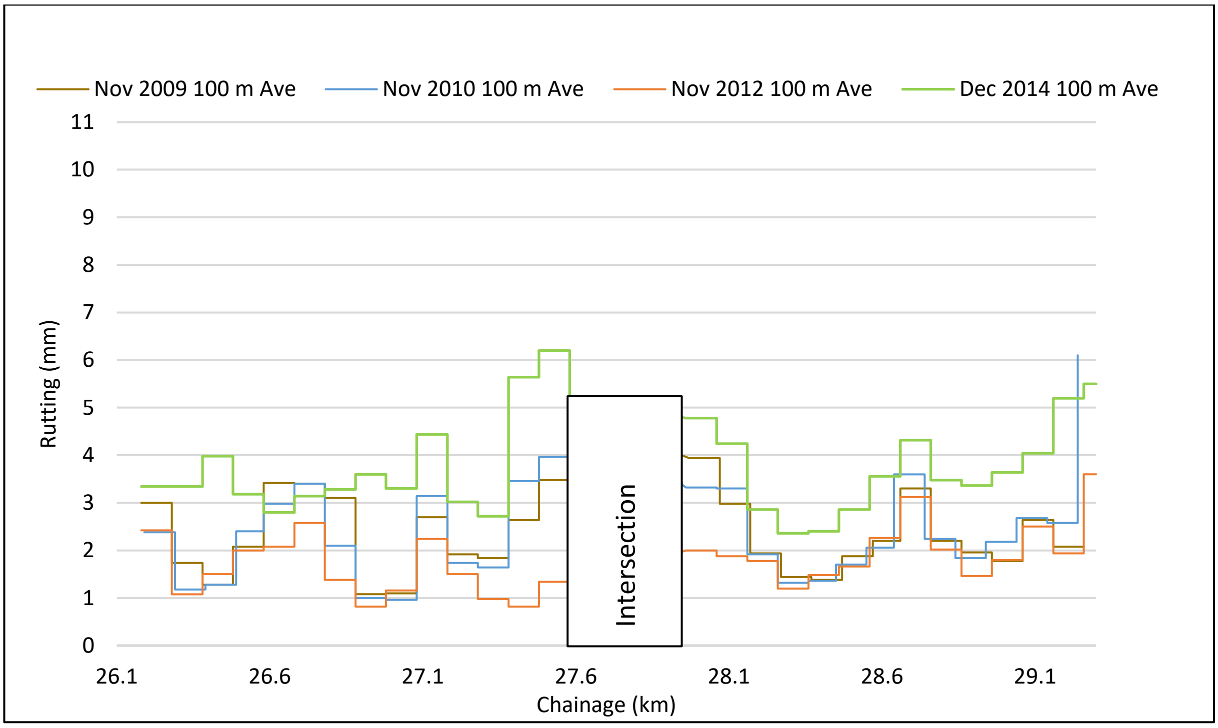
Figure 13.8: Mitchell Freeway rut depths inner wheel path northbound



Source: MRWA IRIS (2017).

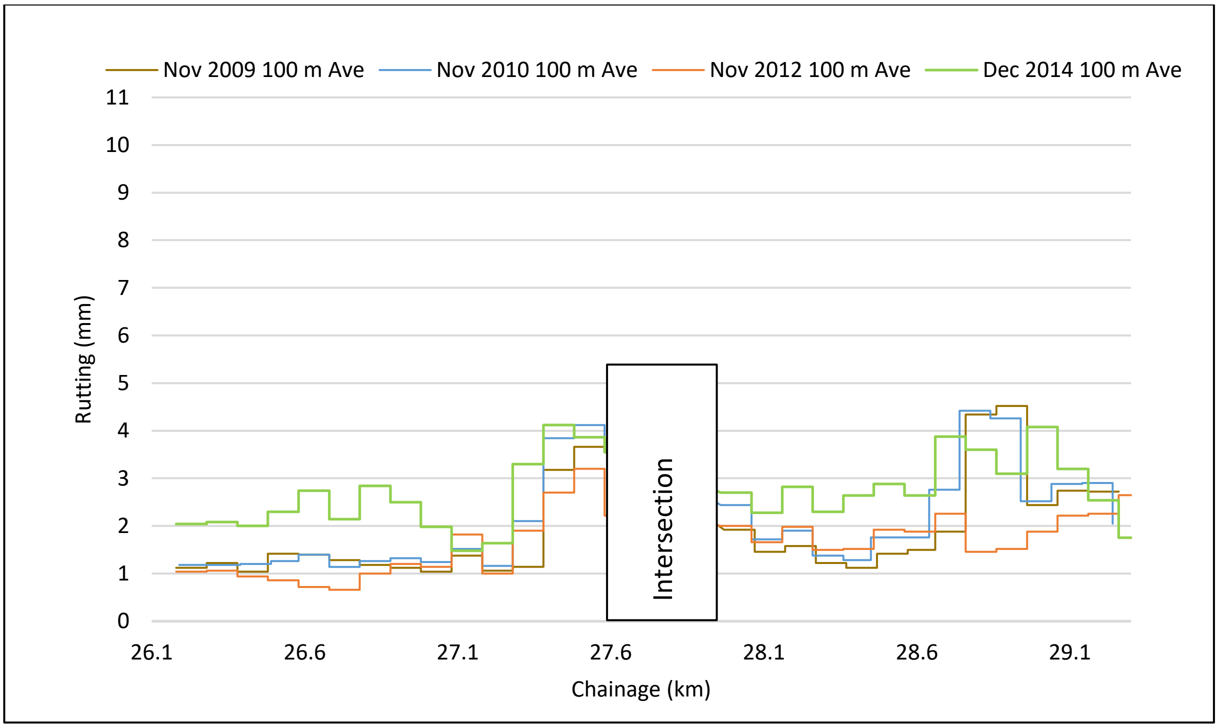
Figure 13.7 and Figure 13.8 represent the average rut depths in the northbound lane for both the inner and outer wheel path. The northbound carriageway average rut depths have not deteriorated in the years from 2009 to 2012, however from 2012 to 2014 the rate of deterioration has been 0.4 mm/year and 0.6 mm/year for the outer wheel path and inner wheel path respectively.

Figure 13.9: Mitchell Freeway rut data outer wheel path southbound



Source: MRWA IRIS (2017).

Figure 13.10: Mitchell Freeway rut data inner wheel path southbound



Source: MRWA IRIS (2017).

Figure 13.9 and Figure 13.10 represent the average rut depths in the southbound lane for both the inner and outer wheel path. Between 2012 and 2014 there was an increase in the rate of deterioration in the outer wheel path of 1 mm/year and 0.5 mm/year in the inner wheel path. It is recommended that roughness be measured again soon to quantify the rate of increase.

Higher rutting values close to the intersection could be caused by breaking of vehicles, resulting in shoving of the asphalt. The average rut depth is still low, being below 4 mm in 2014.

13.9 Summary

For the Mitchell Freeway sections of HCTCRB, the available performance data does not show any deterioration for roughness, deflection and curvature values for the latest survey results. It should be noted that no recent deflection, curvature and roughness data was available to access the current performance after the 2012 results. The latest rut data from 2014 shows an average deterioration of up to 1 mm/year, but additional measurements are required to confirm this deterioration rate. In general, with the available data, the Mitchell Freeway section was still performing well until the latest 2014 condition data and does not yet warrant any remedial actions.

14 APPARENT PERFORMANCE TRENDS

14.1 Introduction

To establish accurate performance trends, complete data sets are required at similar survey conditions and field conditions. For this study, only a few data sets can be used to provide performance trends. For those data sets that are incomplete or outdated, comments can only be made on actual values, and this can be compared to threshold tolerances as indicated in Figure 14.1 for roughness and rutting values. Note that roughness units are represented as m/km or mm/m and not as indicated in the table below as mm/km.

Figure 14.1: Indicative threshold surface performance indicators to initiate targeted discrete network level surveying, Table 2.1 of AGAM05D (2008)

Road type	Typical operating conditions		Performance indicators					
			Roughness		Rutting (1.2 m straight edge)		Cracking	
	Speed (km/h)	AADT (v/day)	Limit (IRI (mm/km))	Rate (IRI/yr)	Limit (mm)	Rate (mm/yr)	Limit (% area)	Rate (% area/yr)
Freeway	≥ 100	> 30,000	3.5	0.05	10	0.3	1	0.1
Highly trafficked arterial road	100	> 10,000	4.2	0.08	10	0.5	2	0.1
Medium trafficked arterial	80 - 100	2,000 – 10,000	4.2	0.2	15	0.6	5	0.5
Low trafficked arterial or main road	Various	< 2,000	5.4	0.3	20	0.8	10	1

Note: Deflection survey to occur when either or any of the above performance indicator limits and/or rates is exceeded.

Source: Austroads (2008).

14.2 Roughness

For the HCTCRB sections in this report, no deterioration trends are evident yet, hence no progression curve or equation can be developed to estimate available service life. Roughness can be caused by both load and non-load sources, for example traffic volume changes or volume changes due to moisture changes. Due to the roughness graphs not showing any clear deterioration, it can be presumed that moisture does not significantly affect the HCTCRB sections, and hence have validated the use of such base layer to reduce moisture sensitivity in pavements.

Mitchell Freeway has the highest roughness values, but in general, all roughness values are still below an IRI of 2 m/km and do not yet warrant any action.

Table 14.1shows the Reid Highway and Kwinana Freeway current roughness values as a percentage of the desirable maximum.Both the Kwinana and Reid Highway sections have roughness values less than 2 m/km and show no signs of deterioration based on roughness. This generally represents a good driving experience and does not warrant any remedial action.

It is strongly recommended that additional roughness and deflection measurements be done on the Mitchell Freeway section. Finally, these roughness measurements do not yet warrant any maintenance action, but they are outdated and they do not necessarily represent the current situation.

Table 14.1: Roughness values as per cent age of desirable maximum

Section	Current lane IRIqc LL	% of desirable max	Current lane IRIqc RL	% of desirable max
Reid Highway (4.2 m/km desirable IRI) 2010 latest data				
1	1.16	28	1.76	42
5	1.19	28	1.2	29
6	1.18	28	1.38	33
Kwinana Freeway (4.2 m/km desirable IRI) 2009 only data set				
1	0.84	24	1.02	29
6	0.94	27	1.44	41
7	0.77	22	2.02	58
8	0.81	23	2.08	59
9	0.8	23	2.0	57
10	0.63	18	1.48	42
14	0.87	25	1.25	36

14.3 Rutting

Some of the uses of rutting data is outlined in Austroads (2007b). For the Reid Highway and Kwinana Freeway trial sections the rut values are still low and do not yet warrant any remedial action or investigation. The Mitchell Freeway has seen a high rate of deterioration from 2012 to 2014. It should be noted that this road has more intersections, which can be prone to increased deterioration. Although rutting values are deteriorating at a higher rate, the extent does not yet warrant any remedial action therefore continuous monitoring is suggested.

Figure 14.2 shows a comparison of deflection measurements and rut measurements for Mitchell Freeway. It is evident from this figure that the rutting and deflection data show an increase at similar chainages. These values are still within acceptable limits, and the shear strength in the upper pavement layers seems to be adequate to withstand loads.

Table 14.2 summarises the most current rut data for Reid Highway and Kwinana Freeway and how they are performing based on desirable maximum rut limits.

Figure 14.2: Rutting and deflection correlations for Mitchell Freeway

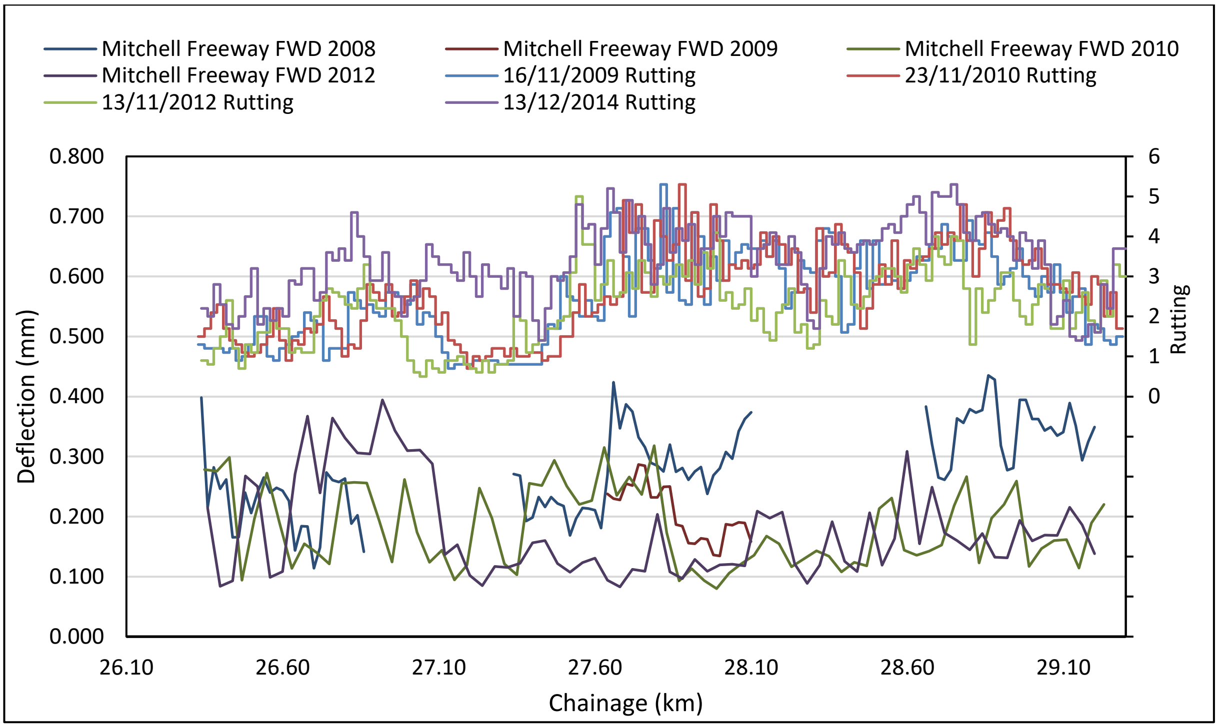


Table 14.2: Rutting values as per cent age of desirable maximum

Section	Rutting depth LL OWP	% of desirable Max	Rutting depth LL IWP	% of desirable max	Rutting depth RL OWP	% of desirable max	Rutting depth RL IWP	% of desirable max
Reid Highway (10 mm desirable maximum rut) 2014 latest data								
1	1.28	13	3.02	30	1.52	15	2.84	28
5	1.46	15	2.2	22	0.98	10	1.86	19
6	1.34	13	2.36	24	1.22	12	1.38	14
Kwinana Freeway (10 mm desirable maximum rut) 2014 latest data								
1	1.6	16	3.74	37	3.24	32	4.46	45
6	2.36	24	4.34	43	2.12	21	2.74	27
7	2.04	20	3.24	32	1.88	19	3.38	34
8	1.28	13	2.8	28	1.36	14	3.42	34
9	1.68	17	3.38	34	1.32	13	4.22	42
10	1.78	18	4.9	49	1.5	15	4.28	43
14	1.46	15	4.18	42	2.8	28	3.62	36

14.4 Deflection and Curvature

Deflection and curvature data is used to assess the structural adequacy of flexible pavements. Assessment can be done by either comparing characteristic deflections to design deflections or using the general mechanistic procedure for overlay thickness design. None of the trial sections currently have maximum deflection values higher than 0.5 mm. High deflections above 1.5 mm may be an indication of weak subgrade conditions. Hence, it can be expected the subgrade is still in a sound condition for all trial sections.

High curvatures imply a lack of pavement upper layer stiffness and reduced asphalt surfacing life with premature fatigue cracking (Butkus 2004).

For Reid Highway, the curvature values are between 25% and 34% of the deflection values for Sections 1 and 5. For Section 6, this value is only between 13% and 18% through the years from 2006 to 2011. In relation to Section 1 westbound lane, the curvatures appear to be increasing, which may be due to the thin (123 mm) HCTCRB reducing in modulus due to traffic loading. Visual inspection of all three pavement sections is required to assess surface cracking.

Kwinana Freeway also shows similar results with curvature values between 22% and 32% of the deflection values for the slow lane. This is a typical result for granular pavements with thin bituminous surfacings according to AGPT05 (2011). Mitchell Freeway however, has curvature values as a percentage of deflection above 35% from chainages 27.35 to 29.20. This could indicate low modulus of the granular base course.

Kwinana Freeway's outer lane shows the highest change in deflection and curvature values from that of the inner lane for Section 1 and 14 only. These sections had an HCTCRB thickness of 180 mm compared to a thickness of 230 mm for the other sections. This could be because of moisture ingress from the side of the road and could also be because of thinner base layers than the rest of the sections. The increase in curvatures in Sections 1 and 14, are consistent with decrease in modulus of the HCTCRB in the heavily trafficked slow lane. Again, visual inspection of all pavements is recommended to confirm whether these possible decreases in HCTCRB moduli have or will result in surface cracking.

15 CONCLUSIONS AND WAY FORWARD

One of the objectives of this report was to review the performance of the sections with HCTCRB on the Reid Highway, Mitchell Freeway and Kwinana Freeway since their construction in 1996, 2008 and 2009 respectively. Information from MRWA database indicates there has been no treatments applied to these pavements, except for the upgrading of the intersection at Reid Highway.

The performance monitoring has yet to include visual inspections for surface cracking. When this cracking is obtained in the future it will assist in clarifying the performance trends summarised below.

The Reid Highway trial sections perform well, given that the sections have carried about 1×10^7 ESA of traffic loadings over the last 21 years and that the HCTCRB in Section 1 was only 123 mm thick. In relation to Section 1 westbound lane, the curvatures appear to be increasing which may be due to the thin HCTCRB reducing in modulus due to traffic loading. Visual inspection of all three pavement sections to assess surface cracking is required. The HCTRCB in one of the Reid Highway sections was produced using only 1% cement, yet it appears to be performing well, albeit that the curvatures are high. Consequently, the Reid Highway trial performance suggests that for moderately trafficked roads, at least the current specification 501 could be reviewed and possibly be amended to be less strict on the cement content and layer thicknesses required for HCTCRB construction.

The more heavily trafficked Mitchell and Kwinana Freeway sections are also performing well after about 9 and 8 years of trafficking respectively, with associated 2017 cumulative traffic loadings of 5×10^6 ESA and 8×10^6 ESA. However, the increase in curvatures in Kwinana Freeway Sections 1 and 14, are consistent with a decrease in modulus of the HCTCRB in the heavily trafficked slow lane. Again, visual inspection of all pavements is recommended to confirm whether these possible decreases in HCTCRB moduli have or will result in surface cracking.

In summary, all three roads have performed satisfactorily so far based on a desktop study on information provided by MRWA. These preliminary findings should provide increased interest in the use of HCTCRB. However, the reader is cautioned that the test results for some sections are outdated, hence the current road condition could be significantly different from the results in this study. In addition, data on surface cracking which is critical to an assessment of HCTCRB has yet to be obtained and reviewed.

The project to date has been biased in that it has not included pavements that have experienced premature cracking. It is recommended that this study be extended to include such HCTCRB pavements that have failed in the past, with more in-depth analysis to identify why it performed differently. If such information can be identified, it may lead to improved confidence in the use of HCTCRB as a cost-effective pavement for heavily trafficked urban roads.

Based on the current performance of the trial sections in this report, the HCTCRB pavement layers seem to be behaving as bound layers. From the review of the Specification 501, UCS limits have been reduced and hydration periods have been increased in the latest revision, in order to have more confidence that the constructed layer will not gain too much strength to behave as a bound layer. None of the above roads have been constructed to these new limits and hence no performance data on the influence of these changes can be reviewed.

Throughout Australia, the definition and limits for modified granular materials differ. TMR has a limit on maximum UCS values of between 1 MPa and 2 MPa for modified granular materials, while

MRWA have lowered this maximum limit to 1 MPa, which is similar to the Austroads 28-day UCS maximum value.

The current practise is to model modified granular material as unbound material when carrying out mechanistic designs. This could have a major influence if the material is behaving as a bound layer and not as an unbound layer. This design process requires further investigation.

It is recommended that further research should be done on classifying HCTCRB material together with a future trial section on current specifications. For this future stage, the following outcomes are proposed:

- Undertake visual inspections of the three trial sections investigated to date to obtain cracking data.
- To better characterise the material to include possible additional testing requirements in order to reduce the uncertainty of whether the material will behave as a bound layer.
- As part of Austroads research, ARRB is developing a large-scale wheel-tracking test to assess the cracking characteristics of lightly-bound materials. Consideration should be given to testing HCTCRB to provide insight into its characteristics.
- To study the effect of lower cement contents and the effect of carbonation on these lower cement contents.
- To develop a specific mechanistic design approach and testing methods for HCTCRB.
- To identify a minimum pavement thickness for HCTCRB with a pavement design life of 40 years and design traffic of up to 1 x 10^8 ESA.
- To develop a scope for a trial pavement section, which should include different design approaches and requirements for surfacing thicknesses, and including potential strain alleviating interlayers.
- To compare these recommended HCTCRB design approaches with alternative design approaches for example, full depth asphalt (FDA), to ascertain the viability of this product.

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APPENDIX A

A.1 RTL and UCS Results from MRWA Report

		Hydration Period																			
		7 days				14 days				30 days				60 days				90 days			
			RLT				RLT				RLT				RLT				RLT		
Cement Content	Curing in Mould (days)	E MPa	DR	MR	UCS MPa	E MPa	DR	MR	UCS MPa	E MPa	DR	MR	UCS MPa	E MPa	DR	MR	UCS MPa	E MPa	DR	MR	UCS MPa
READYMIX GOSNELLS																					
	0	564	97.3%	74%	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	7	827	99.1%	84%	1.23	-	-	-	1.24	-	-	-	1.03	-	-	-	0.84	-	-	-	1.29
2.0%	28	1350	99.6%	83%		1307	98.7%	91%		886	97.3%	92%		721	97.4%	96%		637	98.6%	97%	
		1174	99.3%	83%	1.53	1359	98.9%	84%	1.40	1002	96.1%	83%	1.39	774	97.2%	84%	1.31	680	97.9%	87%	1.61
		2174	98.2%	70%		1389	99.5%	67%		1055	96.8%	65%		766	97.7%	75%		539	97.5%	76%	
	7	-	-	-	1.52	-	-	-	1.25	-	-	-	0.76	-	-	-	1.09	-	-	-	0.87
1.5%	28	979	101.4%	93%		1236	101.3%	72%		742	100.2%	81%		-	-	-		604	99.2	93.6	
		1257	101.8%	88%	1.86	1183	100.6%	86%	1.43	882	100.2%	86%	1.59	637	97.3%	81%	1.45	558	99.6	99.0	0.90
		1266	101.0%	89%		1079	99.8%	86%		632	99.9%	79%		962	98.9%	69%		587	96.7	89.2	
	7	-	-	-	1.23	-	-	-	1.25	-	-	-	1.19	-	-	-	0.99	-	-	-	0.81
1.0%	28	1042	100.0%	89%		888	101.4%	78%		568	101.7%	93%		612	101.3%	91%		403	100.3%	103.1%	
		782	97.7%	80%	1.48	766	102.1%	82%	1.62	671	100.4%	85%	1.07	554	99.2%	87%	1.23	619	98.0%	93.7%	0.74
		1213	98.8%	75%		1028	100.4%	60%		692	98.3%	73%		688	100.2%	86%		567	98.2%	83.9%	
0%	0	277	98.9%	74%		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	28	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
WA LIMESTONE BYFORD COARSE																					
	7	-	-	-	1.43	-	-	-	1.08	-	-	-	0.78	-	-	-	0.61	-	-	-	0.56
2.0%	28	893	99.9%	86.1%		669	100.4%	92.9%		759	97.5%	69%		573	101.7%	101.1%		643	100.0%	94.0%	
		799	99.3%	84.6%	1.55	747	99.1%	83.2%	0.92	783	99.9%	69%	0.77	685	99.9%	84.2%	0.40	678	98.9%	84.8%	-
		1050	100.6%	73.5%		761	99.5%	70.6%		904	101.0%	64%		703	97.8%	68.8%		1019	100.4%	72.3%	
WA LIMESTONE BYFORD FINES ADDED																					
	7	-	-	-	1.31	-	-	-	0.97	-	-	-	1.09	-	-	-	0.61	-	-	-	0.59
2.0%	28	-	-	-		659	99.0%	84.6%		897	101.6%	73%		703	100.4%	82.1%		686	99.0%	87.7%	
		855	98.5%	73.4%	2.31	640	100.7%	81.4%	1.52	762	100.0%	66%	1.12	749	97.9%	70.0%	0.89	784	98.8%	71.9%	0.66
		845	98.0%	67.3%		723	98.3%	70.6%		745	98.5%	61%		749	97.0%	64.2%		723	99.6%	78.7%	

Laboratory RLT & UCS Testing Data

Cement Content	Curing in Mould (days)	7 days E MPa	7 days RLT DR	7 days MR	7 days UCS MPa	14 days E MPa	14 days RLT DR	14 days MR	14 days UCS MPa	30 days E MPa	30 days RLT DR	30 days MR	30 days UCS MPa	60 days E MPa	60 days RLT DR	60 days MR	60 days UCS MPa	90 days E MPa	90 days RLT DR	90 days MR	90 days UCS MPa
BGC LAKES																					
	7	-	-	-	1.40	-	-	-	1.60	-	-	-	1.20	-	-	-	0.90	-	-	-	1.06
2.0%		1400	99.6%	82%		871	100.1%	90%		709	99.0%	100%		631	99.6%	97.0%		668	95.9%	88.8%	
	28	1087	99.9%	89%	2.20	736	97.9%	92%	2.40	977	99.9%	83%	1.60	576	95.9%	100.3%	0.98	619	97.5%	97.1%	0.78
		1795	99.6%	71%		1608	98.1%	73%		960	98.1%	77%		804	97.3%	84.1%		721	99.4%	86.2%	
BORAL ORANGE GROVE																					
	7	-	-	-	-	-	-	-	1.02	-	-	-	0.71	-	-	-	0.52	-	-	-	0.84
2.0%		Bad Data	98.3%	77.2%		730	101.3%	83.0%		657	99.3%	79.9%		573	97.7%	82.9%		953	100.2%	66.5%	
	28	1236	100.4%	58.4%	1.36	972	100.6%	69.0%	1.28	839	98.0%	67.3%	1.1	804?	98.5%	66.2%	1.25	Sample Destroyed			0.92
		1090	99.3%	56.6%		1108?	98.8%	64.4%		717	97.2%	65.4%		740	100.0%	67.9%		685	98.2%	60.8%	
HANSON BYFORD																					
	7	-	-	-	1.2	-	-	-	1.06	-	-	-	0.72	-	-	-	1.1	-	-	-	0.84
2.0%		1448	98.7%	73.6%		988	97.9%	91.0%		Sample Destroyed				970.021	97.6%	96.2%		857	97.7%	99.8%	
	28	1401	96.4%	70.4%	1.56	1187	96.5%	81.2%	1.44	Bad Data			1.27	860	97.7%	89.1%	1.49	853	97.1%	85.5%	1.00
		1300	96.3%	66.3%		1313	96.1%	72.9%		1050	97.1%	77.4%		985	96.0%	84.3%		1055	99.1%	73.0%	
HANSON RED HILL																					
	7	-	-	-	1.42	-	-	-	1.26	-	-	-	0.81	-	-	-	0.89	-	-	-	0.84
2.0%		976	100.8%	81.3%		922	100.3%	78.9%		898	99.1%	79.6%		606	100.6%	85.5%		681	99.1%	73.5%	
	28	1098	99.1%	69.8%	1.68	972	98.5%	73.5%	1.48	906	98.5%	80.1%	1.10	773	98.1%	73.9%	1.21	766	100.7%	69.9%	1.10
		1223	97.8%	57.6%		Bad Data	97.3%	57.5%		992	98.1%	69.4%		877	98.0%	66.0%		717	98.0%	60.4%	

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A.2 Deflection and Curvature Data

A.2.1 Kwinana Freeway FWD Data No Seasonal Corrections

LEFT LANE	KWINANA FREEWAY DEFLECTION AND CURVATURE VALUES LEFT LANE AND LEFT WHEEL PATH (mm)										
DEFLECTION	Sep-09	Sep-10	Mar-11	Oct-11	Oct-12	Oct-13	Apr-14	Mar-15	Oct-15	Mar-16	Oct-16
Section 1 deflection	0.39	0.34	0.32	0.33	0.38	0.39	0.37	0.38	0.38	0.41	0.37
Section 6 deflection	0.37	0.26	0.23	0.25	0.24	0.25	0.23	0.21	0.24	0.25	0.25
Section 7 deflection	0.37	0.28	0.24	0.27	0.26	0.27	0.25	0.25	0.25	0.23	0.25
Section 8 deflection	0.35	0.25	0.19	0.22	0.22	0.22	0.19	0.19	0.21	0.20	0.24
Section 9 deflection	0.39	0.27	0.21	0.24	0.24	0.23	0.21	0.21	0.23	0.20	0.23
Section 10 deflection	0.34	0.26	0.21	0.23	0.23	0.23	0.21	0.21	0.21	0.20	0.24
Section 14 deflection	0.37	0.31	0.29	0.29	0.33	0.34	0.32	0.32	0.32	0.29	0.31
CURVATURE											
Section 1 curvature	0.15	0.10	0.08	0.08	0.11	0.12	0.10	0.12	0.12	0.12	0.11
Section 6 curvature	0.15	0.07	0.06	0.06	0.07	0.07	0.06	0.06	0.07	0.07	0.06
Section 7 curvature	0.14	0.08	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.06	0.06
Section 8 curvature	0.13	0.06	0.04	0.05	0.05	0.04	0.04	0.04	0.05	0.05	0.06
Section 9 curvature	0.16	0.08	0.06	0.06	0.06	0.06	0.05	0.06	0.06	0.04	0.05
Section 10 curvature	0.12	0.06	0.05	0.05	0.05	0.05	0.04	0.05	0.05	0.04	0.05

Section 14 curvature	0.13	0.08	0.08	0.07	0.09	0.10	0.09	0.08	0.09	0.07	0.08
RIGHT LANE											
DEFLECTION	Sep-09	Sep-10	Mar-11	Oct-11	Oct-12	Oct-13	Apr-14	Mar-15	Oct-15	Mar-16	Oct-16
Section 1 deflection	0.37	0.26	0.18	0.23	0.23	0.23	0.18	0.17	0.20	0.17	0.22
Section 6 deflection	0.40	0.24	0.17	0.20	0.21	0.21	0.18	0.17	0.22	0.20	0.26
Section 7 deflection	0.47	0.31	0.25	0.26	0.25	0.23	0.19	0.19	0.22	0.21	0.24
Section 8 deflection	0.41	0.26	0.19	0.21	0.21	0.20	0.17	0.17	0.20	0.17	0.22
Section 9 deflection	0.42	0.27	0.20	0.26	0.22	0.20	0.17	0.17	0.19	0.18	0.22
Section 10 deflection	0.43	0.27	0.20	0.21	0.21	0.20	0.17	0.17	0.20	0.19	0.22
Section 14 deflection	0.38	0.26	0.21	0.24	0.24	0.22	0.18	0.16	0.20	0.17	0.21
CURVATURE											
Section 1 curvature	0.13	0.07	0.04	0.05	0.05	0.05	0.04	0.05	0.04	0.04	0.04
Section 6 curvature	0.15	0.07	0.05	0.04	0.06	0.05	0.06	0.06	0.06	0.06	0.08
Section 7 curvature	0.16	0.09	0.07	0.06	0.06	0.05	0.05	0.05	0.05	0.05	0.05
Section 8 curvature	0.15	0.07	0.05	0.05	0.05	0.04	0.04	0.05	0.05	0.04	0.05
Section 9 curvature	0.16	0.08	0.05	0.09	0.05	0.04	0.04	0.04	0.04	0.04	0.04
Section 10 curvature	0.15	0.07	0.05	0.05	0.04	0.04	0.04	0.04	0.04	0.05	0.04

Section 14 curvature	0.13	0.07	0.05	0.05	0.06	0.05	0.04	0.03	0.04	0.04	0.04