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P2/P14/P16 Stabilisation Practices in Queensland: Cementitious Modification and Foam Bitumen Stabilisation 2013-14/2014-15

SUMMARY

The stabilisation of otherwise unsuitable road construction materials is an economically and environmentally beneficial alternative to importing base course, subbase course, and/or select fill materials. A number of stabilisation technologies are currently available, each with distinct benefits and limitations. In Queensland, the selection of a stabilisation treatment for a given application is heavily influenced by traditional local practice, as a systematic approach for evaluating alternative treatments and electing the optimal solution was not previously available. The *Stabilisation Practices in Queensland* project is a multiyear effort with the objective to develop guidance on the best value for money stabilisation solution to adopt for a given set of climatic, environmental and operational conditions relative to the Queensland state-controlled road network.

The project objective was pursued through documenting and comparing Queensland and international best practices, reviewing the performance of the state-controlled road network, investigating factors significantly influencing in-service performance and comparing the relative cost of selected stabilisation treatments to unbound granular and full depth asphalt alternatives. Learnings were put into practice through the generation and modification (where required) of Department of Transport and Main Roads Queensland technical notes and specifications.

This report documents the learnings resulting from Years 1 and 2 of the project. The technologies investigated included plant-mixed cementitiously modified base (PM-CMB) and in situ mixed foamed bitumen stabilised (I-FBS) base. Findings included variances in national and international practice relative to the fundamental application of the different technologies, in addition to mixture proportioning and structural design. At the time of the initial survey (January 2014), PM-CMB was a component of approximately 109 km and I-FBS base of approximately 156 km of the Queensland state-controlled road network; approximately 99.5% of PM-CMB and 93.9% of I-FBS road sections were in excellent or good condition; and a significant proportion had far exceeded the original design service life.

This investigation revealed that increased resiliency in high-exposure environments can be achieved at a fraction of the cost of full depth asphalt when PM-CMB or I-FBS base technologies are utilised in accordance with best practice. PM-CMB is ideally provisioned in new pavements or for the rehabilitation of pavements where existing granular materials are of poor quality. Ideal conditions include moderate to heavy traffic, wet climatic conditions, nonreactive subgrade soils and a high-quality surfacing (asphalt or reinforced seal) is planned. I-FBS base is optimally selected for the rehabilitation of pavements where additional structural capacity is required and existing granular materials conform to standard specifications. Ideal conditions include moderate to heavy traffic volumes and a variety of climatic conditions and subgrade types, given appropriate measures are taken to stabilise moisture-sensitive soils.

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1 INTRODUCTION

The Queensland state-controlled road network includes approximately 33 300 km of pavement infrastructure connecting an area of roughly 1 850 000 km^2. The Department of Transport and Main Roads Queensland (TMR) is charged with the establishment and maintenance of the road network, significant portions of which are composed of stabilised granular pavement layers. Due to the vastness of the state, TMR operations are divided into nine jurisdictions including Far North Queensland, North Queensland, Central Queensland, Mackay/Whitsunday, Wide Bay/Burnett, Downs South West, North Coast, Metropolitan and South Coast regions. The widely varying prevalent source materials, environmental conditions and historical approaches across the state have purportedly given rise to regional stabilisation practices. As a result, stabilisation technologies with a history of satisfactory utilisation are indiscriminately selected and unfamiliar or previously unsuccessful technologies receive little consideration. Development of a systematic approach for selection of stabilisation technologies, based on site-specific conditions such as material availability, climate, environment and traffic, is critical to ensuring efficient practice in Queensland.

Over the past two decades TMR has played a leading role in the development of pavement stabilisation technology in Australia. Much work has been done in the past to optimise different forms of stabilisation for the Queensland roadbed environment. However, recent evidence suggests that the methodology for selecting the ideal stabilisation technology, in addition to mixture proportioning, structural design and construction techniques, varies across the state and is heavily influenced by historical local practice. Properly designed and implemented stabilised layers can significantly reduce the cost of pavement construction and/or rehabilitation by reducing the required quantities of higher-quality asphalt and Portland cement concrete. The effectiveness and value for money of different stabilisation techniques, including cementitious modification and foamed bitumen bounding, is greatly debated within the pavement engineering community. Research, development and long-term monitoring are needed to establish the most efficient (cost, construction and maintenance) stabilisation solutions under various climatic, environmental and operational conditions.

There may be significant cost savings to be realised by standardising the additive selection, design, construction, and maintenance practices for stabilised pavement layers. There are also significant capability development needs, given the loss of experienced practitioners who traditionally managed the risks associated with these works through their own personal knowledge. It is consequently proposed that the long-term performance of Queensland roads can be enhanced through improved decision making, provisioning and management of pavements incorporating stabilised structural layers.

The investigation of *Stabilisation Practices in Queensland* is a multiyear effort to allow for evaluation of the most commonly utilised stabilisation technologies. The planned program of investigation is presented in Table 1.1. Plant-mixed cement modified base (PM-CMB) and in situ mixed foamed bitumen stabilised (I-FBS) base technologies were selected by the National Asset Centre of Excellence (NACOE) board of directors for investigation during the 2013–15 program cycle. The general investigation format included identification, selection and historical review of representative sections, in addition to validation of physical assessment methodology in 2013–14. The 2014–15 effort included detailed evaluation and characterisation of stabilised materials, investigation of design, laboratory and in-service performance relationships and analysis of relative economic factors.

Table 1.1: Program for investigation of stabilisation practices in Queensland

Material Type	Stabilising Agents	Unbound	Subgrade	Modified		Bound	
		In situ	In situ	In situ	Plant-mixed	In situ	Plant-mixed
Granular	Aggregate	2018–19[2]					
Cementitious	Portland cement				2013–15[1]	2015–16[1]	2016–17[1]
	Portland cement/Fly ash				2013–15[1]	2015–16[1]	2016–17[1]
	Portland cement/Slag				2013–15[1]	2015–16[1]	2016–17[1]
	Portland cement/Fly ash/Slag				2013–15[1]	2015–16[1]	2016–17[1]
Pozzolanic Blends	Lime		2017–18[1]				
	Lime/Fly ash		2017–18[1]	TBD[1]	TBD[1]	TBD[1]	TBD[1]
	Lime/Slag		2017–18[1]	TBD[1]	TBD[1]	TBD[1]	TBD[1]
	Lime/Fly ash/Slag		2017–18[1]	TBD[1]	TBD[1]	TBD[1]	TBD[1]
Bituminous	Bitumen						TBD[2]
	Bitumen (emulsion)					TBD[2]	TBD[2]
	Bitumen (foamed)					2013–15[1]	2016–17[2]
Synthetic Polymer	Polymer		2017–18[2]	TBD[2]	TBD[2]		
	Polymer/Lime		2017–18[2]	TBD[2]	TBD[2]		

1 Queensland standard practice is well documented.
2 New technology and/or standard practice is not well established.

1.1 Purpose

The purpose of this project was to provide technical guidance on the ideal climatic, environmental and operational conditions to maximise the value for money of PM-CMB and I-FBS base stabilisation technologies utilised along the Queensland state-controlled road network. This includes production of a contract report benchmarking *Stabilisation Practices in Queensland*, in addition to outlining the development or amendment of existing TMR technical notes and/or technical specifications (where required) to transition best practice engineering learnings into practice.

1.2 Objectives

The principal outcome for this and future *Stabilisation Practices in Queensland* projects includes the confirmation, modification or creation of technical guidance regarding:

- which stabilisation treatment is most appropriate for a given budget, material characteristic, subgrade, environment, traffic, resilience and performance requirement
- whether stabilised materials are cost-effective alternatives to unbound granular and asphalt concrete layers for Queensland traffic and environmental conditions
- best practice engineering for Queensland - investigating the sensitivity of stabilised layer performance characteristics (e.g. shrinkage, strength, and modulus) to material properties and design practice.

1.3 Approach

The objective to develop technical guidance for selecting the best value for money stabilisation technology relative to Queensland roadbed conditions was accomplished through:

- reviewing available literature to determine current best practice – cementitious modification is discussed in Section 3 and foamed bitumen stabilisation in Section 4

- acquiring and summarising stabilised pavement inventory and condition data – Section 5
- selecting pavement sections representative of standard practice in Queensland – Section 6
- conducting visual condition, structural capacity and material property assessments – Section 7, Section 8 and Section 9 respectively
- investigating relationships between inventory, condition and performance data – Section 10
- evaluating the relative cost factors associated with PM-CMB and I-FBS base stabilisation technologies – Section 11.

Recommendations for modification of current practices, based upon learnings resulting from the above exploratory approach, are presented in Section 12.

2 STABILISATION TECHNOLOGIES

Stabilisation, in the context of pavement materials, refers to the process of modifying the engineering properties of soil or aggregate through the addition of fixed amounts of a stabilising agent. Stabilisation is remarkable in that the addition of a small amount of stabilising agent results in dramatic property changes (Lay & Metcalf 1983). Engineering properties that can be improved through stabilisation include modification of particle size distribution, controlling plasticity, increasing bearing capacity, reducing moisture sensitivity, enhancing workability and decreasing permeability. Additionally, stabilisation processes can reduce in situ moisture content and bind particles together to form a more resilient structure (Gray et al. 2011).

Stabilisation can provide significant economic and environmental benefits. Benefits arise from the utilisation of otherwise unsuitable material that no longer has to be carted and disposed, and reduction or elimination of required high-quality virgin aggregate, in addition to the associated production and transportation (Wilmot 2006). Other benefits of stabilisation include extending pavement service life and reducing whole-of-life (WOL) cost (Austroads 2006a). Stabilising agents typically represent half of the total cost of pavement layer stabilisation. Significant immediate and long-term cost savings can be realised through careful selection, design and construction of stabilising technologies (Austroads 2002c).

Stabilisation is a key technology for delivery of road pavements in Australia due to the extensive network length, shortage of quality aggregate materials and low taxpayer base. However, the use of stabilisation technologies varies considerably between the different states and territories (Wilmot 2006). Variations in practice, including treatment selection, mixture proportioning, structural design and construction, typically stem from different prevalent materials, varying environmental conditions and alternative historical approaches.

2.1 Stabilising Agents

Common stabilising agents include granular materials, Portland/modified cements, lime and bitumen. Other commonly used additives include fly ash, granulated blast furnace slag (GBFS), chemical agents and various combinations of the above. Stabilised materials are divided into four categories depending on the incorporated stabilising agent and developed performance attributes including subgrade, granular, modified and bound materials (Austroads 2006a).

2.1.1 Subgrade Stabilisation

Subgrade stabilisation refers to the in situ mixing of lime and/or cement with natural foundation soils to improve strength, stiffness and reduce potential for moisture-induced volume change (Austroads 2006a). Subgrade stabilisation is most effective for weaker materials with California Bearing Ratio (CBR) less than 8% (Wilmot 2006).

2.1.2 Granular Stabilisation

Granular stabilisation refers to the mixing of high-quality aggregate materials such as crushed rock, gravels and fine-grained sands with a natural material to improve strength, stiffness and resistance to aggregate breakdown (Austroads 2006a).

2.1.3 Modified Stabilisation

Modified stabilisation refers to the addition of small amounts of lime, Portland cement/cementitious blends or chemical stabilising agents to increase stiffness, improve strength, and reduce moisture susceptibility while maintaining the performance characteristics of an unbound granular material (Austroads 2006a).

2.1.4 Bound Stabilisation

Bound stabilisation refers to the blending of significant amounts of Portland cement, cementitious blends or bitumen combinations with granular materials to increase stiffness and tensile strength capacity (Austroads 2006a).

2.2 Characterisation

Laboratory testing is required to determine stabilising agent/aggregate compatibility, optimum stabilising agent content and to develop structural design parameters (Austroads 2006a). Unconfined compressive strength (UCS), indirect tensile strength (ITS) and CBR are commonly used to characterise the behaviour of stabilised materials for design, construction quality control and performance modelling (Gray et al. 2011).

The structural pavement layers (base and subbase) of sealed roads typically employ either the modified or bound stabilisation categories. The Austroads (2006a) *Guide to Pavement Technology-Part 4D: Stabilised Materials*, defines modified materials as having stabilising agent contents between 1 and 3% by mass and 28-day UCS between 0.7 and 1.5 MPa. Modified materials exhibit behaviour similar to unbound granular materials and should be characterised and modelled according to similar methodologies. Bound materials are defined as having stabilising agent contents greater than 3% and 28-day UCS greater than 1.5 MPa (Austroads 2006a). Bound materials exhibit significantly increased stiffness that requires consideration in design due to propensity for shrinkage and fatigue cracking. The interface between modified and bound materials was observed at an approximate resilient modulus (M_r) value of 1500 MPa (Gray et al. 2011).

Currently available methods for characterising stabilised materials are not based on actual performance data. Strength parameters such as UCS, ITS or CBR are commonly used to quantify the degree of modification/binding. However, these measures do not adequately replicate in-pavement conditions. If a material characterised as bound does not develop sufficient strength and durability, rapid structural deterioration can occur. If a material designated as modified develops excessive tensile capacity and stiffness representative of bound materials, fatigue cracking can develop. The result of incorrectly characterising stabilised materials is costly unplanned maintenance or rehabilitation. The benefits of incorporating stabilisation technologies are lost if the materials are not properly characterised and justified in the design methodology (Gray et al. 2011).

2.3 Construction

The production of stabilised pavement materials is accomplished using either centralised mixing plant (plant-mixed) or mobile mixing plant (in situ) (Austroads 2006a). Plant-mixed stabilised materials are generally produced near the source of the parent material (AustStab 2012). The parent material, stabilising agent and small quantities of water are blended in a fixed pugmill mixer, providing a high degree of uniformity. In situ stabilisation is conducted at the project site and typically involves preparation of the existing material, metered application of stabilising agent and mixing. Application and mixing of dry stabilising agents are conducted separately while liquid stabilising agents are automatically applied and mixed using specialised recycling machines. In situ stabilisation requires careful monitoring to achieve a high level of consistency. Regardless of mixing technique, the subsequent construction processes include compaction, finishing and curing, as dictated by the selected stabilising agent and stabilisation category.

2.4 Performance

Stabilised materials provide greater strength, stiffness and durability compared to unbound granular materials. However, stabilised materials also exhibit increased shrinkage cracking potential compared to untreated aggregates (Lay & Metcalf 1983). The improved performance characteristics of stabilised materials degrade over time to a level similar to unbound granular material as a result of cracking (Dunlop 1980). The degradation in structural capacity is indicated by the M_r and is typically high following the initial curing period and degrades under traffic (Lay & Metcalf 1983).

The controlling failure modes for stabilised pavements vary according to stabilising agent and stabilisation category. The primary failure mode for modified materials is permanent deformation characterised by rutting and shoving. Rutting results from excessive deflection of the subgrade and overlying pavement layers as a result of repeated traffic loading. Shoving develops in the upper pavement layers as a result of heavy vehicle loading and insufficient strength or an unstable foundation. The primary failure modes for bound materials include cracking, characterised by longitudinal, transverse, block or densely interconnected (crocodile) fracturing of the pavement surfacing, and permanent deformation. Cracking distresses manifest at the pavement surface as a result of varying moisture or thermal gradients, structural overload and/or fatigue of the bound layer. The origins of permanent deformation in bound materials are similar to those for modified materials.

2.5 Stabilisation in Queensland

Stabilised pavement materials in the TMR system are generally classified as either modified granular, cemented (Category 1 or Category 2) or foamed bitumen. Modified granular materials are combinations of granular material and small quantities of stabilising agent that are considered to behave similarly to unbound granular materials with improved material properties, such as reduced moisture sensitivity, higher strength and stiffness, reduced permeability, reduced erodibility and reduced sensitivity to variable particle size distribution and plasticity (TMR 2009). Cemented materials are combinations of granular materials, cementitious binder and water that form a stiff, bound layer when cured. Cemented materials are further subdivided into Category 1 and Category 2 depending on the parent material quality, stabilising agent content and magnitude of developed strength and stiffness (TMR 2009). Foamed bitumen is a combination of bitumen, a secondary stabilising agent (typically lime), and high-quality granular material. The bitumen binds the granular particles together increasing the strength and stiffness and reducing the permeability and erodibility. Stabilised pavement layers are typically provisioned in Queensland where:

- significant material property improvements can be achieved with relatively low stabilising agent application rates
- additional structural capacity is required and geometric constraints prohibit thick granular layers
- potential for moisture infiltration is high due to geometric constraints, lack of or insufficient drainage provisions, proximity to the watertable or likely inundation (floodway).

3 CEMENTITIOUS MODIFICATION

The objective of cementitious modification is to amend the undesirable properties of problem soils or nonconforming aggregate so that they can be used in a particular engineering application (Garber, Rasmussen & Harrington 2011). Cementitious modification is commonly pursued to improve particle size distribution, reduce plasticity or increase stability of a construction platform. Other improvements resulting from cementitious modification include increased structural and volumetric stability. Cementitious modification is typified by the treatment of soil or aggregate with small amounts of Portland cement and/or supplementary cementitious material (SCM) to improve the properties of an otherwise unsuitable material (ACI 2009). Cementitious modification improves specific properties of the parent material, but does not typically increase the structural capacity (Unified Facilities Criteria 2004).

The addition of small amounts of cementitious stabilising agent provides reduced moisture susceptibility, improved shear strength and cohesion, in addition to increased bearing capacity, durability and stability without increasing tensile strength and stiffness (AustStab 2012). Cementitious modification does not provide the level of engineering property improvement typically expected for cementitiously bound materials. Nevertheless, the potential for shrinkage cracking is also reduced (Garber, Rasmussen & Harrington 2011). Low stabilising agent contents minimise internal cracking potential and allow the material to retain the flexibility of an unbound granular pavement layer (Dunlop 1980). Improvements resulting from cement modification are permanent, making the technology an effective tool for producing strong, durable and sustainable pavements (Halsted 2011). Cementitious modification can also be utilised to rapidly ‘dry out’ aggregates prior to construction (Austroads 2002c).

While cementitious modification facilitates the provision of strong, durable and sustainable pavements, the approach is only effective in pavement layers that are properly designed, constructed and maintained. Cementitious modification provides economic benefits and longer service life compared to conventional unbound aggregate. Additionally, it is a sustainable engineering practice, reducing waste and preserving virgin aggregate sources, minimising the transport of materials, increasing the stability of construction platforms and reducing the volume of required overlying materials (Garber, Rasmussen & Harrington 2011). The use of cementitiously modified material should increase in the future with the growing scarcity of high-quality virgin aggregates and increasing transportation costs (Dunlop 1980).

3.1 Applications

Cementitious modification is typically adopted in pavements to improve marginal aggregate material properties to meet specification requirements (ACI 2009). Cementitiously modified pavement base (CMB) incorporates small amounts of stabilising agent to improve an unacceptable aggregate for use as base course. Incorporation of CMB does not reduce the required thickness of structural layers. However, the process does allow for the use of otherwise unacceptable materials. The use of limited supply high-quality virgin aggregate is non-sustainable when superior base courses can be developed using inferior aggregates and small amounts of stabilising additive (Dunlop 1980). Cementitious modification also eliminates the need for the removal and replacement of marginal or unsound materials, saving time and money (Garber, Rasmussen & Harrington 2011).

Portland cement and/or SCM are effective stabilising agents for a range of materials and applications (AustStab 2012). Granular materials with considerable silt, clay and/or high plasticity fines content are typically selected (Garber, Rasmussen & Harrington 2011). Materials best suited for cementitious stabilisation are well-graded, granular materials free of organics or other deleterious materials (Austroads 2002c). The modification of fine-grained materials (> 35% passing

0.075 mm sieve opening) targets reduced plasticity and volume change potential in addition to increased bearing capacity and stability. The modification of coarse-grained materials (< 35% passing 0.075 mm sieve opening) attempts to reduce the proportion of substandard fines to meet base/subbase specification requirements (Halsted 2011).

Cementitious modification can be used to increase cohesion and reduce moisture susceptibility in base applications (AustStab 2012). However, coarse aggregate can become lightly or fully bound even when small stabilising agent contents (< 3% by mass) are used (Gray et al. 2011). Materials at the boundary between modified and bound characterisation exhibit variable mechanical behaviour and can suffer from fatigue and/or structural overload failure modes. Modified materials develop networks of closely spaced cracks due to drying shrinkage, but not wide-spaced 'open' cracks like bound materials (AustStab 2012). The careful selection of cementitious stabilising agent content is required to ensure the modified material conforms to design assumptions.

3.2 Materials

The type, quality and proportion of materials composing CMB significantly influence the performance characteristics. The primary constituent materials of CMB include soil or aggregate, Portland cement and/or SCM and water. Chemical admixtures are typically not used but can be included to improve handling or modify working time.

3.2.1 Soil or Aggregate

The parent material for cementitious modification can include coarse and/or fine aggregate in addition to industrial by-products such as foundry sand, bottom ash or boiler slag. Coarse aggregates include naturally occurring and manufactured rocks with nominal maximum aggregate size (NMAS) greater than 2.36 mm. Fine aggregates are naturally occurring or manufactured materials with NMAS less than 2.36 mm. Industrial waste products can range in size but are typically on the order of fine aggregate. Care should be observed when employing industrial by-products as the chemistry of the materials can significantly impact the cementitious hydration process. For a fixed stabilising agent content, strength increase is greater for coarse-grained compared to fine-grained materials. Cement-stabilised materials composed of fine-grained soils also exhibit greater shrinkage potential, but with smaller crack widths and intervals, compared to coarse-grained aggregates (ACI 2009).

3.2.2 Cementitious Binder

The modification of soil or aggregate is defined by chemical reactions between clayey minerals and calcium that is provided by the cementitious binder (Halsted 2011). The cementitious stabilising agent is the active agent improving cohesion, reducing plasticity and amalgamating fines. Common cementitious stabilising agents include Portland cement and/or SCM. Blends of Portland cement and SCM (GB) are preferred to straight general purpose (GP) cement in stabilisation works due to slower setting times (increased working time and reduced cracking potential), recycling of waste materials (sustainable practice) and reduced cost (AustStab 2012). The effectiveness of the cementitious stabilising agent is determined by the diffusion of calcium and hydroxyl ions in addition to the moisture content and soil particle size distribution (Lay & Metcalf 1983). Lime should be considered in place of cementitious materials for fine-grained plastic soils (AustStab 2012).

Portland cement

Portland cement is a fine, hydraulic powder composed of calcium, aluminium, ferrite and gypsum. The hydration of Portland cement by water forms calcium-silicate-hydrate (CSH) and calcium-aluminium-hydrate (CAH) that bond clayey particles, forming larger effective size particles (Halsted 2011). GP and GB cements are principally used for stabilisation processes in Australia (Austroads 2006a) and should conform to Australian Standard AS 3972, *General Purpose and Blended Cements* (Standards Australia 2010).

Supplementary cementitious material

SCMs are alternatives to traditional GP cement that provide greater economy and ease of handling (Austroads 2009a). SCMs react slowly, providing increased working time but also greater long-term strength gain and reduced cracking potential (AustStab 2012). The most commonly used SCMs include fly ash and granulated blast furnace slag (GBFS). Fly ash is a fine granular material produced as a by-product of coal combustion. GBFS is also a granular material but of greater size than fly ash. GBFS is produced as a result of iron ore smelting. SCMs used in pavement applications should conform to AS 3582.1, *Supplementary Cementitious Materials for Use with Portland and Blended Cement-Fly Ash* (Standards Australia 2008), and AS 3582.2*, Supplementary Cementitious Materials for Use with Portland and Blended Cement – Slag – Ground Granulated Iron Blast-furnace* (Standards Australia 2001).

3.2.3 Water

Water is essential to the modification process, igniting hydration and contributing to workability. Mixing water should be potable and free of deleterious substances such as salts and other fine minerals. Organic material, sugars, and sulphates should also be avoided due to the influence on reaction time (AustStab 2012).

3.2.4 Admixtures

Chemical admixtures are not typically utilised in cementitious modification, but can be used to extend/shorten working time, increase air void content and/or decrease unit weight.

3.3 Hydration Process

The introduction of water to cementitious material activates hydration and the formation of crystal lattices. CSH and CAH generate bonds between fine particles, improving the overall particle size distribution by forming effectively larger size particles (Halsted 2011). Hydrated lime is also produced (30% by mass of cement) and reacts with available pozzolans in a secondary reaction to form additional cementitious crystals. Another form of secondary modification occurs when free calcium from Portland cement reacts with dissolved silica and alumina from clayey minerals to form additional CSH and CAH (Halsted 2011). Initial reactions occur rapidly and secondary reactions occur over time, similar to the processes of subgrade stabilisation using lime (AustStab 2012). Reactions are temperature sensitive, with reaction processes accelerating with increasing temperature.

The primary processes defining cementitious modification include cation exchange, particle restructuring, cementitious hydration and pozzolanic reaction (Halsted 2011). Cation exchange involves replacing monovalent (sodium) cations on the surface of clay particles with free calcium ions from Portland cement. Particle restructuring refers to the processes of flocculation and agglomeration in which the particle orientations are altered from a flat, parallel structure to an edge-to-face orientation with bonding at the intersection of particles. Cementitious hydration is as defined above. The pozzolanic reaction process involves the reaction of calcium hydroxide, from cementitious hydration, with free silica and alumina from clayey minerals to form additional CSH and CAH (Halsted 2011).

3.4 Properties

The properties of CMB are determined by the parent aggregate, type and relative amount of stabilising agent, moisture content, degree of compaction, uniformity of mixing, curing conditions and the mixture age (ACI 2009). Property modification is a permanent process and can include reducing plasticity, decreasing volume change potential and increasing bearing capacity (Halsted 2011). Soil and aggregate become caked or slightly hardened with the addition of small quantities of cementitious material. However, the mixture still functions as a soil or aggregate, but with improved properties (Halsted 2011).

It is difficult to assess the true benefit of cementitious modification in pavement applications due to uncertainty regarding structural properties (Dunlop 1980). However, material property improvement can be quantified by measuring the strength, modulus, moisture sensitivity, durability, permeability and workability. Other properties that can, and should, be evaluated in the laboratory include particle size distribution, unit weight and stabilising agent content. Laboratory testing should always be performed using the specific materials intended for a particular application (ACI 2009). The production of laboratory testing specimens should be accomplished using standard, as opposed to modified, compactive effort (Austroads 2002c).

3.4.1 Compressive Strength

UCS is the most widely used method for characterising the strength of cementitiously stabilised materials (ACI 2009). UCS is typically measured to provide an indicator of modulus for structural design and also for field quality control. Higher compressive strength correlates directly with durability and exhibits a reverse correlation with shrinkage (Scullion et al. 2005). UCS increases with increasing density, stabilising agent content and curing time. Significant variance between the compressive strength of laboratory and field-produced specimens can occur due to differences in curing conditions (Austroads 2002b).

The UCS of CMB can be determined in accordance with TMR test method Q115, *Unconfined Compressive Strength of Compacted Materials* (TMR 2013c). In this method, a cylindrical specimen of approximately 100 mm diameter and 115 mm height is subjected to increasing vertical axial loading until failure. Cementitiously modified materials should not be soaked prior to UCS testing, as is the standard procedure for bound materials (Austroads 2013a). The UCS is the maximum load achieved before failure divided by the cross-sectional area. Due to large variations in parent material and stabilising agent type, chemistry and content, it is not possible to provide unique strength requirements based upon UCS. Specifying a maximum strength, as opposed to minimum strength gain, may be a more effective approach depending upon the intended application (Austroads 2002c).

3.4.2 Tensile Strength

The tensile strength of CMB defines the ability to maintain structural consistency when subjected to bending stresses. Tensile strength also dictates the mechanical behaviour of CMB under vehicular traffic loading. Stabilised materials transition from unbound to bound structures with increasing tensile strength. The tensile strength of cementitiously modified material should be less than 80 kPa when cured for seven days at 20 °C to maintain unbound material mechanical characteristics (Dunlop 1980). Tensile strength is commonly determined from flexural beam or indirect tension testing and is greatly influenced by the testing method due to stress concentrations and variations in tensile versus compressive modulus (ACI 2009). Based on extensive testing of subgrade and unbound granular materials, CMB should have a maximum flexural strength of 100 kPa, maximum ITS of 150 kPa and maximum UCS of 1.0 to 1.2 MPa (Austroads 2013a).

Flexural beam

The flexural strength of CMB can be determined in accordance with ASTM D1635, *Standard Test Method for Flexural Strength of Soil-Cement using Simple Beam with Third-point Loading* (ASTM International 2012). In this method a prismatic beam of CMB with dimensions 76 mm width by 76 mm height by 290 mm length, is loaded at the third-points with an increasing vertical load until failure. Failure is indicated by fracture of the beam within the middle third of the span length. The flexural strength is calculated according to Equation 1.

$$R = \frac{PL}{bd^2} \qquad 1$$

where

R	=	flexural strength (MPa)
P	=	maximum load before fracture (N)
L	=	beam span length (mm)
b	=	width of beam (mm)
d	=	height of beam (mm)

Flexural strength is typically approximated from UCS as a result of the limited availability of third-point loading equipment in addition to extended testing times. The ratio of flexural to compressive strength is higher (≈ 33%) for low-strength mixtures and lower (≈ 20%) for high-strength mixtures (ACI 2009). Matanovic (2012) found the ratio of flexural strength to UCS ranged from 30% to 35% for CMB incorporating TMR Type 2.1 aggregates.

Indirect tension

The ITS of CMB can be determined in accordance with European Committee for Standardization (CEN) test method 13286–42, Unbound and Hydraulically Bound Mixtures – Part 42: Test Method for the Determination of the Indirect Tensile Strength of Hydraulically Bound Mixtures (CEN 2003). In this method a cylindrical specimen with dimensions of 100 mm diameter and height ranging from 80 mm to 200 mm is subjected to an increasing strip load along the circumference of the specimen until fracture. The loading configuration develops complex multi-axial tensile and compressive stresses within the specimen. The ITS is calculated according to Equation 2.

$$R = \frac{2F}{HD} \qquad 2$$

where

R	=	indirect tensile strength (MPa)
F	=	maximum load before fracture (N)
H	=	height of specimen (mm)
D	=	diameter of specimen (mm)

Determination of the ITS of CMB is not typically pursued due to the structural instability of the material. Therefore, the material is most commonly only characterised according to UCS. The ratio between ITS and UCS is material dependent, but typically ranges from 10–12%.

3.4.3 Modulus

Stiffness as indicated by M_r is a fundamental input for structural design. Resilient modulus increases with increasing stabilising agent content and density and decreases with increasing moisture content. Similarly to UCS, the modulus of field specimens is typically higher than laboratory-prepared specimens. However, modulus results exhibit reduced variability compared to UCS testing results (Austroads 2002b). The M_r of cementitiously modified materials can be determined in accordance with TMR test method Q139, *Resilient Modulus of Stabilised Materials (Indirect Tensile Method)* (TMR 2013f). The method was developed for foamed bitumen stabilised (FBS) materials but can also be applied to cementitiously modified materials. In this method a strip load is repeatedly applied to the circumference of a 150 mm diameter 90 mm height cylindrical specimen until a recoverable horizontal deformation of approximately 50 microstrain is reached. The M_r is calculated according to Equation 3.

$$M_r = \frac{P(\upsilon + 0.27)}{Hh_c} \tag{3}$$

where

M_r	=	resilient modulus (MPa)
P	=	maximum load (N)
v	=	Poisson's ratio
H	=	height of specimen (mm)
h_c	=	recovered horizontal deformation (mm)

While it is preferred to directly measure the Mr of CMB, the secant modulus determined from the double punch test can also be used as a conservative estimate (Dunlop 1980).

3.4.4 Moisture Sensitivity

Drying shrinkage

Cementitiously modified materials undergo shrinkage as a result of self-desiccation during hydration and loss of water during drying (Scullion et al. 2005). The magnitude and rate of drying shrinkage is influenced by stabilising agent type and content, moisture content, parent material characteristics and the curing environment (Austroads 2002c). Drying shrinkage is typically not specified but dictates the transition from a compacted material to a trafficable layer. Drying shrinkage can be reduced by use of nonreactive parent materials with low plasticity index (PI), linear shrinkage and fines content, efficient mixing, compacting slightly dry of optimum moisture content (OMC), use of water-reducing or set-retarding chemical admixtures and the use of slow-setting cements or SCMs (Austroads 2002c).

The shrinkage potential of cementitiously modified materials is indicated by measuring the linear shrinkage of the fine (< 0.075 mm sieve opening) component of the parent material in accordance with TMR test method Q106, *Linear Shrinkage* (TMR 2013a). In this method, linear shrinkage is determined by measuring the reduction in length of a compacted beam specimen with top width of

25 mm, bottom width of 20 mm, height of 15 mm and length of 150 mm, which has been subjected to oven drying. The linear shrinkage is the average percent reduction in specimen length.

Swell potential

The volumetric stability of CMB used in subbase applications should be characterised according to PI, ideally less than five, or CBR, the minimum value of which should satisfy the pavement design requirements (Dunlop 1980). Minimum CBR values typically range from 50% to 100% for subgrade and base course applications respectively. Cementitious modification has a greater effect on swell/expansion potential compared to PI and therefore, the CBR swell test is a better indicator of degree of enhancement (Halsted 2011).

Plasticity greatly influences the performance of cement-stabilised materials. More plastic materials provide greater permanent strain and rutting potential, reduced M_r and improved resistance to erosion (Symons & Poli 1999). The PI of cementitiously modified material can be determined in accordance with TMR test method Q105, *Plastic Limit and Plasticity Index* (TMR 2010a). The PI is the difference between the liquid limit and the plastic limit of the fine proportion (passing 0.425 mm sieve opening) of the parent material. The liquid limit is the relative moisture content at which the material transitions from a plastic to a liquid state. The plastic limit is the relative moisture content at which the material transitions from a semisolid to a plastic state.

Laboratory-soaked CBR testing can be used to characterise both the strength and swell potential of modified materials. The CBR and swell potential of cementitiously modified materials can be determined in accordance with TMR test method Q113A, *California Bearing Ratio (Standard Compactive Effort)* (TMR 2013b). In this method the CBR of the portion of the parent material passing the 19.0 mm sieve opening is determined by measuring the vertical force required to penetrate a 1932 mm^2 plunger 5.0 mm into a compacted specimen confined in a 150 mm diameter by 175 mm height rigid mould. Swell is determined by measuring the height of the compacted specimen before and after submersion in water for 96 hours.

3.4.5 Durability

Durability, or resistance to degradation as a result of variable temperature and moisture environments, is not typically specified but significantly influences performance. Long-term durability is determined by the mixture proportioning and construction practices used. Resistance to degradation correlates with fines content, stabilising agent content and UCS and is inversely related to density. Durability is also influenced by construction practice (curing and compaction) and should be specifically assessed for low stabilising agent content materials (Austroads 2002b). A quantitative test method for measuring the durability of cementitiously modified materials does not currently exist. UCS and moisture affinity (capillary rise) are commonly used to indicate long-term durability. Higher UCS values may improve durability, but alone do not ensure that adequate durability is provided (Scullion et al. 2005).

Moisture affinity can be determined in accordance with TMR test method Q125D, *Capillary Rise of Stabilised Material* (TMR 2013d). In this method the moisture affinity of compacted 100 mm diameter and 115 mm height specimens is determined by submerging the lower 10 mm of the specimen in water and measuring the vertical rise of moisture within the specimen over a 72-hour period. Capillary rise is reported as the ratio of moisture rise to total specimen height.

3.4.6 Permeability

Permeability defines the ease with which gases or liquids can flow through a material. Permeability has a significant influence on the long-term performance of pavement materials. Permeability is determined by particle size distribution, air void content and distribution and relative compaction of the CMB layer. The permeability of cementitiously modified materials can be determined in accordance with TMR test method Q125A, *Permeability of a Soil (Constant Head)* (TMR 2010b). In this method the permeability of the compacted specimen is determined by applying a constant 117 mm pressure differential to force water through the base of the specimen and out through a free surface at the top. The permeability is determined by measuring the volume of water passing through the specimen during a fixed time period.

3.4.7 Workability

Workability defines the ease with which the CMB can be placed, compacted and finished. The critical component for cementitiously modified materials is the working time. The workable time limit for cement-modified material is up to one day, compared to two to four hours for bound materials. Similarly to unbound granular materials, CMB can be immediately opened to construction traffic (Halsted 2011). The working time for cementitiously modified materials can be determined in accordance with TMR test method Q136, *Working Time of Stabilised Materials* (TMR 2013e). In this method the working time of compacted 100 mm diameter and 115 mm height specimens is determined by measuring the maximum dry density (MDD) and UCS after a predetermined conditioning time. The maximum working time is the conditioning time at which 80% of the MDD and design UCS is still achieved.

3.4.8 Particle Size Distribution

The distribution of particle sizes greatly influences the structural strength development in CMB (Dunlop 1980). Significant particle interlock within the parent material is required to resist deterioration and ensure long-term performance (AustStab 2012). Well-graded materials develop higher UCS, all other factors being equal (Symons & Poli 1999). Materials with high fines content exhibit greater potential for shrinkage cracking, erodibility and permanent strain compared to coarse-grained materials (Scullion et al. 2005).

The particle size distribution of the parent material can be determined in accordance with TMR test method Q103B, *Particle Size Distribution of Aggregate (Dry Sieving)* (TMR 1996a). In this method the distribution of particle sizes is determined by shaking of the blended material over a stack of metal sieves with decreasing opening size. The percentage of the specimen passing each of the discrete sieves is plotted relative to opening size to develop the particle size distribution curve.

3.4.9 Density

The relative dry density (RDD) of CMB significantly influences the long-term performance of the pavement. RDD greatly affects modulus, permeability and durability and should be established based on relative compaction using standard effort and a preliminary field trial prior to construction (Austroads 2002b). The MDD and OMC of a cementitiously modified material can be determined in accordance with TMR test method Q142A, *Dry Density-Moisture Relationship (Standard Compaction)* (TMR 2010c). In this method, the mass of material filling a fixed volume is determined by compacting in thin lifts using a drop force of 596 kJ/m^3. The in situ density and moisture content of CMB can be determined in accordance with TMR test method Q171, *Determination of the Moisture Content and Dry Density of a Soil Sample* (TMR 2013g). In this method, the dimensions and mass of a compacted specimen are determined before and after oven drying. The dry density is the unit weight after drying. The moisture content is the difference in specimen mass before and after oven drying.

3.4.10 Stabiliser Content

Stabilising agent content significantly influences the long-term performance of CMB. Sufficient stabilising agent is required to improve the properties of the material for use in a particular engineering application. However, too much stabilising agent can result in excessive shrinkage cracking and potential fatigue failure. The cement content of cementitiously modified materials can be determined in accordance with TMR test method Q116A, *Cement Content of Cement Treated*

Material (EDTA Titration) (TMR 1996b). In this method the combined calcium and magnesium contents of the parent material, cementitious stabilising agent and blended material are determined by ethylenediamminetetraacetic acid (EDTA) titration. The differences in the relative proportions of EDTA used are compared to determine the approximate cementitious stabilising agent content.

3.5 Material Specification

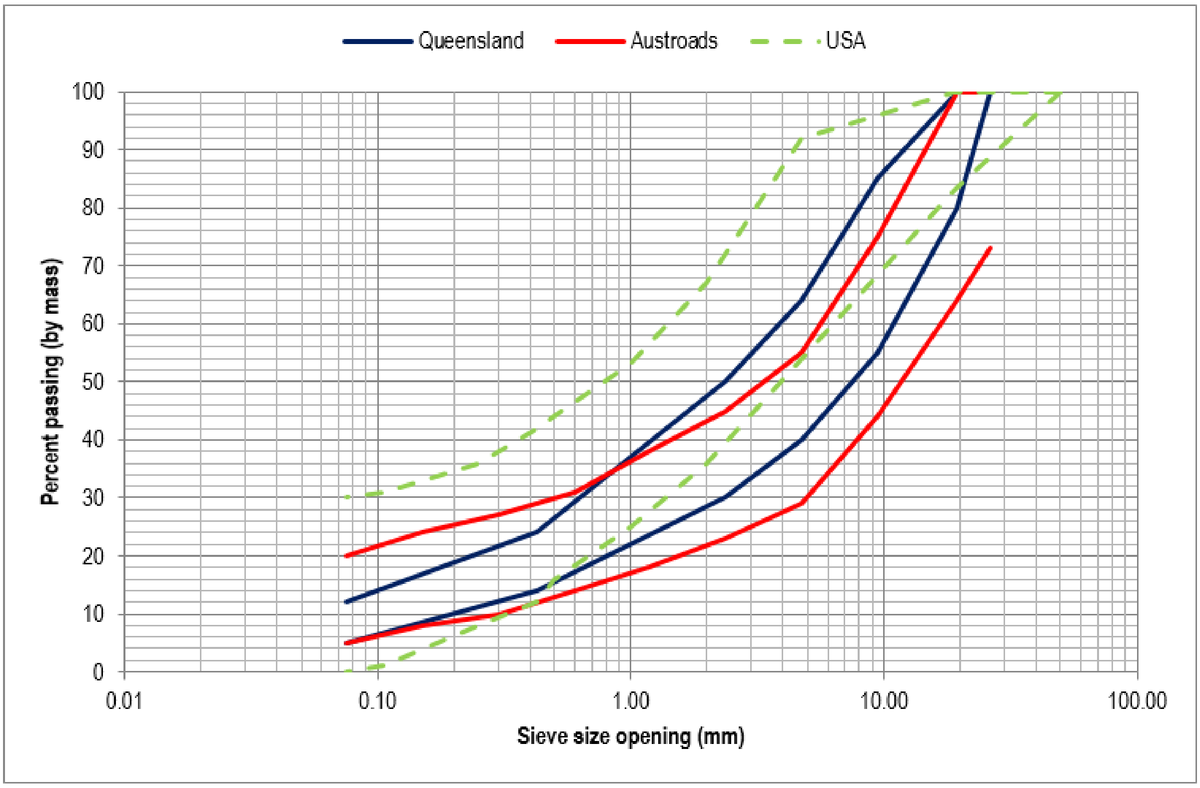
The common approach for specification of PM-CMB includes establishing property limits for the parent aggregate, binder and blended end product. However, the adopted testing methods and limits vary between road agencies both nationally and internationally. TMR has developed a comprehensive specification for PM-CMB as presented in Main Roads Technical Standard (MRTS) ET05C, *Plant-mixed Cement Modified Base (CMB)* (TMR 2012b). MRTS ET05C provides guidance on parent aggregate and binder selection, mixture proportioning, structural design and construction of PM-CMB.

3.5.1 Parent Material

Cementitious modification has been successfully undertaken for a wide range of soils and aggregates, but is optimally used for well-graded, low plasticity materials. As a result, material property requirements for parent materials typically include particle size distribution, fines ratio, and Atterberg limits. PM-CMB layers are regularly provisioned in high moisture environments; thus, measures of moisture sensitivity, such as linear shrinkage (LS), soaked CBR, and Texas triaxial testing, are also commonly specified. Particle size distribution limits for Queensland (MRTS ET05C) are compared to national (Austroads) and international, as represented by the American Concrete Institute (ACI) in the USA, best practice in Figure 3.1.

Sand (4.75–0.425 mm) content should generally be limited to 70% by mass to maximise strength development (Symons & Poli 1999). Sufficient fines (< 0.075 mm) content is also required to increase cohesion, modulus and shear strength, while limiting permeability. However, excessive (> 12% by mass) fines content can negatively impact structural stability (Symons & Poli 1999).

Figure 3.1: Comparison of recommended grading envelopes for parent material



Particle size distribution requirements for cementitiously modified material vary widely, but commonly 100% is required to pass the 37.5 mm sieve opening and a minimum of 30% must pass the 4.75 mm sieve (Halsted 2011). Materials meeting ACI (2009) requirements are finer than Austroads (2006a), with Queensland (TMR 2012b) materials bisecting the two sets of limits. The wide variation in national and international best practice results from the different methods of use. In Australia, CMB is typically surfaced with a thin sprayed bituminous seal. Therefore, the CMB layer requires sufficient coarse material to withstand the direct application of wheel loads, but also sufficient fines to provide a dense smooth surface for the bituminous seal to adhere to. Whereas, in the USA, CMB is typically used in subbase applications and is surfaced with high-quality asphalt or PCC. As a result, the performance and particle size distribution requirements are less stringent and generally conform to the maximum density line.

In addition to the distribution of particle sizes, the inherent properties of the parent material including liquid limit, PI, and LS significantly influence the success of modification. Additionally, in-service performance indicators, such as soaked CBR and Texas triaxial, are also commonly specified. The parent material property limits for a selection of national and international road agencies are provided in Table 3.1.

Table 3.1: Commonly specified parent material properties

Road agency	Maximum liquid limit	PI	LS	Fines ratio	Minimum soaked CBR	Texas classification number
TMR [(1)]	25	2 – 6	1.5 – 3.5	0.30 – 0.55	80	–
RMS [(2)]		0 – 10	–	–	–	0 – 3
VicRoads [(3)]		0 – 10	–	–	–	–
MRWA [(4)]	30	0 – 10	0 – 4	–	30	–

Road agency	Maximum liquid limit	PI	LS	Fines ratio	Minimum soaked CBR	Texas classification number
AustStab [5]	40	0 – 20	–	–	-	–
US DoD [6]	25	0 – 12	–	–	-	–
DTSA [7]		0 – 6	0 – 5	–	15	–

1 Source: Department of Transport and Main Roads Queensland (2012b).
2 Source: Roads and Maritime Services (2013).
3 Source: VicRoads (2008).
4 Source: Main Roads Western Australia (2012).
5 Source: AustStab (2011).
6 Source: Naval Facilities Engineering Command (2008).
7 Source: Committee of Land Transport Officials (1996).

Material property requirements are similar both nationally and internationally. Limits for maximum liquid limit range from 25–30%, maximum PI from 6–20% and maximum LS from 3.5–5.0%. PM-CMB is commonly provisioned in moisture-prone environments. Therefore, characterisation of performance in saturated conditions, such as those accomplished using soaked CBR or Texas triaxial testing, is commonly specified but limiting requirements vary widely.

3.5.2 Cementitious Binder

Cementitious stabilising agents utilised in Australia must conform to AS 3972 (Standards Australia 2010) for GP and GB cements, AS 3582.1 (Standards Australia 2008) for fly ash, and AS 3582.2 (Standards Australia 2001) for GBFS.

3.5.3 Mixing Water

Water used in the production of CMB is ideally of potable quality. However, nonpotable water has been successfully utilised, but must be clean and free of oil, acid, organic matter and other deleterious substances (TMR 2012b).

3.5.4 Modified End Product

Specification of properties for the blended end product is typically limited to stabilising agent content and/or UCS. Sufficient quantities of stabilising agent (> 1% by mass) are required to facilitate uniform distribution throughout the parent material. UCS limits for CMB typically include both upper and lower bounds to ensure sufficient improvement is achieved while minimising the development of tensile capacity. Common limits for a selection of national and international agencies are presented in Table 3.2.

Table 3.2: Range of specification limits for cementitiously modified material

Road agency	Material type	Binder content (%)	Curing time (days)	Unconfined compressive strength (MPa)	Max. design modulus (MPa)
TMR[1]	Modified	1.0 – 2.0	28	1.0 – 2.0	500
RMS[2]	Modified	–	28	1.0 (max)	450
VicRoads[3]	Modified	1.0 – 3.5	7	1.0 – 2.0	500
MRWA[4]	Modified	1.0 – 2.0	7	0.6 – 1.0	1000
	HCTCRB	1.9 – 2.1	7	1.0 (max)	1500
Austroads[5]	Modified	2.0 (max)	28	0.7 – 1.0	1000
US DoD[6]	CTSB	1.5 – 4.0	7	2.0 (min)	–
UKHA[7]	In situ CBGM	3.0 (min)	28	0.4 – 36	–

Road agency	Material type	Binder content (%)	Curing time (days)	Unconfined compressive strength (MPa)	Max. design modulus (MPa)
	Plant-mixed CBGM	2.0 (min)	28	0.4 – 36	–
DTSA[8]	C3	2.0 – 3.0	7	1.5 – 3.5	10 000
	C4	2.0 – 3.0	7	0.8 – 1.5	7000

1 Source: Department of Transport and Main Roads Queensland (2012b).
2 Source: Roads and Maritime Services (2013).
3 Source: VicRoads (2008).
4 Source: Main Roads Western Australia (2012).
5 Source: Austroads (2012).
6 Source: Naval Facilities Engineering Command (2008).
7 Source: Highways Agency (2009).
8 Source: Committee of Land Transport Officials (1996).

Degree of stabilisation (modified vs. bound) and shrinkage cracking potential are indicated by the maximum tensile strength of the material. Limiting the tensile capacity to the recommended 80 kPa for modified materials typically requires Portland cement contents less than 1% (Gray et al. 2011). For the selected specifications reviewed, minimum stabilising agent contents range from 1.0–3.0% and maximum values range from 2.0–4.0%. The UCS measured after 28 days of curing generally increases 1.0 MPa for each additional 1% of Portland cement (Lay & Metcalf 1983). Adequate distribution of stabilising agent at very low contents (< 1%) is difficult to achieve in practice, while usage of high contents (> 3%) can produce a fully bound pavement layer (Gray et al. 2011). Utilisation of SCMs allows for greater stabilising agent contents without the development of significant tensile strength.

Minimum specified UCS values range from 0.4–2.0 MPa and maximum values range from 1.0–3.5 MPa. The values for the Highways Agency in the UK (Highways Agency 2009) range from 0.4–36.0 MPa, but the specification includes both modified and bound materials. Typical design UCS values in Australia range from 1.0–2.0 MPa and are determined by the nature of the parent material and the type and amount of stabilising agent employed (Austroads 2012).

3.6 Mixture Proportioning

The objective of mix design is to proportion a material meeting strength and durability requirements with minimal shrinkage. Due to the increased cost of the cementitious stabilising agent compared to other mixture components, care should be taken in selecting both the type and quantity of stabilising agent for a given application (Austroads 2006a). Selecting the optimal design stabilising agent content involves balancing the competing efforts of providing adequate strength and durability without the development of excessive tensile capacity (Scullion et al. 2005). Understabilising produces a material with insufficient strength, durability or stability. Overstabilising increases the potential for wide shrinkage cracks that can propagate through the pavement surfacing.

Efficient mixture design should focus on proportioning constituent materials to meet the required level of modification measured in terms of strength and moisture sensitivity in addition to other application-specific criteria (Halsted 2011). The proportioning of constituent materials is typically accomplished using an iterative approach in which an initial stabilising agent content is selected based on historical practice or engineering judgement, the material properties are determined, the stabilising agent content is refined, and the process is repeated until a material conforming to the required specifications is produced. The properties of the parent material change incrementally with increasing cementitious binder content (AustStab 2012). The amount of stabilising should be selected so that the desired properties are modified without generating a hardened mass (ACI 2009).

Due to significant variations in material performance resulting from variations in aggregate type and size, stabilising agent type and content, pavement structure, traffic loading and environmental conditions, material combinations should be tested prior to use (Lay & Metcalf 1983). The mixture proportioning of stabilised materials should be based on a standardised laboratory testing program or, when facilities are not available, significant previous experience (Austroads 2006a). Laboratory testing can be used to determine optimum stabilising agent content, moisture content and density to meet the objectives of mixture design (Dunlop 1980). Most agencies use measurements of UCS for the selection of optimum design cementitious content (Scullion et al. 2005). However, selection of design stabilising agent content should also consider tensile strength, stiffness, moisture sensitivity, durability, permeability and workability properties. To simplify the design process without introducing unnecessary risk, only the critical and essential material properties, as dictated by the intended application, require determination (Dunlop 1980).

3.7 Structural Design

The structural design of flexible pavements focuses on the provision of sufficient thickness of layered materials to resist fatigue and permanent deformation (Matanovic 2012). Pavement materials are commonly separated into five distinct categories according to fundamental mechanical behaviour under traffic loading including unbound granular, modified granular, cemented, asphalt and concrete materials (Austroads 2013a). When stabilised materials conform to the definition of modified material, pavements can be designed and modelled as traditional unbound granular flexible structures (Dunlop 1980).

Modified materials meeting the Austroads classification (28-day UCS < 1.0 MPa; M_r < 1000 MPa) typically require stabilising agent contents less than 1% (Gray et al. 2011). Constructing modified materials that behave as stiffened unbound materials is difficult due to issues with uniformly distributing the small (< 1%) amount of stabilising agent. When cementitious stabilising agent contents are between 1.0% and 3.0%, the material should be modelled as 'lightly' bound. Matanovic (2012) observed that modified Type 2.1 specimens subjected to cyclic beam fatigue testing at 40% and 60% of the maximum flexural strength did not reach failure prior to the application of 500 000 loading cycles. Stabilisation with high (> 3%) stabilising agent contents produces bound materials, the characterisation and modelling of which should be accomplished according to Austroads methods (Gray et al. 2011). The design of lightly bound material layers can be accomplished using the current Austroads unbound granular methodology, but the process should also include a check of tensile stresses within the layer (Gray et al. 2011).

3.7.1 Mechanistic Approach

The mechanistic design of flexible pavements is commonly accomplished using multilayer linear elastic models. Materials are commonly described according to modulus and Poisson's ratio when modelled using linear elastic methods. Modified materials designed using a mechanistic approach should be characterised as cross-anisotropic (degree of anisotropy of 2.0) unbound granular layers with Poisson's ratio of 0.35, ITS less than 80 kPa, 28-day UCS less than 1.0 MPa and 700 MPa maximum M_r (TMR 2012b, Dunlop 1980, Austroads 2012).

Typical linear elastic structural design systems consider the primary failure mechanisms for unbound granular pavements, including the principal stresses within the granular layers and the vertical compressive strain at the top of the subgrade layer. The normal and shear stresses at a given location within the pavement structure resulting from idealised traffic loading are determined using the Boussinesq equations and are applied to determine the principal stresses and vertical compressive strain as shown in Equation 4 and Equation 5 respectively.

$$\sigma_{1,2,3} = \frac{(\sigma_z + \sigma_x) \pm \sqrt{(\sigma_z - \sigma_x)^2 + (2\tau_{xz})^2}}{2} \quad (4)$$

where

$\sigma_1, \sigma_2, \sigma_3$ = principal stresses

$\sigma_x, \sigma_y, \sigma_z$ = normal stresses in x, y, and z planes respectively

τ_{xz} = shear stress in the y-plane

$$\varepsilon_z = \frac{1}{M_r}\left(\sigma_z - \mu\sigma_x - \mu\sigma_y\right) \qquad 5$$

where

ε_z = vertical compressive strain

M_r = resilient modulus

$\sigma_x, \sigma_y, \sigma_z$ = normal stresses in x, y, and z planes respectively

μ = Poisson's ratio of modified material

The maximum principal stresses and compressive strains at critical locations within the pavement structure resulting from the idealised traffic loading are typically adopted as the controlling conditions for prediction of in-service performance.

3.7.2 Austroads

Part 2 of the Austroads *Guide to Pavement Technology* (2012) assumes modified materials have negligible tensile strength and therefore can be approximated as unbound granular materials. The structural design of unbound granular materials using the Austroads (2012) mechanistic approach includes:

- selection of trial pavement configuration
- characterisation of elastic parameters for subgrade, subbase and base layers
- determination of design standard axle repetitions (SAR)
- calculation of critical compressive strain at subgrade level
- determination of allowable SAR.

Determination of elastic parameters for the component pavement materials is ideally accomplished using laboratory testing. However, presumptive values can be utilised in the absence of test data. Determination of design SAR requires definition of service period, calculation or estimation of annual average daily traffic (AADT) and percentage of heavy vehicles, selection of growth factor and determination of number of axle groups per heavy vehicle. The equivalent standard axle (ESA) used in the calculation of design SAR is an 80 kN single truck axle with dual tyres.

The principal failure mode for modified material is permanent deformation as a result of shear and/or densification. The Austroads (2012) method does not include provisions for evaluation of shear or densification potential of unbound granular layers. The controlling failure mechanism is permanent deformation resulting from excessive deflection of the subgrade. It is important to note that when using the Austroads (2012) method, the stresses and strains within the unbound granular/modified pavement layers are not directly considered during the structural design process.

The maximum vertical compressive strain measured at the top of the subgrade layer resulting from repeated application of the ESA loading is the key failure criterion. The allowable SAR is calculated using the Austroads (2012) subgrade failure criterion as presented in Equation 6.

$$SAR_{Allow} = \left(\frac{9300}{\mu\varepsilon}\right)^7 \qquad (6)$$

where

SAR_{Allow} = allowable standard axle repetitions

$\mu\varepsilon$ = maximum subgrade strain (μm)

The initial trial pavement configuration is adjusted in an iterative manner until the allowable SAR exceeds the design SAR.

3.7.3 National Cooperative Highway Research Program

The National Cooperative Highway Research Program (NCHRP) *Mechanistic-Empirical Pavement Design Guide* (MEPDG) (2004) recommends that cementitiously modified materials used in pavement base applications should be designed and modelled as unbound granular layers and combined with other unbound layers for evaluation. The structural design of unbound granular materials using the MEPDG includes:

- selection of trial pavement configuration
- characterisation of elastic parameters for subgrade, subbase and base layers
- estimation of equivalent single, tandem, tridem and quad axle loads throughout the design period
- estimation of equilibrium moisture content throughout the design period
- determination of critical compressive strain for each combination of equivalent axle load and equilibrium moisture content
- calculation of permanent deformation at the pavement surface.

The MEPDG utilises the Jacob Uzan Linear Elastic Analysis (JULEA) multilayer elastic model to determine the compressive stresses and strains within the base/subbase layers, in addition to the top of the subgrade. The trial pavement configuration, equivalent axle loading and environmental conditions are utilised to predict the maximum compressive strains. The MEPDG does not provide recommended elastic properties for CMB. However, low quality soil cement in the MEPDG conforms to the Austroads (2012) definition of CMB, including Poisson's ratio ranging from 0.15 to 0.35 and elastic modulus ranging from 50 000–150 000 psi (350–1050 MPa) (NCHRP 2004). The mechanical response model presented in Equation 7 is used to estimate permanent deformation in each sublayer resulting from the design traffic and environmental conditions.

$$\delta_i = \left[\beta_1 \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^{\beta_2}} \varepsilon_{max} h\right] / N \qquad (7)$$

where

δ_i = permanent deformation in sublayer i (in.)

β_1 = layer calibration factor

$(\varepsilon_0/\varepsilon_r)$ = $10^{\left[0.15\left(e^{\rho^{\beta_2}}\right)+20.0\left(e^{(\rho/10^9)^{\beta_2}}\right)\right]/2}$

ρ = $10^9\left[-4.8929/1-(10^9)^{\beta_2}\right]^{1/\beta_2}$

β_2 = $10^{[-0.61119-(0.017638M_c)]}$

M_c = equilibrium moisture content (%)

ε_{max} = maximum compressive strain

h = design layer thickness (in.)

N = number of traffic repetitions

The β_1 value is approximately 1.673 for unbound granular base and subbase layers and 1.350 for the subgrade. M_c is calculated for each month of the design period using the enhanced integrated climate model (EICM). The EICM includes a database of historical climatic data and predicts temperature and precipitation variation within each subseason according to a normal distribution. The estimated equivalent single, tandem, tridem, and quad axles for each month of the design period are used to calculate the equivalent axle loading for each subseason according to a normal distribution. The M_r value used in the calculation is varied for each subseason based on the equivalent moisture content calculated using the EICM.

The principal failure mode for unbound granular pavements within the MEPDG is permanent deformation. The deformation exhibited at the surface of the pavement includes contributions from both the unbound granular and subgrade layers and is determined by summing the relative deformation (RD) of the granular layers and the subgrade. RD is determined as presented in Equation 8.

$$RD = \sum_{i=1}^{n} \delta_i h_i \qquad (8)$$

where

RD = permanent deformation in layer (in.)

n = number of sublayers

δ_i = permanent deformation in sublayer i (in.)

h_i = design thickness of sublayer i (in.)

The initial trial pavement configuration is adjusted in an iterative manner until the predicted surface deformation resulting from the design traffic loading and environmental conditions is within acceptable serviceability limits.

3.7.4 Committee of Land Transport Officials (COLTO)

The South African mechanistic design method (SAMDM) is recommended by the COLTO for the mechanistic design of flexible pavements. The modification of natural gravels (G4, G5, and G6) with relatively low quantities of cementitious stabilising agents is undertaken to improve workability and moisture sensitivity (COLTO 1996). Mechanically, these materials are designed and modelled similarly to unbound granular materials. The structural design of unbound granular materials using the SAMDM includes:

- selection of trial pavement configuration
- characterisation of elastic parameters for subgrade, subbase and base layers
- determination of principal stresses (σ_1 and σ_3) and maximum compressive strain (ε_{max}) at the centre of the granular layers and top of the subgrade respectively
- calculation of the allowable ESA loading.

Unbound granular layers

The principal failure mode for unbound granular material is deformation resulting from densification and gradual shear under traffic loading (COLTO 1996). Unlike the Austroads and NCHRP methods, the SAMDM failure model explicitly addresses shear failure through the inclusion of a safety factor as presented in Equation 9.

$$F = \frac{(\sigma_3 \times \Phi) + C}{(\sigma_1 - \sigma_3)} \qquad 9$$

where

F	=	shear failure safety factor
σ_1	=	major principal stress (kPa)
σ_3	=	minor principal stress (kPa)
Φ	=	material angle of internal friction
C	=	material cohesion

The major and minor principal stresses in the centre of each unbound granular layer resulting from the ESA loading are determined using the linear elastic model. Presumptive values for M_r and Poisson's ratio are provided in TRH4 (COLTO 1996). The angle of internal friction and cohesion are selected according to parent material type and the anticipated moisture environment. The presumptive values for modified materials range from 4.02 to 1.40 and 140 to 60 for angle of internal friction and cohesion respectively. The allowable traffic loading in ESA is determined using transfer functions developed from historical observations of in-service performance and accelerated pavement testing. The applicable transfer function is determined by the road category as presented in Equation 10:

$$N_i = 10^{(2.605122F + j)} \qquad 10$$

where

N	=	number of equivalent standard axle loads
i	=	road category (A, B, C or D)
F	=	shear failure safety factor
j	=	road category constant: A = 3.480098, B = 3.707667, C = 3.983324, D = 4.510819

Subgrade

The failure mode for subgrade materials is also permanent deformation, resulting in rutting at the pavement surface. An in-service performance-validated transfer function, incorporating the maximum compressive strain at the top of the subgrade resulting from repeated ESA loading and a road category/service level constant (A-factor), is utilised to estimate the allowable loading as presented in Equation 11.

$$N = 10^{(A - 10 \log \varepsilon_v)} \quad (11)$$

where

N = number of equivalent standard axle loads

A = road category/service level constant

ε_v = critical compressive strain

The applicable A-factor is determined in consideration of the road category and terminal rut condition as presented in Table 3.3.

Table 3.3: Road category/service level constant for estimation of ESA loading to subgrade failure

Road category	Terminal rut condition (mm)	A-factor
A	10	33.30
	20	36.30
B	10	33.38
	20	36.38
C	10	33.47
	20	36.47
D	10	33.70
	20	36.70

Source: COLTO (1996).

Pavement structure

The initial trial pavement configuration is adjusted in an iterative manner until the allowable ESA loading, for each granular layer and the subgrade, meets or exceeds the design traffic level. Additionally, the capacity of the upper (base and subbase) and lower (subgrade) layers should be balanced to reduce the loading susceptibility of the pavement structure (COLTO 1996).

3.8 Construction

No amount of design effort can compensate for improperly constructed pavement layers (Dunlop 1980). Careful construction control including mixing, compaction, surface finish, curing and priming is required to ensure satisfactory long-term performance (Lay & Metcalf 1983). In-place mixing is typically used in the construction of cementitiously stabilised pavement layers (Garber, Rasmussen & Harrington 2011). However, extra care is required when mixing low binder content materials to ensure uniform mixing and reduce material variability (AustStab 2012). For this reason, centralised plant mixing is often pursued for modification of aggregate and soil materials. Finer pulverisation of fine-grained soils leads to a greater effect of stabilising agents and improved performance (Scullion et al. 2005).

3.8.1 Layer Preparation and Finishing

Layers underlying the modified layer under construction require a minimum bearing capacity of 5% CBR to allow for adequate compaction of the overlying material (Austroads 2002c). For construction over soft subgrades (CBR < 5) provision of a thin (100 mm) aggregate working platform assists in achieving desired compaction levels (Austroads 2002b). PM-CMB layers can be formed using mechanical spreaders (paver) or motor graders to spread staged windrows of material. Lift thicknesses range from 100–300 mm, but the optimal compacted layer thickness is 150–200 mm (TMR 2012b). The degree of compaction significantly influences strength and durability properties. Standard practice is to determine reference density according to relative compaction using standard effort as outlined in test method Q142A (TMR 2010c). The required level of compaction level is typically 100% of MDD determined using standard effort (Austroads 2006a). Following compaction, the material can be opened to construction traffic without delay (Halsted 2011).

The surface finish of the CMB layer has a significant impact on the structural properties of the pavement. Effective bonding of multiple layers is essential to maximise bearing capacity and minimise delamination potential. The surface of the lower CMB layer can be scarified to maximise bonding. Curing procedures are typically not observed for cementitiously modified material, but light water spray or bituminous seal will maximise the benefits of modification (Garber, Rasmussen & Harrington 2011). However, insufficient available moisture during curing can lead to the development of shrinkage cracks and reduced strength (Lay & Metcalf 1983).

3.8.2 Quality Control

Quality control is essential to ensure the final product will achieve the design performance. Specific properties requiring monitoring include gradation, stabilising agent and moisture content, uniformity, degree of compaction, lift thickness and curing environment (ACI 2009). In addition to the above properties, monitoring density and strength development is beneficial for project evaluation. The selection of field samples should be representative of the in situ materials, where great variability can exist in material type, particle size distribution, moisture content and layer thickness (Austroads 2006a). In situ and laboratory performance varies due to differences in moisture content, RDD, temperature and construction method (Matanovic 2012). Variability in laboratory testing results can arise from user sensitivity of compaction equipment, nonstandard specimen age, various curing regimen, soaked versus unsoaked preparation and delay time between mixing and testing (Symons & Poli 1999). Selection of an appropriate end product assessment period is critical to ensuring material quality, but should not significantly delay construction operations (Austroads 2002a).

3.9 Performance

The cementitious modification of aggregate and soil increases resilience, in addition to structural and volumetric stability. The processes of flocculation and agglomeration enhance surface texture and interparticle friction. Plasticity is reduced by neutralising the negative charge on the surface of clay particles (Halsted 2011). Increased weathering resistance and reduced moisture sensitivity are the primary benefits resulting from cementitious modification (Garber, Rasmussen & Harrington 2011). The improvements are permanent and do not revert back, even after repeated traffic and environmental loading (Halsted 2011). However, cementitious modification can increase material cost, construction time, and shrinkage cracking potential (Matanovic 2012).

3.9.1 Structural

The performance of modified materials is defined according to resistance to permanent deformation (Gray et al. 2011). CMB is subject to two forms of permanent deformation, including shoving and rutting, resulting from shallow shear and densification respectively (Dunlop 1980). The rate of distress development can be decreased by reducing the moisture sensitivity (plasticity) of the subgrade, improving the load distribution abilities (stiffness), increasing the pavement thickness and reducing the severity of loading (Dunlop 1980). Saturation of stabilised materials does not induce a critical condition. However, drying of the layer may induce high shrinkage strains, inducing cracking and weakening the layer (Dunlop 1980).

3.9.2 Functional

The primary concern of cementitious modification is overapplication of the cementitious stabilising agent, resulting in the development of excessive tensile strength and the associated shrinkage cracking. The impact of shrinkage cracks on pavement performance is determined by the width and frequency of cracking. Narrow cracking maintains load transfer and minimises moisture infiltration. Wide cracking provides marginal load transfer and allows moisture to infiltrate the structure, accelerating subbase and subgrade degradation (ACI 2009). The severity of shrinkage cracking depends on inter-layer friction, shrinkage potential, tensile strength and extensibility (Scullion et al. 2005).

The development of shrinkage cracks in cement-stabilised material cannot be eliminated, only minimised and managed (Lay & Metcalf 1983). Good construction and quality control procedures are critical to minimising shrinkage cracking (ACI 2009). Techniques for reducing shrinkage cracking include compacting slightly dry of optimum moisture content, limiting fines content, utilisation of interlayers, prolonged curing, increased layer thickness and decreased stabilising agent content, immediate surface application, precracking using roller or construction traffic, use of SCMs, and limiting strength gain and drying shrinkage in material specifications (ACI 2009). However, addressing shrinkage cracking through precracking procedures is a risky endeavour and may lead to overstressing of the pavement structure (Dunlop 1980). Successful modification has been achieved when the 7-day UCS is limited to 1.0 MPa at a minimum stabilising agent dosage rate of 1% by mass (Austroads 2002c).

3.10 Queensland vs. Best Practice

The guiding standard for the provision of PM-CMB pavements in Queensland is MRTS ET05C (TMR 2012b). Australia, and TMR in particular, have been a leader internationally in defining CMB and outlining the requirements for satisfactory performance. The principal differences in Queensland versus national and international best practice are associated with the specification, mixture design and structural design of PM-CMB.

3.10.1 Material Specification

The principal difference between Queensland and both national and international best practice, relative to material specification, is associated with the properties of the parent material including particle size distribution, plasticity properties and bearing capacity as outlined in Section 3.5. TMR requirements for gradation bisect the ACI and Austroads requirements, but plasticity and CBR requirements are the most stringent of those investigated. The quality requirements for the cementitious binder and mixing water vary slightly, but essentially a high-standard, well-controlled product is desired.

3.10.2 Mixture Proportioning

A standard method for proportioning the constituent material for PM-CMB does not currently exist. Typically, the design mix is established based on previous experience and/or a trial-and-error approach. The difference between Queensland and both national and international best practice stems from the minimum recommended stabilising agent contents and target UCS values. Requirements within the Australian authorities investigated are generally in good agreement, stabilising agent content between 1% and 2% and UCS between 1 MPa and 2 MPa. However, the curing period prior to measuring UCS, seven or 28 days, varies and will result in the production of materials with significantly different ultimate strength values. Additionally, the Australian authority requirements are quite low compared to international requirements. This discrepancy probably stems from the differences in technology utilisation as highlighted in Section 3.1.

3.10.3 Structural Design

Queensland practice for the structural design of PM-CMB is based on the Austroads system, and therefore is aligned with the other Australian state and territory road agencies. However, a significant difference exists in the maximum allowable design modulus value, where Western Australia and Austroads allow values twice the magnitude allowed by the other states and territories. The Austroads design methodology is similar to international best practice, in that PM-CMB is modelled as unbound granular material. However, the Austroads system does not directly consider the stresses and strains within the unbound granular/modified pavement layer, as is accomplished in both the NCHRP and COLTO methods. Additionally, damage resulting from trafficking is not considered and the structural design is based solely upon the theoretical deflection of the subgrade surface. Both the NCHRP and COLTO methods include damage models for quantifying susceptibility to permanent deformation within the unbound granular/modified pavement layer. Differences in the determination of structural layer thickness are directly attributable to systemic differences, as outlined in Section 3.7.

4 FOAMED BITUMEN STABILISATION

Foamed bitumen is a mixture of air, water and bitumen that is produced by injecting a small quantity of cold water into a stream of hot bitumen and allowing the material to expand rapidly to approximately 15 times the original volume. The resulting product is a fine mist or foam (Kendall et al. 2001). The foamed bitumen is introduced to granular material and blended to produce a bound, flexible material for use in base and subbase pavement layers (AustStab 2008). The blending of foamed bitumen with aggregate and soil materials is typically accomplished in situ, using a dedicated stabilising/recycling machine. However, the use of centralised plant mixing has also been successfully accomplished.

Foamed bitumen stabilisation can be utilised to improve the strength and stiffness properties of unbound granular materials. Mixture proportioning and structural design practices significantly influence the efficiency and value for money of I-FBS base materials. This review of current practice explores different design philosophies and laboratory testing procedures used for the provision of I-FBS pavement layers by Australian and international road agencies. The review of alternative practice was undertaken to highlight potential relationships between in situ performance and I-FBS base parent material and blended mixture characteristics.

4.1 Applications

FBS is primarily utilised for the rehabilitation of road pavements. I-FBS base should be considered where the road to be treated conforms to one or more of the following situations (AustStab 2008, Austroads 2011):

- the pavement has been repeatedly patched to the extent that pavement repairs are no longer cost-effective
- a weak granular base overlies a reasonably strong subgrade (> 5% CBR)
- structural overlay is not possible due to site constraints (limited access, moisture inundation, adjacent structures)
- conventional reseals or thin asphalt overlays can no longer correct flushing problems.

However, successful foamed bitumen stabilisation requires a suitable parent material with sufficient grading of fines, a purpose-built stabiliser/recycler, experienced operators, and rigorous quality control (AustStab 2008).

Benefits of adopting I-FBS base in the rehabilitation of pavement layers include (Kendall et al. 2001):

- increased shear strength and reduced moisture susceptibility
- strength characteristics approach that of cement-treated materials while remaining flexible and hence relatively fatigue resistant
- lower moisture contents are required during construction, compared to bitumen emulsion, reducing the occurrence of wet spots
- increased construction efficiency due to expedient process, compared to structural overlay, and ability to immediately open to construction traffic
- all-weather construction where compacted layers can withstand heavy rainfall with only minor surface damage under traffic.

4.2 Materials

The long-term performance of I-FBS base layers is significantly influenced by the type, quality and proportion of the constituent materials. The principal materials composing FBS base include bitumen, aggregate, active filler (cement or lime) and water.

4.2.1 Bitumen

The bitumen utilised in the production of I-FBS base is referred to as expanded bitumen. Expanded bitumen is hot bituminous binder that has been temporarily converted from a liquid to a foam consistency through the addition of a small volume of water (Muthen 1999). The foamed bitumen is the primary binding agent for the production of bound I-FBS base material. However, unlike asphalt mixtures where the grade of the bitumen, commonly indicated by penetration value, has great influence on the end properties, the penetration value does not qualify bitumen for use in FBS. Abels and Hines (1978) found that lower-viscosity bitumen foamed more readily than high-viscosity types, providing foams with greater expansion ratios and half-lives. However, high-viscosity bitumen provided improved coating of aggregates. Wirtgen GmbH (2012) suggests hard bitumen should be avoided due to the poor quality of foam produced, which leads to insufficient dispersion of binder in the FBS mixture.

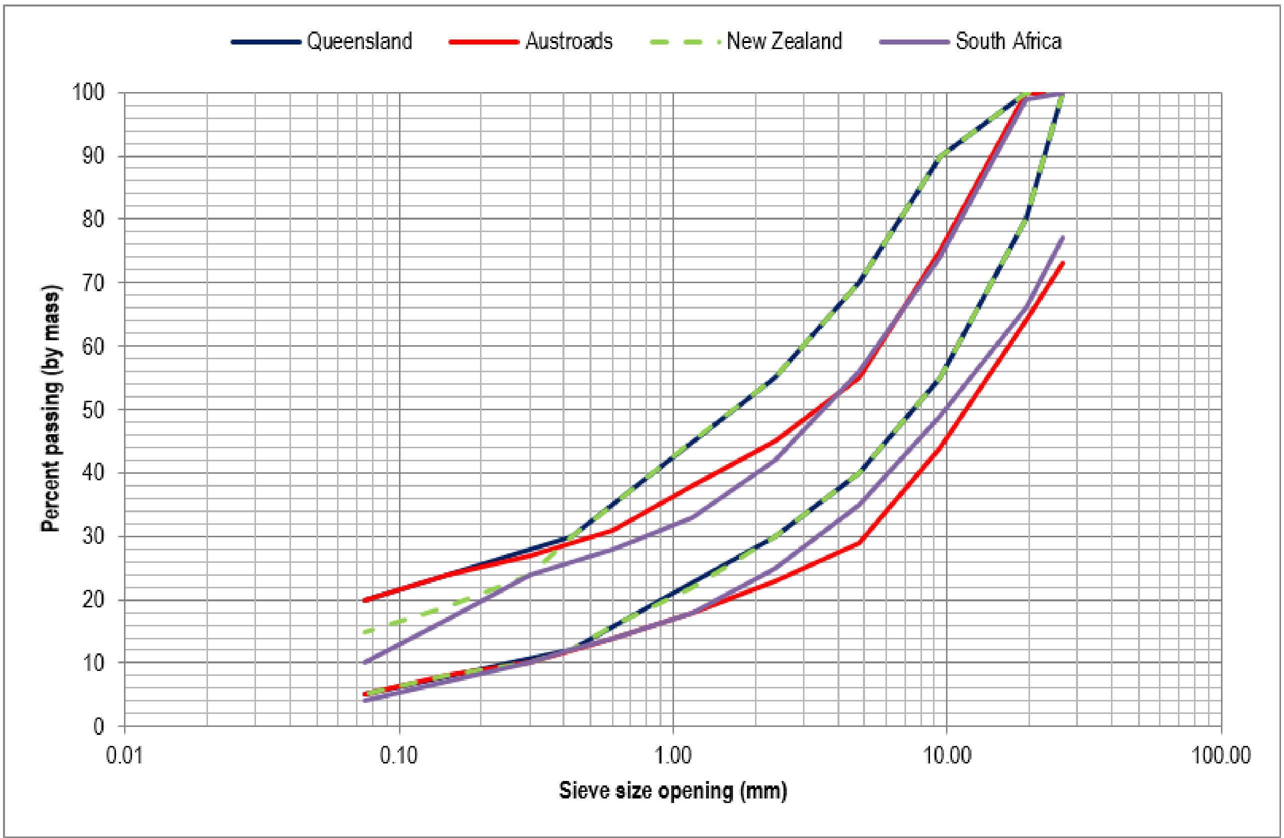
Foaming agent

Due to the sourcing of bitumen from numerous refineries worldwide, Australian suppliers cannot ensure a consistent product is available (AustStab 2011). Some bitumen will not provide the required foam characteristics. Bitumen commonly available in Australia is produced in 'lube oil' refineries where the addition of 0.5% silicon doubles the throughput. However, the addition of silicone reduces the foamability of the bitumen. When insufficient foam characteristics are provided, foaming agents can be utilised to ensure the bitumen develops the required foaming properties (Kendall et al. 2001). The foaming properties of bitumen should be determined inclusive of any proposed additives to confirm the desired characteristics are maintained (Austroads 2011).

4.2.2 Aggregate

The parent or source material for FBS includes aggregates of varying quality, ranging from premium quarried to marginal materials (Roading New Zealand 2007). However, FBS is optimally utilised on material conforming to established basecourse requirements. Typical requirements include well-graded particle size distribution (5–20% passing 0.075 mm sieve opening) and PI less than 15 (Browne 2008). However, Ramanujam and Jones (2008) and the Asphalt Academy (2009) suggest that parent materials with PI greater than ten be subjected to pretreatment with lime. TMR (Ramanujam, Jones & Janosevic 2009) typically utilises 1.5–2.0% (by mass) hydrated lime, while the Asphalt Academy (2009) imposes an upper limit of 1.5% for the pretreatment of high (> 10%) PI aggregates. A comparison of parent material grading requirements for Queensland (Ramanujam, Jones & Janosevic 2009), Austroads (2006a), New Zealand (Transit New Zealand 2008) and South Africa (Asphalt Academy 2009) are presented in Figure 4.1.

Figure 4.1: Comparison of particle size distribution requirements



4.2.3 *Secondary Binder (Active Filler)*

The benefits of including active fillers (cement, lime, fly ash) in FBS mixtures are well documented, including:

- modifying the fines fraction of the parent material
- promoting bitumen-aggregate adhesion
- assisting the uniform dispersion of bitumen
- reducing moisture sensitivity and
- improving early-life shear strength (Asphalt Academy 2009).

The most commonly used secondary binders for FBS base are lime (quick or hydrated) and Portland cement. In Australia, hydrated lime (1% to 2% by mass) is typically used as a secondary binder to improve particle coating. As mentioned previously, lime may also be used as a preliminary treatment to render high PI aggregates more amenable to foamed bitumen stabilisation (AustStab 2011). In New Zealand, 95% of I-FBS projects incorporate Portland cement as the secondary binder (AustStab 2011). The optimum bitumen and secondary binder contents should be selected in consideration of laboratory modulus and rut-resistance performance testing (Austroads 2013b).

4.2.4 *Water*

Water is one of the three principal components, along with bitumen and aggregate, of FBS base and plays a significant role during the foaming process. Water utilised for the production of FBS base is ideally of potable quality. Although acceptable foam characteristics may be achieved using water containing impurities, such practice should be avoided (Wirtgen 2012).

4.3 Foaming Process

The foaming of bitumen is accomplished by injecting a small volume of cold water into a stream of hot bitumen. The rapid evaporation of water produces tiny water vapour droplets that are encapsulated by the bitumen, producing an instantaneous volumetric expansion of the bitumen (Austroads 2011). Two criteria are commonly used to characterise the quality of the foamed bitumen including expansion ratio and half-life time.

4.3.1 Expansion Ratio

Expansion ratio is the quotient of the maximum binder volume achieved in the foamed state to the final binder volume once the foam has dissipated (Austroads 2011).

4.3.2 Half-life Time

Half-life is the time, in seconds, between the moment the foam achieves maximum volume and the time it dissipates to half of the maximum volume (Austroads 2011).

4.3.3 Foam Characterisation

Sufficient expansion ratio and half-life time are required to ensure uniform distribution of foamed bitumen binder and thorough coating of the aggregate particles. An inferior FBS product will result due to the lack of adequate bitumen dispersion within the material (Ramanujam & Jones 2008). Standard laboratory testing methods can be used to assess the foaming characteristics of candidate bitumen products. Should the half-life or expansion ratio be unsatisfactory, the test is repeated with differing percentages of water and/or the addition of foaming agents until the desired foam properties are achieved (Austroads 2011). It is imperative to consider the differences between laboratory and in situ performance. However, it was noted that for the foaming process, the foam produced in the expansion chambers of the stabiliser spraybar is always of higher quality than that produced in the laboratory. This is a result of higher operating pressure and warmer water temperature in the stabiliser compared to typical laboratory testing conditions (AustStab 2011).

Class 170 bitumen is commonly used for FBS in Queensland and typically provides a minimum expansion ratio of 12 and half-life of 45 seconds. Threshold values for successful performance include limits of ten seconds and 20 seconds, for expansion ratio and half-life respectively (Ramanujam & Jones 2008, Ramanujam, Jones & Janosevic 2009). The foaming moisture content is typically 2.5% unless the minimum expansion ratio or half-life requirements cannot be achieved (Austroads 2011). In South Africa, the recommended minimum expansion ratio ranges from eight to ten when the parent material is at temperatures of greater than 25 °C and 10–25 °C respectively. An absolute minimum half-life requirement of six seconds is recommended irrespective of aggregate temperature (Asphalt Academy 2009).

4.4 Mixture Proportioning

Optimal primary (foamed bitumen) and secondary (Portland cement/lime) stabilising agent contents are determined by the properties of the parent material and the application-specific requirements. Methodologies for determining optimal stabilising agent content, in addition to volume of filler, vary between TMR, Austroads and other international road agencies. Due to the high relative cost of stabilising agents compared to the total cost of provisioning I-FBS pavement layers, great potential for savings exists from optimisation of specified contents. Selection of the optimal primary and secondary stabilising agent contents to provide the required stiffness, strength, durability and workability is the focus of the mixture proportioning process.

4.4.1 Characterisation Methods

UCS, ITS and indirect tensile resilient modulus (ITM_R) are the most commonly used test methods to assess the appropriateness of FBS materials for a given application. The methodologies of UCS, ITS and ITM_R tests were introduced previously in Section 3.4.1, Section 3.4.2 and Section 3.4.3 respectively. In addition to the standard characterisation methods, the rut resistance of FBS materials can be determined in accordance with Austroads (2006b) test method AGPT/T231, *Deformation Resistance of Asphalt Mixtures by the Wheel Tracking Test*. However, the method was developed for asphalt mixtures and a standard protocol, including specimen preparation, for FBS materials has not yet been developed. TMR has developed interim specification requirements for the rut resistance of FBS materials based on historical observations of in-service performance (Austroads 2013b).

4.5 Structural Design

A review of I-FBS base design specifications adopted by road agencies in Queensland, City of Canning (Western Australia), New Zealand, South Africa and the UK was undertaken to establish current best practice. The review explored design methods and associated development procedures in addition to the fundamental underlying philosophy.

4.5.1 Queensland

Although field data in Queensland was limited when the design equation was first suggested, there was sufficient anecdotal evidence to suggest that the primary distress mechanism of FBS layers was load-induced fatigue (Jones & Ramanujam 2004). TMR has adopted the Shell Petroleum asphalt fatigue relationship for FBS structural layers; the same approach is widely utilised for the design and modelling of asphalt pavement layers. The fatigue equation provided in Equation 12 includes the volumetric properties and ITM_r of the FBS material and the critical tensile strain at the bottom of the FBS material layer, determined using linear elastic structural modelling.

$$N = \frac{6918(1.08 + 0.856V_b)}{S_{mix}^{0.36} \times \mu\epsilon} \qquad 12$$

where

N	=	the number of load cycles during the effective fatigue life
V_b	=	volumetric bitumen content (normally between 6% and 8%)
S_{mix}	=	stiffness of foamed bitumen mix, measured using the ITM_r (MATTA testing) on soaked specimens
$\mu\epsilon$	=	induced horizontal tensile strain at bottom of foamed bitumen layer

The fatigue relationship, as utilised by TMR, is limited to binder contents less than 8% (by volume) and ITM_r values less than 2500 MPa. The design modulus (S_{mix}) is based on the soaked ITM_r at the nominated design binder content.

4.5.2 City of Canning

The City of Canning, Western Australia, also uses a fatigue relationship for the FBS layer based on testing data from flexural beams prepared and compacted in the field and tested in the laboratory. The results of the testing showed that the performance of individual beams varied widely. While bitumen content and stiffness would be considered to influence fatigue life, due to the scatter of test results, no significant relationships were observed between modulus or bitumen content and fatigue life (Leek 2009). The City of Canning fatigue equation is presented in Equation 13.

$$N = \left(\frac{1558}{\mu\epsilon}\right)^6 \tag{13}$$

where

N = the number of load cycles during the effective fatigue life

$\mu\epsilon$ = induced horizontal tensile strain at bottom of foamed bitumen layer

Although the fatigue relationship adopted by the City of Canning is presented in a different form, when compared with the TMR fatigue relationship the methods yield similar results (Austroads 2012).

4.5.3 New Zealand

In contrast to the design methods adopted by TMR and the City of Canning, the New Zealand Transport Agency (NZTA) does not consider fatigue resistance and models the FBS layer as an unbound granular material. This is similar to the approach commonly followed for modified materials. The design of FBS layers is conducted in accordance with the Austroads mechanistic approach. FBS layers can be characterised using the following material properties:

- anisotropic ($E_{vertical}$ = 2 * $E_{horizonal}$)
- M_r = 800 MPa
- Poisson's ratio = 0.3
- no sublayering.

The principal failure criterion for unbound granular materials is permanent deformation as a result of vertical compressive strain. The allowable traffic loading (ESAs) is calculated using the Austroads subgrade criterion presented in Equation 6. The limitation of this design methodology is that it does not directly consider the performance (fatigue, permanent deformation) of the FBS layer. This is inconsistent with the observations of Long (2001) where two-thirds of total pavement rutting was measured in a FBS base layer.

4.5.4 South Africa

Technical guideline (TG) 2, *Design and use of Foamed bitumen Treated Materials* (Asphalt Academy 2002), is based on full-scale accelerated testing and extensive laboratory investigation of FBS materials in South Africa. The core of the methodology is that the behaviour of the FBS can be divided into two distinct phases. The first phase is defined as the effective bound phase (fatigue controlled) and the second phase is defined as the equivalent granular phase (deformation controlled). Initially, the material provides some fatigue resistance. Repeated load applications reduce the tensile capacity until the material reverts to an unmodified granular material (Asphalt Academy 2002). The allowable traffic loading is determined by summing the load cycles to failure for phases one and two as presented in Equation 14 and Equation 15 respectively.

$$N_{F,FB} = 10^{\left(A - 0.708\frac{\varepsilon}{\varepsilon_b}\right)} \qquad 14$$

where

$N_{F,FB}$	=	the number of load cycles during the effective fatigue life
ε	=	calculated horizontal tensile strain at the bottom of the layer
ε_b	=	strain-at-break measured in the monotonic strain-at-break beam testing
A	=	a coefficient related to the category of the road or reliability (risk) of the design

$$N_{PD,FB} = \frac{10\text{^}[A + 11.9.38(RD) + 0.0726(PS) - 1.628(SR) + 0.691(cem/bit)]}{30} \qquad 15$$

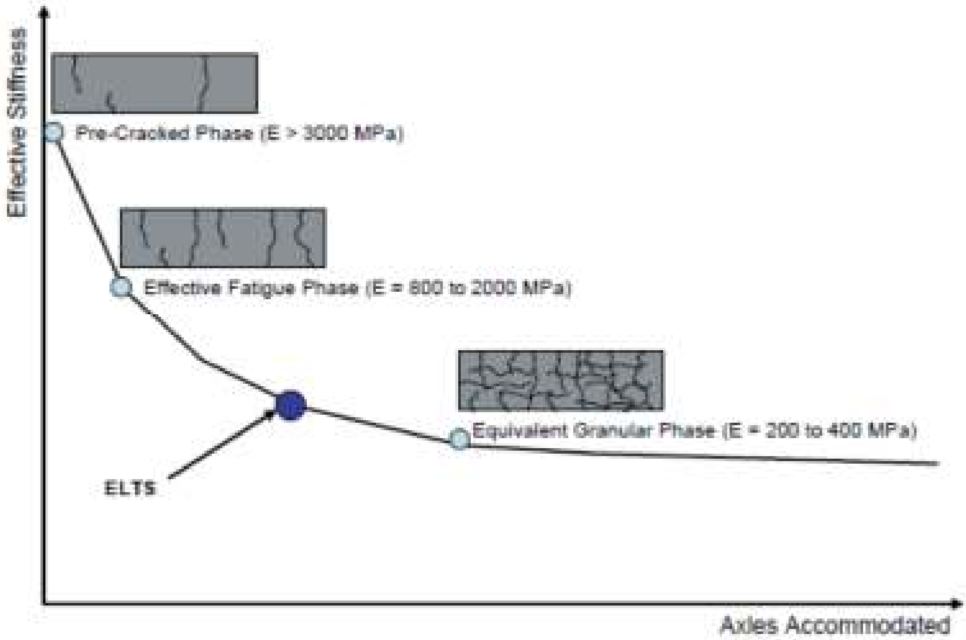
where

$N_{PD,FB}$	=	the number of load cycles
A	=	a coefficient related to the category of the road or reliability (risk) of the design
RD	=	relative density of the foamed bitumen mix
PS		plastic strain expressed as percentage
SR		stress ratio
cem/bit		cement to bitumen ratio

Although the method is widely utilised in South Africa, a number of limitations were discovered. Given constant cement content, increasing foamed bitumen content results in a reduction in allowable loading. It was also observed that the second phase is highly sensitive to the RDD of the mixture. Increasing RDD from 75% to 80% results in an increase in allowable loading from approximately 1 million ESAs to more than 17 million ESAs (Jenkins, Collings & Jooste 2008). These limitations resulted in the revision of TG2 in 2009.

The current TG2 (Asphalt Academy 2009) methodology uses an empirical method to determine FBS base structural layer thickness. A pavement number (PN) is determined by summing the product of the effective material stiffness and equivalent layer thickness for each pavement component. This approach is similar to the AASHTO structural number (SN) method. The PN and design traffic loading are utilised to determine the design thickness of the pavement. The determination of effective long-term stiffness (ELTS) is presented in Figure 4.2.

Figure 4.2: Determination of effective long-term stiffness

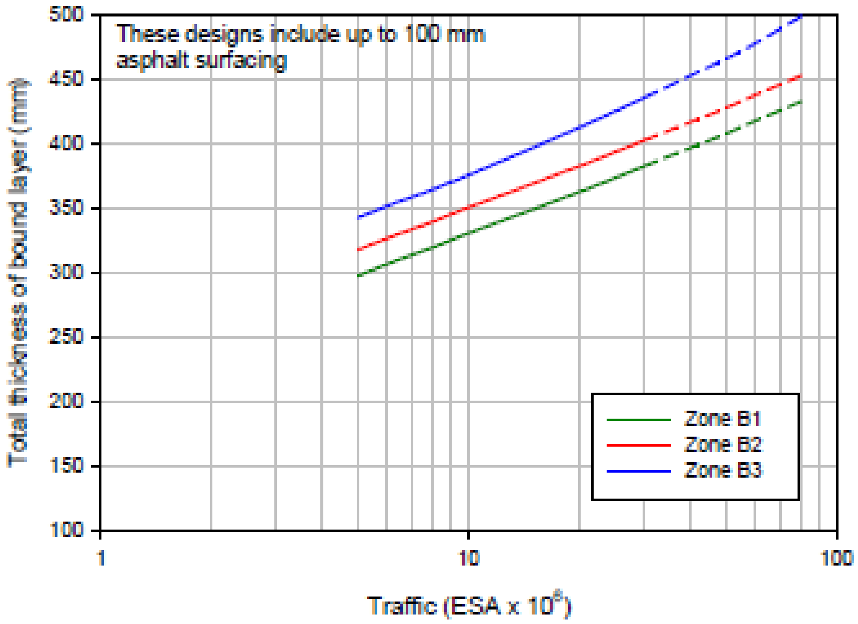


Source: Asphalt Academy (2009).

4.5.5 United Kingdom

The Transport Research Laboratory (TRL) in the United Kingdom adopts fatigue as the principal distress mode for FBS base. However, an empirical approach, based on a number of tables and charts, is utilised to determine the layer thickness relative to the design subgrade condition, traffic loading and foamed bitumen type. Similar to the Queensland approach, the empirical charts were developed using asphalt fatigue relationships (Nunn 2004). Figure 4.3 shows an example of the design curves recommended by TRL.

Figure 4.3: Design curves for bitumen-bound cold-recycled material, Foundation Class 1



Source: Merrill, Nunn and Carswell (2004).

4.5.6 Summary of Design Methods

Review of the selected design methods highlights the significant differences in the design philosophy for FBS base of different road agencies. TMR, Austroads and TRL assume the FBS layer will behave similarly to an asphalt pavement layer, where fatigue is the principal failure mechanism. The City of Canning uses a similar fatigue relationship calibrated using local observations of in-service performance. NZTA is the only road agency that ignores fatigue and assumes rutting and shape loss as the key distress mode.

Table 4.1 summarises the differences in design and assessment methodologies between the selected road agencies highlighted in this literature review (Austroads 2011, Browne 2012, Muthen 1999). An anecdotal review was conducted by Austroads (2013b) to evaluate the sensitivity of the various design methods. The TMR and TRL methods yield similar FBS base design layer thickness. Similar results were obtained, but with slightly reduced thickness, when utilising the City of Canning method. The NZTA method yielded the thinnest layer thickness due to the fact that fatigue distress is ignored. The 2009 TG2 method was observed to be the most conservative design approach.

Table 4.1: FBS specification, characterisation and design methodology summary

Elements		Austroads specification	South African specification (TG2 2009)	TMR specification	New Zealand specification
Foaming properties	*Expansion rate*	> = 15	8–12	12 as desired; 10 as absolute minimum	> = 10
	Half time (second)	30–45	> = 6	45 as desired; 20 as absolute minimum	6
Aggregate	*Particle size*	Refer to Figure 4.1			
	PI	< = 10	< = 10	< = 10	< = 15
Binder	*Secondary binder*	1–2% lime or cement	0–1.5% hydrated lime and/or 0–1% cement	2.0% lime for 10% ≥ PI ≥ 6% 1.5% lime for PI < 6%	< = 1.5% cement
	Bitumen content	2% or 3% or 4% depends on lab testing outcome	1.7–2.5%	2.5–4.0%, typically 2.5–3.0%	2.7–3.5%, typically 2.7–3%
Lab testing	*Moisture content*	–	65% to 95% of OMC	Depending on PI of the aggregates; usually 70% of OMC	–
	Trial compaction	80 cycles of Gyropac compaction	Vibration hammer	50 blows per layer using Marshall hammer	Vibration hammer
	Curing condition	3-day curing at 60 °C for dry samples and soaked for 10 minutes at 0.95 kPa vacuum for wet samples	24 hours at 25 °C unsealed before sample is sealed and cured for further 48 hours at 40 °C	Oven cured for three days at 40 °C Satisfies both modulus and rutting resistance requirements that vary according to the traffic level land layer to be stabilised	6-days curing period at room temperature (around 20 °C)
	Determination of binder content	Select the binder content that has the highest modulus while considering the wet/dry modulus ratio and local experience	Satisfy the ITS, triaxial and moisture sensitivity test with respect to material properties	Satisfy both modulus and rutting resistance requirements that vary according to the traffic level and layer to be stabilised	–
Design philosophy		Effective fatigue phase: mechanistic design	Knowledge-based method: pavement number empirical or mechanistic design	Effective fatigue phase: mechanistic design	Unbound granular material mechanistic design

4.6 Construction

In addition to general construction best practice, I-FBS base layers require that construction be undertaken in a sequential and methodical manner utilising equipment specifically designed for the task (AustStab 2011). The construction of I-FBS base layers is defined by the principal activities of existing formation preparation, spreading secondary binder, in situ stabilisation, compaction, curing and quality control testing.

4.6.1 Formation Preparation

The in situ stabilisation of existing pavement structures requires significant site preparation before the actual stabilisation works commence. The formation preparation works include premilling, additional of granular material (as required), removal of thick asphalt or cementitious patches, and stockpiling of material for two-layer stabilisation (AustStab 2011). The adequate preparation of the existing pavement formation is critical to the long-term performance of the I-FBS base.

4.6.2 Spreading Secondary Binder

Secondary binders (active filler) utilised in I-FBS are typically lime (quick and hydrated) and/or Portland cement. It is important to ensure that secondary binders are uniformly applied at the specified rate. Spreading using a purposely built machine is recommended (Asphalt Academy 2009). Additionally, the in situ spread rate should be verified prior to progression of stabilisation works. The two most commonly used methods to verify spread rate include the use of trays or mats that are weighted before and after application of binder and the use of an automated spreader incorporating electronic load cells with real-time monitoring and adjustment (AustStab 2011).

4.6.3 In situ Stabilisation

Foamed bitumen stabilisation requires utilisation of specialised construction equipment due to the short amount of time during which the foamed bitumen can be effectively blended with the parent material. With respect to I-FBS, blending of the pulverised existing pavement material, correction course (where required), active filler, water, and foamed bitumen takes place in the mixing chamber of the stabiliser machine. The quality of the foamed bitumen is directly related to the bitumen and water temperature, quality, and supply system pressure (AustStab 2011). Stabilising using two passes of the mixing machine is recommended, where the parent material, secondary binder and water are blended on the first pass and the foamed bitumen is introduced and blended on the subsequent pass.

4.6.4 Compaction

Achieving adequate compaction is critical, due to the influence of RDD on stiffness, strength, and long-term durability. Compaction should commence as soon as practicable after mixing and should be completed within the working time of the binder (Austroads 2009a). TMR requires that initial compaction be completed within two hours after the secondary binder has been spread on the prepared surface to ensure the active filler does not begin to stiffen the material (Ramanujam & Jones 2000).

4.6.5 Curing

Curing defines the gradual reduction in moisture content of the FBS material due to evaporation. Curing is associated with an increase in stiffness and tensile strength. Ensuring sufficient curing takes place is critical to long-term performance, as sufficient stiffness and cohesion between particles is required to withstand traffic loading (Asphalt Academy 2009). TMR requires at least four hours to elapse from time of incorporation of the foamed bitumen until trafficking is permitted (Kendall et al. 2001). NZTA has established a similar requirement, except when cement is included as secondary binder the curing period is reduced to two hours (Transit New Zealand 2008).

4.6.6 Quality Control Testing

Construction quality assurance testing requires bulk samples to be taken from the freshly hoed FBS layer and compacted in the same manner to confirm that treated pavement properties comply with pavement and mix design requirements. It has been emphasised by Browne (2011) that the time between obtaining sample material after pulverising and compaction by the testing agency should be less than one hour. It has been observed that a significant reduction in strength can occur between samples compacted within 20 minutes and those compacted within two hours.

4.7 Performance

A review of the primary factors influencing long-term I-FBS base performance revealed material composition and construction practice are critical. A number of research institutes and road agencies have conducted studies on the influence of parent material fines component quality, active filler type and content, in addition to bitumen content on the long-term performance of FBS materials. Generally, the relative influence of the above material properties was observed to be dependent on the nature of the parent material. Significant variation in testing outcomes is probably due to differences in the properties of parent materials, bitumen type and content, as well as specimen production and testing procedures. Therefore, the observations of one study may not be generally applicable and the information provided in this review should be used as a guide only.

4.7.1 Fines Content

The fines component refers to the portion of the parent material passing the 0.075 mm sieve size opening. Wirtgen (2012) observed that FBS base requires strict adherence to established grading requirements, especially the fine material fraction. Research work performed during the past decade has concluded that the fine aggregate content (FAC) should be greater than 3% (by mass), with 5% being optimal, and 20% being the upper limit. Browne (2008) indicates that the ideal FAC ranges between 5% and 20%. TG2 (Asphalt Academy 2009) recommends FAC values between 4% and 12% for crushed stone and 5% and 15% for natural gravels. Huan, Jitsangiam & Nikraz (2011) investigated the influence of FAC on the performance of FBS, relative to MDD, ITS and ITM_R. Parent aggregates consisting of crushed rock base or crushed limestone and varying amounts of fines were mixed with fixed amounts of foamed bitumen. The authors observed a clear trend of increased bitumen content and FAC resulting in improved density, ITS and ITM_R (Huan, Jitsangiam & Nikraz 2011). All the testing specimens reached peak values at bitumen contents between 2% and 4% and FAC between 15% and 20%.

4.7.2 Secondary Binder Content

Active filler is included in FBS materials to improve the quality and content of fines, which promotes uniform dispersion of foamed bitumen. Additional benefits include:

- improved bitumen-to-aggregate adhesion
- reduced parent material PI
- increased stiffness and rate of strength gain
- accelerated curing of the compacted mixture.

Lime and Portland cement are the most commonly utilised secondary binders in FBS materials. Huan, Jitsangiam & Nikraz (2011) investigated the influence of secondary binder content (hydrated lime, quicklime and Portland cement) on UCS, ITS, and ITM_R. The authors recommend that the amount of the secondary binder should be limited to 3% to minimise potential for development of a brittle mixture (Huan, Jitsangiam & Nikraz 2011). Xu et al. (2011) suggest that the optimal Portland cement content for FBS mixtures is approximately 1.5% (by mass). The Asphalt Academy (2009) recommends maximum secondary binder contents of 1% and 1.5% for Portland cement and hydrated lime respectively. Wirtgen (2012) also recommends a maximum

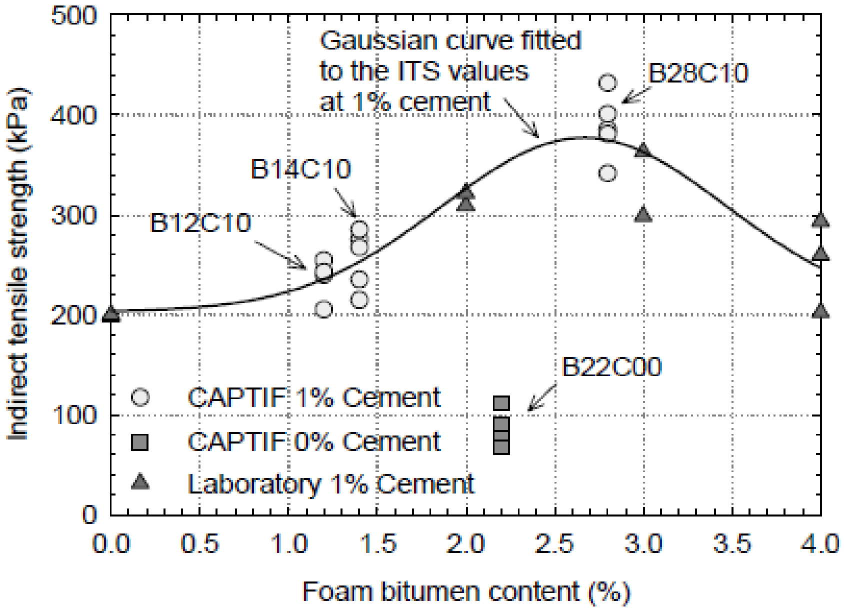
Portland cement content of 1%. In Queensland, hydrated lime is typically utilised as the active filler in FBS materials. TMR has developed guidance for selection of optimal contents based on the particle size distribution and plasticity of the parent material. For in situ stabilisation works, the following hydrated lime contents are generally used (Ramanujam & Jones 2008):

- 2.0%; 6% ≤ PI < 10%
- 1.5%; PI < 6%.

4.7.3 Bitumen Content

A number of research studies have investigated the influence of foamed bitumen content on long-term FBS base performance. The ITS test conducted under both oven-dry and soaked conditions is commonly used to predict the in-service performance of FBS materials. Long and Theyse (2002) observed that increasing bitumen content (1.8–3.0%) produced a corresponding increase in ITS (up to 43%). Browne (2008) observed an improvement in ITS (5–10%) resulting from increased foamed bitumen content. Gonzalez et al. (2009) performed a study to determine the optimum bitumen content for a common parent aggregate with varying Portland cement content. The authors observed a parabolic trend between ITS and bitumen content that was not significantly influenced by the content of active filler (Figure 4.4).

Figure 4.4: Bitumen content vs. ITS



Source: Gonzalez et al. (2009).

The Asphalt Academy (2009) indicates that the optimal bitumen content ranges from 1.7–2.5% and is parent material dependent. Browne (2008) determined the optimal foamed bitumen content is approximately 2.8%. Austroads (2009a) states that 3% is the most commonly utilised bitumen content in Australia. A review of the design specifications for FBS base layers provisioned along the TMR road network revealed that, for the majority of pavements, the target bitumen content was 3.5% with a number of projects specifying contents as high as 4%.

4.8 Queensland vs. Best Practice

The guiding standard for the provision of I-FBS pavements in Queensland is MRTS 07C, *Insitu Stabilised Pavements Using Foamed Bitumen* (TMR 2014a). TMR has been a leader both nationally and internationally in developing and implementing foamed bitumen stabilisation technology. Other significant international contributors to the state of practice include New

Zealand, South Africa and the USA. Australian practice, as defined by Austroads, is generally in agreement with Queensland practice. The principal differences in Queensland versus international best practice are associated with the specification, mixture design and structural design of I-FBS. A summary of the key differences is provided in Table 4.1 and expounded below.

4.8.1 Material Specification

The differences in material specification between Queensland and international best practice are related to the properties of the parent material and the type of secondary stabilising agent. Requirements for gradation are similar between Queensland and New Zealand, but Austroads and South Africa specify a coarser parent material. Additionally, plasticity requirements for the parent material are identical for Queensland, Austroads and South Africa. However, New Zealand allows a slightly more plastic (> PI) material. The type of secondary stabilising agent varies internationally, with Austroads and South Africa allowing either Portland cement or lime, New Zealand requiring cement and Queensland typically only using lime. Differences in secondary stabilising agents have probably evolved from differences in prevalent source materials and the different structural design methodologies. The quality requirements for the secondary stabilising agent and mixing water vary slightly, but essentially a high-standard, well-controlled product is desired.

4.8.2 Mixture Proportioning

The procedure for proportioning of constituent materials varies significantly internationally. All of the investigated authorities assess the bitumen quality according to half-life and expansion ratio. However, the performance measures used to optimise the bitumen content vary widely. The Austroads method requires selection of the binder content which has the highest M_r while considering the retained modulus and historical performance. South African materials must satisfy ITS, triaxial compression and moisture sensitivity performance requirements. Queensland requirements include satisfying both M_r and rutting resistance performance thresholds, which vary according to traffic volume and the position of the FBS layer in the pavement structure.

4.8.3 Structural Design

The most significant difference in the application of I-FBS materials internationally stems from the methods for characterising and modelling the material during structural design. Consensus on the controlling failure mode for FBS materials, either fatigue or permanent deformation, has not yet been reached. Therefore, the most reliable design approach is that which most closely resembles the failure modes observed in-service.

The Austroads/Queensland method includes a mechanistic approach with consideration of fatigue within the FBS layer and permanent deformation within the composite pavement structure. The City of Canning and Austroads approaches yield similar results despite differences in the equation for determining the fatigue resistance. The current South African method, Asphalt Academy (2009) is an empirical approach validated by accelerated loading and observations of in-service performance. The South African methodology is the only approach, of those investigated, that has been validated by field performance observations. The New Zealand method utilises a mechanistic approach, but unlike the Austroads/Queensland methodology, is based on permanent deformation within the composite pavement structure only. Because the method does not consider fatigue behaviour, the New Zealand approach generally provides the thinnest design thickness compared to the other methods.

5 IDENTIFICATION OF PAVEMENT SECTIONS

Benchmarking the stabilisation practices across Queensland required a review of the material selection, mixture proportioning, structural design, construction and maintenance processes utilised throughout the state. However, before beginning the review, the extent and general condition of pavements incorporating PM-CMB or I-FBS base needed to be determined. The Queensland state-controlled road network consists of approximately 33 300 km of national highways, state-controlled roads and local roads of regional significance. The network covers a number of varying environmental and climatic zones and caters to a wide range of traffic loadings. Significant portions (≈ 15%) of the network are composed of stabilised structural layers, including PM-CMB and I-FBS base pavements. Fortunately, the inventory, condition and maintenance data for the entire Queensland road network is accumulated and maintained centrally in the *A Road Management Information System* (ARMIS) database, allowing for a state-wide review of technology selection, design practice and maintenance programming.

5.1 Network Inventory

The extent, composition and historical performance of PM-CMB and I-FBS sections of the Queensland state-controlled road network were determined by referencing the ARMIS database. The database contains historical pavement information including construction date, location, extent, configuration, condition (roughness, rutting, texture and deflections), traffic and resurfacing/rehabilitation date. This valuable information was provided to the ARRB Group project team by the Pavement Rehabilitation Section of the TMR Engineering and Technology Branch. The data was extracted from the ARMIS database in January 2014 and all data presented in this report reflect the state of the database at the end of the 2013 calendar year. The ARMIS information was instrumental to the investigation, as the PM-CMB and I-FBS sections of the road network were identified and categorised according to region, environmental zone, age, stabilised layer thickness, traffic volume and design stabilising agent content. The PM-CMB and I-FBS base sections identified from the ARMIS database are presented in Table 5.1 and Table 5.2 respectively. It should be noted that these sections are only representative of the state-controlled road network and do not include recent works such as those completed as part of the Transport Network Reconstruction Program (TNRP).

Table 5.1: PM-CMB network composition

Region	Road ID	Length (km)	Environmental zone	Pavement age (years)	Stabilised layer thickness (mm)	Cumulative traffic (ESA)	Stabiliser content (%)
Far North	10N	11.4	WNR	3	250	1.70E+06	3.0
Mackay/Whitsunday	516	0.9	WNR	5	250	9.90E+05	1.0
Mackay/Whitsunday	530	0.9	WNR	5	250	4.53E+06	1.0
Mackay/Whitsunday	530	0.7	WNR	5	250	4.53E+06	1.0
Mackay/Whitsunday	854	0.8	WNR	12	250	9.17E+05	2.0
Mackay/Whitsunday	856	2.9	WNR	5	250	4.54E+06	2.0
Mackay/Whitsunday	856	2.9	WNR	33	250	9.05E+06	–
Mackay/Whitsunday	856	1.9	WNR	8	250	1.08E+07	1.5
Mackay/Whitsunday	856	1.9	WNR	37	250	1.57E+07	–
Mackay/Whitsunday	857	1.1	WNR	3	250	2.21E+06	–
Mackay/Whitsunday	33B	6.9	WNR	10	250	7.00E+06	2.0
Northern	10K	3.1	WNR	4	250	2.02E+06	3.0
Northern	10K	3.3	WNR	4	400	2.02E+06	3.0

Region	Road ID	Length (km)	Environmental zone	Pavement age (years)	Stabilised layer thickness (mm)	Cumulative traffic (ESA)	Stabiliser content (%)
Northern	10K	3.7	WNR	4	250	2.02E+06	3.0
Northern	10K	0.3	WNR	20	375	1.29E+07	1.5
Northern	10K	0.4	WNR	11	375	1.00E+07	2.0
Northern	10K	0.9	WNR	11	375	1.05E+07	2.0
Northern	10K	0.1	WNR	14	250	4.86E+06	–
Northern	10L	0.9	WNR	28	200	4.83E+06	–
Northern	10L	1.3	WNR	1	200	3.93E+05	1.5
Northern	10L	4.0	WNR	1	300	8.33E+05	1.5
Northern	10L	3.0	WNR	10	350	7.20E+06	–
Northern	10L	7.9	WNR	4	400	3.33E+06	3.0
Northern	10L	2.3	WNR	1	250	8.73E+05	1.5
Northern	10L	1.2	WNR	1	250	8.73E+05	4.0
Northern	10L	1.6	WNR	3	250	2.55E+06	3.0
Northern	10L	3.5	WNR	3	250	2.55E+06	3.0
Northern	10L	0.1	WR	28	250	1.47E+07	–
Northern	10L	1.1	WNR	3	250	3.29E+06	3.0
Northern	10L	0.7	WNR	3	250	3.29E+06	3.0
Northern	10L	4.0	WNR	3	250	3.29E+06	3.0
Northern	10L	0.9	WNR	3	250	3.29E+06	3.0
Northern	10L	0.3	–	4	250	4.39E+06	–
Northern	10M	0.4	WNR	4	250	9.68E+06	–
Northern	10M	0.2	WNR	15	250	1.39E+07	–
Northern	10M	1.7	WNR	9	250	1.94E+07	–
Northern	10M	13.3	WNR	10	350	2.07E+07	1.5
Northern	10M	1.4	WNR	7	340	1.11E+07	1.5
Northern	10M	1.7	WNR	4	150	5.02E+06	2.0
Northern	10M	3.3	WNR	6	350	9.70E+06	2.5
Northern	10M	4.2	WNR	4	180	6.56E+06	3.0
Northern	10M	2.1	WNR	5	180	2.94E+06	3.0
Northern	10M	1.6	WNR	5	180	2.94E+06	1.5
Northern	10M	3.1	WNR	1	460	8.75E+05	3.5
Northern	10M	1.1	WNR	4	175	2.82E+06	2.0
Northern	10M	0.3	WNR	6	175	3.06E+06	1.5
Northern	10M	3.0	WNR	6	150	5.33E+06	2.0
Northern	10N	1.4	–	3	310	1.89E+06	3.0

Table 5.2: FBS network composition

Region	Road ID	Length (km)	Environmental zone	Pavement age (year)	Stabilised layer thickness (mm)	Cumulative traffic (ESA)	Stabiliser content
Darling Downs	17B	1.8	DNR	16	200	1.22E+07	3.0% B+2.0% L
Darling Downs	17C	1.5	DNR	18	200	7.58E+06	–
Darling Downs	22B	21.3	DR	17	200	5.22E+06	3.5% B+2.0% L
Far North	641	3.0	WNR	4	250	9.72E+05	3.5% B+2.0% L
Far North	641	1.1	WNR	4	250	6.30E+05	3.5% B+2.0% L
Far North	641	1.3	WNR	16	250	6.30E+05	3.5% B+2.0% L
Far North	642	1.2	WNR	13	180	2.00E+06	3.5% B+2.0% L
Far North	645	1.8	WNR	8	250	3.71E+06	3.5% B+1.5% C
Far North	647	0.2	WNR	13	200	1.56E+07	3.5% B+1.5% C
Far North	649	1.5	WNR	15	270	5.29E+06	3.5% B+1.5% C
Far North	649	0.1	–	15	210	6.87E+06	3.5% B+1.5% C
Far North	653	0.7	WNR	13	170	1.11E+06	3.5% B+1.5% C
Far North	809	4.7	WNR	14	200	1.34E+07	3.5% B+2.0% L
Far North	814	0.1	WNR	19	200	2.26E+06	3.5% B+2.0% L
Far North	10N	1.2	WNR	6	250	2.59E+06	3.0% B+1.0% L
Far North	10N	2.9	WNR	7	250	2.59E+06	3.5% B+1.5% L
Far North	10N	4.2	WNR	3	250	8.20E+05	3.5% B+2.0% L
Far North	10N	11.9	WNR	9	250	5.54E+06	3.5% B+1.5% L
Far North	10N	3.6	WNR	8	250	5.06E+06	3.5% B+1.5% L
Far North	10N	1.9	WNR	13	250	9.13E+06	3.5% B+1.5% L
Far North	10P	0.9	WNR	15	250	1.41E+07	3.5% B+1.5% L
Far North	10P	10.3	WNR	5	250	1.92E+06	3.5% B+1.5% L
Far North	10P	2.7	WNR	10	250	4.94E+06	3.5% B+1.5% L
Far North	10P	1.2	WNR	3	250	1.32E+06	3.5% B+1.5% C
Far North	10P	3.4	WNR	3	250	1.32E+06	3.5% B+1.5% L
Far North	10P	4.1	WNR	3	250	1.32E+06	3.5% B+2.0% L
Far North	10P	0.8	WNR	14	300	7.26E+06	3.0% B+2.0% L
Far North	10P	1.3	WNR	20	250	1.42E+07	3.0% B+2.0% L
Far North	10P	0.4	WNR	12	250	1.86E+07	–
Far North	10P	0.3	WNR	12	250	1.86E+07	–
Far North	20A	4.6	WNR	15	250	1.97E+07	3.5% B+2.0% L
Far North	20A	2.2	WNR	8	250	6.82E+06	–
Far North	20A	2.1	WNR	8	250	3.55E+06	–
Far North	20A	3.0	WNR	14	200	5.13E+06	–
Far North	20A	3.1	WNR	11	250	4.70E+06	3.5% B+2.0% L
Far North	21A	9.0	WNR	15	250	4.26E+06	3.5% B+2.0% L
Far North	21A	0.5	WNR	15	170	4.26E+06	3.0% B+2.0% L
Far North	21A	10.3	WNR	14	200	4.00E+06	3.0% B+2.0% L

Region	Road ID	Length (km)	Environmental zone	Pavement age (year)	Stabilised layer thickness (mm)	Cumulative traffic (ESA)	Stabiliser content
Far North	21A	0.4	WNR	11	250	1.67E+06	3.5% B+1.5% C
Far North	21A	1.3	WNR	6	150	9.01E+05	3.0% B+2.0% L
Fitzroy	16A	1.0	WR	9	250	1.20E+07	–
Fitzroy	16A	2.4	WR	9	250	1.20E+07	3.5% B+2.0% L
Fitzroy	16A	0.5	WNR	16	250	1.72E+07	3.5% B+2.0% L
Fitzroy	16B	1.8	DR	11	270	3.82E+06	3.5% B+2.0% L
Fitzroy	16B	0.8	DR	11	270	3.82E+06	3.5% B+2.0% L
Metropolitan	18A	1.9	WR	16	210	4.75E+07	3.0% B+2.0% L
North Coast	141	3.3	WNR	17	300	2.64E+05	–
Northern	840	0.4	WNR	10	200	5.32E+06	3.5% B+2.0% L
South Coast	205	2.4	WNR	7	200	1.11E+06	–
South Coast	208	0.8	WR	16	250	1.09E+07	–
South Coast	212	0.9	WR	11	300	4.11E+06	3.5% B+2.0% L
South Coast	1003	1.8	WNR	11	300	6.49E+06	3.0% B+2.0% L
South Coast	2020	1.7	WNR	7	200	2.57E+05	–
South Coast	25A	2.0	DNR	6	300	6.84E+06	3.5% B+2.0% L
South Coast	25A	0.6	WNR	14	250	9.94E+06	3.5% B+2.0% L
South Coast	25B	3.3	DNR	13	300	2.59E+06	3.5% B+2.0% L
Wide Bay/Burnett	40C	3.8	WNR	17	150	6.06E+06	3.0% B+2.0% L
Wide Bay/Burnett	45A	4.8	DR	17	150	2.85E+06	3.5% B+2.0% L
Wide Bay/Burnett	45A	2.5	DNR	17	150	3.93E+06	3.0% B+2.0% L

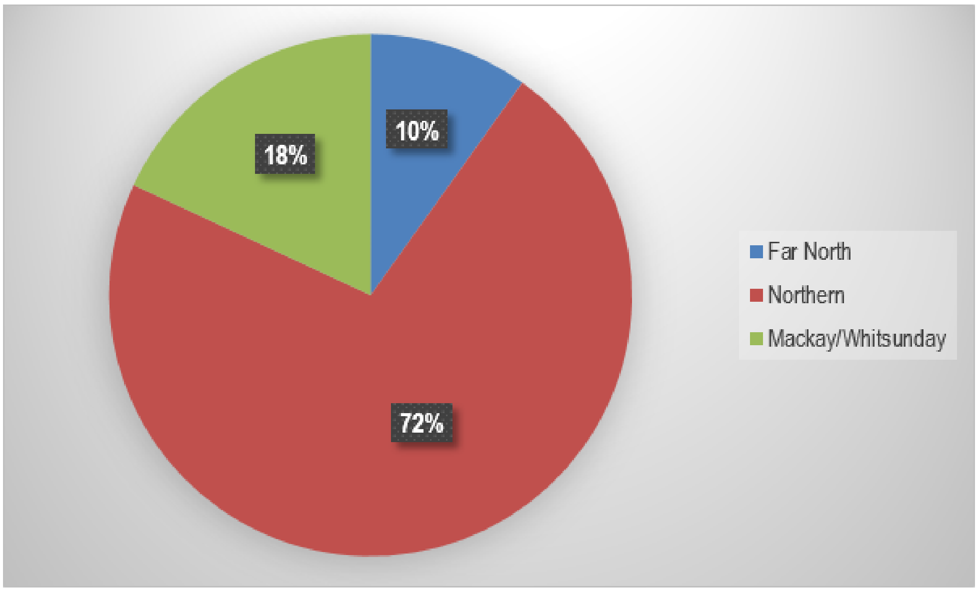
Note: Stabiliser content notations B, C and L represent the proportion of foamed bitumen, cement and lime stabilising agents by mass.

In reviewing Table 5.1 and Table 5.2, the utilisation of PM-CMB appears to be isolated within the northern coastal regions. However, I-FBS is widely used with sections identified in eight of the nine TMR regions. The provision of PM-CMB or I-FBS pavement sections is primarily pursued in wet/nonreactive (WNR) environmental zones. However, a few I-FBS sections are also located in wet/reactive (WR), dry/reactive (DR) and dry/nonreactive (DNR) zones. Both the specified stabilising agent content and structural layer thickness for both stabilisation technologies vary widely between the different regions, highlighting differences in utilisation approaches. There are a number of sections with incomplete stabilising agent content information. This is one of the limitations of the ARMIS system, as the accuracy, extent, detail and level of aggregation of data can vary (TMR 2012a).

5.1.1 PM-CMB

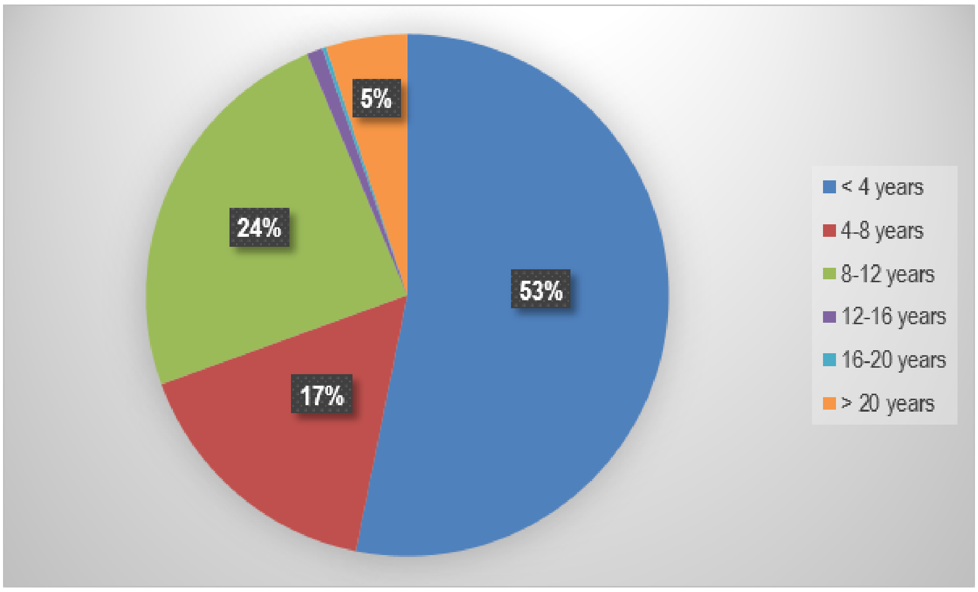
A large proportion, approximately 72%, of the PM-CMB pavement structures along the Queensland state-controlled road network are located in the Northern TMR region (Townsville). Smaller proportions of the network can be found in the Far North and Mackay/Whitsunday TMR regions as shown in Figure 5.1. Almost all (> 99%) of the PM-CMB pavements are located in WNR environmental zones, that is areas with median annual rainfall greater than 800 mm and subgrade soils with low potential for significant moisture-induced volumetric change (shrink/swell).

Figure 5.1: Regional distribution of PM-CMB pavements



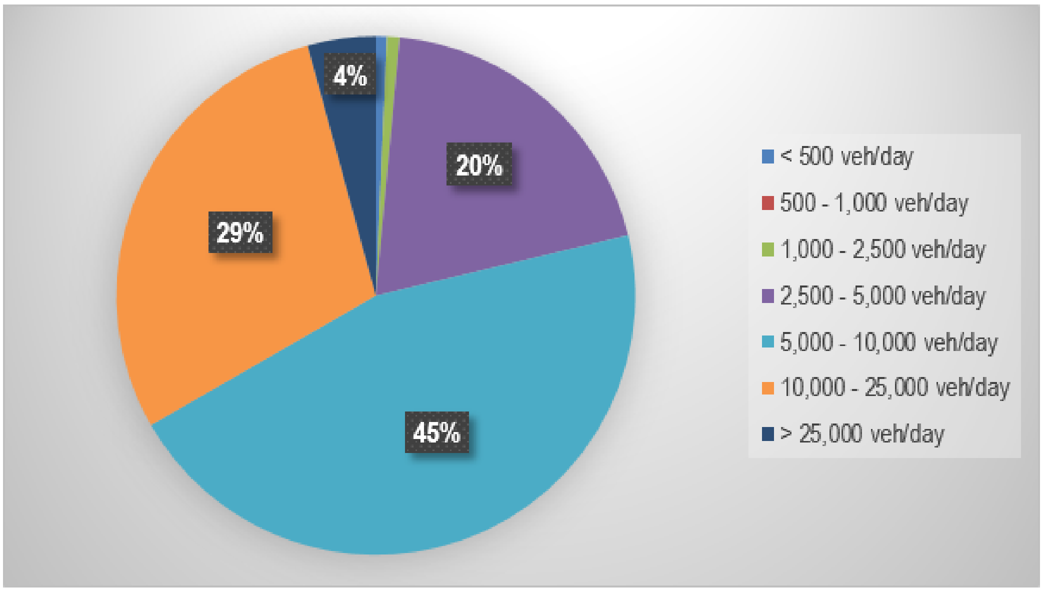
The age of the PM-CMB stabilised base layers along the state-controlled road network range from one year to 37 years with the vast majority (≈ 53%) having been constructed after 2010. As mentioned previously, the pavements identified in the ARMIS database do not include the recent works accomplished as part of the TNRP programme. The PM-CMB pavement structures are relatively young, with approximately 94% of the network less than 12 years old, as shown in Figure 5.2.

Figure 5.2: Age distribution of PM-CMB pavements



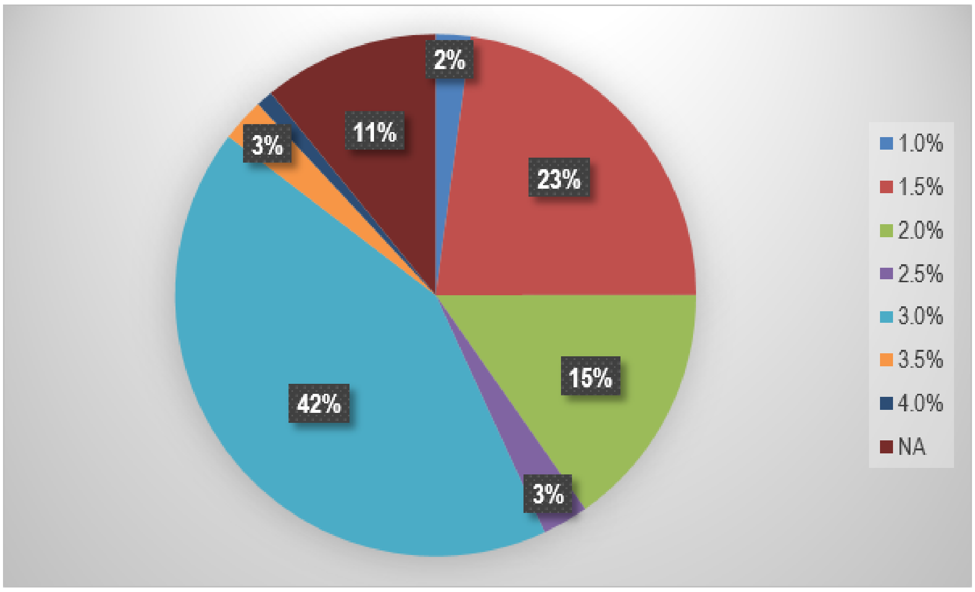
PM-CMB pavements are selected for a wide range of operating conditions, as represented by AADT. PM-CMB is utilised in applications with traffic ranging from fewer than 500 vehicles per day to more than 25 000 vehicles per day. However, the vast majority (≈ 95%) of PM-CMB structures are selected for moderate to heavily trafficked pavements with AADT values measured in 2013 ranging from 1000 vehicles per day to 25 000 vehicles per day as shown in Figure 5.3.

Figure 5.3: Traffic distribution of PM-CMB pavements



The proportions of stabilising agent specified in PM-CMB pavements ranges from 1% to 4%. However, the type of cementitious stabilising agent is not recorded in the ARMIS database. The distribution of design stabilising agent contents are presented in Figure 5.4, with 3.0% being the dominant value. Stabilising agent contents of 3.0% would typically produce a bound material where GP or GB cements are used. However, 3.0% may also produce an effective modified material where blends of SCMs are used.

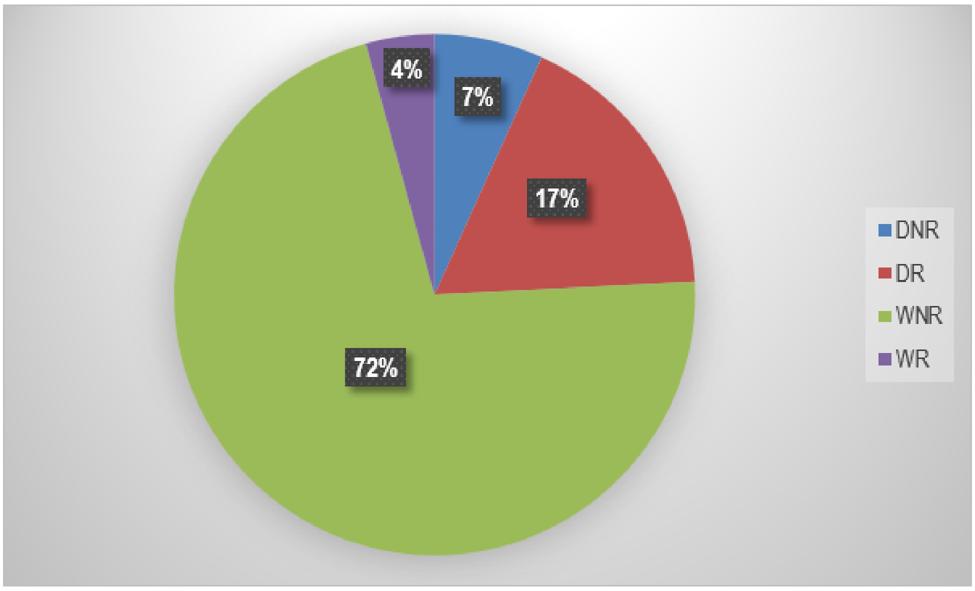
Figure 5.4: Stabiliser content distribution of PM-CMB pavements



5.1.2 I-FBS Base

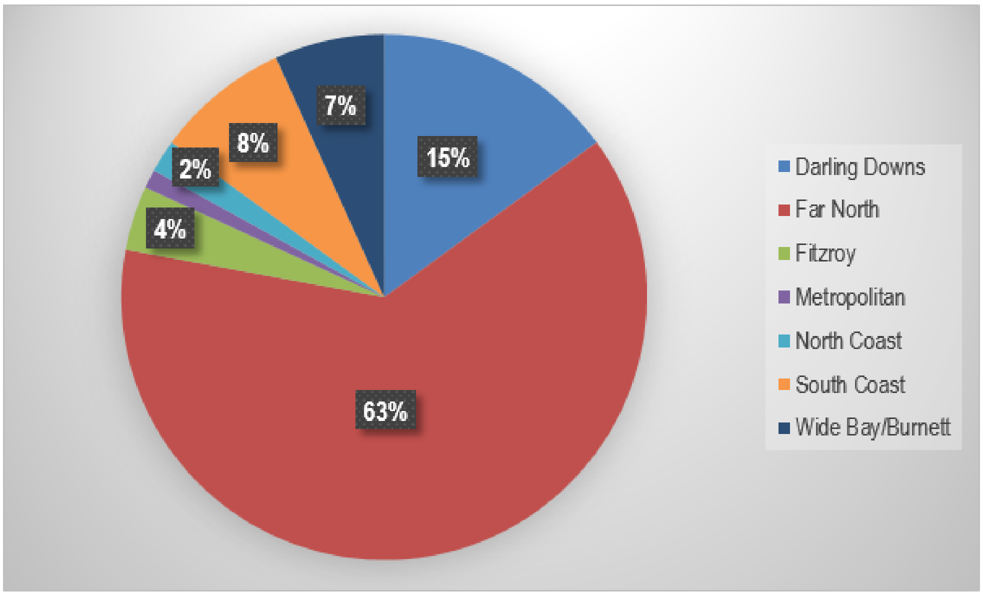
In situ stabilisation with FBS is selected in a much wider range of operating environments, compared to PM-CMB. Similar to PM-CMB, the majority (≈ 72%) of I-FBS base pavements exist in WNR environmental zones. However, significant proportions of the network also exist in DNR, DR and WR environmental zones as shown in Figure 5.5.

Figure 5.5: Environmental distribution of I-FBS pavements



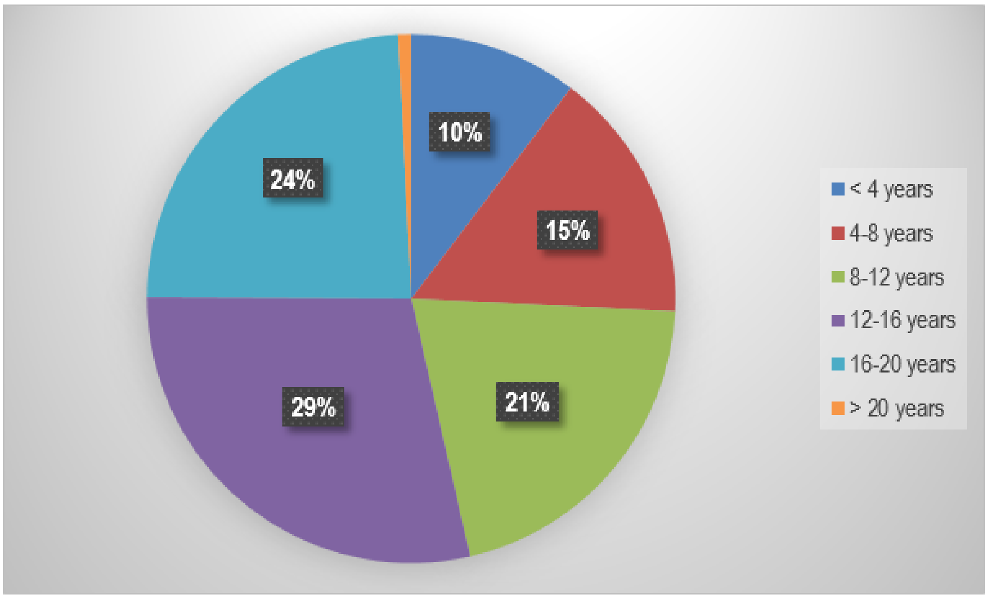
The majority (≈ 63%) of the I-FBS state-controlled road network exists within the Far North TMR region. Similarly to the environmental zone, I-FBS base pavements have been provisioned widely across the State with significant proportions of the network also in the Darling Downs, Fitzroy, North Coast, South Coast and Wide Bay/Burnett TMR regions, as shown in Figure 5.6.

Figure 5.6: Regional distribution of I-FBS pavements



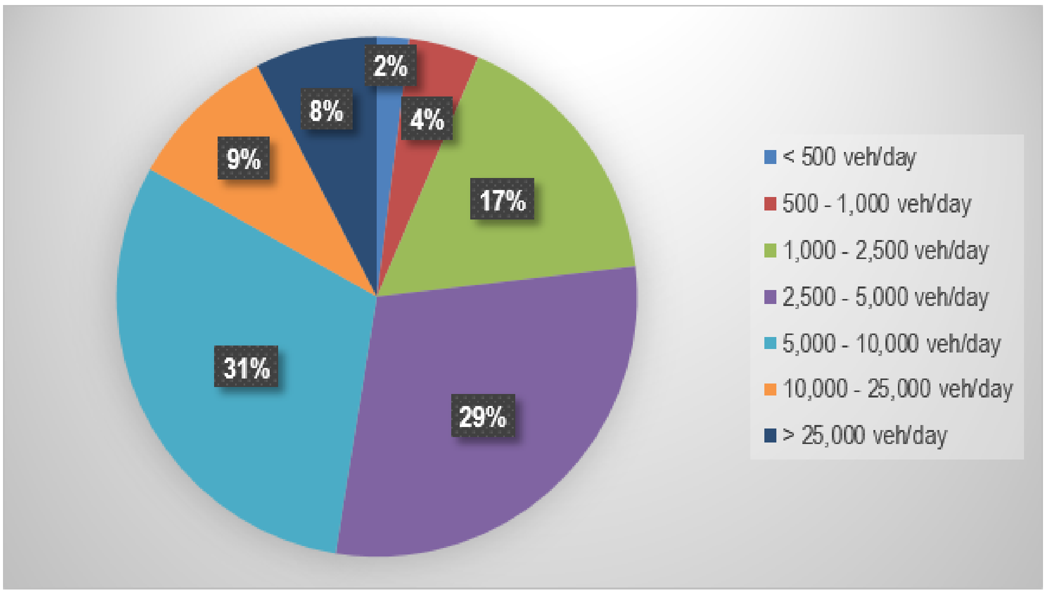
The age of the I-FBS pavements along the state-controlled road network range from three years to 20 years. As mentioned previously, the pavements identified in the ARMIS database do not include the recent works accomplished as part of the TNRP programme. The distribution of pavement age varies widely from less than four years to greater than 20 years with more than half of the pavements between 12 years and 20 years old, as shown in Figure 5.7.

Figure 5.7: Age distribution of I-FBS pavements



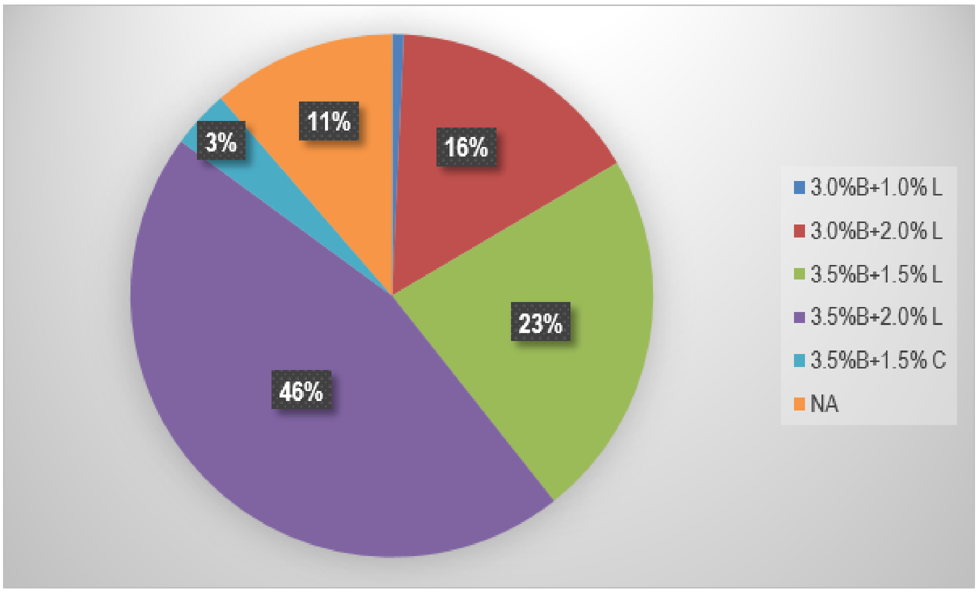
The general utility of I-FBS pavement structures across a wide range of operating conditions, as demonstrated by the environmental zone, region and age distribution, is also reflected in the distribution of traffic as represented by AADT. I-FBS is utilised in applications with traffic ranging from fewer than 500 vehicles per day to more than 25 000 vehicles per day. However, a significant majority (≈ 77%) of I-FBS structures are selected for moderately trafficked pavements with AADT values measured in 2013 ranging from 1000 vehicles per day to 10 000 vehicles per day, as shown in Figure 5.8.

Figure 5.8: Traffic distribution of I-FBS pavements



The specified bitumen content in addition to type and proportion of secondary stabilising agent used in I-FBS pavements varies, but typically ranges from 3.0% to 3.5% foamed bitumen and 1.0% to 2.0% of either Portland cement or lime. The distribution of stabilising agent contents are presented in Figure 5.9, with 3.5% foamed bitumen and 2.0% lime being the dominant target primary and secondary stabilising agent contents.

Figure 5.9: Stabiliser content distribution of I-FBS pavements



In total, 109 km of PM-CMB and 156 km of I-FBS pavement sections were identified, exclusive of the recent TNRP works. Given the cumulative size and the geographic distribution of the identified sections, it would be impractical and uneconomical to perform a detailed assessment of every project. Therefore, a desktop review of the current condition (based on ARMIS data) of the identified sections was undertaken to provide an additional categorisation parameter. The further categorisation allowed for investigation of PM-CMB and I-FBS base subnetworks, according to current condition and historical performance in addition to region, environmental zone, age, stabilised layer thickness, traffic volume and design stabilising agent content.

5.2 Network Condition

The objective of the desktop condition review was to further categorise the identified PM-CMB and I-FBS pavement sections into subnetworks based on general performance, as indicated by the development of surface distress. The subnetwork refinement was accomplished by categorising and sorting the identified pavement sections according to the severity and development rate of pavement distresses. Referenced condition data included historical roughness, rutting and macrotexture measurements from laser profiler surveys and cracking assessments from video survey images. This information allowed for a detailed condition assessment of 100 m pavement sections by examining the extent and severity of current pavement distresses in addition to trends in road surface distress development. The current condition and deterioration rates were evaluated against standard TMR condition criteria to categorise the identified PM-CMB and I-FBS sections as excellent, good, mediocre or poor, in the case of current condition, and good or poor, according to deterioration rate. A summary of the current condition (January 2013) of the PM-CMB and I-FBS road sections along the Queensland state-controlled road network according to rutting and roughness is provided in Table 5.3.

Table 5.3: Summary of I-FBS and PM-CMB current condition

	FBS		CMB	
	Rutting (mm)	Roughness (IRI)	Rutting (mm)	Roughness (IRI)
Maximum	39.4	7.1	25.5	7.3
Average	5.9	1.9	4.2	1.6
Standard deviation	3.4	0.66	2.63	0.64

5.2.1 Condition Data

Roughness, rutting and macrotexture

The roughness, rutting and macrotexture condition data available from the ARMIS database are most commonly obtained using a high-speed survey vehicle (HSV) equipped with an array of laser profilometers, accelerometers, displacement transducers and cameras that continually monitor the pavement surface to assess the condition.

Roughness, in terms of International Roughness Index (IRI), is determined according to the quarter-car model and indicates suspension displacement accumulation in m/km. Roughness was a key component in the condition evaluation process as it is the most common objective measure of the general condition of a road. Roughness is a measure of the serviceability of a pavement and is also indicative of the structural condition. In general, roughness and structural condition are inversely related. Additionally, excessive roughness increases dynamic loading, accelerating pavement deterioration.

Rutting is determined through measurement of the distance between a fixed horizontal datum and the pavement surface and is commonly presented in millimetres. Rutting measurement generally includes the maximum rut in each wheelpath in addition to the lane maximum. Rutting data obtained from ARMIS was the second key analysis element, in addition to roughness, as it directly reflects the structural condition of the pavement. Well-established rutting criteria are generally utilised when judging the performance of a pavement.

Sensor measured texture depth (SMTD) is a continuous measure of surface profile divided laterally into discrete segments. The SMTD is the root mean square of the residuals between the measurements and a second-order polynomial representing the pavement surface. SMTD has been shown to correlate well with traditional measures of macrotexture including the sand patch test. While SMTD is a critical measure of serviceability, it was not utilised as part of the condition categorisation effort in this investigation.

Cracking

The cracking data available within the ARMIS database is commonly collected along with other surface defect data, such as patching, through the review of video images of the pavement surfaces collected during HSV runs. The review of survey images can be conducted either manually (visual assessment by a trained professional) or automatically (electronic review using specialised video processing software). The type (either longitudinal, transverse, fatigue or block cracking), extent and severity are determined on a frame-by-frame basis. Fatigue cracking was the focus of this study as it is indicative of structural failure precipitated by inadequate pavement strength or instability in the supporting pavement layers. However, longitudinal, transverse and block cracking were also taken into account as part of the condition categorisation.

Survey video images

A continuous video of the road surface was also collected by the HSV during surface condition assessment. The video is commonly used to quantify cracking and verify the measurements of roughness, rutting and surface texture. The video, when available, was invaluable during validation of the categorisation approach. Review of the survey video images allowed for investigation of sections eliminated by data aggregation, validation of ARMIS surface condition data and preliminary assessment of the consistency of distresses across the identified pavement sections.

5.2.2 Other ARMIS Data

Traffic volume

Traffic information, including both AADT and annual cumulative heavy-vehicle axle groups (HVAG), was obtained from the ARMIS system for the identified road sections. The cumulative traffic over the service life was calculated and applied as an additional categorisation parameter. The performance of a pavement section can be directly determined through consideration of both the current condition and the volume of traffic the pavement has served relative to the design traffic.

Maintenance cost

The TMR ARMIS database does not include information relative to maintenance operations or expenditure in 100 m detail for any pavement section. The annual maintenance cost presented in ARMIS is the average of the total maintenance expenditure divided by the total length of road. This data is insufficient for detailed investigation of either regional maintenance programming or relative stabilisation technology expenditure. Due to the unsuitability of the data, the maintenance cost information was only used as supplementary information to complement the conclusions made based on other parameters.

Other information

In addition to the data elements described in Section 5.2.1 and Section 5.2.2, other pavement asset information available in ARMIS, such as surfacing age and pavement structural composition, were utilised for categorisation where additional refinement categories were required.

5.2.3 Limitations of ARMIS Data

The dataset obtained from the ARMIS database was provided in 100 m aggregation, where the condition and other data elements provided were statistically significant approximations of the actual measurements collected for the 100 m pavement sections. While this aggregation greatly simplifies network-level asset management practices, it creates challenges when the data is applied for project-level assessments. For example, the 100 m intervals resulted in the omission of a number of road sections during the condition categorisation effort. Condition data within the ARMIS database begins at a chainage of 0.0 km and is subsequently presented in increments of 0.1 km. For road sections with starting and ending chainage values that do not fall exactly on a 100 m interval, portions of the section on either end will not have any associated condition data and will consequently be excluded from analysis. Additionally, a number of short road sections (100–200 m) will be completely removed from investigation as a result of the data aggregation. Road sections with lengths between 200 m and 300 m will be characterised by a single aggregated measurement of condition. Examples of the impact of the 100 m aggregation of the condition categorisation of PM-CMB and I-FBS road sections are presented in Table 5.4.

Table 5.4: Example of road section with little or no condition data

Road ID	Road name	Carriageway #	Lane #	Start (km)	End (km)	Length (km)	Stabiliser content (%)	Section number (100 m)
10N	Ingham – Innisfail	2	1	121.82	123.00	1180.00	3.5B+2.0L	11
20A	Cairns – Mossman	1	1	21.34	21.43	0.09	3.5B+2.0L	0
641	Millaa Millaa – Malanda	1	1	2.42	2.56	0.14	3.5B+2.0L	0
10P	Innisfail – Cairns	1	1	30.035	30.267	0.232	3.5B+1.5C	1

As presented in Table 5.4, 80 m of the Bruce Highway between Ingham and Innisfail (10N) would not be assigned condition data because it falls between 100 m aggregation intervals. Due to the short length (< 100 m), sections of the Captain Cook Highway between Cairns and Mossman (20A) and Millaa Millaa-Malanda Road (641) would not have any condition data attributed. Finally, 232 m of the Bruce Highway between Innisfail and Cairns (10P) would be characterised by a single condition assessment measure. In total, six kilometres of PM-CMB (≈ 5%) and seven kilometres of I-FBS (≈ 4%) were excluded from condition categorisation due to data limitations.

5.2.4 Initial Categorisation Criteria

The PM-CMB and I-FBS sections of the state-controlled road network were categorised according to current condition by referencing the TMR ARMIS database and established performance criteria. Classification of sections as excellent, good, mediocre or poor allowed for refinement of the identified sections into subnetworks based on extent and severity of pavement distress. The adopted condition categorisation criteria relative to roughness, rutting and fatigue cracking are presented in Table 5.5. The sections identified as either excellent or poor were subject to further investigation to identify trends between material properties, structural configuration, environmental conditions and traffic relative to the observed performance.

Table 5.5: Initially set evaluation criteria

Evaluation criteria	Roughness (counts/km)	Rutting (mm)	Fatigue cracking (%)
Excellent	≤ 60	≤ 10	≤ 5
Good	60–110	10–15	5–10
Mediocre	110–200	15–20	10–20
Poor	> 200	≥ 20	≥ 20

Source: TMR (2012a), Austroads (2009b).

Historical performance data from the ARMIS database including HSV measurements of rutting and roughness, in addition to manual fatigue cracking assessments, were utilised to establish both the current condition and relative rate of deterioration of the identified PM-CMB and I-FBS sections. Three condition indices based on roughness, rutting and fatigue cracking were initially selected for the study. The condition evaluation criteria were established to clearly reflect the serviceability of the pavement sections. For example, a section of road in excellent condition should exhibit performance at a similar level to a newly constructed road. A road in poor condition should be close to, or exceed, the performance level indicating that rehabilitation or reconstruction is required. The initial condition criteria presented in Table 5.5 were established in accordance with the TMR *Pavement Rehabilitation Manual* (2012a) and the Austroads *Guide to Asset Management Part 5H: Performance Modelling* (2009b). It should be noted that the network condition analysis included all the road sections initially identified in ARMIS, although the material and condition information for a number of sections was incomplete.

The condition of the PM-CMB and I-FBS pavement sections identified in Table 5.1 and Table 5.2 respectively were categorised according to the criteria presented in Table 5.5. The most recent assessment data available in ARMIS (2013) was utilised to categorise the roughness, rutting and fatigue cracking of each of the sections as excellent, good, mediocre or poor. The condition distribution with respect to roughness, rutting and fatigue cracking of the Queensland state-controlled PM-CMB and I-FBS road sections is presented in Table 5.6 and Table 5.7 respectively.

Table 5.6: CMB network condition distribution with initial evaluation criteria

Evaluation criteria	Roughness (%)	Rutting (%)	Fatigue cracking (%)
Excellent	91.6	98.4	99.1
Good	8.4	1.4	0.4
Mediocre	0.0	0.1	0.4
Poor	0.0	0.0	0.1

Table 5.7: FBS network condition distribution with initial evaluation criteria

Evaluation criteria	Roughness (%)	Rutting (%)	Fatigue cracking (%)
Excellent	79.8	91.8	92.3
Good	19.9	6.0	1.6
Mediocre	0.3	0.8	2.1
Poor	0.0	1.4	4.0

The majority of the road sections were determined to be in excellent condition according to the criteria presented in Table 5.5. Although the roughness of some 20% of the I-FBS road sections was categorised as good, which is between 60 and 110 National Association of Australian State Road Authorities (NAASRA) counts per kilometre. This data shows that, in general, the PM-CMB and I-FBS road sections are performing exceptionally well, with 99.5% and 93.9% of the state-controlled road network categorised as either excellent or good condition. The generally good condition is particularly telling when the median pavement age and traffic volume distributions presented in Section 5.1.1 and Section 5.1.2 are considered. However, a robust determination of performance efficiency also requires consideration of stabilising agent content, pavement configuration, cumulative traffic loading, prevalent environmental conditions and maintenance expenditure.

During the evaluation it was discovered that the fatigue cracking information was incomplete for a significant number of the identified pavement sections. As a result, the total length of network assessed according to the fatigue cracking criteria was much less than that assessed according to either roughness or rutting. The TMR project team advised that the survey video images used to manually assess cracking for input into the ARMIS database are not regularly processed. Therefore, the cracking data for a large portion of the road network was either incomplete or out-of-date. Additionally, resurfacing works can temporarily improve the cracking assessment immediately after construction. However, if the source of the distress is not addressed, the cracks will eventually reflect through the newly applied surfacing. A number of the identified road sections were discovered to have recently benefitted from resurfacing works. These works significantly affect the accuracy of any condition assessment. Therefore, due to the unreliability of the cracking data, roughness and rutting were the primary condition categorisation criteria utilised in the investigation and fatigue cracking was only used as a complementary evaluation parameter.

5.2.5 Revised Categorisation Criteria

The generally good condition of the PM-CMB and I-FBS road sections along the Queensland state-controlled road network provides an indication that the stabilisation technologies are performing exceptionally well. However, categorisation of the identified road sections based on the condition criteria presented in Table 5.5 did not provide a sufficient number of pavement sections in the mediocre and poor condition categories. The revised evaluation criteria (more stringent) presented in Table 5.8 were developed to establish a greater distribution of pavement sections between the different condition categories.

Table 5.8: Revised evaluation criteria

Evaluation criteria	Roughness (counts/km)	Rutting (mm)	Fatigue cracking (%)
Excellent	≤ 60	5	0
Good	60–80	5–7	0–5
Mediocre	80–100	7–10	5–10
Poor	> 100	> 10	> 10

The revised criteria were developed to further discretise the pavement sections identified as good condition, allowing for investigation of factors separating 'nearly excellent' and 'almost mediocre' pavement sections. Pavements categorised as good condition using the standard criteria presented in Table 5.5 could be categorised as good, mediocre or poor using the revised criteria presented in Table 5.8. However, pavements categorised as mediocre or poor condition using the standard criteria would be categorised as poor using the revised criteria. The current condition of the PM-CMB and I-FBS road sections on the Queensland state-controlled road network, when assessed using the revised condition criteria presented in Table 5.8, are presented in Table 5.9 and Table 5.10 respectively.

Table 5.9: CMB network condition distribution with new evaluation criteria

Evaluation criteria	Roughness (%)	Rutting (%)	Fatigue cracking (%)
Excellent	91.6	79.9	98.8
Good	7.5	12.5	0.3
Mediocre	0.8	6.0	0.4
Poor	0.1	1.6	0.5

Table 5.10: FBS network condition distribution with new evaluation criteria

Evaluation criteria	Roughness (%)	Rutting (%)	Fatigue cracking (%)
Excellent	79.8	47.8	91.7
Good	14.4	28.6	0.6
Mediocre	5.2	15.4	1.6
Poor	0.5	8.2	6.1

Despite the application of more stringent criteria, the majority of PM-CMB road sections were still categorised as excellent condition, with a small portion (≈ 13%) of the sections exhibiting good rutting condition. The roughness and fatigue cracking condition of the majority of the I-FBS sections were categorised as either excellent or good according to the revised criteria. However, a greater distribution of performance was established based on the revised rutting criteria. It should be noted that the majority of the PM-CMB and I-FBS road network, approximately 92.4% and

76.4% respectively, was still categorised as either excellent or good condition. The principal distress mechanism for both stabilisation technologies appears to be permanent deformation, as considerable portions of the PM-CMB and I-FBS sections, 7.6% and 23.6% respectively, are exhibiting more than 7.0 mm of rutting. The details of the condition categorisation for each of the identified 100 m PM-CMB and I-FBS road sections, inclusive of region, stabilising agent content, structural layer thickness and environmental zone are included in Table A 1 and Table A 2 of Appendix A.

The revised roughness, rutting and fatigue cracking condition criteria were established solely for the purpose of identifying representative pavement sections in this study and should not be applied for general network condition assessment. The revised condition categories (excellent, good, mediocre and poor) used in this study do not reflect best practice, as established by TMR (2012a) and Austroads (2009b), for the assessment of road pavement networks.

5.2.6 Condition Deterioration Rate

In addition to categorisation according to current condition, as represented by the most recent roughness, rutting and fatigue cracking measurements, categorisation according to rate of deterioration was also investigated. The change in condition over the service life of the pavement surface was examined as an indicator of stabilised material performance. It is acknowledged that the rate of condition deterioration is subject to a number of influencing factors including structural composition, construction practices, environmental conditions and variations in traffic loading. The condition deterioration rate for each of the primary distresses (roughness, rutting and fatigue cracking) was determined by fitting a linear trend to available ARMIS data. A number of methods have been successfully utilised by engineers to evaluate network performance based on condition deterioration rate. The linear rate of progression (LRP) methodologies proposed by both Martin, Hoque and Roper (2004) and Hunt (2002) were considered as part of this investigation.

Refinement of the identified PM-CMB and I-FBS road sections into a subnetwork of good and poor performing sections, according to current condition, identifies road sections at either end of the traditional pavement serviceability decay model. However, refinement according to deterioration rate identifies road sections both at the ends (low rate) and at the critical break-over point (high rate) of the serviceability trendline. Condition assessment according to deterioration rate is particularly valuable for the identification of intermediate age road sections undergoing increasing damage, but with surface distress measurements that have not yet triggered traditional asset management condition criteria.

The development of accurate and reliable rates of deterioration requires detailed time-series condition data. Ideally, condition data would include initial post-construction assessments and each annual measurement through to the present day. The available ARMIS data for the PM-CMB and I-FBS road sections contained a significant number of information gaps where single or multiple years of condition data are incomplete. A manual manipulation process was conducted to populate the missing data by conducting linear interpolation between previous and subsequent condition records, where available. The roughness, rutting and fatigue cracking deterioration rates for each of the identified 100 m road sections were calculated by deriving the slope of a linear trend of condition variation. The condition variation was calculated using the change in condition relative to the previously available assessment measurement and the associated time interval (years).

The condition deterioration rate criteria presented in Table 5.11 were established according to best practice, as presented in the TMR *Pavement Rehabilitation Manual* (2012a), for the categorisation of the PM-CMB and I-FBS road sections along the Queensland state-controlled road network as either good or poor according to roughness and rutting surface distresses.

Table 5.11: Deterioration rate evaluation criteria

Performance level	Roughness (counts/year)		Rutting (mm/year)	
	I-FBS	PM-CMB	I-FBS	PM-CMB
Good	< 3	< 2	< 2	< 1
Poor	≥ 3	≥ 2	≥ 2	≥ 1

Categorisation of the identified I-FBS and PM-CMB road sections according to the deterioration rate criteria presented in Table 5.11 resulted in the distribution of good and poor performing pavements, relative to roughness and rutting, shown in Table 5.12.

Table 5.12: I-FBS and PM-CMB network condition distribution with deterioration rate criteria

Performance level	Roughness (%)		Rutting (%)	
	I-FBS	PM-CMB	I-FBS	PM-CMB
Good	96.3	98.9	90.3	98.6
Poor	3.7	1.1	9.7	1.4

Similar to the condition-based categorisation, the majority of PM-CMB and I-FBS road sections, approximately 90.3% and 98.6% respectively, were observed to provide good performance. However, significantly more sections were categorised as poor, including 3.7% and 9.7% of the I-FBS sections according to roughness and rutting respectively, compared to 0.0% and 1.4% assessed according to the standard condition criteria presented in Table 5.5. On closer inspection, the identified sections categorised as poor, according to condition deterioration rate, included the majority of sections classified according to current condition, in addition to a number of relatively young (< 5 years) road sections. Review of survey video images for these early-life, high deterioration rate sections was conducted to validate the current condition and ensure that the calculated deterioration rates were confirmed by surface distresses. Unanimously, the occurrence of either functional or structural pavement distress was not observed for the early-life, high deterioration rate sections. It is proposed that new stabilised pavements undergo a post-construction ‘settling in’ period, during which roughness and rutting increase rapidly for a short time before levelling off. Care must be exercised when assessing newly constructed road sections according to condition deterioration rate to avoid triggering superfluous treatments.

The general lack of consistent datasets, in addition to the misidentification of early-life sections, resulted in adoption of the condition-based categorisation (Table 5.9 and Table 5.10) for the selection of representative sections and consideration of condition deterioration rate as a secondary screening measure.

6 SELECTION OF REPRESENTATIVE SECTIONS

Determination of the project constraints maximising the performance of PM-CMB and I-FBS base pavements required investigation of the design assumptions, in situ constraints and as-constructed pavement properties relative to long-term performance. The vast extents of the Queensland state-controlled road network prevented the detailed investigation of all of the pavements incorporating the PM-CMB or I-FBS base layers. Information within the ARMIS database was examined to identify and categorise PM-CMB and I-FBS sections according to region, environmental zone, age, stabilised layer thickness, traffic volume, design stabilising agent content and general condition (excellent, good, mediocre or poor) based on 2013 surveys of roughness and rutting, as presented in Section 5. The categorisation allowed for sorting of the identified PM-CMB and I-FBS base pavements into subnetworks with similar project variables and performance. The grouping of common pavement sections allowed for the selection and detailed investigation of a reduced number of pavement sections that were still representative of the range of stabilisation practices in Queensland.

6.1 Methodology

Documentation of the stabilisation practices throughout Queensland required evaluation of the relative performance of the range of parent materials, stabilising agent types and contents, pavement configurations, prevalent environments and traffic loading typically encountered throughout the state. The principal differences in the utilisation of both CMB and FBS base technologies in Queensland stem from the type and quantity of stabilisation agent, structural thickness of the stabilised layer and the maximum lift thickness allowed during construction. Additionally, opportunities for significant process improvements, including reducing cost and inherent conservatism, may arise from reductions in stabilising agent content and/or layer thickness. Therefore, the selection of representative sections combined elements of mixture proportioning, structural design and performance to identify road sections with the following properties:

- thin stabilised layers (< 200 mm) with roughness **and** rutting condition categorised excellent or good
- thick stabilised layers (≥ 200 mm) with roughness **or** rutting categorised poor
- low stabiliser content (PM-CMB < 2%; I-FBS < 3.5%) with roughness **and** rutting categorised excellent or good
- high stabiliser content (PM-CMB ≥ 2%; I-FBS ≥ 3.5%) with roughness **or** rutting categorised poor.

Identified PM-CMB and I-FBS road sections conforming to the above criteria were subjected to further review including calculation of cumulative traffic, review of condition deterioration rates and estimation of relative maintenance cost. Additionally, the most recent and available survey video images were examined to validate the condition of the road sections before selection for further investigation.

6.2 Representative Sections

Thirteen PM-CMB and fourteen I-FBS road sections were identified as meeting the criteria for further study according to the methodology outlined in Section 6.1. The representative PM-CMB and I-FBS sections with associated TMR road ID, environmental zone, start and end chainage, design layer thickness and stabiliser content, stabilised layer construction year and the year of the most recent surfacing are presented in Table 6.1 and Table 6.2 respectively.

Table 6.1: Summary of selected CMB network

Region	Road ID	Environmental zone	Start chainage (km)	End chainage (km)	Layer thickness (mm)	Stabiliser content (%)	Construction year	Surfacing year
Far North	10N	WNR	17.712	18.052	250	3.0	2011	2011
Far North	10N	WNR	18.448	21.002	250	3.0	2011	2011
Northern	10L	WNR	70.2	71.1	250	3.0	2010	2010
Northern	10M	WNR	121.453	121.705	200	1.5	2008	2008
Northern	10M	WNR	8.361	8.615	180	2.0	2005	2012
Northern	10M	WNR	20.115	20.302	150	2.0	2009	2009
Northern	10M	WNR	20.302	20.435	150	1.0	2009	2009
Mackay/Whitsunday	33B	WNR	69.199	72.495	150	2.0	2003	2004
Mackay/Whitsunday	33B	WNR	72.762	74.114	150	2.0	2003	2004
Mackay/Whitsunday	530	WNR	1.054	3.112	340	1.0	2009	2009
Mackay/Whitsunday	854	WNR	0.943	1.668	200	2.0	2001	2002
Mackay/Whitsunday	856	WNR	4.426	5.912	330	1.5	2005	2005
Mackay/Whitsunday	856	WNR	5.912	6.291	330	1.5	2005	2005

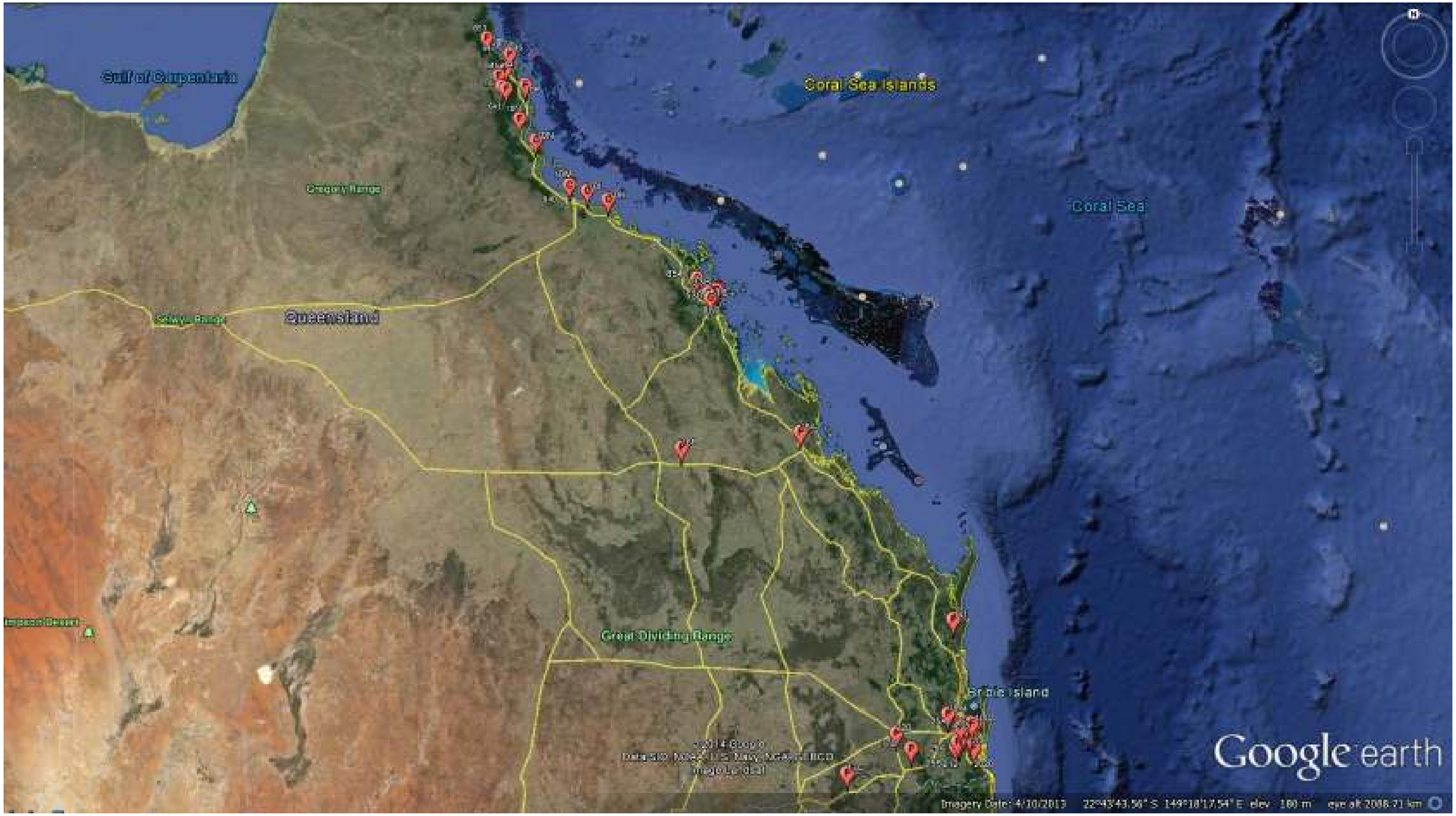
Table 6.2: Summary of selected FBS network

Region	Road ID	Environmental zone	Start chainage (km)	End chainage (km)	Layer thickness (mm)	Stabiliser content (%)	Construction year	Surfacing year
Far North	10P	WNR	50.33	54.328	250	3.5B+1.5C	2013	2013
Fitzroy	16B	DR	122.0	122.98	270	3.0B+1.0L	2005	2007
Darling Downs	17C	DNR	104.21	105.61	200	4.0B+2.0C	1998	2007
Darling Downs	22B	DR	36.812	38.8	200	3.5B+1.5L	1999	2011
Darling Downs	22B	DR	38.8	42.75	200	3.5B+1.5L	1999	2011
Darling Downs	22B	DR	42.75	43.61	200	3.5B+1.5L	1999	2011
Darling Downs	22B	DR	43.61	46.87	200	3.5B+1.5L	1999	2011
Darling Downs	22B	DR	46.87	47.68	200	3.5B+1.5L	1999	2011
Darling Downs	22B	DR	52.74	55.66	200	3.5B+1.5L	1999	2011
South Coast	25B	DNR	10.16	13.42	300	3.5B+2.0L	2003	2009

Region	Road ID	Environmental zone	Start chainage (km)	End chainage (km)	Layer thickness (mm)	Stabiliser content (%)	Construction year	Surfacing year
South Coast	208	WR	3.16	3.732	250	3.5B+2.0L	2000	2000
South Coast	212	WR	3.8	3.9	300	3.0B+2.0L	2000	2008
South Coast	1003	WNR	3.21	4.92	300	3.5B+2.0L	2005	2009
South Coast	2020	WNR	8.75	10.35	200	3.5B+2.0L	2009	2009

The selected road sections are representative of the range of PM-CMB pavements in Queensland covering three TMR regions with variations in project length (133 m to 3296 m), stabilised layer design thickness (150 mm to 340 mm), target stabilising agent content (1.0% to 3.0%) and pavement age (2 years to 12 years). The selected I-FBS road sections are also representative of the variation in the application of the stabilisation technology covering four TMR regions, all of the environmental zone categories and variations in project length (100 m to 3998 m), stabilised layer design thickness (200 mm to 300 mm), bitumen content (3.0% to 4.0%), secondary stabilising agent type (cement and lime) and content (1.0% to 2.0%), in addition to pavement age (< 1 year to 15 years). The geographic locations of the representative sections are presented in Figure 6.1. In addition to the fourteen I-FBS road sections selected according to the methodology described in Section 6.1, three I-FBS sections nominated by the TMR project team were also included for further investigation. These sections were proposed for inclusion due to accelerated distress (rutting and shoving) development.

Figure 6.1: Relative location of selected representative road sections



Source: Google Earth (2014).

7 IN SITU EVALUATION

A well-maintained asset management system, such as the ARMIS database, can provide extremely valuable data on road pavement infrastructure including region, environmental zone, age, stabilised layer thickness, traffic volume, design stabilising agent content and condition. These parameters are essential for understanding the assumptions, constraints and properties that constitute the general pavement provisioning practice. However, differences in the design and as-constructed details routinely vary, sometimes significantly, with respect to road pavement infrastructure. The representative PM-CMB and I-FBS road sections selected in accordance with the methodology outlined in Section 6.1 were subject to further investigation including visual condition inspection (Section 7), determination of structural capacity (Section 8) and laboratory characterisation of stabilised and subgrade materials (Section 9) to validate the inventory and condition data extracted from the ARMIS database and investigate relationships between mixture proportioning, structural design and construction practices with in-service performance.

7.1 Visual Inspection

The purpose of the site inspection was to confirm the current condition of the selected road sections, identify any abnormal features and refine the extents prior to subsequent structural capacity assessment. Every effort was made to ensure accurate assessment of the condition of the sections using available ARMIS data and video survey images. However, some distresses such as rutting, shoving, depression, corrugation and fine cracking can be difficult to distinguish from video images. The 100 m aggregation of ARMIS data can also result in the extrapolation of isolated defects, significantly affecting the reported condition of the entire section. Finally, due to the significant cost of structural capacity assessment using a falling weight deflectometer (FWD), opportunities to reduce the size of the selected network were explored.

Prior to inspection, a number of representative road sections were eliminated from consideration due to logistical constraints. Mount Ossa – Seaforth Road (854), the Capricorn Highway (225) between Duaringa and Emerald and Beechmont Road (230) are isolated in remote locations requiring significant mobilisation investment for further study. Additionally, the Cunningham Highway (231) between Warwick and Inglewood was found to be under reconstruction in late March 2014.

Visual inspection of the remaining 16 combined PM-CMB and I-FBS road sections was conducted by the ARRB Group project team on 2 and 7–10 April 2014. Site inspections on 2 April included I-FBS road sections in South-East Queensland (SEQ). The inspections conducted on 7–10 April included both PM-CMB and I-FBS sections in northern Queensland, starting in the Far North and finishing in the Mackay/Whitsunday region. The site inspections consisted of locating the road section, correlating TMR road network chainage and GPS (latitude and longitude) coordinates, detailing features and distresses, in addition to photo documentation. A condition summary for the selected road sections based on the findings of the visual inspection is presented in Table 7.1.

Table 7.1: Summary of visual inspection assessment

Road name	Road ID	Start chainage	End chainage	Surface age (years)	Rutting	Roughness	Surface condition
Stapylton-Jacobs Well	1003	3210	4920	5	Low	Low	Good; isolated longitudinal cracking
Beenleigh Connection	208	3060	3732	14	Medium	Medium	Poor; low severity block, longitudinal & transverse cracking; patching
Beaudesert-Boonah	212	3240	4020	3	Low	Low	Good; light flushing

Road name	Road ID	Start chainage	End chainage	Surface age (years)	Rutting	Roughness	Surface condition
New England Hwy	22B	36812	55660	11	NA	NA	NA
Mt. Lindesay Hwy	25B	10300	13500	5	Low	Medium	Mediocre; isolated longitudinal cracking; medium flushing
Bruce Hwy	10P	50330	54828	2	Medium	Medium	Mediocre; medium flushing
Bruce Hwy	10P	64110	65250	8	High	Medium	Mediocre; medium flushing
Bruce Hwy	10N	123400	124400	-	Low	High	Poor; isolated longitudinal cracking; medium severity shoving; heavy flushing
Bruce Hwy	10N	103400	104400	-	High	High	Poor; high severity shoving; heavy flushing
Bruce Hwy	10N	110800	111800	-	Low	High	Poor; low severity fatigue cracking; medium severity shoving; potholing; heavy flushing
Bruce Hwy	10N	18448	21002	4	Low	High	Mediocre; low severity longitudinal cracking; extensive patching; light flushing
Bruce Hwy	10M	118680	121710	6	Low	Low	Good; medium flushing
Bruce Hwy	10L	35938	37692	0	Nil	Low	Good; new pavement
Mackay Bypass	530	1054	3112	5	Low	Medium	Good; isolated longitudinal & transverse cracking; patching
Mackay-Bucasia	856	4426	6291	4	Low	Medium	Mediocre; medium severity longitudinal & transverse cracking
Peak Downs Hwy	33B	69199	74114	4	High	High	Poor; high severity shoving; patching; heavy flushing

7.1.1 Stapylton-Jacobs Well Road (1003: 3.21 km to 4.92 km)

Stapylton-Jacobs Well Road is a secondary route in SEQ approximately 45 km south-east of the Brisbane central business district (CBD). The roadway serves commuter traffic between the Pacific Motorway and the Stapylton and Gilberton communities with AADT of 3539 and 11% heavy vehicles measured in 2013. The road section was constructed in 2005 and included a 300 mm I-FBS layer (3.5% bitumen and 2% lime) overlying a 300 mm select-fill subgrade. The road section received a 60 mm asphalt structural overlay in 2009. The pavement was designed to service 6.90×10^6 ESA, 94% of which had been consumed by January 2013. All 16 of the 100 m road sections were categorised as excellent condition in the desktop review.

Figure 7.1: Stapylton-Jacobs Well Road overview (left) and surface condition (right)

During the visual inspection the road section was observed to be in generally good condition with no significant distress as shown in Figure 7.1. Some low severity wheelpath rutting and longitudinal cracking were discovered at the beginning of the road sections (chainage 3.2 km to 3.3 km). A high volume of heavy commercial traffic was also observed. At the time of inspection, the road section was a good example of an intermediate life (ten years) pavement in good condition.

7.1.2 Beenleigh Connection Road (208: 3.06 km to 3.732 km)

Beenleigh Connection Road is a main route in SEQ approximately 35 km south-east of the Brisbane CBD. The arterial road serves traffic between the Pacific Motorway and the Mount Warren Park community with AADT of 17 819 and 7% heavy vehicles measured in 2013. The road section was constructed in 2000 and consists of a 50 mm asphalt surfacing and 250 mm I-FBS layer (3.5% bitumen and 2% lime) overlying the natural subgrade. The pavement was designed to service 3.30 x 10^6 ESA. However, the cumulative traffic measured in January 2013 had exceeded the design life by 332%. The maintenance cost for this section since 2005 is high compared to the other selected road sections. The condition was categorised as mediocre during the desktop review due to the frequent occurrence of low-severity rutting and cracking.

Figure 7.2: Beenleigh Connection Road overview (left) and surface condition (right)

During the site inspection (Figure 7.2), regularly spaced low to medium severity transverse cracking and low-severity block cracking were noted. A significant volume of traffic, consisting primarily of passenger vehicles, was observed. Signs of several isolated repair treatments were noted, including a number of small patches. The overall condition of the road section at the time of inspection was poor and approaching failure. The road section was a good example of a pavement exhibiting a range of conditions (excellent to poor) that is approaching the end of the service life.

7.1.3 Beaudesert-Boonah Road (212: 3.24 km to 4.02 km)

Beaudesert-Boonah Road is a main route in SEQ approximately 70 km west of the Gold Coast. The intraregional roadway connects the rural communities in SEQ and north-east New South Wales with the urban centres of Brisbane, Toowoomba, and the Gold Coast. The road section served an AADT of 2768 and 19% heavy vehicles in 2013. The road section was built in 2000 and consists of a sprayed seal surfacing, 300 mm I-FBS layer (3.5% bitumen and 2% lime), and 200 mm cement-treated subbase overlying the natural subgrade. The pavement was designed to service 2.0 x 10^6 ESA. However, the cumulative traffic measured in January 2013 had exceeded the design life by 250%. The section received resurfacing treatments in 2011 including high-pressure retexturing and a polymer modified binder (PMB) reseal. Following the treatments, the annual maintenance costs were still high relative to the surrounding road sections. Most of the road sections were categorised as excellent during the desktop condition categorisation, with three 100 m sections categorised as good.

Figure 7.3: Beaudesert-Boonah Road overview (left and right)

The road section was noted as generally good condition with some isolated low-severity rutting and light wheelpath flushing during the site inspection as shown in Figure 7.3. No signs of extensive maintenance were observed, indicating the maintenance cost information may be inaccurate. Indication of recent flooding was detected in addition to a significant volume of heavy vehicles. The road section is a good example of a high-performing resilient structure given the excellent condition despite the high volume of heavy vehicle traffic and suspected drainage issues.

7.1.4 New England Highway (22B: 36.812 km to 55.66 km)

The New England Highway is a state highway connecting Yarraman in SEQ with Newcastle in eastern New South Wales. The New England Highway is a major interregional route catering to both rural and urban communities. The roadway served an AADT of 4003 and 15% heavy vehicles in 2013. The road section was constructed in 2003 and consists of a geotextile reinforced sprayed seal surfacing, 200 mm I-FBS layer (3.5% bitumen and 1.5% lime), 200 mm unbound granular subbase, and 100 mm select fill subgrade. The pavement was designed to service 3.0 x 10^6 ESA. However, the cumulative traffic measured in January 2013 had exceeded the design life by 166%. The New England Highway was the subject of a recent Austroads (2013b) study, *Improved Design of Bituminous Stabilised Pavements*. The study included a detailed review of site conditions, mixture proportioning, structural design, construction and maintenance practices, in addition to surface condition and structural assessment. To avoid duplication of effort, a site inspection was not conducted for the New England Highway, but it was subjected to structural capacity assessment and consideration for laboratory material characterisation.

7.1.5 Mount Lindesay Highway (25B: 10.3 km to 13.5 km)

The Mount Lindesay Highway is a state highway connecting the Logan Motorway in SEQ to Woodenbong in north-east New South Wales. The roadway is a significant intraregional route connecting rural communities with the Brisbane metropolitan area. Traffic volumes for 2013 included an AADT of 1770 and 14% heavy vehicles. The road section was built in 2003 and consists of a sprayed seal surfacing, 350 mm I-FBS layer (3.5% bitumen and 2% lime), and 200 mm select fill subgrade. The pavement was designed to service 3.20 x 10^6 ESA, 81% of which had been consumed by January 2013. The section received a high-pressure retexturing treatment in 2009 but the original surfacing is intact. The majority of the road sections were categorised as excellent or good condition during the desktop review, with a few poor sections as a result of high-severity rutting.

Figure 7.4: Mount Lindesay Highway overview (left) and surface condition (right)

The road section was observed to be in an overall mediocre condition with some low-severity rutting, low-severity longitudinal cracking and patching in isolated areas. The general condition of the investigated section is presented in Figure 7.4. Additionally, localised high-severity cracking associated with edge breaking was noted. These issues are most probably the result of differential volumetric changes as opposed to structural overload. At the time of the inspection, the road section was a good example of a satisfactorily performing intermediate life (ten years) pavement exhibiting variable surface condition.

7.1.6 Bruce Highway (10P: 50.33 km to 54.828 km)

The Bruce Highway is a 1700 km major state highway running adjacent to the Queensland coastline, connecting Brisbane at the southern end to Cairns at the northern end. The road section under investigation is located approximately 35 km south of the Cairns CBD. The interregional route caters to both commuter and commercial traffic including an AADT of 5593 and 14% heavy vehicles in 2013. The road section was constructed in 2012 and consists of a spray seal surfacing, 250 mm I-FBS layer (3.5% bitumen and 2% lime), and 60 mm lime-stabilised subgrade overlying the natural foundation. The original design life for the pavement section is unknown, but in 2013 the cumulative traffic to date was 1.32 x 10^6 ESA. In the first two years of service, two consecutive 100 m sections were categorised as poor, in addition to a number of sections categorised as mediocre due to high-severity (11.4 mm and 12.0 mm) and medium-severity (8.9 mm and 8.7 mm) rutting.

Figure 7.5: Bruce Highway (10P) overview (left and right)

As a result of the site inspection (Figure 7.5), the road section was recategorised as mediocre condition with a few isolated areas with low-severity and medium-severity rutting. A close inspection of the trouble areas revealed that the measurement of deformation indicative of high-severity rutting may be inflated due to the compound curvature of the road surface. The one metre wide 'hump' straddling the centreline may lead to nonrepresentative measurements of rutting in the inside wheelpath. In addition to the rutting, low to medium severity flushing was also observed in isolated areas. This road section is an example of a young pavement with isolated areas of accelerated deterioration as a result of poor construction practice.

7.1.7 Bruce Highway (10P: 64.11 km to 65.25 km)

The Bruce Highway is a 1700 km major state highway running adjacent to the Queensland coastline, connecting Brisbane at the southern end to Cairns at the northern end. The road section under investigation is located outside of Gordonvale, approximately 24 km south of the Cairns CBD. The interregional route caters to both commuter and commercial traffic. This part of the Bruce Highway served an AADT of 15 077 and 8% heavy vehicles in 2013. The road section was built in 2006 and consists of a spray seal surfacing, 250 mm I-FBS layer (3.0% bitumen and 2% lime) and 100 mm unbound granular subbase overlying the natural subgrade. The original design life for the pavement section is unknown, but in 2013 the cumulative traffic to date was 1.0×10^7 ESA. The desktop condition evaluation categorised the majority of the road sections as mediocre in addition to a number of poor sections according to the rutting criteria. While only a few road sections were categorised as poor, the remaining sections categorised as mediocre exhibited absolute rutting values nearing the poor criteria limits. The sections classified as mediocre exhibited rutting values ranging from 6.7 mm to 8.3 mm.

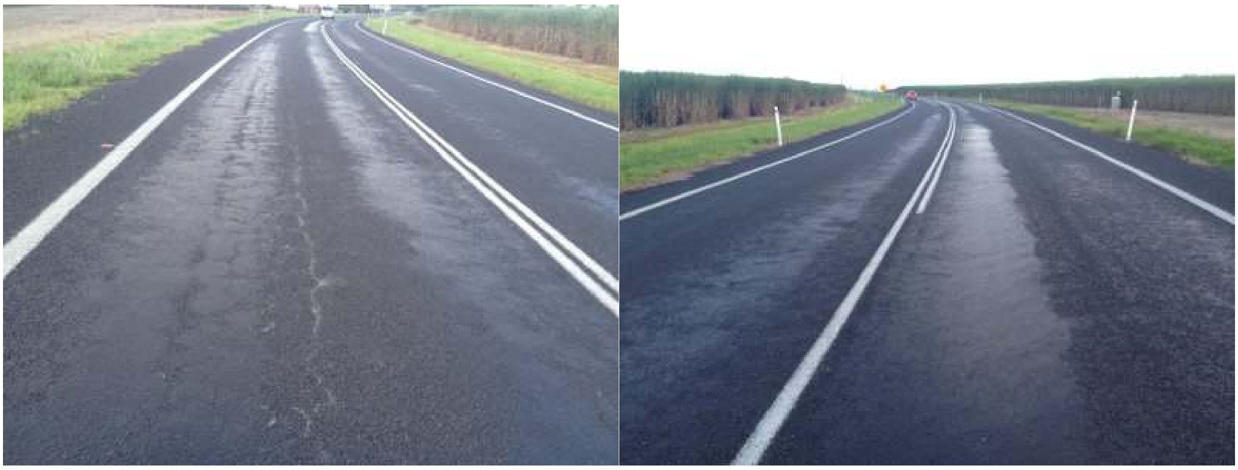
Figure 7.6: Bruce Highway (10P) overview (left and right)

The medium-severity and high-severity rutting in the condition data was confirmed during the site inspection. The identified road section is along a shallow curve with one-way crossfall sloping to the east as shown in Figure 7.6. The condition of the northbound and southbound lanes was found to differ significantly, with the medium-severity and high-severity rutting localised within the southbound lane. A poorly maintained drainage structure on the east side of the alignment was also noted. The northbound lane was observed to be in generally good condition with some light flushing in the wheelpaths. Considering the age of the pavement and the high traffic volume, this road section is a good example of a rapidly deteriorating structure as a result of the combined influence of environmental conditions and poor maintenance.

7.1.8 Bruce Highway (10N: 123.4 km to 124.4 km)

The Bruce Highway is a 1700 km major state highway running adjacent to the Queensland coastline, connecting Brisbane at the southern end to Cairns at the northern end. The road section under investigation is located approximately 112 km south of the Cairns CBD. The interregional route caters to both commuter and commercial traffic. This road section was one nominated by the TMR project team due to the occurence of high-severity distresses. The AADT on this part of the Bruce Highway was 5588 with 16% heavy vehicles in 2013. Details on the pavement are limited, but it is thought to consist of a spray seal surfacing, 250 mm I-FBS base, 300 mm unbound granular subbase, and 300 mm select fill subgrade. The original design life for the pavement section is also unknown, but in 2013 the cumulative traffic to date was 2.31 x 10^6 ESA. Due to the limited availability of inventory, material and performance information, condition categorisation for this road section was not undertaken.

Figure 7.7: Bruce Highway (10N) overview (left and right)



During the visual inspection (Figure 7.7) the condition of the road section was observed to be poor as a result of extensive high-severity rutting and cracking. Roughness could not be quantitatively measured but was expected to be categorised as poor condition as well. The degradation of some areas, such as located at TMR road network chainage 123.5 km, is so severe as to pose immediate danger to the travelling public due to limited skid resisitance resulting from high-severity rutting and flushing. At the time of the site inspection, this road section was an example of a pavement at terminal failure condition.

7.1.9 *Bruce Highway (10N: 103.4 km to 104.4 km)*

The Bruce Highway is a 1700 km major state highway running adjacent to the Queensland coastline, connecting Brisbane at the southern end to Cairns at the northern end. The road section under investigation is located between Djarawong and Midgenoo townships, approximately 132 km south of the Cairns CBD. The interregional route caters to both commuter and commercial traffic. This road section was nominated by the TMR project team due to the observation of high-severity distresses. Details on the I-FBS material and structural composition were not available. The design serivce life for the pavement is also unknown, but the cumulative traffic to date measured in 2013 was 1.49 x 10^7 ESA. Due to the limited availability of inventory, material and performance information, condition categorisation for this road section was not undertaken.

Figure 7.8: Bruce Highway (10N) overview (left) and surface condition (right)

During the visual inspection the road section was discovered to be in poor condition as a result of high-severity rutting and heavy flushing as shown in Figure 7.8. High-severity shoving was also identified in a number of locations and may be contributing to the severity of the observed rutting. Fatigue cracking and patching were also noted in a number of locations. Evidence of previous sampling was also observed including patched core holes and trenches. At the time of the site inspection, this road section was an example of a pavement at terminal failure condition.

7.1.10 Bruce Highway (10N: 110.8 km to 111.8 km)

The Bruce Highway is a 1700 km major state highway running adjacent to the Queensland coastline, connecting Brisbane at the southern end to Cairns at the northern end. The road section under investigation is located outside El Arish, approximately 125 km south of the Cairns CBD. The interregional route caters to both commuter and commercial traffic. This road section was nominated by the TMR project team due to the observation of high-severity distresses. Details on the I-FBS material and structural composition were not available. The design serivce life for the pavement is also unknown, but the cumulative traffic to date measured in 2013 was 3.71×10^6 ESA. Due to the limited availability of inventory, material and performance information, condition categorisation for this road section was not undertaken.

Figure 7.9: Bruce Highway (10N) overview (left) and surface condition (right)

During the visual inspection the condition of the road section was observed to be poor as a result of high-severity rutting and high-severity fatigue cracking as shown in Figure 7.9. Evidence of significant repair and maintenance works in the form of patching and sealing was also discovered. At the time of the site inspection, this road section was an example of a pavement at terminal failure condition.

7.1.11 Bruce Highway (10N: 18.448 km to 21.002 km)

The Bruce Highway is a 1700 km major state highway running adjacent to the Queensland coastline, connecting Brisbane at the southern end to Cairns at the northern end. The road section under investigation is located outside of Rungoo, approximately 133 km north of the Townsville CBD. The interregional route caters to both commuter and commercial traffic including an AADT of 2705 and 20% heavy vehicles in 2013. This road section was built in 2010 and consists of a spray seal surfacing, 250 mm PM-CMB layer (3% stabilising agent), 100 mm CMB subbase, and 180 mm select fill subgrade. The original design life for the pavement section is unknown, but in 2013 the cumulative traffic to date was 7.22×10^6 ESA. The condition of the road section was categorised as poor due to prevalent high-severity rutting during the desktop review.

Figure 7.10: Bruce Highway (10N) overview (left and right)

A detailed assessment of the pavement condition at the time of the site inspection was not possible due to the presence of moisture at the pavement surface as a result of heavy rain (Figure 7.10). The occurrence of high-severity rutting could not be validated. Significant portions of the wheelpaths in both directions had been patched with asphalt. It was suspected that the patching resulted from recent repair works to address the high-severity rutting. Considering the reported traffic volumes, the accelerated deterioration of this road section was surprising. The recent corrective overlays and inability to validate the condition limits the value of this road section as a representative example.

7.1.12 Bruce Highway (10M: 118.68 km to 121.71 km)

The Bruce Highway is a 1700 km major state highway running adjacent to the Queensland coastline, connecting Brisbane at the southern end to Cairns at the northern end. The road section under investigation is located outside of Ingham, approximately 110 km north of the Townsville CBD. The interregional route caters to both commuter and commercial traffic including an AADT of 6991 and 15% heavy vehicles in 2013. This road section was constructed in 2008 and consists of a spray seal surfacing, 300 mm PM-CMB layer (2% stabilising agent), 150 mm unbound granular subbase, and 150 mm select fill subgrade. The pavement was designed to service 1.19 x 10^7 ESA, 66% of which had been consumed by January 2013. The overall condition of the road section was categorised as good during the desktop review, with the majority of the sections excellent, eight sections categorised as good and five categorised as mediocre, all as a result of the rutting criteria.

Figure 7.11: Bruce Highway (10M) overview (left and right)

The overall good condition of the road section was confirmed during the site inspection. Isolated areas of low-severity rutting and light wheelpath flushing were observed as shown in Figure 7.11. Signs of recent inundation were detected but without any associated structural pavement distress. The road section is an example of an intermediate life (six years) pavement exhibiting good performance in a high-risk environment.

7.1.13 Bruce Highway (10L: 35.938 km to 37.692 km)

The Bruce Highway is a 1700 km major state highway running adjacent to the Queensland coastline, connecting Brisbane at the southern end to Cairns at the northern end. The road section under investigation is located approximately 51 km south of the Townsville CBD. The interregional route caters to both commuter and commercial traffic including an AADT of 5211 and 16% heavy vehicles in 2013. This road was constructed in 2004 and consists of a spray seal surfacing, 200 mm PM-CMB layer (2% stabilising agent), 185 mm cement-bound subbase and 300 mm stabilised subgrade. The pavement was designed to service 9.47 x 10^6 ESA. However, the cumulative traffic measured in January 2013 had exceeded the design life by 179%. The road has provided 10 years of service and the desktop condition assesssment categorised the road sections as good to mediocre, both as a result of the rutting criteria.

Figure 7.12: Bruce Highway (10L) overview (left) and surface condition (right)

Upon locating the road section, the ARRB Group project team discovered that a resurfacing had been concluded the previous day (Figure 7.12). However, TMR North Region staff were on site and advised that the road was in good condition prior to resurfacing and no structural capacity improvement treatments were applied. The road section had required minimum maintenance and the resurfacing was part of the programmed maintenance plan. The road section is a good example of a well-performing pavement incorporating reduced stabilising agent content and structural layer thickness compared to other representative PM-CMB road sections.

7.1.14 Mackay Bypass Road (530: 1.054 km to 3.112 km)

Mackay Bypass Road is a main route on the eastern coast in central Queensland approximately 5.0 km west of the Mackay CBD. The road provides an alternative route for commuter traffic travelling along the Bruce Highway. The bypass serves both urban and rural communities in western Mackay and the Meadowlands with a 2013 AADT of 15 477 and 6% heavy vehicles. The road was constructed in 2009 and consists of 50 mm asphalt surfacing, 340 mm PM-CMB layer (1% cement) and 250 mm stabilised subgrade. The pavement was designed to service 7.10 x 10^6 ESA. However, the cumulative traffic measured in January 2013 had exceeded the design life by 142%. The desktop condition assessment categorised the majority of the road sections as excellent. However, there were two consecutive 100 m sections on either end categorised as mediocre according to the rutting criteria. The sections categorised as mediocre are exhibiting medium to high severity rutting approaching the poor condition rutting criteria.The location of these sections at the extents of the project suggests potential construction issues.

Figure 7.13: Mackay Bypass Road overview (left) and surface condition (right)

The road section is relatively young (four years) and unlike the majority of representative sections, has been surfaced with a thin (45 mm) asphalt surfacing. The site inspection confirmed the overall good condition of the pavement (Figure 7.13). Additionally, the medium-severity rutting was only observed at the project extents. Low-severity longitudinal cracking was also observed throughout the section and appeared to be associated with underlying construction joints. The road section is an example of utilisation of reduced stabilising agent content and incrased structural thickness in a high-volume traffic application.

7.1.15 Mackay-Bucasia Road (856: 4.426 km to 6.291 km)

Mackay-Bucasia Road is a main route in central coastal Queensland approximately 11.0 km north of the Mackay CBD. The road serves commuter traffic in the northern suburbs of Richmond, Beaconsfield, Rural View, Eimeo and Bucasia with an AADT of 26 323 and 6% heavy vehicles in 2013. This road was buit in 2010 and consists of a 70 mm asphalt surfacing and 330 mm PM-CMB layer (1.5% stabilising agent) overlying the natural subgrade. The pavement was designed to service 6.20 x 10^6 ESA. However, the cumulative traffic measured in January 2013 had exceeded the design life by 219%. The desktop condition assessment categorised the majority of the road sections as excellent with a few isolated good and mediocre sections as a result of the limiting rutting criteria.

Figure 7.14: Mackay-Bucasia Road overview (left and right)

No evidence of rutting was observed during the site inspection. However, low to medium severity longitudinal and transverse cracking was noted throughout the road section as shown in Figure 7.14. A number of the cracks were observed to be pumping fines from the subgrade, indicative of cracks that propagate through the entire structural pavement. It was suspected that the source of the cracking may be differential volumetric change in the subgrade, which generates and propagates cracking through the surfacing. The road section is an example of an early (four years) life pavement with increased potential for early failure due to high traffic volumes and potential foundation issues.

7.1.16 Peak Downs Highway (33B: 69.199 km to 74.114 km)

Peak Downs Highway is a state highway in central coastal Queensland approximately 20 km south-west of the Mackay CBD. The roadway is a rural route connecting the hinterland communities as far inland as Clermont to the Mackay metropolitan area with an AADT of 7461 and 12% heavy vehicles in 2013. This road was reconstructed in 2010 and consists of a spray seal surfacing, 150 mm PM-CMB layer (2% cement), and 200 mm cement-stabilised subbase overlying the natural subgrade. The pavement was designed to service 7.20 x 10^6 ESA. However, the cumulative traffic measured in January 2013 had exceeded the design life by 111%. The condition of approximately 90% of the road sections was categorised as poor during the desktop review according to the rutting criteria. However, it was also noted that the roughness condition for the majority of the road sections was either excellent or good.

Figure 7.15: Peak Downs Highway overview (left) and surface condition (right)

During the visual inspection the condition of the road sections was found to be poor as a result of medium to high severity rutting, isolated medium to high severity shoving and medium to heavy flushing in the wheelpaths. The general condition of the pavement surface is presented in Figure 7.15. Several patches were also observed indicative of previous repair and maintenance activities. Signs of recent inundation were also detected. The road section is an example of a rapidly deteriorating pavement, potentially as a result of environmental conditions.

7.2 Inspection Outcome

The objective of the site inspections was to validate the current condition of the representative sections, eliminate isolated defects from consideration and refine the subnetwork for further investigation. Generally, the condition categorisation based on ARMIS data was confirmed on site. In a few cases, the road surface geometry influenced the HSV measurements or isolated defects were extrapolated across an entire road section, reducing the overall condition. The 20 road sections presented in Table 7.2 have been selected for structural capacity assessment using the FWD. The sections and extents have been selected based on the criteria presented in Section 6.1, with the exception of those nominated by the TMR project team, and refined according to the findings of the site inspections. The selected sections include early, intermediate and late-life pavements covering a range of conditions and exhibited distresses. In total, 26.1 km of road alignment or 52.2 km of pavement when both traffic lanes are considered, were selected for further investigation.

Table 7.2: Roads selected for further investigation

Road Name	Road ID	Region	Material	Number of lanes	Direction	Length (km)	Start chainage	End chainage
Bruce Hwy	10L	Northern	PM-CMB	2	Both	1.8	35900	37700
Bruce Hwy	10M	Northern	PM-CMB	2	Both	1.0	118700	119700
Bruce Hwy	10N	Far North	PM-CMB	2	Both	2.3	19100	21400
Peak Downs Hwy	33B	Whitsunday	PM-CMB	2	Both	4.8	69200	74000
Mackay Bypass Rd	530	Whitsunday	PM-CMB	2	Both	0.3	1100	1400
Mackay Bypass Rd	530	Whitsunday	PM-CMB	2	Both	0.3	1500	1800
Mackay Bypass Rd	530	Whitsunday	PM-CMB	2	Both	0.4	2400	2800
Mackay-Bucasia Rd	856	Whitsunday	PM-CMB	2	Southbound	0.7	2600	3300
Mackay-Bucasia Rd	856	Whitsunday	PM-CMB	2	Southbound	1.9	4400	6300
Bruce Hwy	10N	Far North	I-FBS	2	Both	1.0	103400	104400
Bruce Hwy	10N	Far North	I-FBS	2	Both	1.0	110800	111800
Bruce Hwy	10N	Far North	I-FBS	2	Both	1.0	123000	124000
Bruce Hwy	10P	Far North	I-FBS	2	Both	1.0	51300	52300
Bruce Hwy	10P	Far North	I-FBS	2	Both	1.1	64100	65200
New England Hwy	22B	Darling Downs	I-FBS	2	Both	1.0	34400	35400
New England Hwy	22B	Darling Downs	I-FBS	2	Both	1.0	54700	55700
Mount Lindesay Hwy	25B	South Coast	I-FBS	2	Both	2.0	11500	13500
Beenleigh Connection Rd	208	South Coast	I-FBS	2	Both	0.8	3000	3800
Beaudesert-Boonah Rd	212	South Coast	I-FBS	2	Both	0.9	3200	4100
Stapylton-Jacobs Well Rd	1003	South Coast	I-FBS	2	Both	1.8	3200	5000

8 STRUCTURAL ASSESSMENT

The structural assessment of the selected representative road sections Table 7.2) was undertaken to provide a measure of the structural properties of the as-constructed pavements and an indication of future performance. The selected road sections included 26.1 km of combined PM-CMB and I-FBS pavements, representing approximately 10% of the Queensland state-controlled inventory for each respective stabilisation technology. The structural assessment included measurement of surface deflection under an impulse load using the FWD, determination of average characteristic deflection values and backcalculated layer moduli, in addition to estimation of allowable loading in ESAs for each 100 m section. An ARRB Group FWD was utilised for the evaluation and included testing at 25 m intervals along both the outer wheelpath (OWP) and between wheelpath (BWP) testing control lines. The assessments were carried out for the SEQ I-FBS sections during the periods 31 March to 1 April 2014 and 14 to 20 May 2014. The remaining PM-CMB and I-FBS sections in northern and central Queensland were assessed during the period 14 May to 13 June 2014.

8.1 Surface Deflection

The structural assessment of the selected pavement sections for both the PM-CMB and I-FBS portions of the state-controlled road network was undertaken using a FWD. The FWD imparts an impulse loading to the pavement surface, achieved by dropping a known weight a fixed distance onto rubber buffers that transmit the load to a circular plate (diameter = 300 mm) in contact with the pavement surface, and measures the resulting surface deflection at the centre and at fixed radial distances, commonly 200 mm, 300 mm, 450 mm, 600 mm, 750 mm, 900 mm, 1200 mm and 1500 mm, away from the loading plate. The FWD testing was accomplished for each of the selected road sections at a longitudinal offset of 25 m with a 12.5 m stagger between the OWP and BWP testing control lines. The testing protocol resulted in approximately eight sets of deflection measurements for each 100 m pavement section.

Each set of deflection measurements was normalised to a 40 kN load and averaged within each 100 m section to generate a characteristic deflection bowl. In addition to the deflection measurements, the curvature function, or the difference between the deflection at the centre of the loading plate (D0) and the deflection 200 mm away (D200), and the deflection ratio (DR), or the ratio of the deflection 250 mm away from the loading plate (D250) and D0, were calculated. As the D250 was not directly measured, a linear interpolation between the D200 and the deflection 300 mm away from the loading plate (D300) was utilised. The maximum deflection (D0), deflection 900 mm from the centre of the loading plate (D900), curvature function and DR for each 100 m segment of the selected pavement sections are presented in Appendix B. The D900 value is commonly utilised as an indicator of the subgrade bearing capacity (TMR 2012a) and was calculated in accordance with Equation 16.

$$CBR(\%) = 0.5883(D_{900})^{-1.479} \quad 16$$

where

CBR = the subgrade bearing capacity

D_{900} = the surface deflection resulting from a 40 kN FWD impulse load measured 900 mm away from the centre of the loading plate

Characteristic deflection values for each road section were determined by averaging the D0, D900, curvature and DR values for each 100 m segment within the selected pavement section and are presented in Table 8.1 and Table 8.2 for the PM-CMB and I-FBS pavement sections respectively.

Table 8.1: Summary of PM-CMB average deflections and allowable loading estimates

Road ID	Start chainage (m)	End chainage (m)	Average normalised deflection				Estimated subgrade CBR (%)	Estimated allowable loading (ESA)
			D0 (µm)	D900 (µm)	Curvature (µm)	DR		
10L	36 100	36 600	264	75	48	0.74	25	1.00E+08
10L	36 600	37 700	206	75	30	0.80	25	1.00E+08
10M	118 700	119 000	158	58	23	0.83	25	1.00E+08
10M	119 000	119 800	282	61	60	0.70	25	1.00E+08
10N	19 100	19 600	618	73	130	0.65	23	5.59E+07
10N	19 600	19 700	689	72	146	0.66	25	< 1.00E+05
10N	19 700	21 400	460	75	86	0.75	24	1.00E+08
33B	69 200	74 000	372	71	73	0.74	23	9.23E+07
530	1 100	1 200	170	65	27	0.79	25	1.00E+08
530	1 200	1 500	98	55	14	0.81	25	1.00E+08
530	1 500	1 700	87	29	17	0.73	25	1.00E+08
530	1 700	1 900	101	46	18	0.77	25	1.00E+08
530	2 400	2 500	181	91	20	0.82	20	1.00E+08
530	2 500	2 600	157	89	16	0.86	21	1.00E+08
530	2 600	2 800	135	67	18	0.81	25	1.00E+08
856	2 600	3 400	508	42	117	0.65	25	6.93E+07
856	4 400	4 500	486	67	98	0.70	25	1.00E+08
856	4 500	6 300	92	33	16	0.75	25	1.00E+08

Table 8.2: Summary of I-FBS average deflections and allowable loading estimates

Road ID	Start chainage (m)	End chainage (m)	Average normalised deflection				Estimated subgrade CBR (%)	Estimated allowable loading (ESA)
			D0 (µm)	D900 (µm)	Curvature (µm)	DR		
10N	103 400	104 400	407	75	75	0.74	24	1.00E+08
10N	110 800	111 200	143	51	28	0.77	25	1.00E+08
10N	111 200	111 400	216	76	34	0.81	23	1.00E+08
10N	111 400	111 800	379	84	72	0.76	23	1.00E+08
10N	123 000	124 000	423	112	61	0.81	17	8.76E+07
10P	51 300	52 300	148	60	17	0.85	25	1.00E+08
10P	64 100	65 200	260	67	48	0.76	24	1.00E+08
22B	34 400	34 600	244	50	53	0.73	25	1.00E+08
22B	34 600	34 800	234	54	40	0.79	25	1.00E+08
22B	34 800	35 400	327	121	50	0.84	15	9.52E+07
22B	54 700	55 600	279	100	46	0.81	18	1.00E+08
22B	55 600	55 700	344	110	57	0.78	15	1.00E+08
25B	11 500	13 400	345	124	48	0.84	15	9.76E+07
25B	13 400	13 500	156	80	11	0.90	25	1.00E+08
208	3 000	3 200	187	28	50	0.67	25	1.00E+08

Road ID	Start chainage (m)	End chainage (m)	Average normalised deflection				Estimated subgrade CBR (%)	Estimated allowable loading (ESA)
			D0 (µm)	D900 (µm)	Curvature (µm)	DR		
208	3 200	3 700	261	35	72	0.68	25	1.00E+08
208	3 700	3 800	367	41	115	0.60	25	1.00E+08
212	3 200	4 000	128	58	26	0.77	25	1.00E+08
212	4 000	4 100	229	62	78	0.63	25	1.00E+08
1003	3 200	3 300	320	47	89	0.64	25	1.00E+08
1003	3 300	4 600	249	80	58	0.73	23	1.00E+08
1003	4 600	4 900	233	86	48	0.76	23	1.00E+08
1003	4 900	5 000	449	101	120	0.67	17	1.00E+08

The TMR *Pavement Rehabilitation Manual* (2012a) provides a method for determining the allowable loading in ESA, for unbound granular pavements where the maximum deflection (D0) and subgrade bearing capacity in %CBR are known. This approach has be followed to estimate the remaining structural life of the representative PM-CMB and I-FBS pavement sections investigated in this study. It should be noted that the TMR approach is intended for unbound granular pavements where the principal failure mode is permanent deformation as a result of repeated deflection of the subgrade and overlying layers and will provide a conservative estimate of structural capacity when applied to pavements incorporating bound structural layers. The estimated allowable loading, determined in accordance with the TMR *Pavement Rehabilitation Manual* (2012a), for the selected PM-CMB and I-FBS pavement sections is presented in Table 8.1 and Table 8.2 respectively.

According to the TMR method, significant structural capacity remains for the selected pavement sections investigated as part of this study. The estimate of allowable loading for the majority of pavement sections is the maximum value possible using the TMR *Pavement Rehabilitation Manual* (2012a) of 1.0×10^8 ESA. Four PM-CMB pavement sections and three I-FBS pavement sections did not achieve the maximum prediction for remaining structural life. The Bruce Highway (10N) sections between chainages 19 100 and 19 700, the Peak Downs Highway (33B) and Mackay-Bucasia Road between chainages 2600 and 3400 displayed relatively high (> 300 µm) D0 values despite a very stiff subgrade (> 20% CBR). However, only the Bruce Highway (10N) section between chainages 19 600 and 19 700 is of concern, as the other sections all have estimates of allowable loading in excess of 1.0×10^7 ESA. The Bruce Highway (10N) section between chainages 123 000 and 124 000, the New England Highway (22B) section between chainages 34 800 and 35 400 and the Mount Lindesay Highway (25B) section between chainages 11 500 and 13 400 displayed relatively high (> 300 µm) D0 values despite a very stiff subgrade (> 15% CBR). However, none of these sections are of concern, as the estimates of allowable loading for all of the sections is in excess of 1.0×10^7 ESA.

8.2 Backcalculated Layer Moduli

In addition to the prediction of remaining life, the FWD deflection measurements obtained as part of the structural assessment of the selected PM-CMB and I-FBS pavement sections were used to backcalculate layer moduli and estimate allowable loading using a linear elastic analysis program. The sets of deflection measurements, or deflection bowls, and the pavement configuration at each testing location were used to estimate the elastic/resilient modulus values for each layer composing the pavement structure. The structural configuration details were obtained from the ARMIS database and are presented in Table 8.3 and Table 8.4 for the selected PM-CMB and I-FBS pavement sections respectively.

Table 8.3: Structural configuration of PM-CMB pavement sections

Road ID	Start chainage (m)	End chainage (m)	Stabilised material type	Surfacing type	Surfacing thickness (mm)	Base thickness (mm)	Subbase thickness (mm)	Subgrade thickness (mm)
10L	36 100	36 600	PM-CMB	Asphalt	60	350	–	∞
10L	36 600	37 700	PM-CMB	Asphalt	60	460	–	∞
10M	118 700	119 000	PM-CMB	Asphalt	95	175	–	∞
10M	119 000	119 800	PM-CMB	Asphalt	80	150	–	∞
10N	19 100	19 600	PM-CMB	Asphalt	125	270	–	∞
10N	19 600	19 700	PM-CMB	Asphalt	65	380	–	∞
10N	19 700	21 400	PM-CMB	Asphalt	60	50	250	∞
33B	69 200	74 000	PM-CMB	Asphalt	60	50	250	∞
530	1 100	1 200	PM-CMB	Asphalt	60	50	250	∞
530	1 200	1 500	PM-CMB	Asphalt	40	200	–	∞
530	1 500	1 700	PM-CMB	Asphalt	75	250	–	∞
530	1 700	1 900	PM-CMB	Asphalt	75	250	–	∞
530	2 400	2 500	PM-CMB	Asphalt	45	200	–	∞
530	2 500	2 600	PM-CMB	Asphalt	80	275	150	∞
530	2 600	2 800	PM-CMB	Asphalt	60	250	–	∞
856	2 600	3 400	PM-CMB	Asphalt	60	50	250	∞
856	4 400	4 500	PM-CMB	Asphalt	60	50	250	∞
856	4 500	6 300	PM-CMB	Asphalt	60	50	250	∞

Table 8.4: Structural configuration of I-FBS pavement sections

Road ID	Start chainage (m)	End chainage (m)	Stabilised material type	Surfacing type	Surfacing thickness (mm)	Base thickness (mm)	Subbase thickness (mm)	Subgrade thickness (mm)
10N	103 400	104 400	I-FBS	Asphalt	100	150	175	∞
10N	110 800	111 200	I-FBS	Spray seal	15	425	–	∞
10N	111 200	111 400	I-FBS	Spray seal	20	250	–	∞
10N	111 400	111 800	I-FBS	Spray seal	55	225	–	∞
10N	123 000	124 000	I-FBS	Spray seal	20	250	300	∞
10P	51 300	52 300	I-FBS	Spray seal	20	250	60	∞
10P	64 100	65 200	I-FBS	Spray seal	15	250	200	∞
22B	34 400	34 600	I-FBS	Spray seal	65	100	350	∞
22B	34 600	34 800	I-FBS	Spray seal	30	200	325	∞
22B	34 800	35 400	I-FBS	Spray seal	30	200	375	∞
22B	54 700	55 600	I-FBS	Spray seal	20	200	250	∞
22B	55 600	55 700	I-FBS	Spray seal	20	200	250	∞
25B	11 500	13 400	I-FBS	Spray seal	15	300	–	∞
25B	13 400	13 500	I-FBS	Spray seal	15	300	–	∞
208	3 000	3 200	I-FBS	Asphalt	65	300	–	∞

Road ID	Start chainage (m)	End chainage (m)	Stabilised material type	Surfacing type	Surfacing thickness (mm)	Base thickness (mm)	Subbase thickness (mm)	Subgrade thickness (mm)
208	3 200	3 700	I-FBS	Asphalt	65	250	–	∞
208	3 700	3 800	I-FBS	Asphalt	55	420	–	∞
212	3 200	4 000	I-FBS	Spray seal	20	300	200	∞
212	4 000	4 100	I-FBS	Spray seal	60	245	200	∞
1003	3 200	3 300	I-FBS	Asphalt	150	300	–	∞
1003	3 300	4 600	I-FBS	Asphalt	150	300	–	∞
1003	4 600	4 900	I-FBS	Asphalt	150	300	–	∞
1003	4 900	5 000	I-FBS	Asphalt	80	150	300	∞

The backcalculation of elastic/resilient layer moduli values was accomplished using the Dynatest ELMOD6 pavement analysis software package. The ELMOD6 program estimates layer moduli by matching the theoretical deflections of an idealised pavement of a given configuration by adjusting the layer modulus values in an iterative fashion until the 'best fit' is achieved. To maximise the reliability of the analysis, the potential modulus values of layers that were not of principal interest, that is, not PM-CMB or I-FBS, were bounded. The limiting values included 1000–10 000 MPa for asphalt layers, 500–2000 MPa for sprayed bituminous seals, 1000–20 000 MPa for cement-bound layers, 100–1000 MPa for unbound granular layers and 10–250 MPa for subgrade materials. The modulus values for the PM-CMB and I-FBS layers were not bound for the analysis.

Layer moduli were estimated for each set of deflection data obtained using the FWD. Representative values for each 100 m section were calculated by averaging the estimated modulus values and a characteristic value for each road section of interest was obtained by averaging the 100 m representative values. The backcalculated layer moduli obtained using the approach outlined above are presented in Table 8.5 and Table 8.6 for the PM-CMB and I-FBS representative pavement sections respectively.

Table 8.5: Summary of PM-CMB average backcalculated layer moduli and allowable loading estimates

Road ID	Start chainage (m)	End chainage (m)	Average backcalculated modulus (MPa)				Estimated allowable loading (ESA)
			Surfacing	Base	Subbase	Subgrade	
10L	36 100	36 600	2 000	1 647	2 160	120	1.03E+08
10L	36 600	37 700	6 672	1 331	-	78	1.16E+08
10M	118 700	119 000	8 835	11 580	635	164	7.22E+06
10M	119 000	119 800	3 540	1 405	182	65	3.07E+05
10N	19 100	19 600	9 776	408	2 712	233	6.06E+09
10N	19 600	19 700	4 457	190	-	30	9.55E+04
10N	19 700	21 400	6 311	172	-	32	1.16E+05
33B	69 200	74 000	10 000	517	-	34	1.25E+04
530	1 100	1 200	10 000	865	5 881	166	2.21E+08
530	1 200	1 500	10 000	13 486	1 933	143	2.00E+08
530	1 500	1 700	10 000	8 584	-	164	4.68E+08
530	1 700	1 900	10 000	15 725	-	182	1.71E+10
530	2 400	2 500	10 000	9 255	-	130	1.37E+07
530	2 500	2 600	10 000	6 096	5 326	163	5.56E+07

Road ID	Start chainage (m)	End chainage (m)	Average backcalculated modulus (MPa)				Estimated allowable loading (ESA)
			Surfacing	Base	Subbase	Subgrade	
530	2 600	2 800	10 000	6 174	-	176	3.05E+07
856	2 600	3 400	10 000	116	378	30	2.30E+04
856	4 400	4 500	10 000	251	601	42	6.35E+04
856	4 500	6 300	10 000	832	903	156	1.23E+08

Table 8.6: Summary of I-FBS average backcalculated layer moduli and allowable loading estimates

Road ID	Start chainage (m)	End chainage (m)	Average backcalculated modulus (MPa)				Estimated allowable loading (ESA)
			Surfacing	Base	Subbase	Subgrade	
10N	103 400	104 400	6 541	1 037	-	56	1.93E+07
10N	110 800	111 200	2 000	5 214	4 404	202	5.32E+07
10N	111 200	111 400	2 000	3 556	-	168	8.29E+05
10N	111 400	111 800	2 000	4 619	5 159	182	1.39E+06
10N	123 000	124 000	1 813	3 504	351	90	4.10E+05
10P	51 300	52 300	1 962	3 271	712	252	7.42E+05
10P	64 100	65 100	1 821	4 273	300	71	1.22E+05
22B	34 400	34 600	5 316	5 701	678	123	2.38E+05
22B	34 600	34 800	1 529	6 300	282	102	2.00E+05
22B	34 800	35 400	1 774	6 889	483	124	7.53E+05
22B	54 700	55 600	1 444	2 471	545	76	1.86E+05
22B	55 600	55 700	1 618	3 040	483	79	1.01E+05
25B	11 500	13 400	2 000	4 044	-	79	2.82E+05
25B	13 400	13 500	2 000	7 156	-	112	1.17E+06
208	3 000	3 200	7 339	1 737	621	144	1.85E+08
208	3 200	3 700	7 549	6 955	-	155	1.88E+06
208	3 700	3 800	7 637	618	367	88	1.00E+06
212	3 200	4 000	1 889	2 960	7 131	125	4.93E+09
212	4 000	4 100	7 081	1 482	11 535	119	3.16E+11
1003	3 200	3 300	1 805	685	-	59	1.19E+06
1003	3 300	4 600	2 411	903	-	67	1.16E+06
1003	4 600	4 900	4 087	1 761	-	108	4.34E+06
1003	4 900	5 000	3 875	635	3 364	77	3.25E+08

The 100 m representative backcalculated values for the PM-CMB layers ranged from 110 MPa to 11 000 MPa with a standard deviation of 2330 MPa. The mean value for the PM-CMB layers was 2640 MPa, which is significantly larger than the maximum design value of 500 MPa proposed in TMR MRTS ET05C (TMR 2012b). The 100 m representative backcalculated values for the I-FBS base layers ranged from 690 MPa to 9010 MPa with a standard deviation of 1870 MPa. The mean value for the I-FBS base layers was 3440 MPa, which is in good agreement with maximum design value of 2500 MPa recommended by Jones and Ramanujam (2004).

In addition to indicating the relative stiffness of the PM-CMB and I-FBS structural layers, the backcalculated modulus values were also used to estimate the remaining bearing capacity of the representative pavement sections. The pavement configurations and backcalculated modulus values were utilised with CIRCLY 5.0 pavement design software package to predict the allowable loading of the selected pavement sections. The pavement sections were modelled in the CIRCLY 5.0 program in accordance with provisions of the Austroads *Guide to Pavement Technology Part 2: Pavement Structural Design* (Austroads 2012). The PM-CMB layers were modelled as unbound granular to capture the dominant failure mode of permanent surface deformation, and the I-FBS layers were modelled as asphalt layers to capture both permanent deformation and fatigue response. The estimated allowable loading for the PM-CMB and I-FBS representative pavement sections are presented in Table 8.5 and Table 8.6 respectively.

The estimates of allowable loading for the selected PM-CMB pavement sections range from 12 500 ESA to 4.21×10^{13} ESA. The majority of the pavement sections are anticipated to provide greater than 1.0×10^6 ESA without significantly deteriorating serviceability. However, sections of the Bruce Highway (10N) between chainages 19 600 and 19 700, the Peak Downs Highway (33B) between chainages 69 200 and 74 000 and Mackay-Bucasia Road between chainages 2600 and 3400 and 4400 and 4500 are estimated to have less than 100 000 ESA of structural capacity remaining and should be the focus of further investigations. It is worth noting that the Bruce Highway (10N), Peak Downs Highway (33B) and Mackay-Bucasia Road sections were identified as containing potentially weak pavements in both the approaches based on the deflection (Section 8.1) and backcalculated modulus (Section 8.2).

The estimates of allowable loading for the selected I-FBS pavement sections range from 37 300 ESA to 3.16×10^9 ESA. Approximately 40% of the representative pavement sections are estimated to have less than 1.0×10^6 ESA of structural capacity remaining. However, the controlling distress mode for all of the pavement sections, with the exception of Beaudesert-Boonah Road (212), was fatigue cracking. Additional structural capacity may be available, as a conservative estimate of fatigue life is currently used, as outlined in Section 4.5.1. The controlling distress mode for Beaudesert-Boonah Road is permanent deformation, probably resulting from the very stiff cement-bound subbase layer underlying the I-FBS base. No sections were identified as being subject to imminent (< 100 000 ESA) structural failure.

8.3 Allowable Loading Comparison

For a number of the selected pavement sections, significantly different estimates of allowable loading were obtained when using the approaches based on either the deflection (Section 8.1) or backcalculated modulus (Section 8.2). A comparison of the estimates of allowable loading using both methods in addition to the design traffic loading (where available) and the cumulative traffic through January 2013 for the selected PM-CMB and I-FBS pavement sections is presented in Figure 8.1 and Figure 8.2 respectively.

Figure 8.1: Comparison of design, cumulative and estimated allowable loading for PM-CMB pavement sections

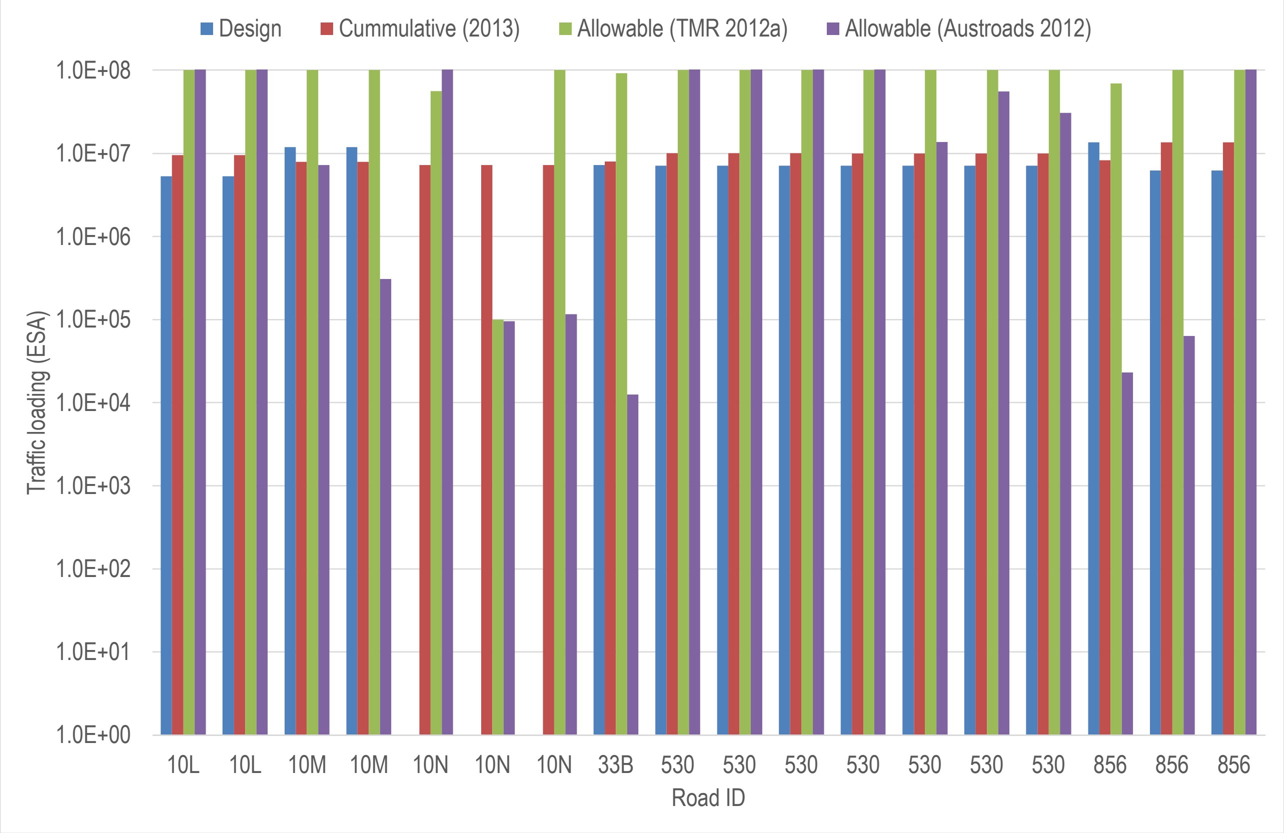
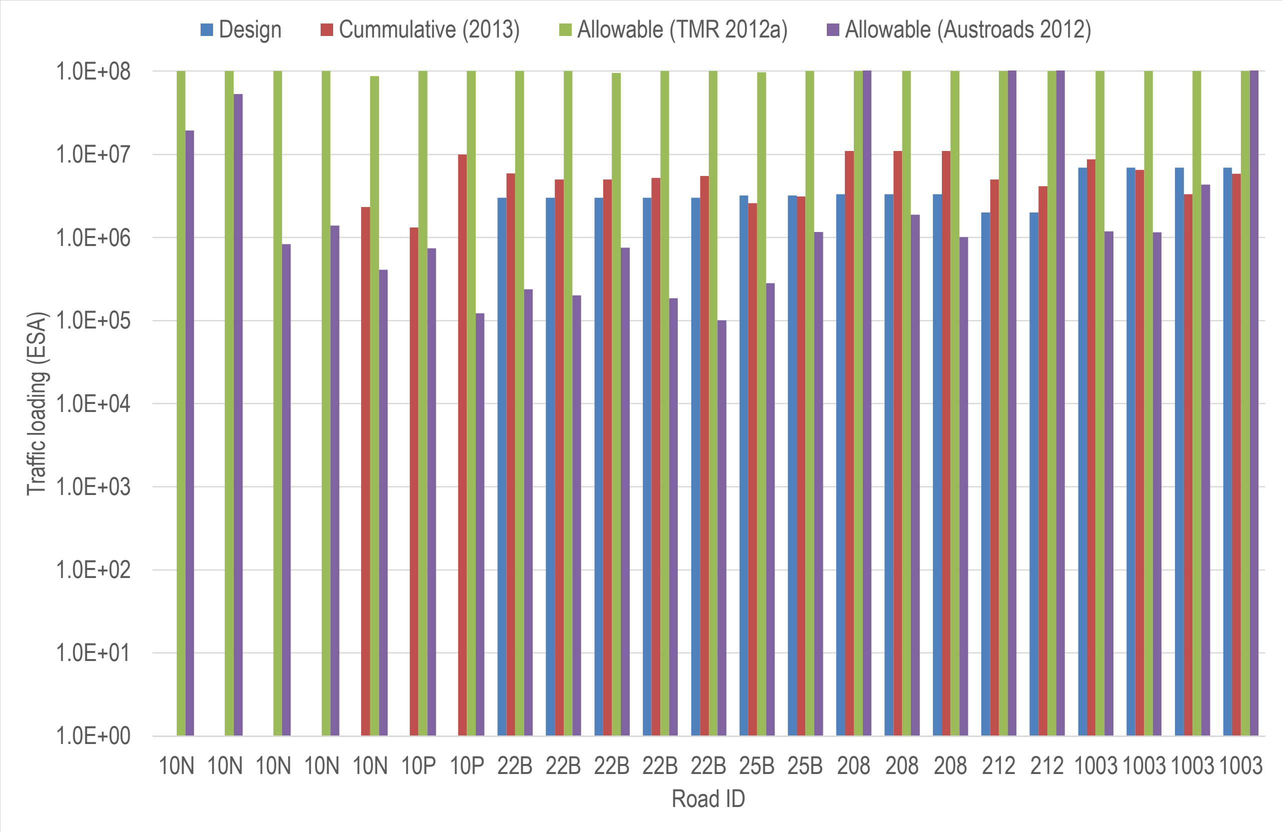


Figure 8.2: Comparison of design, cumulative and estimated allowable loading for I-FBS pavement sections



Reviewing the estimates of allowable loading based on the approach outlined in the TMR *Pavement Rehabilitation Manual* (2012a) and utilising FWD measurements of D0 and D900 under a normalised 40 kN impulse loading, all of the selected road sections, with the exception of the Bruce Highway (10N) between chainages 19 600 and 19 700, should accommodate significant (> 5.5 x 10^7 ESA) traffic loading before a terminal serviceability condition based on structural distress is reached. However, review of the estimates based on backcalculated layer moduli and utilising the entire deflection bowl and the pavement configuration, indicates that a number of the selected pavement sections are nearing a terminal serviceability condition based on structural distress (< 1.0 x 10^6 ESA). The estimated design life for PM-CMB pavements ranges from 5.0 x 10^6 ESA to 1.0 x 10^7 ESA and for I-FBS pavements ranges from 3.0 x 10^6 ESA to 6.0 x 10^6 ESA. It is also worthwhile to note that a number of the representative pavement sections have already significantly exceeded the original design life.

9 LABORATORY CHARACTERISATION

Laboratory testing of field-recovered specimens was undertaken to characterise the engineering properties of selected PM-CMB and I-FBS base materials, validate the design mixture, examine material variability across projects and investigate relationships between material properties and in situ performance. Due to the significant cost and duration of standardised laboratory testing, only a limited number of the selected pavement sections subjected to structural assessment were also subjected to laboratory characterisation. For the sections sampled, material characteristics determined through laboratory testing included strength, stiffness, durability and volumetric properties. The laboratory characterisation testing was conducted during the period September 2014 to June 2015 at the TMR Materials Laboratory in Herston.

Six core specimens of 100 mm or 150 mm diameter were recovered from two locations along each of the selected PM-CMB or I-FBS pavement sections. The extraction of 100 mm cores was attempted for the PM-CMB and 150 mm for the I-FBS pavements to facilitate standard UCS and M_r testing requirements respectively. In addition to the recovery of core specimen, dynamic cone penetrometer (DCP) testing and sampling was undertaken on the subgrade material to characterise the underlying support conditions. The recovered PM-CMB and I-FBS base core specimens were subjected to the laboratory testing protocols presented in Table 9.1 and Table 9.2 respectively.

Table 9.1: Preliminary laboratory testing protocol for PM-CMB specimens

Laboratory characterisation	Testing standard	Core specimens required
Apparent particle density	TMR Q109	2(1)
Unconfined compressive strength	TMR Q115	2
Particle density and water absorption	TMR Q214B	2(1)
Maximum density	TMR Q307A	2(1)
Volume of voids	TMR Q311	2(1)
Cement content	TMR Q116A	2(1)
Capillary rise	TMR Q125D	2
Permeability	TMR Q304A	2

1 Specimens can be composed of recycled material discarded from alternative testing.

Table 9.2: Preliminary laboratory testing protocol for I-FBS specimens

Laboratory characterisation	Testing standard	Core specimens required
Apparent particle density	TMR Q109	2(1)
Resilient modulus	TMR Q139	2
Particle density and water absorption	TMR Q214B	2(1)
Maximum density	TMR Q307A	2(1)

Laboratory characterisation	Testing standard	Core specimens required
Volume of voids	TMR Q311	2(1)
Lime content	TMR Q117A	2(1)
Binder content and gradation	TMR Q308A	2(1)
Capillary rise	TMR Q125D	2
Permeability	TMR Q304A	2

1 Specimens can be composed of recycled material discarded from alternative testing.

9.1 Coring Site Selection

The objective of the laboratory characterisation testing was to determine the strength, stiffness, durability and volumetric properties of selected PM-CMB and I-FBS base materials. Due to the significant cost and duration of standard laboratory testing, only a limited number of the selected pavement sections presented in Table 7.2 could be subjected to the laboratory testing protocols presented in Table 9.1 and Table 9.2. The sampling site selection focused on identifying locations likely to yield the greatest learnings. The guiding methodology included identifying locations where differences in observed pavement performance, as indicated by measurements of roughness and rutting, were probably due to differences in material composition (parent material, stabilising agent content, moisture and air void distribution) and not differences in structural properties such as pavement configuration (layer thickness) and/or underlying support conditions. Additionally, sites were selected where the probability of successfully recovering either 100 mm or 150 mm cores specimen was high. Coring sites were selected where:

- consistent structural configuration exists (based on ARMIS data)
- variability in roughness **does not** mirror variability in D0 or D900 values
- variability in rutting **does not** mirror variability in D0 or D900 values
- deflection curvature function < 0.20 mm
- deflection ratio (DR) > 0.80.

The 2013 measurements of rutting and roughness (Section 5.2) and the FWD deflection measurements (D0 and D900) obtained as part of this study (Section 8.1) were used to construct combined structural and surface condition plots as demonstrated in Figure 9.1 and Figure 9.2 for the New England Highway (22B) and Beaudesert-Boonah Road (212) respectively. The New England Highway (22B) section between chainage 34 400 and 35 400 (Figure 9.1) was not selected for further investigation, as variation in observed performance clearly resembles variation in structural capacity as indicated by increased rutting measurements corresponding with increased D0 and D900 values. However, Beaudesert-Boonah Road (212) between chainage 3200 and 4100 (Figure 9.2) was selected for further investigation as a two-fold increase in rutting measurements can be observed where consistent D0 and D900 were also measured. The evaluation process outlined above was replicated for each of the selected road sections subjected to structural capacity assessment (Table 7.2). In the case of Beaudesert-Boonah Road (212), after reviewing the deflection curvature and DR values, coring locations were established in both the eastbound and westbound lanes at chainages 3450 and 3750.

Figure 9.1: Combined structural and condition data for New England Highway (22B)

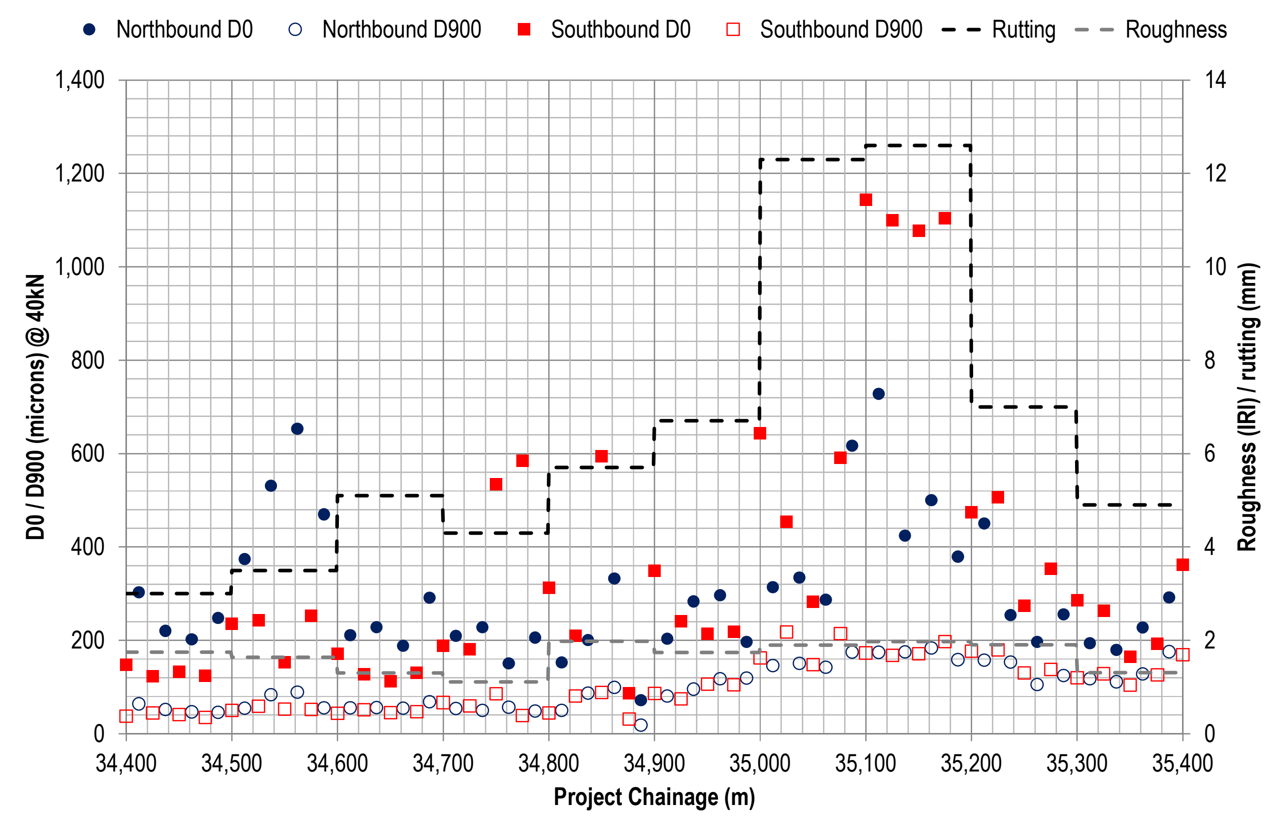
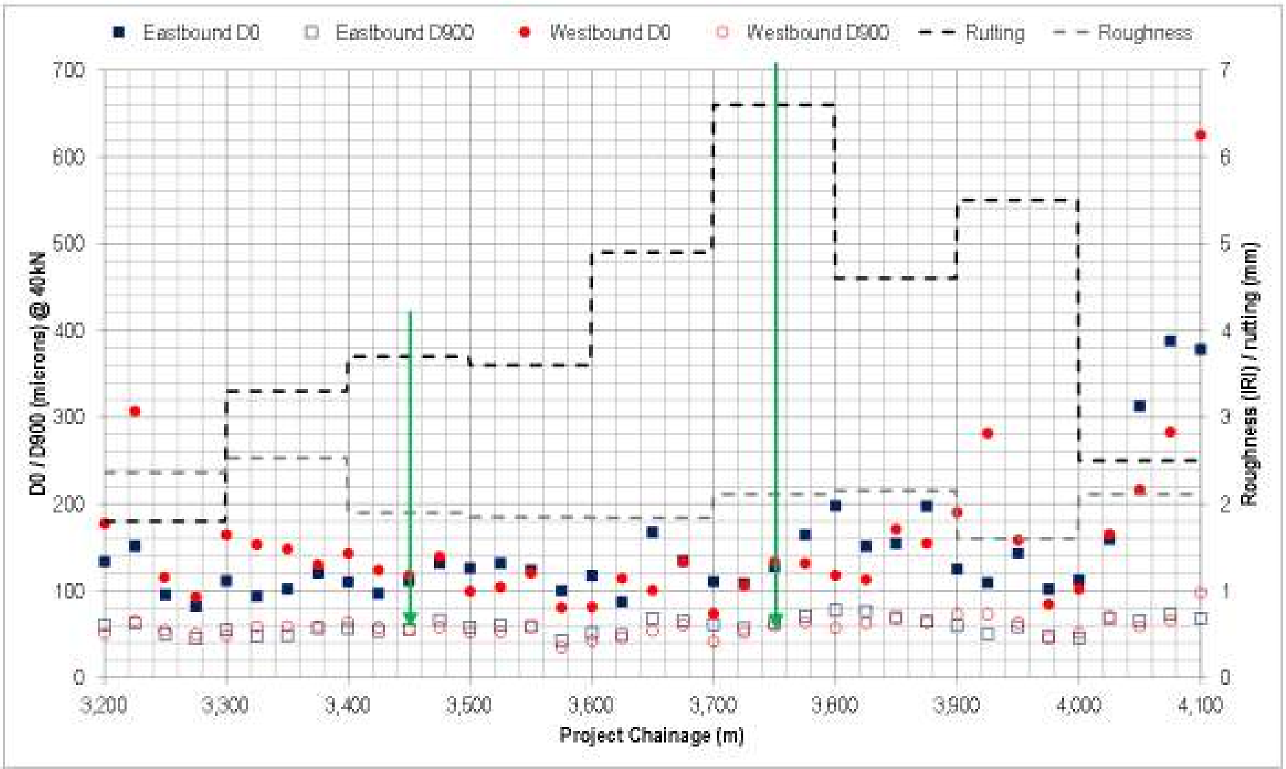


Figure 9.2: Combined structural and condition data for Beaudesert-Boonah Road (212)



Coring locations were identified along the Bruce Highway (10M), Peak Downs Highway (33B), Mackay-Bucasia Road (856), the Bruce Highway (10N), Beaudesert-Boonah Road (212) and the Mount Lindesay Highway (25B) where consistent pavement configuration and structural capacity existed, but where significant differences in pavement performance could be observed. The coring locations selected for the laboratory investigation are presented in Table 9.3. In addition to the locations selected through referencing both the structural capacity and surface condition data, a number of I-FBS sections along the Bruce Highway (10N) of interest to the TMR Pavement Rehabilitation Team including chainages 8140, 36 000, 100 760, 103 790 and 111 360 were also sampled and subjected to the testing protocol outlined in Table 9.2.

Table 9.3: Summary of coring locations for laboratory test samples

Road name	Road ID	Stabilised material	Chainage	Latitude	Longitude	Age (years)	Cumulative traffic (ESA)	Subgrade material
Bruce Hwy	10M	PM-CMB	118 850	-18.6762	146.1521	7	2 943 360	SC-SM
Bruce Hwy	10M	PM-CMB	119 500	-18.6703	146.1526	7	2 943 360	SC-SM
Peak Downs Hwy	33B	PM-CMB	68 500	-21.2256	149.0069	12	5 213 222	SC
Peak Downs Hwy	33B	PM-CMB	72 000	-21.1964	149.0197	12	7 000 700	SC
Peak Downs Hwy	33B	PM-CMB	72 900	-21.1909	149.0260	12	7 000 700	SC
Mackay-Bucasia Rd	856	PM-CMB	3 125	-21.0946	149.1610	10	9 425 906	GC
Mackay-Bucasia Rd	856	PM-CMB	5 100	-21.0782	149.1572	10	15 670 326	SC
Mackay-Bucasia Rd	856	PM-CMB	5 925	-21.0708	149.1565	10	10 807 650	SC
Bruce Hwy	10N	I-FBS	8 140	–	–	–	–	–
Bruce Hwy	10N	I-FBS	36 000	–	–	–	–	–
Bruce Hwy	10N	I-FBS	78 470	-18.0846	145.9073	5	1 323 035	–
Bruce Hwy	10N	I-FBS	78 750	-18.0818	145.9073	5	1 323 035	–
Bruce Hwy	10N	I-FBS	97 110	–	–	4	1 961 520	–
Bruce Hwy	10N	I-FBS	97 120	–	–	4	1 961 520	–
Bruce Hwy	10N	I-FBS	100 760	–	–	5	2 031 208	–
Bruce Hwy	10N	I-FBS	103 790	-17.8750	145.9752	5	2 031 208	–
Bruce Hwy	10N	I-FBS	111 360	-17.8164	146.0042	15	6 150 362	–
Mt Lindesay Hwy	25B	I-FBS	11 800	-28.0655	152.9365	11	2 586 974	SC
Mt Lindesay Hwy	25B	I-FBS	12 800	-28.0743	152.9354	11	2 586 974	SC
Beaudesert-Boonah Rd	212	I-FBS	3 450	-27.9810	152.9650	14	4 990 390	CL
Beaudesert-Boonah Rd	212	I-FBS	3 750	-27.9806	152.9620	14	4 990 390	CH

9.2 Sample Recovery Procedure

The recovered samples included 100 mm core specimens for the PM-CMB and 150 mm core specimens for the I-FBS, taken through the depth of the structural pavement using an electric coring drill. Core specimens were recovered between the wheelpaths (BWP) within the trafficked lanes, three in either direction, spaced one metre apart (longitudinally) and centred at the nominated location. In addition to the core specimen recovery, visual assessment of layer material type and thickness, DCP testing of the subgrade, in addition to collection of approximately 20 kg of subgrade material were undertaken. Soil sampling was conducted in accordance with TMR standard method Q061 (TMR 2014b) and DCP testing in accordance with Q114B (TMR 2014c). Following specimen recovery and sampling, the pavement structure was reconstituted to an equivalent pretest condition.

9.3 PM-CMB Laboratory Testing Results

The recovery of PM-CMB core specimens was attempted along the Bruce Highway (10M), Peak Downs Highway (33B) and Mackay-Bucasia Road (856). As outlined in Section 9.2, the specimen recovery included collection of three BWP cores at the nominated location, documentation of the pavement configuration with depth, determination of the subgrade bearing capacity via DCP and collection of a subgrade material sample. This process was repeated in either both lanes of travel or, in the case Mackay-Bucasia Road, both the slow and fast southbound lanes. In addition to the collection of core specimens for the laboratory material characterisation, the field assessment allowed for verification of the pavement configuration and the subgrade bearing capacity as shown in Table 9.4. Significant variations in the design layer thickness, as presented in the ARMIS database, and the in situ pavement configuration can be observed.

Table 9.4: Verification of design parameters for PM-CMB materials

Road name	Road ID	Chainage	% Design life	Design stabiliser content (%)	Measured stabiliser content (%)	Design base thickness (mm)	Measured base thickness (mm)	Measured subbase thickness (mm)	In situ subgrade CBR (%)
Bruce Hwy	10M	118 850	25	2.0	7.0	200	300	400	–
Bruce Hwy	10M	119 500	25	2.0	5.2	300	220	210	10
Peak Downs Hwy	33B	68 500	72	1.0	–	150	170	220	18
Peak Downs Hwy	33B	72 000	97	2.0	–	150	90	250	12
Peak Downs Hwy	33B	72 900	97	2.0	–	150	170	190	10
Mackay-Bucasia Rd	856	3 125 (SL)	70	2.0	–	350	150	150	30
Mackay-Bucasia Rd	856	3 125 (FL)	70	2.0	–	350	–	–	–
Mackay-Bucasia Rd	856	5 100 (SL)	253	1.5	11.4	330	100	240	22
Mackay-Bucasia Rd	856	5 100 (FL)	253	1.5	11.0	330	–	–	–
Mackay-Bucasia Rd	856	5 925 (SL)	174	1.5	11.5	330	100	200	16
Mackay-Bucasia Rd	856	5 925 (FL)	174	1.5	11.5	330	–	–	–

The sampled PM-CMB pavement sections varied in both age and consumption of design life with the Bruce Highway early in the service life, the Peak Downs Highway nearing the terminal state and sections of Mackay-Bucasia Road having significantly (174% to 253%) exceeded the original design life. All of the sampled sections are constructed in flood-prone areas, in WNR environments, with clayey sand or silty-clayey sand subgrades. The in situ bearing capacity ranged from 10% to 30% CBR with the thickness of the overlying stabilised layers ranging from 300 mm to 700 mm. Comparison of the design and in situ stabilising agent content was attempted. The use of cementitious slurry interlayers to increase the bonding of subsequent lifts of PM-CMB material resulted in inflated estimates of stabilising agent content, as the CMB structural layer and interlayer materials were combined during sampling and the high cementitious materials content of the interlayer significantly influenced the results. However, examination of the relative values indicates that poor and excellent stabilising agent distribution control was achieved during the construction of the Bruce Highway and Mackay-Bucasia Road sections respectively.

Due to the unbound nature of PM-CMB, great difficulty was encountered in the recovery of intact core specimens. In all locations, it was not possible to obtain consistent cores through the depth of the PM-CMB stabilised pavement layer. For the Bruce Highway and Peak Downs Highway sites, the PM-CMB material 'crumbled' during coring and large fragmented chunks were recovered, as shown in Figure 9.3. The PM-CMB layers for Mackay-Bucasia Road were constructed using multiple lifts of 100 mm to 150 mm thickness bonded with a cementitious slurry interlayer. During coring the specimens fractured at the bonded interlayer. As a result, the cores recovered from Mackay-Bucasia Road were typically 100 mm diameter by approximately 100 mm height, as shown in Figure 9.4.

Figure 9.3: PM-CMB core specimen from Bruce Highway (10M) chainage 118 850

Figure 9.4: PM-CMB core specimens from Mackay-Bucasia Road (856)

Due to the difficulties encountered in specimen recovery, the testing protocol for the PM-CMB materials was significantly impacted. UCS and permeability testing could not be carried out on any of the specimens, as the minimum 1.1:1.0 height to diameter ratio was not achieved for any of the cores recovered after trimming. Capillary rise, water absorption and density testing could only be carried out on cores from Mackay-Bucasia Road. Cement content testing was carried out on both the Bruce Highway and Mackay-Bucasia Road materials and is presented in Table 9.4. The intact cores from Mackay-Bucasia Road were also subjected to the ITM_r testing in accordance with TMR test method Q139, which is typically pursued for bitumen-bound materials to get an indication of strength despite the core height limitations. The results of the capillary rise, absorption, M_r and density testing are presented in Table 9.5.

Table 9.5: Determination of material properties for PM-CMB materials

Road name	Road ID	Chainage	Permeability (µm/s)	Capillary rise (%)	Water absorption (g)	As-received modulus (MPa)	Soaked modulus (MPa)	Retained modulus (%)	Bulk density (t/m³)
Bruce Hwy	10M	118 850	–	–	–	–	–	–	–
Bruce Hwy	10M	119 500	–	–	–	–	–	–	–
Peak Downs Hwy	33B	68 500	–	–	–	–	–	–	–
Peak Downs Hwy	33B	72 000	–	–	–	–	–	–	–
Peak Downs Hwy	33B	72 900	–	–	–	–	–	–	–
Mackay-Bucasia Rd	856	3 125 (SL)	–	–	–	–	–	–	–
Mackay-Bucasia Rd	856	3 125 (FL)	–	–	–	–	–	–	–
Mackay-Bucasia Rd	856	5 100 (SL)	–	100	49.1	–	–	–	–
Mackay-Bucasia Rd	856	5 100 (FL)	–	–	–	–	–	–	–
Mackay-Bucasia Rd	856	5 925 (SL)	–	100	49.3	9 374	5 329	58	2.300

Road name	Road ID	Chainage	Permeability (µm/s)	Capillary rise (%)	Water absorption (g)	As-received modulus (MPa)	Soaked modulus (MPa)	Retained modulus (%)	Bulk density (t/m³)
Mackay-Bucasia Rd	856	5 925 (FL)	–	100	51.4	8 274	5 201	64	2.298

Referencing the project documentation for the Mackay-Bucasia Road duplication (120/856/13), the stabilising agent content measured during construction in the lower and upper PM-CMB layers between chainages 5920 and 5937 ranged from 1.5% to 1.8% and the UCS measured in accordance with TMR testing method Q115 at 28 days ranged from 2.6 MPa to 2.8 MPa. Applying the average of the as-received resilient modulus tests, the ratio of 28-day UCS to ITM_r is approximately 3.0×10^{-4}.

9.4 I-FBS Laboratory Testing Results

Due to the bound nature of FBS, compared to CMB, the recovery of core specimens was much more successful. The recovery of I-FBS core specimens was attempted at numerous locations along the Bruce Highway (10N), Mount Lindesay Highway (25B) and Beaudesert-Boonah Road (212). The recovery of core samples from the Bruce Highway sections was limited to extraction of the cores and was undertaken at isolated locations. The specimen recovery for the Mount Lindesay Highway and Beaudesert-Boonah Road was as outlined in Section 9.2 and summarised in Section 9.3. These samples were also recovered from both lanes of travel. Due to the differences in sampling, verification of the pavement configuration and the subgrade bearing capacity was only conducted for the Mount Lindesay Highway and Beaudesert-Boonah Road, as shown in Table 9.6. Unlike the PM-CMB pavement sections, good agreement can be observed between the design layer thickness, as presented in the ARMIS database, and the in situ pavement configuration.

Table 9.6: Verification of design parameters for I-FBS materials

Road name	Road ID	Chainage	% Design life	Design stabiliser content (%)	Measured stabiliser content (%)	Design layer thickness (mm)	Measured base thickness (mm)	In situ subgrade CBR (%)
Bruce Hwy	10N	8140	–	–	3.0 + 4.0	–	–	–
Bruce Hwy	10N	36 000	–	–	2.9	–	–	–
Bruce Hwy	10N	78 750	–	3.5 + 2.0	6.5	250	–	–
Bruce Hwy	10N	78 470	–	3.5 + 2.0	4.0 + 2.3	250	–	–
Bruce Hwy	10N	97 110	–	–	6.9	250	–	–
Bruce Hwy	10N	97 120	–	–	6.2	250	–	–
Bruce Hwy	10N	100 760	–	–	4.3	–	–	–
Bruce Hwy	10N	103 790	–	–	7.6	–	–	–
Bruce Hwy	10N	111 360	–	–	5.1	250	–	–
Mt Lindesay Hwy	25B	11 800 (NB)	81%	3.5 + 2.0	3.8 + 4.4	300	280	30
Mt Lindesay Hwy	25B	11 800 (SB)	81%	3.5 + 2.0	3.6 + 2.8	300	290	24
Mt Lindesay Hwy	25B	12 800 (NB)	81%	3.5 + 2.0	4.1 + 3.2	300	280	17
Mt Lindesay Hwy	25B	12 800 (SB)	81%	3.5 + 2.0	3.8 + 3.5	300	250	22
Beaudesert-Boonah Rd	212	3 450 (EB)	250%	3.5 + 2.0	1.9 + 2.4	300	250	6
Beaudesert-Boonah Rd	212	3 450 (WB)	250%	3.5 + 2.0	3.3 + 2.6	300	250	11
Beaudesert-Boonah Rd	212	3 750 (EB)	250%	3.5 + 2.0	3.8 + 4.1	300	300	11

Road name	Road ID	Chainage	% Design life	Design stabiliser content (%)	Measured stabiliser content (%)	Design layer thickness (mm)	Measured base thickness (mm)	In situ subgrade CBR (%)
Beaudesert-Boonah Rd	212	3 750 (WB)	250%	3.5 + 2.0	3.1 + 4.7	300	300	8

The investigated I-FBS pavement sections varied in both age and consumption of design life with the Bruce Highway sections generally early in the service life, the Mount Lindesay Highway nearing the terminal state and Beaudesert-Boonah Road having significantly (250%) exceeded the original design life. The sections are constructed in a variety of environments including WNR for the Bruce Highway, NDR for the Mount Lindesay Highway and WR for Beaudesert-Boonah Road. The in situ soils for the Mount Lindesay Highway are clayey sands with bearing capacity ranging from 17% to 30% CBR. The in situ soils for Beaudesert-Boonah Road are low to high plasticity clays with bearing capacity at the time of testing ranging from 6% to 11% CBR.

The design thickness of the investigated sections was either 250 mm or 300 mm and good agreement was observed between design and as-built infrastructure. Comparison of the design and in situ stabilising agent content was not nearly as favourable. The bitumen contents of the Mount Lindesay Highway and Beaudesert-Boonah Road sections were in close agreement with the design values. However, significant variations in design and measured stabilising agent content for the Bruce Highway sections were observed. The average bitumen content for the Bruce Highway sections was in excess of 5.0%. The significant difference in bitumen content is clearly observed, as shown in Figure 9.5. In addition to the bitumen content, the secondary stabilising agent (lime) contents also varied greatly between the design and measured values. In some cases, such as Beaudesert-Boonah Road (212), secondary stabilising agent contents twice the design value were measured. However, examination of the relative results with pavement sections indicates good control of secondary stabilising agent distribution for all of the sites investigated.

Figure 9.5: I-FBS core specimens from Mt Lindesay Highway (25B) (left) and Bruce Highway (10N) (right)

The testing protocol for the recovered I-FBS samples included determination of permeability, capillary rise, water absorption, ITM_r and density, in addition to the primary and secondary stabilising agent contents discussed previously. The results of the laboratory testing are presented in Table 9.7.

Table 9.7: Determination of material properties for I-FBS materials

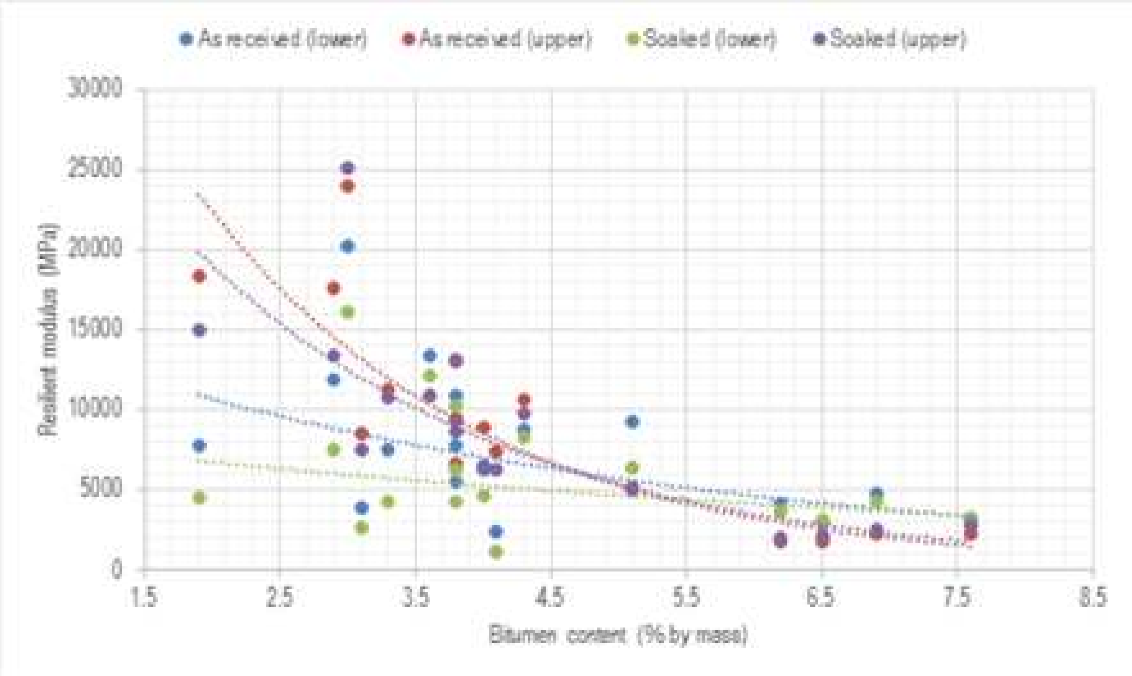
Road name	Road ID	Chainage	Permeability (μm/s)	Capillary rise (%)	Water absorption (g)	As-received modulus (MPa)	Soaked modulus (MPa)	Retained modulus (%)	Bulk density (t/m^3)
Bruce Hwy	10N	8140	0.00	21	29.7	21 408	22 139	103	2.283
Bruce Hwy	10N	36 000	0.00	35	44.0	15 825	9961	63	2.220
Bruce Hwy	10N	78 750	0.00	15	6.6	2972	3103	104	2.163
Bruce Hwy	10N	78 470	0.00	29	22.5	8121	6576	81	2.126
Bruce Hwy	10N	97 110	0.00	28	15.9	4789	4315	90	2.102
Bruce Hwy	10N	97 120	0.00	60	63.1	4175	4056	97	2.094
Bruce Hwy	10N	100 760	0.01	23	19.3	13 077	12 561	96	2.222
Bruce Hwy	10N	103 790	0.00	10	2.9	3120	3314	106	2.290
Bruce Hwy	10N	111 360	0.00	58	81.4	9463	8532	90	2.107
Mt Lindesay Hwy	25B	11 800 (NB)	0.00	66	49.4	10 979	11 473	104	2.248
Mt Lindesay Hwy	25B	11 800 (SB)	0.01	29	20.0	17 227	16 520	96	2.395
Mt Lindesay Hwy	25B	12 800 (NB)	0.02	54	109.6	4959	2569	52	2.285
Mt Lindesay Hwy	25B	12 800 (SB)	0.00	10	9.9	13 249	13 289	100	2.303
Beaudesert-Boonah Rd	212	3 450 (EB)	0.00	53	58.7	11 898	8513	72	2.174
Beaudesert-Boonah Rd	212	3450 (WB)	0.92	28	50.7	7484	4230	57	2.153
Beaudesert-Boonah Rd	212	3750 (EB)	0.00	49	56.9	9260	7369	80	2.134
Beaudesert-Boonah Rd	212	3750 (WB)	0.01	66	75.0	8473	6319	75	2.058

The I-FBS samples were effectively impermeable with permeability measurements ranging from 0.00 μm/s to 0.02 μm/s. A single core obtained from Beaudesert-Boonah Road (212) was observed to have low permeability as indicated by a measurement of 0.92 μm/s. This specimen was also observed to have very high air voids contents of 17.5%, 21.4% and 24.7% for the upper, middle and lower third of the core respectively. The high permeability and voids is probably the result of poor compaction. The capillary rise for the I-FBS specimens ranged from 10% to 66% and indicates that these materials have a low affinity for moisture and are ideally placed in wet environments. The bulk density (upper third) of the I-FBS samples ranged from 2.10 t/m^3 to 2.40 t/m^3 and was relatively consistent when comparing between cores obtained from the same pavement sections.

The M_r of I-FBS cores is a common measure of material strength and resistance to permanent deformation. The M_r of the upper, middle and lower third of each core was determined for untreated as-received and soaked conditions. Soaking consisted of saturation under vacuum for a period of ten minutes. Where the trimmed height of cores was less than 150 mm, only upper and lower sections were tested. In addition to the as-received and soaked M_r, the retained modulus or ratio of soaked to as-received is presented in Table 9.7. The as-received modulus (upper third) of the I-FBS core samples ranged from 3120 MPa to 21 410 MPa and the soaked modulus ranged from 2570 MPa to 22 140 MPa. The impact of soaking, as indicated by the retained modulus, was variable both interproject and intraproject with some materials gaining up to 6.0% and others losing up to 48% of the as-received M_r.

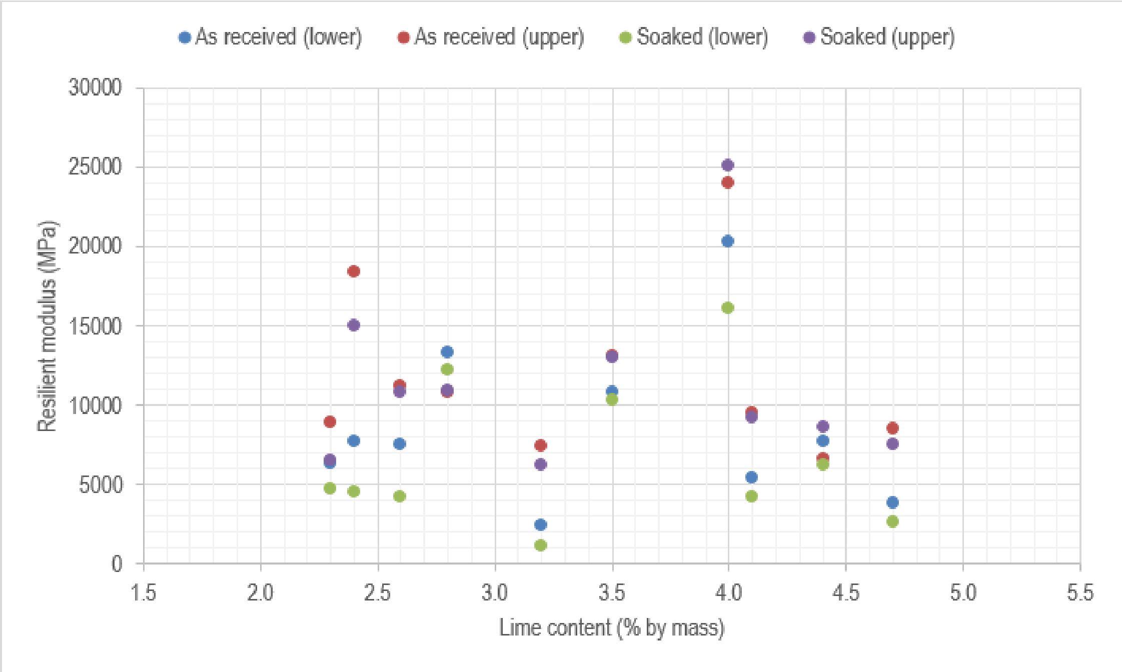
As the strength and stiffness properties of FBS material significantly influences the in-pavement performance, the impact of material properties on measured M_r was also examined. The material properties considered included bitumen content, secondary stabilising agent content and bulk density, which are significantly influenced by both design and construction practices. The relationship between bitumen content and M_r for the materials investigated in this study is presented in Figure 9.6.

Figure 9.6: Influence of bitumen on I-FBS resilient modulus



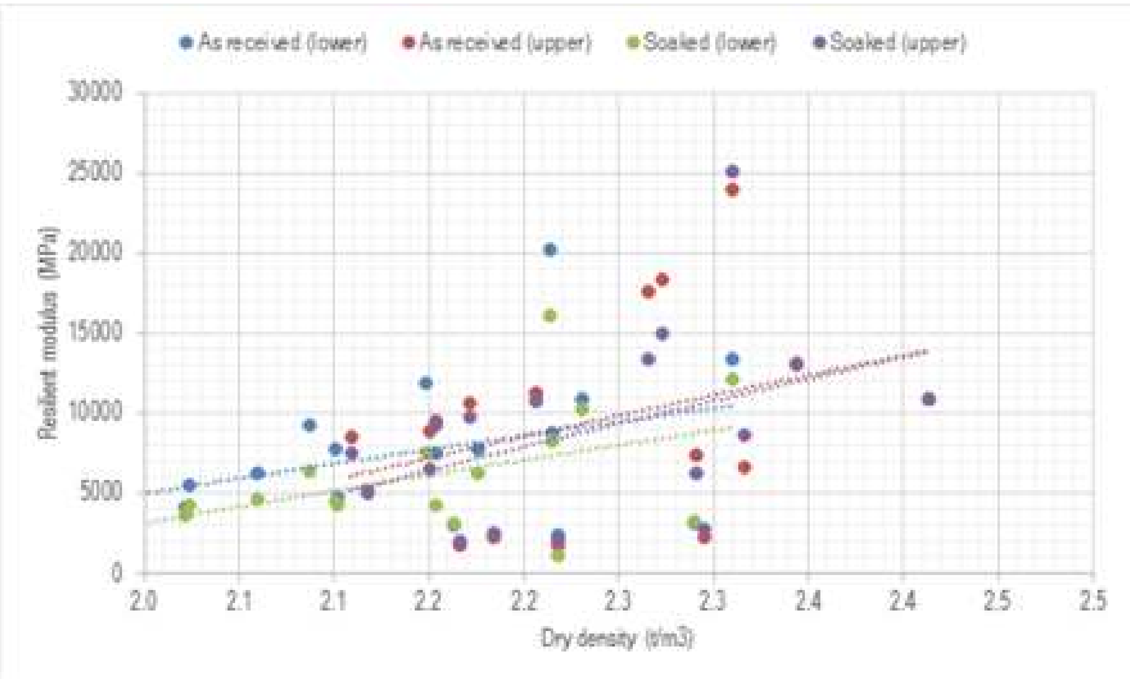
A definitive relationship, that appears to take an exponential form, can be observed between bitumen content and M_r for both the upper and lower third of core specimens, with and without soaking. The influence of the secondary stabilising agent, lime in the case of the materials evaluated in this study, on the as-received and soaked M_r is not as significant as presented in Figure 9.7.

Figure 9.7: Influence of lime content on I-FBS resilient modulus



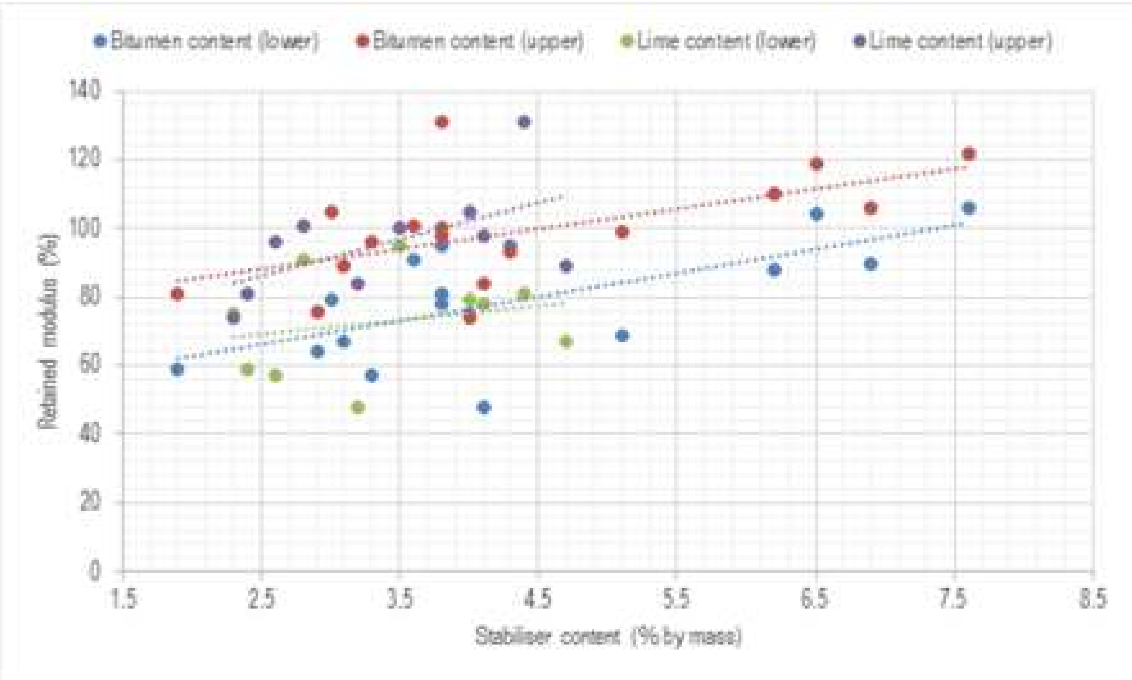
The M_r does not appear to be impacted by variations in lime content for both the upper and lower thirds of cores with and without soaking. However, the bulk dry density does have a significant impact on M_r as shown in Figure 9.8. The relationship between density and M_r appears to take a logarithmic form, with modulus significantly increasing with greater dry density.

Figure 9.8: Influence of density on I-FBS resilient modulus



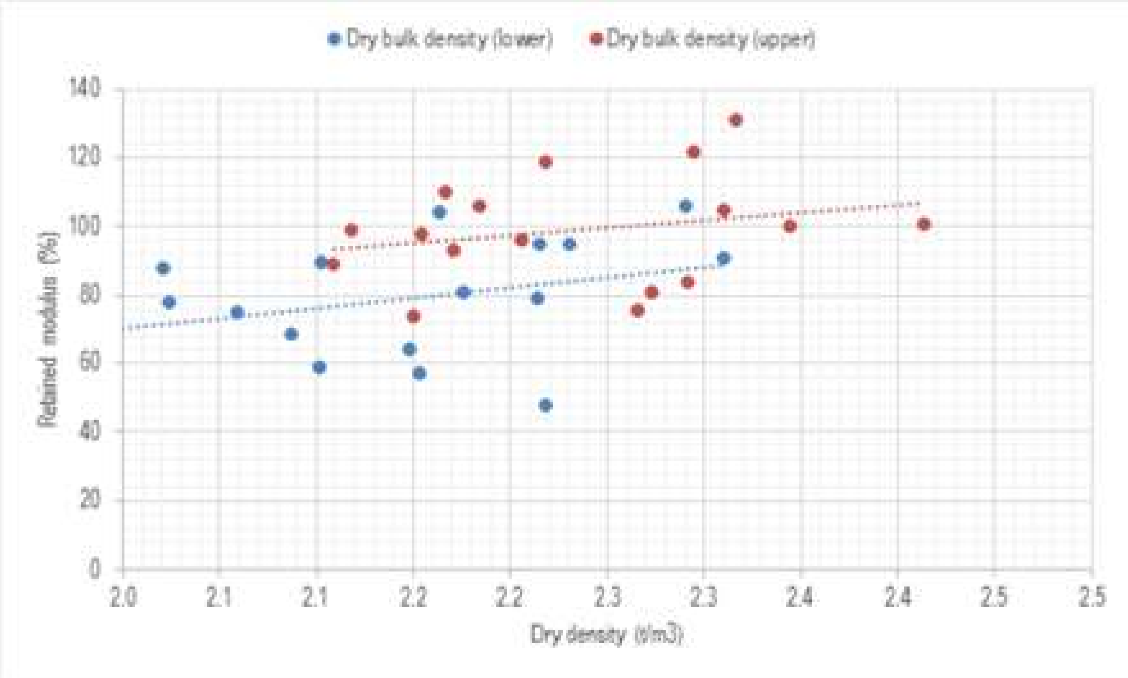
Pavements provisioned within wet environments along the Queensland state-controlled road network must resist moisture-induced reductions in bearing capacity. This is particularly important in the coastal regions of Queensland where both heavy precipitation and high volumes of truck traffic can occur simultaneously. The retained modulus of FBS material is commonly measured as an indication of resistance to moisture-induced strength change. The influence of bitumen content, secondary stabilising agent content and bulk density on retained modulus was investigated to quantify the influence of these design and construction parameters on in-service performance. The relationship between bitumen content and retained modulus, in addition to secondary stabilising agent (lime) content and retained modulus, is presented in Figure 9.9. A significant linear relationship between the bitumen and lime contents and the retained modulus can be readily discerned.

Figure 9.9: Influence of stabiliser content on I-FBS retained modulus



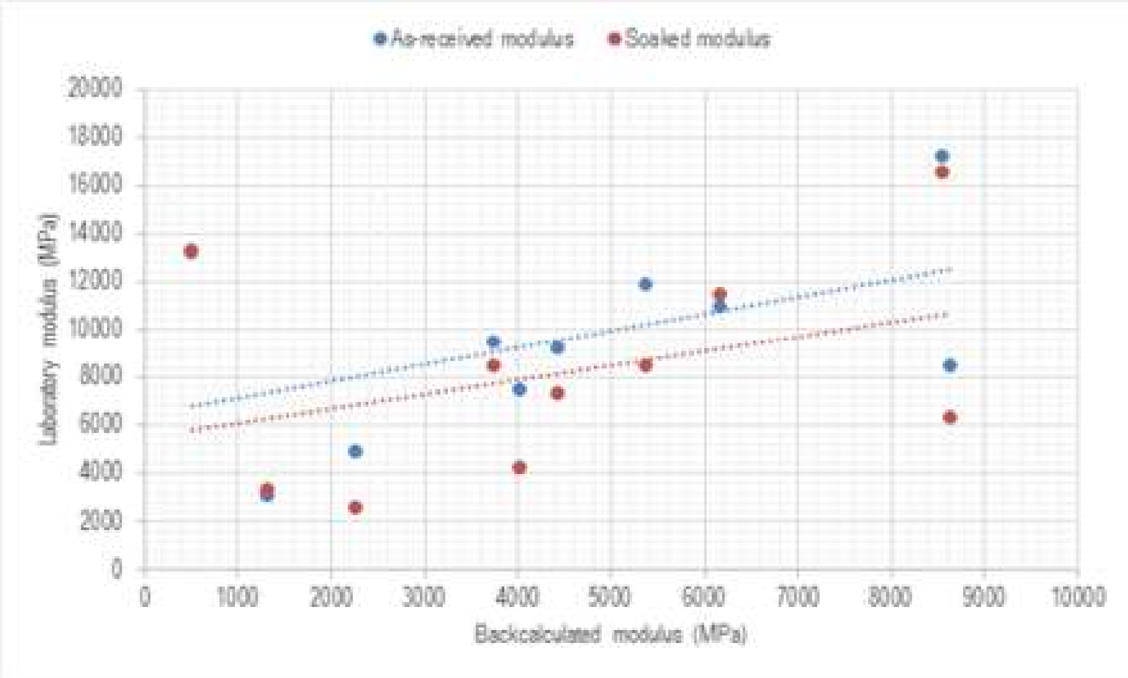
In addition to the influence of stabilising agent content, the impact of I-FBS material dry density on retained modulus was investigated. A significant linear relationship can be observed in Figure 9.10. It is also interesting to note the significant difference in dry density between the upper and lower third of core specimens.

Figure 9.10: Influence of density on I-FBS retained modulus



In addition to relationships between laboratory measurements, the correlation between modulus values backcalculated from in situ FWD measurements and laboratory measurements of modulus determined in accordance with TMR testing method Q139 were examined. The in situ modulus values were determined as outlined in Section 8.2. The ability to relate laboratory and field measurements of material stiffness would be extremely valuable and significantly increase the reliability of mixture proportioning and pavement rehabilitation efforts. The relationships between backcalculated and both as-received and soaked laboratory M_r values are presented in Figure 9.11.

Figure 9.11: Correlation between backcalculated and laboratory modulus vales



A linear trend can be observed between both as-received and soaked modulus and modulus values backcalculated using in situ FWD deflection measurements. A positive trend can be observed where increasing laboratory modulus corresponds with increasing backcalculated modulus. While all the variability in the predicted measurements (laboratory modulus) cannot be explained by the known value (backcalculated modulus), a rough relationship can be discerned, where laboratory modulus is approximately 1.4 to 1.6 times greater than the backcalculated value.

9.5 Laboratory Testing Summary

Laboratory testing was undertaken on core specimens recovered from selected PM-CMB and I-FBS pavements in an attempt to identify material properties significantly influencing in-service performance. Substantial issues were encountered in the recovery of PM-CMB material samples due to the ‘unbound’ nature of the material. While this limitation impacted the overall testing objectives, it was valuable to confirm that PM-CMB pavements ranging in age from seven years to 12 years did not develop “bound” material characteristics. Additionally, it was confirmed that the M_r of PM-CMB materials can be successfully measured using TMR test method Q139, which is specified for the measurement of asphalt and FBS materials. The laboratory testing effort was much more successful for the I-FBS materials, as a number of intact core specimens were recovered. Strong correlations were not discovered. However, general trends between bitumen content, secondary stabilising agent content and density and as-received modulus, soaked modulus and retained modulus were observed. Additionally, a relationship between backcalculated and laboratory-measured modulus was observed, with laboratory-measured values generally 1.4 to 1.6 times greater than the values backcalculated from in situ FWD measurements.

10 PERFORMANCE RELATIONSHIPS

The principal objective of the *Stabilisation Practices in Queensland* project is to develop a systematic approach for the selection of stabilisation technologies, based on project-specific operational conditions such as material availability, climate, environment and traffic. Pursuant to the project objective, development of a standardised selection methodology requires an understanding of the influence of these operational conditions on the performance of stabilised pavements. The preceding review of ARMIS inventory data and subsequent condition categorisation, visual inspection, structural capacity assessment, and laboratory material characterisation for PM-CMB and I-FBS base pavements along the Queensland state-controlled road network were conducted to facilitate an investigation of the relative significance of each of the operational conditions on in-service performance.

The PM-CMB and I-FBS state-controlled road network inventory and condition data, as presented in Section 5, structural capacity assessment, as presented in Section 8, and laboratory characterisation testing, as presented in Section 9, were subjected to a detailed statistical analysis to highlight factors contributing to good or poor performance. A combined analysis of variance (ANOVA) and multivariate linear regression approach was undertaken to explore relationships between the performance indicators of roughness and rutting and independent operational variables. Key variables included environmental zone, surfacing type, stabilising agent type and content, total pavement thickness and traffic volume (AADT). Since the independent variables in the dataset consisted of both categorical and continuous variables, ANOVA was utilised for nominal, ordinal and interval variables, and linear regression was used for continuous and select ordinal variables.

It should be noted that the data used in the statistical analysis was resolved in 100 m increments denoted by chainage. Some information associated with a 100 m road section may not have been normalised across the entire distance nor represent the entire distance, but that a particular characteristic is located within the identified 100 m segment.

10.1 Analysis of Variance

The ANOVA method was chosen to examine relationships between the performance indicators (roughness or rutting) and the nominal, ordinal and interval (categorical) variables presented in Table 10.1 and Table 10.2. These variable were selected due to the well-documented significance of environmental zone, surfacing type, primary and secondary stabilising agent type and content, structural pavement thickness and traffic volume (AADT) on the long-term performance of stabilised pavements. The bins or levels within the categorical variable groups are representative of the range of application of the PM-CMB and I-FBS pavement sections along the Queensland state-controlled network. The sample sizes were unevenly distributed in the majority of levels for each factor, hence a one-way ANOVA method was selected over two-way method with or without replication methods.

Table 10.1: Nominal variables

Factors	Levels
Environmental zones	▪ WNR ▪ WR ▪ DNR ▪ DR
Surfacing type	▪ Asphalt ▪ Spray seal ▪ Concrete

Factors	Levels
Primary stabilising agent content	▪ 1.0% ▪ 1.5% ▪ 2.0% ▪ 2.5% ▪ 3.0% ▪ 3.5% ▪ 4.0%
Secondary stabilising agent type and content	▪ 1.0% lime ▪ 1.5% lime ▪ 2.0% lime ▪ 1.0% cement ▪ 1.5% cement ▪ 2.0% cement

Table 10.2: Ordinal and interval variables

Factors	Levels
Total pavement thickness (mm)	▪ 0–200 ▪ 201–400 ▪ 401–600 ▪ 601–800 ▪ > 800
AADT	▪ 0–1500 ▪ 1501–5000 ▪ 5001–10000 ▪ > 10000

The 2013 measurements of roughness and rutting obtained from the ARMIS database, as dependent variables, represented the overall population for performance indicators. The samples for ANOVA were obtained by grouping the roughness and rutting into the factors and levels given in Table 10.1 and Table 10.2, but using the factors in Table 10.3 as the independent variables, to identify emerging trends. The continuous variables presented in Table 10.3 are factors related to either the composite pavement structure or the stabilised pavement layer and were selected from the available dataset for statistical analysis as they were considered most likely to independently influence the performance indicators of roughness and rutting to a significant degree.

Table 10.3: Continuous variables

Factors
Cumulative traffic (ESA)
Surfacing thickness (mm)
Stabilised layer thickness (mm)
Backcalculated modulus of layers 1 to 5 (MPa)
Estimated subgrade CBR (%)
Surfacing age (years)
Average annual maintenance expenditure ($)
Estimated vertical strain above subgrade layer (με)
Estimated horizontal strain under I-FBS layer (με)

Factors
Average normalised deflection curvature (µm)
Pavement design life (ESA)
Estimated allowable loading (ESA)

ANOVA was used to examine the means and variances of roughness and rutting between the different categorical groups and levels for each of the independent variables. The null hypothesis states that there is no significant difference between the sample populations (i.e. between environmental zones WR, WNR, DR and DNR) in affecting rutting or roughness other than error inherent to the data or pure chance. To reject the null hypothesis and show a statistically significant relationship, a p-value ≤ 0.05 is required.

10.1.1 Assumptions

The dataset is assumed to conform to the following rules for the results of the ANOVA analysis to be meaningful:

1. Error values in a cell should not equal 0 (variance should not equal 0).
2. Cell variances are roughly similar (wide variance is a problem).
3. Measurements must be independent.
4. Distribution of the variables should be roughly normal.

Assumption 1 and Assumption 4 can be controlled as sample sizes of 30 or over tend to fall into a normal distribution. However, for certain instances a violation of Assumption 2 and Assumption 3 may occur. The ANOVA method is largely resilient to assumption violation but the results may indicate interactions with other factors. For example, where Assumption 2 exhibits violation, this may be due to vastly different sample sizes.

The results may also imply other interactions. The averages of the different total pavement thickness groups are distinct. However, the variance of each group was vastly different, which violates Assumption 2. This shows that for a total pavement thickness of over 800 mm, the variance for rutting is very low, compared with the 201–400 mm and other groups. This leads to the question of whether other factors such as binder type and content or AADT contributes to the huge variances observed in rutting.

To explore potential relationships in detail, the other interactions must be considered. In keeping with the above example, statistical significance was observed for the relationship between total pavement thicknesses and estimated vertical strain as well as backcalculated layer moduli in affecting the maximum rutting. As such, these relationships would need to be explored in a more detailed analysis. Since the variables of estimated vertical strain and backcalculated modulus are continuous, a linear regression may be conducted.

10.1.2 PM-CMB

The results of the ANOVA analysis between categorical and continuous variables relative to the rutting performance of PM-CMB pavements are shown in Table 10.4. While the relative influence of each combination of categorical and continuous variables was investigated, only relationships with p-values < 0.05 are shown in the following ANOVA summary tables.

Table 10.4: P-value table for PM-CMB rutting ANOVA analysis

Continuous variables / Nominal variables	Surfacing age	Vertical subgrade strain	FWD deflection curvature	Cumulative traffic	Maintenance expenditure	Surfacing modulus E1	PM-CMB modulus E2	Subgrade modulus E5
Total pavement thickness	2.00E-02	1.50E-06	4.60E-06	6.00E-03	4.00E-02	2.00E-03	1.80E-02	–
Stabiliser content	–	4.30E-07	6.50E-07	–	1.70E-02	–	6.50E-07	–
Surfacing type	–	5.00E-09	3.30E-09	6.00E-05	1.20E-13	–	–	–
AADT	2.00E-02	2.30E-08	5.80E-08	4.00E-02	5.20E-03	–	1.40E-05	9.90E-08

A number of factors can be observed to significantly influence rutting in PM-CMB pavements including the age of the surfacing, estimated vertical compressive strain at subgrade level, curvature function from FWD deflection testing, the cumulative traffic to date (2013), average annual maintenance expenditure, and the modulus values backcalculated from FWD deflection testing for the surfacing, PM-CMB layer and subgrade. The categorical factors of environmental zone and secondary stabilising agent were invalid for the ANOVA analysis, as the majority (> 99%) of identified pavement sections were located in WNR regions and the secondary stabilising agent category only applied for the I-FBS pavements. Of the valid categorical variables, total structural pavement thickness and traffic volume (AADT) appear to be the most influential.

The results of the ANOVA analysis between categorical and continuous variables relative to the roughness performance of PM-CMB pavements are shown in Table 10.5.

Table 10.5: P-value table for PM-CMB roughness ANOVA analysis

Continuous variables / Nominal variables	Total pavement thickness	Surfacing age	Maintenance expenditure	Cumulative traffic	Surfacing modulus E1	Subbase modulus E4
Total pavement thickness	–	–	–	–	4.00E-02	4.70E-02
Stabiliser content	–	–	4.00E-06	–	–	–
Surfacing type	2.60E-02	–	3.00E-06	–	–	3.60E-02
AADT	2.00E-03	3.70E-02	1.40E-04	3.60E-02	–	–

Fewer continuous variables were observed to have an influence on the roughness of PM-CMB pavements compared to rutting performance. However, a number of factors were found to significantly influence roughness including total pavement thickness, the age of the surfacing, average annual maintenance expenditure, the cumulative traffic to date (2013) and the modulus values backcalculated from FWD deflection testing for the surfacing and subbase. The traffic volume presented in AADT appears to be the most influential of the categorical variables.

10.1.3 I-FBS Base

The results of the ANOVA analysis between categorical and continuous variables relative to the rutting performance of I-FBS pavements are shown in Table 10.6. Similarly to the PM-CMB ANOVA analysis, the relative influence of each combination of categorical and continuous variables was investigated. However, only relationships with p-values < 0.05 are shown in the following ANOVA summary tables.

Table 10.6: P-value table for I-FBS base rutting ANOVA analysis

Continuous variables / Nominal variables	Surfacing age	Vertical subgrade strain	FWD deflection curvature	Cumulative traffic	Surfacing modulus E1	I-FBS modulus E2	Subgrade modulus E5
Environmental zone	–	–	2.20E-02	–	–	–	–
Total pavement thickness	–	1.60E-05	6.50E-04	–	3.60E-03	1.00E-03	2.00E-03
Bitumen content	–	–	4.00E-04	–	4.90E-03	3.90E-04	4.00E-04
Surfacing type	1.50E-06	–	–	4.60E-07	–	–	–
AADT	1.30E-02	1.30E-04	2.60E-02	–	1.60E-02	5.00E-03	8.90E-03

A number of factors can be observed to significantly influence rutting in I-FBS pavements including the age of the surfacing, estimated vertical compressive strain at subgrade level, curvature function from FWD deflection testing, the cumulative traffic to date (2013) and the modulus values backcalculated from FWD deflection testing for the surfacing, I-FBS base layer and subgrade. Total structural pavement thickness, bitumen content and traffic volume (AADT) appear to be the most influential of the categorical variables. The secondary stabilising agent type and content were not observed to significantly influence the rutting performance of the identified I-FBS base pavements. The lack of influence may be the result of the majority of pavement sections (> 85%) utilising between 1.5% and 2.0% lime for the secondary stabilising agent.

The results of the ANOVA analysis between categorical and continuous variables relative to the roughness performance of I-FBS pavements are shown in Table 10.7.

Table 10.7: P-value table for I-FBS base roughness ANOVA analysis

Continuous variables / Nominal variables	Total pavement thickness	Surfacing thickness	Subgrade CBR	Vertical subgrade strain	FWD deflection curvature	Cumulative traffic	Surfacing modulus E1	I-FBS modulus E2	Subgrade modulus E5
Environmental zone	–	–	–	2.00E-04	7.00E-04	–	3.70E-04	2.30E-04	–
Total pavement thickness	–	–	9.00E-03	–	–	–	–	–	–
Bitumen content	1.30E-02	9.00E-03	5.80E-03	–	1.10E-06	6.00E-03	4.30E-06	3.00E-07	2.80E-07
AADT	–	–	–	3.00E-03	1.10E-02	–	–	–	–

Contrary to the findings for PM-CMB pavements, a greater number of continuous variables were observed to have an influence on the roughness of I-FBS pavements compared to rutting performance. The influential factors included total pavement thickness, surfacing thickness, subgrade CBR estimated from D900 values from FWD deflection testing, estimated vertical compressive strain at subgrade level, curvature function from FWD deflection testing, the cumulative traffic to date (2013) and the modulus values backcalculated from FWD deflection testing for the surfacing, I-FBS base and subgrade. The surfacing type and secondary stabilising agent type and content were not observed to significantly influence roughness. The design bitumen content appears to be the most influential of the categorical variables.

10.2 Linear Regression

The in-depth evaluation of relationships between the performance indicators and both categorical and continuous variables was more clearly conducted after reviewing the ANOVA analysis results, as the number of potentially significant relationships was reduced to just those factors with p-values ≤ 0.05. However, the ANOVA analysis clearly showed that no one factor significantly influences roughness or rutting. In general, a weak or no correlation is observed when measurements of roughness and rutting are plotted against any one independent variable. However, when the dataset is grouped by categorical factors and levels, the correlations between performance indicators and continuous variables become more pronounced, if still evidently weak ($R^2 < 0.75$). It is important to note that a limitation of the linear regression approach is that the underlying relationship (where one exists) may not follow a linear trend. As a number of factors affect the development of roughness and rutting distresses, a multiple variable linear regression approach was utilised to maximise the probability of sufficiently capturing pavement performance.

10.2.1 Multivariate Linear Regression

To model the development of roughness and rutting distresses in PM-CMB and I-FBS pavements it may be necessary to conduct a multivariate linear regression involving two or more independent variables, as roughness and rutting are affected by multiple factors simultaneously. However, a regression analysis can only be applied to continuous variables. Therefore, categorical factors known to influence performance, such as environmental zone, stabilising agent type and surfacing type, require an alternative approach to model, but must still be taken into consideration.

Extensive preparation of the full dataset is required before a multivariate linear regression can be conducted. Preparations undertaken in this study included checking for collinearity between independent variables and removing outliers that may skew the results. Considering 14 continuous independent variables were used, 91 different relationships needed to be explored to eliminate collinear variables. Additionally, there were five nominal variables with possibly over 21 categories in total. Hence the analysis required to be conducted would amount to over 1000 data plots. Given time and budgeting constraints, a sensitivity analysis was first conducted to eliminate insignificant variables before undertaking the multivariate analysis.

10.2.2 Sensitivity analysis

The objective of the sensitivity analysis was to examine the extent of influence of each level within a nominal category on the performance indicators when plotted against influential continuous variables. Tables outlining linear regression correlations (positive, negative or none) for each level of each nominal category gave an indication of possible linear relationships. Resolving the levels into further sublevels showed whether the roughness or rutting was 'sensitive' to changes within the category. The sensitivity analysis procedure is further described below:

1. Identify the statistically significant continuous factors for each nominal variable for each performance indicator (e.g. FWD deflection curvature for total pavement thickness bins).

2. If two or more nominal variables are affected by the same continuous factors, sort them from lowest number of categories to highest in a pivot table (e.g. FWD deflection curvature is related to environmental zone, surfacing type, stabilising agent type, total pavement thickness and AADT). For this example, one would use a maximum of three layers to avoid removing too many data points per category.
3. Plot the performance indicator sorted by nominal variable category against the continuous variables and determine if there is a linear relationship and whether it is positive or negative.
4. Identify any trends between the subcategories.

One limitation of the sensitivity analysis approach presented above is that, with each successive layer of categorisation the number of total data points is reduced. As the full dataset does not contain the same number of data points per category, the results can vary considerably.

10.2.3 PM-CMB

Rutting Performance Factors

Using the combined ANOVA and linear regression approaches outlined in Section 10.1 and Section 10.2 respectively, the continuous variables observed to significantly influence rutting in PM-CMB pavements along the Queensland state-controlled road network included:

- surfacing age
- cumulative traffic
- average maintenance expenditure
- FWD deflection curvature function
- backcalculated modulus of PM-CMB layer
- vertical subgrade strain (calculated from backcalculated layer moduli as presented in Section 8.2).

These factors interact with the nominal categories in different ways, with the greatest number of continuous variables found to interact with total pavement thickness and traffic volume. Multivariate linear regression was conducted for these two categories and the correlations are outlined in Table 10.8 and Table 10.9. Extremely weak correlations (where the coefficient is much closer to zero compared to the other correlations) are considered to have no correlation. None of the observed relationships are considered strong, as indicated by coefficient of determination (R^2) values much less than 0.75. The categorical variable levels with the greater number of sample points also provided the weaker correlations. However, it is expected that the correlations would be stronger when they are resolved into further levels.

Table 10.8: Correlations between influential continuous variables and rutting sorted by total pavement thickness

Total pavement thickness (mm)	Influential factors					
	Surfacing age	Vertical subgrade strain	FWD deflection curvature	Cumulative traffic	Maintenance expenditure	PM-CMB modulus E2
< 200	N/A	N/A	N/A	N/A	None	N/A
201–400	None	Positive	None	None	None	Negative
401–600	Positive	None(1)	None(3)	None(4)	None	Negative(2)
601–800	Positive	Negative(2)	Negative(2)	Positive	None	Positive(5)

Total pavement thickness (mm)	Influential factors					
	Surfacing age	Vertical subgrade strain	FWD deflection curvature	Cumulative traffic	Maintenance expenditure	PM-CMB modulus E2
800 +	None	None	N/A	Negative(2)	None	N/A

1 Surfacing ages past 20 years were excluded from dataset.
2 Weak negative.
3 Could be positive: cluster concentrated near the lower end with an outlier.
4 Weak positive with no points below a line.
5 Clustered under 5000 MPa.

Table 10.9: Correlations between influential continuous variables and rutting sorted by AADT

Traffic volume (AADT)	Influential factors					
	Surfacing age	Vertical subgrade strain	FWD deflection curvature	Cumulative traffic	Maintenance expenditure	PM-CMB modulus E2
< 2000	None	NA	NA	Negative	Negative	NA
2001–4000	Positive(1)	Positive	Positive	Positive	Negative(2)	Negative
4001–6000	None	Positive	Positive(1)	Negative(2)	Positive(1)	Positive(1)
6001–10 000	Positive(1)	Positive	Positive	Positive(1)	None	Negative
> 10 000	Positive(1)	None	Positive(1)	None	None	Negative(2)

1 Weak positive.
2 Weak negative.

The number of data points between levels for the total pavement thickness category were inconsistent as sample sizes were not reflective of a consistent range of rutting over the range of the independent continuous variables they were plotted against. The relationships between continuous variables determined to significantly influence performance and rutting, when sorted by total pavement thickness (Table 10.8), were highly contradictory between levels except in the case of annual maintenance expenditure where no correlation was observed. Overall, the continuous variables appear to have a tenuous relationship to total pavement thickness where linearity is concerned and may thus be good candidates for further analysis.

The selected AADT categorical levels were representative of the range of AADT values for the PM-CMB pavements as the number of data points were evenly distributed between the levels. The relationships between the influential continuous variables (identified by ANOVA) and rutting when sorted by AADT (Table 10.9) were generally in keeping with expectation. The rutting against surfacing age, vertical subgrade strain, FWD deflection curvature and PM-CMB modulus show a dominant trend between the levels, either positive or negative. However, both the cumulative traffic and average annual maintenance expenditure show mixed responses that are probably the result of very few data points between the levels.

Roughness Performance Factors

The continuous variables observed to significantly influence the roughness of PM-CMB pavements included:

- total pavement thickness
- surfacing age
- cumulative traffic
- average maintenance expenditure
- backcalculated modulus of PM-CMB layer.

These factors interact with the nominal categories in different ways, with the greatest number of influential factors found to interact with surfacing type and traffic volume. Multivariate linear regression was conducted for these two categories and the correlations are outlined in Table 10.10 and Table 10.11. The relationships described in the following sections are not considered strong, as most have R^2 values much less than 0.75. The levels with the greater number of sample points also contained the weaker correlations, thus when they are resolved into further levels, it is expected that the correlations would be stronger.

Table 10.10: Correlations between influential continuous variables and roughness sorted by surfacing types

Surfacing type	Influential factors		
	Total pavement thickness	Maintenance expenditure	PM-CMB modulus E2
Asphalt	None	None	None
Spray seal	None	None	None

Table 10.11: Correlations between influential continuous variables and roughness sorted by AADT

Traffic volume (AADT)	Influential factors			
	Surfacing age	Total pavement thickness	Cumulative traffic	Maintenance expenditure
< 2000	Positive	Positive	Negative(2)	Positive(1)
2001–4000	Positive	Negative	Positive(1)	None
4001–6000	Positive(1)	None	None	None
6001–10 000	Positive(1)	None	Positive(1)	Negative(2)
> 10 000	Positive	None	Positive(1)	None

1 Weak positive.
2 Weak negative.

Significantly fewer continuous variables were observed to influence the roughness of PM-CMB pavements compared to rutting performance. No relationships between the influential continuous variables (identified by ANOVA) and roughness were observed when sorted by surfacing type (Table 10.10). This may be due to the existence of two populations (spray seal and asphalt) with large sample sizes. The relationships between the influential continuous variables and roughness when sorted by AADT (Table 10.11) showed a dominant trend for the age of the surfacing, an inconclusive mixed trend for cumulative traffic and generally no trend for total structural pavement thickness and average annual maintenance expenditure. Compared to rutting performance prediction for PM-CMB pavements, roughness was much more inconclusive.

10.2.4 I-FBS Base

Rutting Performance Factors

Using the combined ANOVA and linear regression approaches outlined in Section 10.1 and Section 10.2 respectively, the continuous variables observed to significantly influence rutting in I-FBS pavements along the Queensland state-controlled road network included:

- surfacing age
- cumulative traffic
- FWD deflection curvature function
- backcalculated modulus of I-FBS layer
- backcalculated modulus of subbase layer (where provisioned)

- vertical subgrade strain.

These factors interact with the nominal categories in different ways, with the greatest number of influential continuous variables found to interact with total pavement thickness and traffic volume presented in AADT. Multivariate linear regression was conducted for these two categories and the correlations are outlined in Table 10.12 and Table 10.13 below. Extremely weak correlations (where the R^2 value is much closer to zero compared to the other relationships) were considered to have no correlation. The relationships observed were not considered strong, as most have R^2 values much less than 0.75. The categorical variable levels with the greater number of sample points also exhibited the weaker correlations. Therefore, it is expected that the correlations would be stronger when resolved into further levels.

Table 10.12: Correlations between influential continuous variables and rutting sorted by AADT

Traffic volume (AADT)	Influential factors			
	Surfacing age	Vertical subgrade strain	FWD deflection curvature	I-FBS modulus E2
< 2000	None	N/A	N/A	N/A
2001–5000	Negative	Positive	Negative	Positive
5001–10000	Negative	Positive	Positive	Negative
> 10 000	Positive	Positive	Positive	None

Table 10.13: Correlations between influential continuous variables and rutting sorted by total pavement thickness

Total pavement thickness (mm)	Influential factors			
	Vertical subgrade strain	FWD deflection curvature	I-FBS modulus E2	Subgrade modulus E5
< 200	N/A	N/A	N/A	N/A
201–400	None	None	Positive	Positive
401–600	Positive	None	Positive	Negative
601–800	Negative	Negative	Positive	Positive
800+	Positive	Positive	Negative	Negative

The relationships between continuous variables determined to significantly influence performance and rutting when sorted by AADT (Table 10.12) were highly contradictory between levels except in the case of vertical compressive strain at subgrade level, where a dominant positive correlation was observed. Additionally, at the < 2000 AADT categorical level, insufficient sample points data points were available to develop a relationship against vertical subgrade strain, FWD deflection curvature or I-FBS base modulus backcalculated from FWD deflection testing. The relationship found between vertical subgrade strain and rutting is in agreement with traditional pavement theory that states rutting will increase with increasing strain magnitude. The relationships between rutting and surfacing age, FWD deflection curvature function and backcalculated modulus of the I-FBS base layer are expected to be positive, positive and negative respectively. However, no dominant trend among the categorical levels was observed.

The relationships between the influential continuous variables and rutting when sorted by total structural pavement thickness (Table 10.13) were highly contradictory between levels except in the case of the backcalculated modulus of the I-FBS base layer, where a generally positive correlation was observed. Similarly to the observation when I-FBS base rutting performance was sorted by AADT, at the < 200 mm categorical level insufficient sample points were available to develop significant relationships. The relationship observed between backcalculated I-FBS base modulus and rutting magnitude is in conflict with traditional pavement theory, as rutting magnitude is expected to decrease with increasing material stiffness. The relationships between rutting and vertical subgrade strain, FWD deflection curvature function and backcalculated modulus of the subgrade are expected to be positive, positive and negative respectively. However, no dominant trend among the categorical levels was observed.

Roughness Performance Factors

The continuous variables observed to significantly influence the roughness of I-FBS pavements included:

- surfacing layer thickness
- total pavement thickness
- cumulative traffic
- FWD deflection curvature function
- backcalculated modulus of I-FBS layer
- vertical subgrade strain
- subgrade bearing capacity (%CBR).

These factors interact with the categorical variables in different ways, with the greatest number of influential factors found to interact with primary stabilising agent (bitumen) content and secondary stabilising agent type and content. Multivariate linear regression was conducted for these two categories and the correlations are outlined in Table 10.14. Extremely weak correlations (where the R^2 value is much closer to zero compared to the other relationships) were considered to have no correlation. The relationships presented in Table 10.14 are not considered strong, as most have R^2 values much less than 0.75. The categorical levels with the greater number of sample points also contained the weaker correlations. Therefore, it is expected that the correlations would be stronger when resolved into further levels.

Table 10.14: Correlations between influential continuous variables and roughness sorted by stabilising agent type

Stabilising agent type	Influential factors					
	Total pavement thickness	Surfacing thickness	FWD deflection curvature	Subgrade CBR(2)	I-FBS modulus E2	Cumulative traffic
3.5%B + 1.5%C	Positive	None	N/A	N/A	N/A	Positive
3.5%B + 2.0%C	N/A	N/A	Negative	N/A	Positive	N/A
3.0%B + 2.0%L	None	Positive	None	Negative	Negative	Positive
3.5%B + 2.0%L	None	None	Positive	None	Negative	Negative
3.5%B + 1.5%L	None	Positive	Positive	None	Negative	Positive
4.5%B + 2.0%L	Positive(1)	Negative	None	N/A	None	Positive

1 Three sample points only.
2 CBR data only available for sections subjected to FWD testing.

A larger number of continuous variables were observed to influence the roughness of I-FBS pavements compared to rutting performance. The development of significant relationships was challenging, as the sample sizes were not consistent across the categorical levels nor was the range of continuous variables uniformly distributed. The relationships between the influential continuous variables and roughness, when sorted by stabilising agent types (Table 10.14), showed general trends for the backcalculated modulus of the I-FBS base layer and the cumulative traffic, and inconclusive mixed trends for total pavement thickness, surfacing thickness, FWD deflection curvature function and estimated subgrade CBR. The trends for backcalculated I-FBS base modulus and cumulative traffic are both in keeping with traditional performance expectations, as rutting magnitude is expect to increase with decreasing layer stiffness and increasing traffic load applications. The relationships between roughness and total pavement thickness, surfacing layer thickness, FWD deflection curvature function and subgrade CBR are expected to be negative, negative, positive and negative respectively. However, no dominant trend among the categorical levels was observed. The lack of correlation may be due to the fact that the absolute roughness value may not be a good indicator of deterioration, as there is assumed to be some roughness built in during construction and absolute roughness may vary over time due to differences in measurement.

10.3 Summary of Statistical Analysis

A combined ANOVA and multivariate linear regression approach was undertaken to investigate relationships between operational factors and the observed performance of PM-CMB and I-FBS pavement sections along the Queensland state-controlled road network. The operational factors investigated are presented in Table 10.1, Table 10.2 and Table 10.3 for the nominal, ordinal and continuous variables respectively. Measurements of roughness (IRI) and rutting (mm) obtained from the ARMIS database (2013) were used as the performance indicators for the analysis. The effect of each variable on the measured rutting and roughness was assessed by first separating the dataset into smaller samples according to the categorical factors and grouping according to the various levels presented in Table 10.1 and Table 10.2. An ANOVA analysis was then conducted for each factor group and level to investigate the existence of relationships between the continuous factors (Table 10.3) and the performance indicators, with p-values < 0.05 considered statistically significant. The results from the ANOVA analysis showed differences in what operational factors significantly influenced the rutting and roughness for the I-FBS and PM-CMB pavement sections, as shown in Table 10.5. Relative to the set of continuous factors and pavement sections selected in this study, there appear to be more factors influencing rutting, compared to roughness.

Table 10.15: Influential variables summary for I-FBS and PM-CMB as determined from ANOVA

Nominal factors / Continuous factors		I-FBS					PM-CMB				
		Total pavement thickness	Environmental zone	Stabilising agent	Surfacing type	AADT	Total pavement thickness	Environmental zone	Stabilising agent	Surfacing type	AADT
Rutting	Total pavement thickness										
	Vertical subgrade strain	X				X	X		X	X	X
	FWD deflection curvature	X	X	X		X	X		X	X	X
	Cumulative traffic (ESA)				X		X			X	X
	Surfacing age				X	X	X				X
	Stabilised material M_r	X		X		X	X		X		X
	Subgrade CBR								X		
	Surfacing thickness										
	Annual maintenance cost						X			X	X

Continuous factors \ Nominal factors		I-FBS					PM-CMB				
		Total pavement thickness	Environmental zone	Stabilising agent	Surfacing type	AADT	Total pavement thickness	Environmental zone	Stabilising agent	Surfacing type	AADT
Roughness	Total pavement thickness			X						X	X
	Vertical subgrade strain		X			X					
	FWD deflection curvature		X	X		X					
	Cumulative traffic (ESA)			X							X
	Surfacing age										X
	Stabilised material M_r		X	X			X			X	
	Subgrade CBR	X		X							
	Surfacing thickness			X							
	Annual maintenance cost			X					X	X	X

Surprisingly, total pavement thickness and thickness of the surfacing layer were not observed to significantly influence the development of rutting distress for either PM-CMB or I-FBS pavements. Additionally, the environmental zone did not significantly correlate with either rutting or roughness for PM-CMB pavements. However, this finding reflects the fact that almost all (> 99%) of the identified PM-CMB pavement sections are located in WNR regions. Therefore, no variability in performance could be observed as a result of different environmental zones. Where a statistically significant relationship was detected (p-value < 0.05) between a continuous variable and either roughness or rutting, a multivariate linear regression analysis was performed.

10.3.1 Correlation of PM-CMB

Not unexpectedly, surfacing age, cumulative traffic, average annual maintenance expenditure and stabilised layer M_r were observed to correlate with the measurements of both rutting and roughness. The age of the surfacing and the annual maintenance expenditure are maintenance factors that can be directly influenced by the effective programming of treatments in accordance with asset management best practice. The traffic volume (AADT) is determined by the location and class of road and is difficult to control. However, similarly to surfacing age and maintenance expenditure, the effective programming of rehabilitation works can minimise the cumulative trafficking, and subsequent damage, sustained by in-service pavements. The M_r of the stabilised layer, as indicated by modulus values backcalculated from FWD deflection measurements, can be controlled by the pavement designer and is optimised through the use of mixture proportioning and structural design best practice as presented in Section 3.6 and Section 3.7 respectively. Further details on the statistical analysis of the PM-CMB pavement sections are presented in Appendix C.

10.3.2 Correlation of I-FBS Base

In agreement with traditional pavement performance considerations, the cumulative traffic, M_r of the I-FBS material and critical compressive strain at subgrade level were observed to significantly influence both rutting and roughness. As mentioned above for the PM-CMB pavements, the cumulative traffic is often out of the control of the pavement designer, but can be managed through the programming of efficient pavement rehabilitation treatments. Also, similarly to PM-CMB materials, the M_r of I-FBS base materials can be optimised through the use of mixture proportioning and structural design best practice as presented in Section 4.4 and Section 4.5

respectively. The vertical compressive strain at subgrade level is determined by the traffic load, thickness of the pavement structure and stiffness of the composing material layers. The development of high vertical subgrade strains can be managed through accurate characterisation of anticipated traffic type and frequency during the structural design process. Further details on the statistical analysis of I-FBS pavement sections are presented in Appendix C.

11 ECONOMIC ASSESSMENT

In addition to the technical advantages of PM-CMB and I-FBS base, including maintaining the serviceability (Section 5.2) and prolonging the structural life (Section 8.3) of road pavements, the stabilisation technologies also provide significant economic benefits, where appropriately selected, designed, constructed and maintained. A key objective of this project was to investigate whether stabilised materials are cost-effective alternatives to unbound granular and asphalt concrete structural layers, relative to Queensland traffic and environmental conditions. To facilitate an assessment of the value for money of PM-CMB and I-FBS base, construction cost information was gathered from the Far North, Northern, Mackay/Whitsunday and South-eastern regions of the state. It is commonly accepted that several key factors affect the cost of pavement stabilisation treatments, including:

- project location (distance from material sources and qualified contractors)
- size or value of the project (low quantities often attract higher unit rates due to fixed establishment costs)
- pavement design life and associated layer thickness
- stabilising agent type and application rate
- on site material storage requirements
- required curing regime.

These factors produce a wide range of project cost values and it is important for site-specific unit cost rates to be developed for each job in order to compare pavement design options.

The project cost data obtained from the TMR regions was broken down into cost items to establish high, medium and low values for the principal budget considerations for PM-CMB and I-FBS pavement layers. Additionally, a 'standard design application' or representative pavement configuration was established to allow for comparison of PM-CMB and I-FBS base with unbound granular and hot-mix asphalt (HMA) base layers across a range of structural design lives (ESAs). The minimum design thickness was determined using a linear elastic analysis in accordance with Austroads (2012) methods and an estimate of capital investment for each alternative was determined by applying the minimum, average and maximum values of the project cost data supplied by the TMR regions. The underlying pavement structure was kept constant between the base technology alternatives and design traffic levels to allow differences in cost to be attributed directly to the selected base material. The underlying pavement structure included:

- 200 mm existing granular pavement providing a support layer with a modulus of 150 MPa
- infinite depth CBR 5% subgrade with a modulus of 50 MPa.

The outcome of the economic assessment of PM-CMB and I-FBS technologies is presented in Section 11.1 and Section 11.2 respectively. It is important to note that the economic benefits of stabilisation technologies extend beyond initial capital investment and are more pronounced when also examining the reduced maintenance expenditure. However, due to the limitations of the annual maintenance expenditure data obtained from the ARMIS database (Section 5.2.2), the economic assessment was constrained to the capital investment data.

11.1 PM-CMB

The cost of PM-CMB varied between \$124 and \$347 per m^3, with a typical unit rate experienced in the regions of around \$200 per m^3. These prices included the cost of the unbound granular parent material (typically TMR Type 2), supply of the cementitious stabiliser, mixing of the cementitious stabiliser and aggregate in a pugmill, transport to the project site, formation of the base layer using a paver, compaction, trimming and curing. The basic cost items for PM-CMB collected as part of this investigation are summarised in Table 11.1.

Table 11.1: Indicative cost of principal PM-CMB budget items

Cost item	Low cost	Medium cost	High cost
Cementitious binder ($/tonne)	350.00	500.00	750.00
Parent aggregate, mixing, transport and placement ($/m^3)	100.00	140.00	180.00
Water curing ($/m^2)	1.00	2.00	3.00

A structural design was undertaken in accordance with Austroads (2012) methods for a standard design application incorporating TMR Type 2 unbound granular, PM-CMB and HMA base layers with four design loading conditions including 10^5 ESA, 10^6 ESA, 10^7 ESA and 10^8 ESA. A summary of the modulus values utilised in the analysis and the minimum design thickness calculated is presented in Table 11.2.

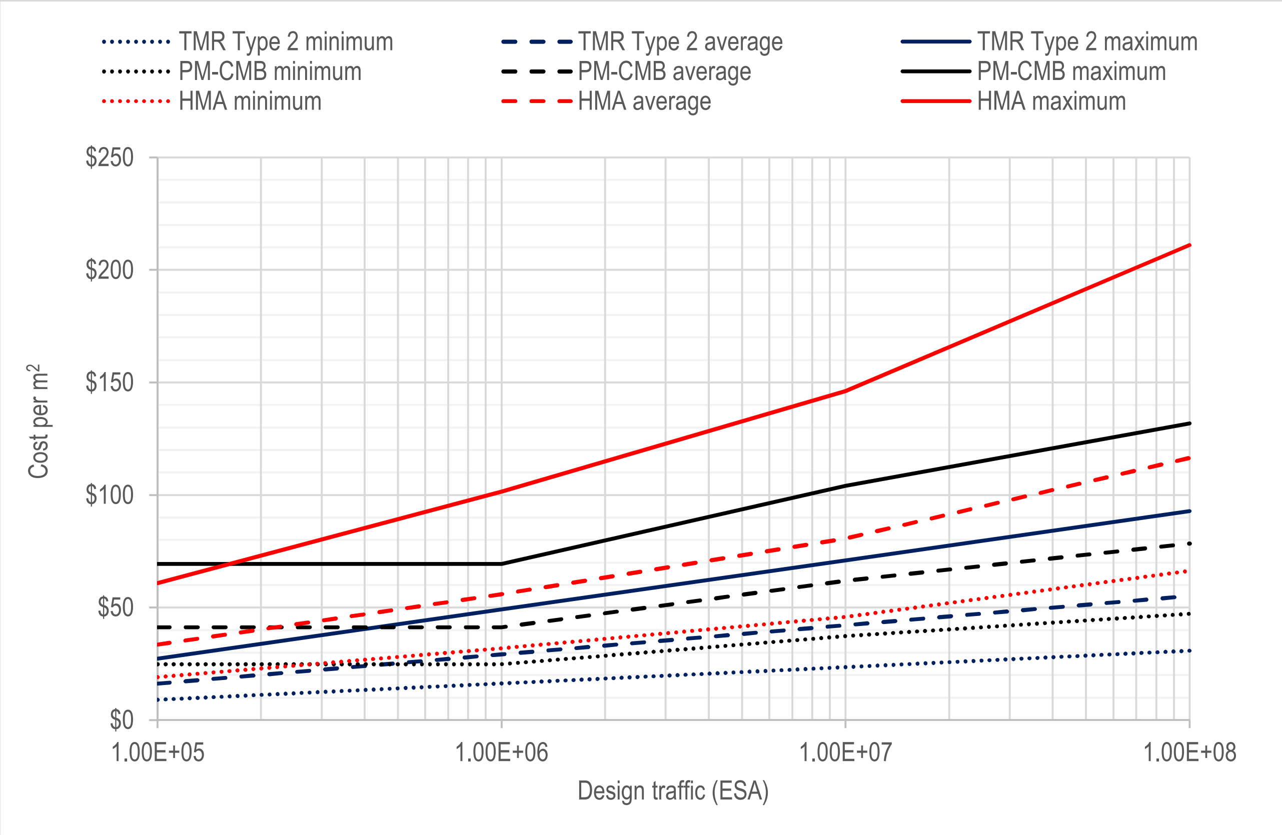
Table 11.2: Representative pavement configuration for PM-CMB cost analysis

Design traffic (ESA)	Unbound granular base		PM-CMB		HMA base		Subbase(2)		Subgrade(2)	
	Modulus (MPa)(1)	Minimum thickness (mm)	Modulus (MPa)(1)	Minimum thickness (mm)	Modulus (MPa)	Minimum thickness (mm)	Modulus (MPa)	Minimum thickness (mm)	Modulus (MPa)	Minimum thickness (mm)
1.00E+05	300	125	450	200	2600	75	150	200	50	∞
1.00E+06	350	225	450	200	2600	125	150	200	50	∞
1.00E+07	350	325	500	300	2600	180	150	200	50	∞
1.00E+08	350	425	500	380	2600	260	150	200	50	∞

1 Modulus value presented is representative of the uppermost sublayer.
2 The underlying pavement structure was consistent for each alternative base material type evaluated.

Applying the minimum layer thickness values presented in Table 11.2 and the minimum, average and maximum values of the project cost data supplied by the TMR regions allows for the comparison of capital investment cost requirements presented in Figure 11.1.

Figure 11.1: Estimated cost of PM-CMB vs. unbound granular and HMA base



The cost for provisioning PM-CMB generally bisects the costs of unbound granular and HMA base layers. Due to the minimum layer thickness of 200 mm proposed in MRTS ET05C (TMR 2012b), at design traffic levels below 3.0 x 10^5 ESA, either unbound granular or HMA base layers may be more appropriate. When comparing the cost of the alternative base layer materials, it is important to consider the level of serviceability and resilience associated with each technology. While unbound granular base is the cheapest option of those considered, it generally does not provide the ride quality nor resistance to environmental and traffic loading effects compared to PM-CMB or HMA base. Disregarding performance and maintenance cost considerations, at design traffic levels greater than 10^6 ESA, PM-CMB generally costs 43% more than unbound granular base and 27% less than HMA base in a standard pavement design application.

11.2 I-FBS Base

The cost of I-FBS base varies between $59 and $123 per m^3 when the in situ unbound granular base is of sufficient quality (Section 4.2.2) or between $132 and $342 per m^3 when TMR Type 2 aggregate also has to be supplied. Typical unit rates observed in the regions were approximately $80 per m^3 for I-FBS base using in situ material and $200 per m^3 for I-FBS base with additional TMR Type 2 aggregate. These prices include the cost of bitumen and secondary stabilising agent (typically lime), unbound granular material (where required), transport to site, in situ mixing of lime and bitumen with aggregate using a dedicated stabilising machine, compaction, trimming and curing. The basic cost items for I-FBS base collected as part of this investigation are summarised in Table 11.3.

Table 11.3: Indicative cost of principal I-FBS budget items

Cost item	Low cost	Medium cost	High cost
C170 bitumen ($/litre)	1.10	1.10	1.10

Cost item	Low cost	Medium cost	High cost
Type 2.1/3.1 parent aggregate ($/m^3)	100.00	150.00	200.00
2% lime, mixing and trimming ($/m^2)	15.00	20.00	26.00

Identically to the approach followed for PM-CMB, a structural design was undertaken in accordance with Austroads (2012) methods for a standard design application incorporating TMR Type 2 unbound granular, I-FBS and HMA base layers with four design loading conditions including 10^5 ESA, 10^6 ESA, 10^7 ESA and 10^8 ESA. A summary of the modulus values utilised in the analysis and the minimum design thickness calculated is presented in Table 11.4.

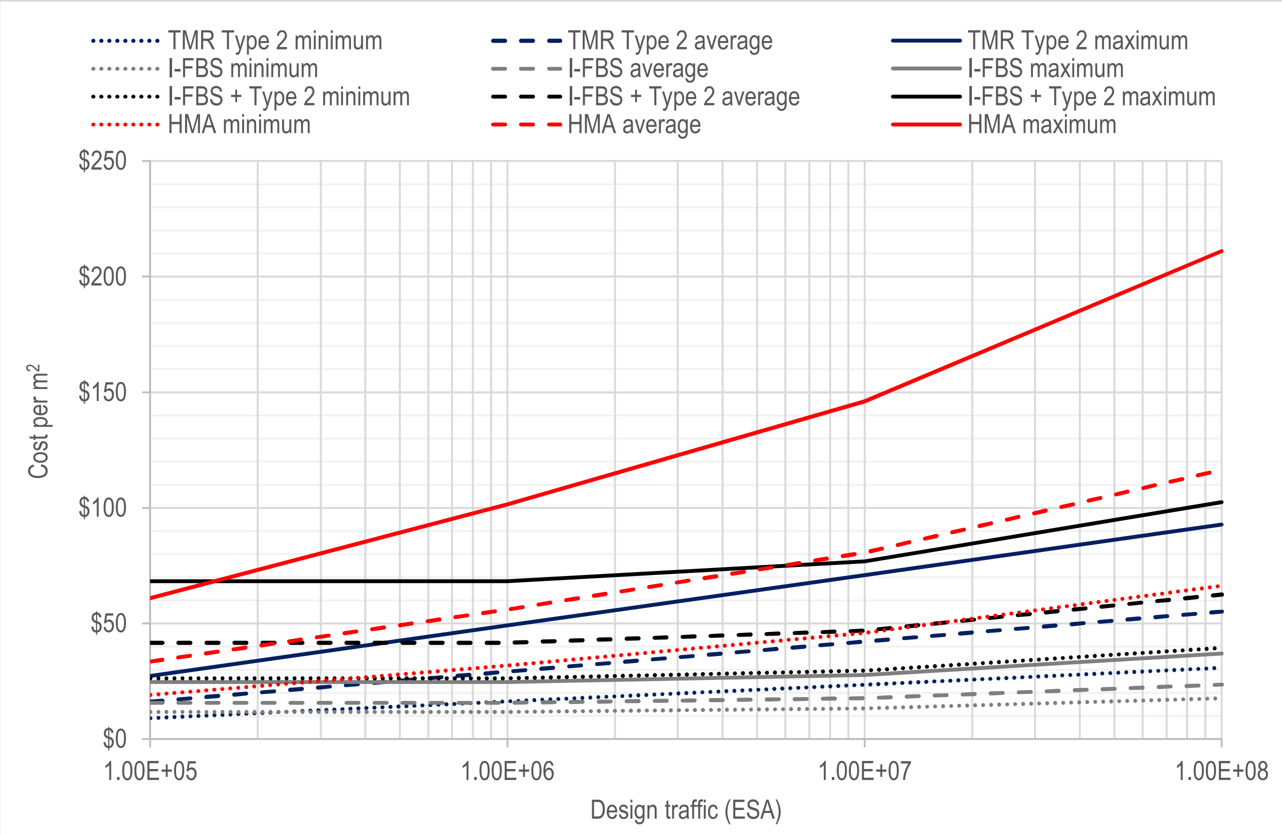
Table 11.4: Representative pavement configuration for I-FBS cost analysis

Design traffic (ESA)	Unbound granular base		I-FBS base		HMA base		Subbase(1)		Subgrade (1)	
	Modulus (MPa)	Minimum thickness (mm)	Modulus (MPa)	Minimum thickness (mm)	Modulus (MPa)	Minimum thickness (mm)	Modulus (MPa)	Minimum thickness (mm)	Modulus (MPa)	Minimum thickness (mm)
1.00E+05	300	125	2000	200	2600	75	150	200	50	∞
1.00E+06	350	225	2000	200	2600	125	150	200	50	∞
1.00E+07	350	325	2000	225	2600	180	150	200	50	∞
1.00E+08	350	425	2000	300	2600	260	150	200	50	∞

1 The underlying pavement structure was consistent for each alternative base material type evaluated.

Applying the minimum layer thickness values presented in Table 11.4 and the minimum, average and maximum values of the project cost data supplied by the TMR regions allows for the comparison of capital investment cost requirements presented in Figure 11.2.

Figure 11.2: Estimated cost of I-FBS vs. unbound granular and HMA base



Where the in situ unbound granular material is of sufficient quality (Section 4.2.2), I-FBS base can be provisioned at a much lower cost than replacing the existing material with TMR Type 2 unbound granular material or HMA. Because the investment in parent material has already been made (> 90% by volume), the cost of I-FBS base only includes acquisition and transport of the bitumen and secondary stabilising agent and utilisation of a dedicated stabilising machine. The other construction activities, including compaction, trimming and curing are required, to some extent, for all of the alternative base materials. Effectually, a pavement base layer can be stabilised with foamed bitumen for 54% less than the cost of TMR Type 2 unbound granular material or 77% less than the cost of HMA in a standard pavement design application.

Where the in situ unbound granular material is of poor quality and virgin TMR Type 2 unbound aggregate has to be acquired and transported, I-FBS base can still be provisioned at a significantly lower cost than HMA. Additionally, at design traffic levels >10^7 ESA, I-FBS base can be established at effectively the same cost as unbound granular base, but with significantly enhanced performance properties. Disregarding performance and maintenance cost considerations, at design traffic levels greater than 10^6 ESA, I-FBS base with virgin TMR Type 2 aggregate generally costs 22% more than unbound granular base and 38% less than HMA base in a standard pavement design application.

11.3 Summary of Economic Assessment

The initial capital investment required for PM-CMB and I-FBS pavements in Queensland has been investigated for a standard pavement design application by reviewing recent (2014) project costing data from regions across the state. As stabilised pavements are generally provisioned in Queensland for applications where moisture inundation is likely and thick unbound granular layers are not feasible, both PM-CMB and I-FBS base are resilient pavement solutions that can be provisioned at a fraction, 85% and 78% respectively, of the cost of asphalt layers. The condition assessment of pavement sections investigated in this study indicated that approximately 99.5% and 93.9% of PM-CMB and I-FBS pavements respectively, along the state-controlled road network are in good to excellent condition despite the high-risk operating environment. This finding supports the value for money of the respective pavement technologies, particularly when considering that a significant number of the pavement sections have exceeded the original design service life.

In light traffic volume applications (< 3.0×10^5 ESA) the use of PM-CMB or I-FBS base structural layers may not be appropriate. Ideal stabilised material construction practice suggests that minimum layer thickness should be limited to 200 mm. In light traffic applications, thin (≈ 100 mm) layers of asphalt can be utilised with better performance outcomes. Where the risk of moisture inundation is low, unbound granular layers can be used at significantly reduced cost.

The economic analysis suggests that PM-CMB provides good value for money in new construction or structural rehabilitation for applications with moderate to heavy traffic, wet climatic conditions, nonreactive subgrade soils and a high-quality surfacing (asphalt or reinforced seal) is planned. I-FBS pavements provide significant value for money, particularly when existing materials are of sufficient quality, for rehabilitation applications with moderate to heavy traffic, a variety of climatic conditions and subgrade types and appropriate measures are taken to stabilise moisture sensitive soils.

The true economic impact of road pavement infrastructure is ideally assessed relative to WOL cost where initial capital investment, maintenance costs and user costs are considered. WOL costing of representative PM-CMB and I-FBS base pavements has not been attempted in this research due to data limitations, but such an approach may add significant value at a later date when sufficient surfacing treatment and routine maintenance data is available.

12 CONCLUSIONS AND RECOMMENDATIONS

A review of stabilisation technologies in Queensland, in particular PM-CMB and I-FBS base, was undertaken to develop technical guidance on the selection, design and construction of stabilised pavement layers in accordance with international best practice and relative to Queensland roadbed conditions. Establishment of best practice was achieved through a review of literature, to include both national and international specifications and other technical documents. Documentation of current practices in Queensland was accomplished by querying the ARMIS database and summarising inventory and performance data, categorising the relevant state-controlled road network according to current condition and evaluating the surface condition, structural capacity and material properties of selected pavements representative of both good and poor performing PM-CMB and I-FBS base stabilised pavements. Confirmation of best practice relative to Queensland roadbed conditions was pursued through a statistical analysis of the influence of material properties, pavement configuration, in addition to traffic, environment and climatic factors on the in-service performance of PM-CMB and I-FBS base pavements. General conclusions resulting from the investigation include:

- The review of literature highlighted differences in the implementation of PM-CMB and I-FBS both nationally and internationally. Queensland practices for mixture proportioning and construction generally align with international best practice. However, significant discrepancies were identified in the fundamental application, stabilisation binder selection and structural design of both PM-CMB and I-FBS pavements. The differences in practice stem from the controlling failure mode and underlying design assumptions.
- The TMR ARMIS database contains valuable inventory, condition assessment and maintenance data. This information is critical to understanding the nature, current condition and performance trends of the road network. However, a number of discrepancies between the database and actual conditions were identified.
- PM-CMB is a component of approximately 109 km and I-FBS base approximately 156 km of the Queensland state-controlled road network. A total of 20 road sections, representing approximately 10% of the identified sections for each stabilisation technology, were selected for detailed visual and structural capacity assessment. Twelve of these sections were also subjected to laboratory material characterisation.
- The majority of the PM-CMB and I-FBS pavements along the state-controlled road network, 99.5% and 93.9% respectively, are in good to excellent condition including several that have exceeded the original service life estimates. Structural assessment of a representative proportion of the network indicated that many of these pavements have significant (> 10^6 ESA) bearing capacity remaining.
- PM-CMB and I-FBS road sections are typically provisioned in moderate to high traffic volume, weak subgrade, expansive foundation and/or potential flood inundation applications. The generally good condition of these pavements after a number of years of service suggests that these stabilisation technologies may be more resilient than unbound granular base in high exposure (traffic, subgrade and/or moisture) environments.
- The increased resilience of PM-CMB and I-FBS stabilisation technologies in high-exposure environments can be achieved at a fraction, 85% for PM-CMB and 78% for I-FBS base, of the cost of full depth asphalt. Greater utilisation of these technologies, in accordance with best practice, can result in significant reductions in both construction and maintenance expenditure for TMR.

12.1 PM-CMB

Significant findings relative to the PM-CMB technology include:

- The majority of PM-CMB pavements on the state-controlled road network are located in northern coastal Queensland, are less than 12 years old, service AADT volumes ranging from 2500 to 25 000 vehicles per day and have highly variable stabilisation binder types and contents.
- Condition categorisation according to best practice revealed that approximately 99.5% of PM-CMB road sections along the Queensland state-controlled road network are in good condition. The average condition for the network included rutting of 4.2 mm and IRI of 1.6.
- For the representative pavements investigated as part of this study, the PM-CMB layer thickness ranged from 100 mm to 500 mm with backcalculated in situ resilient modulus values ranging from 100 MPa to 12 000 MPa.
- The key performance indicators of rutting and roughness were observed to be significantly influenced by maximum vertical strain at subgrade level, the deflection curvature function, PM-CMB resilient modulus, surfacing age, cumulative traffic and total pavement thickness.
- The relative cost of provisioning PM-CMB stabilised pavement layers ranges from $124 to $347 per m^3 according to the location of project, transport of additive, location of additive geographic base, total quantities, storage on site, curing regime, depth, additive type and quarry source.

Recommendations specific to the PM-CMB technology include:

- The current definition for modified pavement materials in Queensland includes a 28-day UCS of 1.0–2.0 MPa. However, the bound nature of pavement materials is better defined by the tensile capacity. Therefore, investigation of an alternative characterisation method, such as indirect tensile strength (Q315) or indirect tensile resilient modulus (Q139), should be undertaken.
- The structural design of PM-CMB in Queensland and Australia is based solely upon the vertical compressive strain at subgrade level. Supplementary models for consideration of permanent deformation within the modified layer, such as implemented by NCHRP and COLTO, should be investigated.
- Generally, the practices utilised for provision of PM-CMB pavement layers is highly variable across the state, particularly with respect to selection of binder type and content. A Technical Note should be drafted in collaboration with TMR, the ARRB Group and industry leaders outlining a standard technology selection methodology and highlighting best practices for the utilisation of PM-CMB.

12.2 I-FBS Base

Significant findings relative to the I-FBS base technology include:

- I-FBS base pavements on the state-controlled road network are widely distributed throughout the state and are highly variable with respect to age and traffic volume. Typically, between 3.0% and 3.5% of type C170 bitumen is used with 1.0-2.0% lime as the secondary stabilising agent.
- Discrepancies were observed between the design and actual stabilising agent contents and stabilised layer thickness. The relative magnitude of the differences varied regionally. These variations may be reduced through the incorporation of construction best practice into TMR technical specifications and active enforcement at the project level.

- Condition categorisation according to best practice revealed that approximately 93.9% of I-FBS road sections along the Queensland state-controlled road network are in good condition. The average condition for the network included rutting of 5.9 mm and IRI of 1.9.
- For the representative pavements investigated as part of this study, the I-FBS base layer thickness ranged from 100 mm to 300 mm with backcalculated in situ resilient modulus values ranging from 700 MPa to 9000 MPa.
- For the limited pavements subjected to laboratory material characterisation as part of this study, bitumen contents ranged from 1.9% to 7.6%, as-received and soaked resilient modulus values ranged from 3000 MPa to 21 000 MPa and 2500 MPa to 22 000 MPa respectively, and retained modulus values ranged from 52% to 106%.
- Primary and secondary stabilising agent content (typically C170 bitumen and lime) in addition to bulk density directly correlated with both as-received and soaked resilient modulus in addition to retained modulus.
- The key performance indicators of rutting and roughness were observed to be significantly influenced by maximum vertical strain at subgrade level, the deflection curvature function, I-FBS base and subbase resilient modulus, surfacing age, cumulative traffic, subgrade CBR, surfacing thickness and total pavement thickness.
- The relative cost of provisioning I-FBS stabilised pavement layers ranges from $59 to $123 per m^3 when the in situ material is of sufficient quality or from $132 to $342 per m^3 including virgin TMR Type 2 aggregate according to the location of project, transport of additive, location of additive geographic base, total quantities, storage on site, curing regime, depth, additive type and quarry source.

Recommendations specific to the I-FBS base technology include:

- The structural design of I-FBS pavements, both nationally and internationally, is based on fatigue of the I-FBS layer and/or permanent deformation of the composite pavement structure. However, the principal distress modes observed during the field inspections carried out as part of this investigation included rutting, shoving and flushing. Further investigation of the controlling failure mode for I-FBS pavements is required.
- Laboratory testing results on cores extracted from in-service I-FBS pavements in the Far North Region indicate high bitumen contents (≈ 6%) were utilised. Accelerated distress development (rutting & shoving) was also observed in these pavements. Excessive binder contents may be the result of improper mixture design, poor construction practice, or both. The best practices documented throughout this project should be referenced by Far North Region practitioners prior to any future works.
- I-FBS stabilisation practices are variable across the state, with south-eastern projects generally conforming to best practice, but other regions deviating, in some cases significantly, to the detriment of the in-service performance. A Technical Note should be drafted in collaboration with TMR, the ARRB Group and industry leaders outlining a standard technology selection methodology and highlighting best practices for the utilisation of I-FBS base.

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APPENDIX A DETAILS OF CONDITION ASSESSMENT

Table A 1: CMB network evaluation summary

Region name	Layer thickness (mm)	Cement content (%)	Construction year	Environmental zone	Excellent roughness and rutting	Excellent rutting	Good rutting	Mediocre rutting	Poor rutting	Excellent roughness	Good roughness	Mediocre roughness	Poor roughness	Poor roughness deterioration	Poor rutting deterioration
Far North	250	3.0	2011	WNR	50	87	9	1	3	57	23	15	1	27	0
Mackay/Whitsunday	150	1.0	2006	WNR	8	9	1	0	0	8	2	0	0	0	0
Mackay/Whitsunday	150	1.0	2008	WNR	4	6	1	0	0	4	2	1	0	1	0
Mackay/Whitsunday	150	2.0	2003	WNR	10	42	10	1	7	11	24	11	0	5	0
Mackay/Whitsunday	200	2.0	2001	WNR	0	1	3	2	2	2	1	1	0	0	0
Mackay/Whitsunday	320	(blank)	2011	WNR	3	3	5	1	0	9	0	0	0	0	0
Mackay/Whitsunday	330	1.5	2005	WNR	11	15	1	0	0	12	2	2	0	1	0
Mackay/Whitsunday	340	1.0	2009	WNR	15	15	4	1	0	18	1	1	0	3	0
Mackay/Whitsunday	350	2.0	2009	WNR	21	21	2	0	0	23	0	0	0	3	0
Northern	125	1.5	2008	WNR	1	2	0	0	0	1	1	0	0	0	0
Northern	125	1.5	2012	WNR	27	27	1	0	0	28	0	0	0	0	0
Northern	140	2.0	2012	WNR	16	16	0	0	0	16	0	0	0	0	0
Northern	150	1.5	2012	WNR	14	21	0	0	0	14	7	0	0	0	0
Northern	150	(blank)	2009	WNR	5	5	0	0	0	5	0	0	0	0	0
Northern	180	1.5	2008	WNR	0	0	2	0	0	2	0	0	0	0	0
Northern	180	1.5	2012	DNR	15	15	4	0	0	18	1	0	0	0	0
Northern	180	(blank)	2005	WNR	2	2	0	0	0	2	0	0	0	0	0
Northern	200	1.5	2008	N/A	1	2	1	0	0	2	1	0	0	1	0
Northern	200	1.5	2008	WNR	7	8	0	0	0	7	1	0	0	2	0
Northern	200	2.0	2004	WNR	6	15	0	0	0	6	3	6	0	0	0

Region name	Layer thickness (mm)	Cement content (%)	Construction year	Environmental zone	Excellent roughness and rutting	Excellent rutting	Good rutting	Mediocre rutting	Poor rutting	Excellent roughness	Good roughness	Mediocre roughness	Poor roughness	Poor roughness deterioration	Poor rutting deterioration
Northern	200	2.0	2008	WNR	0	2	0	0	0	0	0	2	0	0	0
Northern	200	(blank)	2004	WNR	0	0	0	0	0	0	0	0	0	0	0
Northern	220	(blank)	2011	WNR	10	10	0	0	0	10	0	0	0	0	0
Northern	240	(blank)	2009	N/A	0	0	0	0	0	0	0	0	0	0	0
Northern	240	(blank)	2009	WNR	0	0	0	0	0	0	0	0	0	0	0
Northern	250	1.5	2012	WNR	4	4	0	0	0	4	0	0	0	0	0
Northern	250	3.0	2010	DNR	61	61	1	0	0	62	0	0	0	1	0
Northern	250	3.0	2010	WNR	127	135	2	0	0	129	6	2	0	8	6
Northern	250	3.0	2011	N/A	12	12	1	0	0	13	0	0	0	2	0
Northern	250	4.0	2012	WNR	2	7	4	0	0	3	8	0	0	0	0
Northern	250	(blank)	2010	N/A	9	11	0	0	0	9	1	1	0	3	0
Northern	250	(blank)	2010	WNR	0	1	0	0	0	0	0	1	0	1	0
Northern	270	2.0	2003	WNR	2	3	3	0	0	2	3	1	0	1	0
Northern	290	1.5	2009	WNR	3	3	0	0	0	3	0	0	0	0	0
Northern	300	2.0	2008	WNR	9	15	0	0	0	9	5	1	0	1	0
Northern	300	(blank)	2012	WNR	6	6	0	0	0	6	0	0	0	0	0
Northern	320	1.5	2009	WNR	2	2	0	0	0	2	0	0	0	0	0
Northern	340	(blank)	2012	WNR	25	25	0	0	0	25	0	0	0	0	0
Northern	380	2.0	2003	WNR	1	1	1	0	0	2	0	0	0	0	0
Northern	410	1.5	2006	WNR	0	0	1	0	0	0	0	1	0	0	0
Northern	420	(blank)	2009	WNR	8	8	0	0	0	8	0	0	0	5	0
Northern	430	(blank)	2009	WNR	18	20	0	0	0	18	2	0	0	2	0
Northern	460	(blank)	2009	WNR	48	49	0	0	0	48	1	0	0	6	1

Region name	Layer thickness (mm)	Cement content (%)	Construction year	Environmental zone	Excellent roughness and rutting	Excellent rutting	Good rutting	Mediocre rutting	Poor rutting	Excellent roughness	Good roughness	Mediocre roughness	Poor roughness	Poor roughness deterioration	Poor rutting deterioration
Northern	460	(blank)	2012	WNR	2	2	0	0	0	2	0	0	0	0	0
Northern	480	(blank)	2009	WNR	8	8	0	0	0	8	0	0	0	1	1
Northern	500	1.5	2009	WNR	0	0	0	0	0	0	0	0	0	0	0
Northern	520	(blank)	2009	WNR	1	1	0	0	0	1	0	0	0	0	0
Northern	540	1.5	2009	WNR	1	1	0	0	0	1	0	0	0	0	0
Northern	580	1.5	2009	WNR	0	0	0	0	0	0	0	0	0	0	0

Table A 2: FBS network evaluation summary

Region name	Stabiliser content (%)	Layer Thickness (mm)	Construction year	Environmental zone	Excellent roughness and rutting	Excellent roughness	Good roughness	Mediocre roughness	Poor roughness	Excellent rutting	Good rutting	Mediocre rutting	Poor rutting	Poor roughness deterioration	Poor rutting deterioration
Darling Downs	3.5 B+2.0C	200	1997	DNR	3	3	9	4	0	16	0	0	0	0	0
Darling Downs	3.5B+1.5L	200	1999	DR	111	117	43	6	0	156	10	0	0	0	0
Darling Downs	4.0B+2.0C	200	1998	DNR	1	1	4	4	0	4	7	0	2	2	0
Far North	3.0B	250	2007	WNR	11	11	4	1	0	16	0	0	0	0	0
Far North	3.0B+2.0L	200	2002	WNR	2	2	0	0	0	2	0	0	0	0	0
Far North	3.0B+2.0L	250	2001	WNR	5	5	1	1	0	7	0	0	0	0	0
Far North	3.0B+2.0L	250	2006	WNR	40	45	3	0	0	43	5	0	0	0	0
Far North	3.0B+2.0L	250	2008	N/A	3	3	4	2	0	10	0	0	0	1	0
Far North	3.0B+2.0L	250	2008	WNR	38	38	3	0	0	41	0	0	0	3	0
Far North	3.0B+2.0L	250	2010	WNR	17	17	0	1	0	18	0	0	0	1	0
Far North	3.0B+2.0L	250	2011	WNR	16	16	1	0	0	17	0	0	0	0	0
Far North	3.0B+2.0L	250	2010	WNR	13	13	0	0	0	13	0	0	0	0	4
Far North	3.5B+1.5C	250	2013	WNR	69	71	5	3	0	77	2	0	0	0	0
Far North	3.5B+1.5C	300	2006	WNR	2	2	1	0	0	3	0	0	0	0	0
Far North	3.5B	170	2003	WNR	4	5	0	0	0	4	1	0	0	0	0
Far North	3.5B+1.5C	250	2013	WNR	7	8	10	11	0	26	3	0	0	0	0
Far North	3.5B+1.5C	250	2013	WNR	3	4	1	1	0	5	1	0	0	0	0
Far North	3.5B+2.0L	170	2001	WNR	29	29	1	0	0	30	0	0	0	0	0
Far North	3.5B+2.0L	200	2005	WNR	0	0	8	3	0	8	3	0	0	3	0
Far North	3.5B+2.0L	200	2011	WNR	6	6	0	0	0	6	0	0	0	0	2
Far North	3.5B+2.0L	230	2001	WNR	7	7	0	0	0	7	0	0	0	0	0

Region name	Stabiliser content (%)	Layer Thickness (mm)	Construction year	Environmental zone	Excellent roughness and rutting	Excellent roughness	Good roughness	Mediocre roughness	Poor roughness	Excellent rutting	Good rutting	Mediocre rutting	Poor rutting	Poor roughness deterioration	Poor rutting deterioration
Far North	3.5B+2.0L	250	2003	WNR	34	34	1	0	0	35	0	0	0	0	0
Far North	3.5B+2.0L	250	2006	WNR	5	5	2	0	0	7	0	0	0	0	0
Far North	3.5B+2.0L	250	2007	WNR	14	14	1	0	0	15	0	0	0	1	0
Far North	3.5B+2.0L	250	2008	WNR	96	104	10	2	0	108	8	1	0	4	3
Far North	3.5B+2.0L	250	2009	WNR	2	2	0	0	0	2	0	0	0	0	0
Far North	3.5B+2.0L	250	2010	WNR	36	37	0	0	0	36	1	0	0	1	0
Far North	3.5B+2.0L	250	2011	WNR	55	56	0	0	0	55	1	0	0	2	0
Far North	3.5B+2.0L	250	2012	WNR	15	15	1	0	0	16	0	0	0	0	0
Far North	3.5B+2.0L	280	2001	WNR	4	5	1	0	0	5	1	0	0	0	0
Far North	(blank)	120	2002	WNR	0	0	0	0	0	0	0	0	0	0	0
Far North	(blank)	150	2002	WNR	8	8	2	1	0	11	0	0	0	2	0
Far North	(blank)	180	2003	WNR	5	5	5	0	0	10	0	0	0	0	0
Far North	(blank)	200	2001	WNR	32	32	4	0	0	36	0	0	0	1	0
Far North	(blank)	200	2002	WNR	117	131	11	1	0	126	8	4	5	5	0
Far North	(blank)	200	2003	WNR	0	0	0	0	0	0	0	0	0	0	0
Far North	(blank)	210	2001	N/A	1	1	0	0	0	1	0	0	0	0	0
Far North	(blank)	220	2001	WNR	3	4	1	0	0	4	1	0	0	0	0
Far North	(blank)	220	2002	WNR	4	4	1	0	0	5	0	0	0	0	0
Far North	(blank)	223	2008	WNR	0	0	0	0	0	0	0	0	0	0	0
Far North	(blank)	230	2001	WNR	1	1	0	0	0	1	0	0	0	0	0
Far North	(blank)	240	2001	WNR	3	3	0	0	0	3	0	0	0	0	0
Far North	(blank)	250	2001	WNR	7	7	0	0	0	7	0	0	0	0	0
Far North	(blank)	250	2003	WNR	7	7	0	0	0	7	0	0	0	0	0

Region name	Stabiliser content (%)	Layer Thickness (mm)	Construction year	Environmental zone	Excellent roughness and rutting	Excellent roughness	Good roughness	Mediocre roughness	Poor roughness	Excellent rutting	Good rutting	Mediocre rutting	Poor rutting	Poor roughness deterioration	Poor rutting deterioration
Far North	(blank)	250	2004	WNR	17	18	0	0	0	17	1	0	0	0	0
Far North	(blank)	250	2007	WNR	0	0	0	0	0	0	0	0	0	0	0
Far North	(blank)	270	2001	WNR	7	7	0	0	0	7	0	0	0	0	0
Far North	(blank)	280	2001	WNR	9	10	0	0	0	9	1	0	0	0	0
Fitzroy	3.0B+1.0L	270	2005	DR	4	8	8	2	0	14	3	1	0	0	0
Fitzroy	3.0B+2.0L	250	2007	N/A	14	17	5	0	0	18	4	0	0	0	0
Fitzroy	3.0B+2.0L	250	2007	WNR	1	1	2	0	0	3	0	0	0	0	0
Fitzroy	3.0B+2.0L	250	2007	WR	7	7	1	0	0	8	0	0	0	1	0
Metropolitan	(blank)	210	2000	WR	13	17	0	0	0	13	4	0	0	0	0
North Coast	(blank)	250	1999	WNR	1	8	4	0	0	2	8	1	1	0	0
North Coast	(blank)	300	1999	WNR	3	5	13	0	0	3	3	1	11	0	3
North West	(blank)	150	1992	DNR	0	0	0	0	0	0	0	0	0	0	0
Northern	(blank)	200	2006	WNR	0	0	0	0	0	0	0	0	0	0	0
South Coast	3.0B+2.0L	300	2010	DNR	16	16	1	0	0	17	0	0	0	1	1
South Coast	3.5B+1.0L	250	2002	WNR	0	0	1	3	0	4	1	0	0	1	0
South Coast	3.5B+2.0L	200	2009	WNR	0	0	3	12	0	14	1	0	0	10	1
South Coast	3.5B+2.0L	250	2000	WR	1	2	2	1	0	3	1	1	0	0	0
South Coast	3.5B+2.0L	300	2000	WR	6	6	1	0	0	7	0	0	0	0	0
South Coast	3.5B+2.0L	300	2003	DNR	28	29	2	1	0	31	0	1	0	1	0
South Coast	3.5B+2.0L	300	2005	WNR	16	16	0	0	0	16	0	0	0	0	0
South Coast	4.5B+2.0L	200	2009	WNR	4	4	9	9	0	22	0	0	0	7	0
South Coast	(blank)	250	2013	WR	0	0	0	0	0	0	0	0	0	0	0
South West	(blank)	70	1994	DNR	0	0	0	0	0	0	0	0	0	0	0

Table A 3: Condition detail for CMB network selected for further study

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Total rutting (mm/year)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Construction year	Surfacing year	Annual maintenance cost ($)	Cumulative traffic (ESA)
10M	3	2	121.4	121.5	Northern	Townsville – Ingham	0.00	-0.24	51	-0.41	2008	2008		5.33E+06
10M	3	2	121.5	121.6	Northern	Townsville – Ingham	0.00	-0.44	62	-2.62	2008	2008		5.33E+06
856	3	2	4.4	4.5	Mackay/Whitsunday	Mackay-Bucasia Rd	5.70	0.10	54	0.46	2004	1997	35	1.57E+07
856	3	2	4.5	4.6	Mackay/Whitsunday	Mackay-Bucasia Rd	3.30	-0.85	47	1.23	2005	2005	71	1.08E+07
856	3	2	4.6	4.7	Mackay/Whitsunday	Mackay-Bucasia Rd	5.00	-0.20	42	1.05	2005	2005	71	1.08E+07
856	3	2	4.7	4.8	Mackay/Whitsunday	Mackay-Bucasia Rd	4.50	0.01	50	2.56	2005	2005	71	1.08E+07
856	3	2	4.8	4.9	Mackay/Whitsunday	Mackay-Bucasia Rd	3.00	-0.20	60	-0.74	2005	2005	71	1.08E+07
856	3	2	4.9	5	Mackay/Whitsunday	Mackay-Bucasia Rd	3.40	-0.28	40	1.25	2005	2005	71	1.08E+07
856	3	2	5	5.1	Mackay/Whitsunday	Mackay-Bucasia Rd	8.70	0.57	31	0.27	2005	2005	63	1.08E+07
856	3	2	5.1	5.2	Mackay/Whitsunday	Mackay-Bucasia Rd	9.30	0.61	42	0.27	2005	2005	63	1.08E+07
856	3	2	5.2	5.3	Mackay/Whitsunday	Mackay-Bucasia Rd	3.40	-0.24	41	-0.17	2005	2005	63	1.08E+07
856	3	2	5.3	5.4	Mackay/Whitsunday	Mackay-Bucasia Rd	3.20	-0.31	44	0.58	2005	2005	63	1.08E+07
856	3	2	5.4	5.5	Mackay/Whitsunday	Mackay-Bucasia Rd	2.70	-0.40	52	0.44	2005	2005	63	1.08E+07
856	3	2	5.5	5.6	Mackay/Whitsunday	Mackay-Bucasia Rd	5.10	-0.15	33	-0.51	2005	2005	63	1.08E+07
856	3	2	5.6	5.7	Mackay/Whitsunday	Mackay-Bucasia Rd	5.10	-0.02	52	1.94	2005	2005	63	1.08E+07
856	3	2	5.7	5.8	Mackay/Whitsunday	Mackay-Bucasia Rd	3.60	-0.19	65	1.47	2005	2005	63	1.08E+07
856	3	2	5.8	5.9	Mackay/Whitsunday	Mackay-Bucasia Rd	2.00	-0.23	54	1.76	2005	2005	63	1.08E+07
856	3	2	5.9	6	Mackay/Whitsunday	Mackay-Bucasia Rd	2.40	-0.13	58	2.44	2005	2005	63	1.08E+07
856	3	2	6	6.1	Mackay/Whitsunday	Mackay-Bucasia Rd	2.10	-0.16	51	1.73	2005	2005	38	1.08E+07
856	3	2	6.1	6.2	Mackay/Whitsunday	Mackay-Bucasia Rd	1.80	-0.36	38	0.53	2005	2005	38	1.08E+07
856	3	2	6.2	6.3	Mackay/Whitsunday	Mackay-Bucasia Rd	3.00	-0.41	60	1.20	2005	2005	38	1.08E+07

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Total rutting (mm/year)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Construction year	Surfacing year	Annual maintenance cost ($)	Cumulative traffic (ESA)
10N	1	1	18.4	18.5	Far North	Ingham – Innisfail	2.50	-1.20	66	-3.12	2010	2010	3	1.70E+06
10N	1	1	18.5	18.6	Far North	Ingham – Innisfail	2.10	-5.05	54	-20.88	2010	2010	3	1.70E+06
10N	1	1	18.6	18.7	Far North	Ingham – Innisfail	1.80	-4.80	46	-52.08	2010	2010	3	1.70E+06
10N	1	1	18.7	18.8	Far North	Ingham – Innisfail	1.80	-6.20	47	-53.52	2010	2010	3	1.70E+06
10N	1	1	18.8	18.9	Far North	Ingham – Innisfail	1.50	-2.70	33	-90.48	2010	2010	3	1.70E+06
10N	1	1	18.9	19	Far North	Ingham – Innisfail	1.60	-2.30	35	-91.92	2010	2010	3	1.70E+06
10N	1	1	19	19.1	Far North	Ingham – Innisfail	1.40	-3.45	48	-64.56	2010	2010	48	1.70E+06
10N	1	1	19.1	19.2	Far North	Ingham – Innisfail	5.00	-1.20	67	-7.68	2010	2010	48	1.70E+06
10N	1	1	19.2	19.3	Far North	Ingham – Innisfail	6.20	-0.55	47	2.40	2010	2010	48	1.70E+06
10N	1	1	19.3	19.4	Far North	Ingham – Innisfail	7.60	-0.60	29	-8.40	2010	2010	48	1.70E+06
10N	1	1	19.4	19.5	Far North	Ingham – Innisfail	8.70	-0.05	41	1.20	2010	2010	48	1.70E+06
10N	1	1	19.5	19.6	Far North	Ingham – Innisfail	8.70	-0.25	34	-1.92	2010	2010	48	1.70E+06
10N	1	1	19.6	19.7	Far North	Ingham – Innisfail	8.70	0.15	31	-2.64	2010	2010	48	1.70E+06
10N	1	1	19.7	19.8	Far North	Ingham – Innisfail	10.50	0.15	34	0.96	2010	2010	48	1.70E+06
10N	1	1	19.8	19.9	Far North	Ingham – Innisfail	8.50	-0.55	44	2.64	2010	2010	48	1.70E+06
10N	1	1	19.9	20	Far North	Ingham – Innisfail	9.60	0.15	34	0.00	2010	2010	48	1.70E+06
10N	1	1	20	20.1	Far North	Ingham – Innisfail	9.20	-0.20	36	0.48	2010	2010	3	1.70E+06
10N	1	1	20.1	20.2	Far North	Ingham – Innisfail	10.10	0.25	33	0.96	2010	2010	3	1.70E+06
10N	1	1	20.2	20.3	Far North	Ingham – Innisfail	9.50	-0.65	37	1.68	2010	2010	3	1.70E+06
10N	1	1	20.3	20.4	Far North	Ingham – Innisfail	6.10	-2.16	62	2.26	2008	2006	2	4.05E+06
10N	1	1	20.4	20.5	Far North	Ingham – Innisfail	9.00	-1.34	45	-1.65	2006	2000	132	6.95E+06
10N	1	1	20.5	20.6	Far North	Ingham – Innisfail	4.00	-0.80	36	-16.08	2010	2010	3	1.70E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Total rutting (mm/year)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Construction year	Surfacing year	Annual maintenance cost ($)	Cumulative traffic (ESA)
10N	1	1	20.6	20.7	Far North	Ingham – Innisfail	3.60	-3.30	36	-12.00	2010	2010	3	1.70E+06
10N	1	1	20.7	20.8	Far North	Ingham – Innisfail	4.20	-0.85	35	-9.12	2010	2010	3	1.70E+06
10N	1	1	20.8	20.9	Far North	Ingham – Innisfail	3.90	-0.50	40	-0.72	2010	2010	3	1.70E+06
10N	1	1	20.9	21	Far North	Ingham – Innisfail	4.10	-0.60	45	-3.36	2010	2010	3	1.70E+06
10M	2	1	8.3	8.4	Northern	Townsville – Ingham	8.60	0.06	38	1.08	2011	2002	54	2.23E+07
10M	2	1	8.4	8.5	Northern	Townsville – Ingham	3.30	-0.37	30	-0.22	2011	2004	67	1.94E+07
10M	2	1	8.5	8.6	Northern	Townsville – Ingham	1.70	-0.41	46	-0.75	2011	2004	67	1.94E+07
10M	2	1	8.6	8.7	Northern	Townsville – Ingham	2.20	0.45	34	-3.12	2011	2010	119	5.93E+06
10M	1	1	20.302	20.4	Northern	Townsville – Ingham	1.50	-0.44	31	-3.07	2008	2008	436	6.83E+06
10M	1	1	20.4	20.5	Northern	Townsville – Ingham	2.30	0.38	41	3.36	2009	2009	436	4.93E+06
854	1	1	0.9	1	Mackay/Whitsunday	Mt Ossa-Seaforth Rd	9.00	0.22	90	0.52	2002	2001	315	9.17E+05
854	1	1	1	1.1	Mackay/Whitsunday	Mt Ossa-Seaforth Rd	5.10	0.11	52	-0.18	2002	2001	219	9.17E+05
854	1	1	1.1	1.2	Mackay/Whitsunday	Mt Ossa-Seaforth Rd	12.20	0.90	77	1.27	2002	2001	219	9.17E+05
854	1	1	1.2	1.3	Mackay/Whitsunday	Mt Ossa-Seaforth Rd	10.40	0.52	86	1.49	2002	2001	219	9.17E+05
854	1	1	1.3	1.4	Mackay/Whitsunday	Mt Ossa-Seaforth Rd	4.30	-0.06	72	0.67	2001	2001	219	9.17E+05
854	1	1	1.4	1.5	Mackay/Whitsunday	Mt Ossa-Seaforth Rd	4.20	-0.14	83	0.98	2002	2001	219	9.17E+05
854	1	1	1.5	1.6	Mackay/Whitsunday	Mt Ossa-Seaforth Rd	9.10	0.40	73	1.33	2002	2001	219	9.17E+05
854	1	1	1.6	1.7	Mackay/Whitsunday	Mt Ossa-Seaforth Rd	8.90	-0.46	86	-0.72	2002	1991	147	1.39E+06
530	1	1	1.197	1.2	Mackay/Whitsunday	Glenella Connection Rd	3.90	-0.30	73	-5.66	2008	2008	12	4.53E+06
530	1	1	1.2	1.3	Mackay/Whitsunday	Glenella Connection Rd	2.40	0.05	45	-0.82	2008	2008	12	4.53E+06
530	1	1	1.3	1.4	Mackay/Whitsunday	Glenella Connection Rd	4.40	0.86	43	1.46	2008	2008	12	4.53E+06
530	1	1	1.4	1.5	Mackay/Whitsunday	Glenella Connection Rd	3.80	0.65	56	2.66	2008	2008	12	4.53E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Total rutting (mm/year)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Construction year	Surfacing year	Annual maintenance cost ($)	Cumulative traffic (ESA)
530	1	1	1.5	1.6	Mackay/Whitsunday	Glenella Connection Rd	2.60	0.20	40	0.55	2008	2008	12	4.53E+06
530	1	1	1.6	1.7	Mackay/Whitsunday	Glenella Connection Rd	2.80	0.27	40	0.46	2008	2008	12	4.53E+06
530	1	1	1.7	1.8	Mackay/Whitsunday	Glenella Connection Rd	2.30	0.22	54	1.75	2008	2008	12	4.53E+06
530	1	1	1.8	1.9	Mackay/Whitsunday	Glenella Connection Rd	3.80	0.38	66	0.91	2008	2008	12	4.53E+06
530	1	1	1.9	2	Mackay/Whitsunday	Glenella Connection Rd	4.20	0.34	52	1.51	2008	2008	12	4.53E+06
530	1	1	2	2.1	Mackay/Whitsunday	Glenella Connection Rd	4.00	0.39	45	0.41	2008	2008	23	4.53E+06
530	1	1	2.1	2.2	Mackay/Whitsunday	Glenella Connection Rd	4.10	0.30	49	1.01	2008	2008	23	4.53E+06
530	1	1	2.2	2.3	Mackay/Whitsunday	Glenella Connection Rd	3.70	0.30	54	1.68	2008	2008	23	4.53E+06
530	1	1	2.3	2.4	Mackay/Whitsunday	Glenella Connection Rd	3.30	0.23	62	0.60	2008	2008	23	4.53E+06
530	1	1	2.4	2.5	Mackay/Whitsunday	Glenella Connection Rd	3.30	-0.54	36	0.10	2008	2008	23	4.53E+06
530	1	1	2.5	2.6	Mackay/Whitsunday	Glenella Connection Rd	2.20	-0.23	43	1.13	2008	2008	23	4.53E+06
530	1	1	2.6	2.7	Mackay/Whitsunday	Glenella Connection Rd	3.00	-0.03	60	2.06	2008	2008	23	4.53E+06
530	1	1	2.7	2.8	Mackay/Whitsunday	Glenella Connection Rd	2.60	-0.16	53	0.62	2008	2008	23	4.53E+06
530	1	1	2.8	2.9	Mackay/Whitsunday	Glenella Connection Rd	3.60	0.06	45	1.73	2008	2008	23	4.53E+06
530	1	1	2.9	3	Mackay/Whitsunday	Glenella Connection Rd	5.40	-0.06	61	0.60	2009	2008	23	4.53E+06
530	1	1	3	3.1	Mackay/Whitsunday	Glenella Connection Rd	9.20	-0.54	83	2.30	2009	2008	151	4.53E+06
10N	1	1	17.7	17.8	Far North	Ingham – Innisfail	2.70	-2.11	80	5.95	2009	2006	186	4.05E+06
10N	1	1	17.8	17.9	Far North	Ingham – Innisfail	10.40	0.00	77	0.00	2010	2010	3	1.70E+06
10N	1	1	17.9	18	Far North	Ingham – Innisfail	7.30	-0.65	105	0.00	2010	2010	3	1.70E+06
10N	1	1	18	18.1	Far North	Ingham – Innisfail	2.60	-2.25	51	-25.92	2010	2010	3	1.70E+06
33B	1	1	69.2	69.3	Mackay/Whitsunday	Nebo – Mackay	6.50	0.22	66	0.15	2004	2003	72	7.00E+06
33B	1	1	69.3	69.4	Mackay/Whitsunday	Nebo – Mackay	6.00	0.10	63	-0.09	2004	2003	72	7.00E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Total rutting (mm/year)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Construction year	Surfacing year	Annual maintenance cost ($)	Cumulative traffic (ESA)
33B	1	1	69.4	69.5	Mackay/Whitsunday	Nebo – Mackay	5.60	0.05	62	0.32	2004	2003	72	7.00E+06
33B	1	1	69.5	69.6	Mackay/Whitsunday	Nebo – Mackay	9.20	0.46	99	4.54	2004	2003	72	7.00E+06
33B	1	1	69.6	69.7	Mackay/Whitsunday	Nebo – Mackay	7.00	0.46	73	2.49	2004	2003	72	7.00E+06
33B	1	1	69.7	69.8	Mackay/Whitsunday	Nebo – Mackay	16.20	1.36	68	1.89	2004	2003	72	7.00E+06
33B	1	1	69.8	69.9	Mackay/Whitsunday	Nebo – Mackay	14.40	1.06	65	1.72	2004	2003	72	7.00E+06
33B	1	1	69.9	70	Mackay/Whitsunday	Nebo – Mackay	6.20	0.08	52	-0.33	2004	2003	72	7.00E+06
33B	1	1	70	70.1	Mackay/Whitsunday	Nebo – Mackay	4.50	0.00	49	-0.15	2004	2003	51	7.00E+06
33B	1	1	70.1	70.2	Mackay/Whitsunday	Nebo – Mackay	5.30	0.05	45	-0.34	2004	2003	51	7.00E+06
33B	1	1	70.2	70.3	Mackay/Whitsunday	Nebo – Mackay	4.90	-0.03	44	-0.19	2004	2003	51	7.00E+06
33B	1	1	70.3	70.4	Mackay/Whitsunday	Nebo – Mackay	6.50	0.20	38	-0.60	2004	2003	51	7.00E+06
33B	1	1	70.4	70.5	Mackay/Whitsunday	Nebo – Mackay	5.90	0.16	37	0.02	2004	2003	51	7.00E+06
33B	1	1	70.5	70.6	Mackay/Whitsunday	Nebo – Mackay	4.10	-0.04	37	0.39	2004	2003	51	7.00E+06
33B	1	1	70.6	70.7	Mackay/Whitsunday	Nebo – Mackay	5.00	0.00	53	1.36	2004	2003	51	7.00E+06
33B	1	1	70.7	70.8	Mackay/Whitsunday	Nebo – Mackay	7.20	0.22	50	-0.36	2004	2003	51	7.00E+06
33B	1	1	70.8	70.9	Mackay/Whitsunday	Nebo – Mackay	8.90	0.56	46	0.92	2004	2003	51	7.00E+06
33B	1	1	70.9	71	Mackay/Whitsunday	Nebo – Mackay	8.60	0.51	50	0.72	2004	2003	51	7.00E+06
33B	1	1	71	71.1	Mackay/Whitsunday	Nebo – Mackay	7.50	0.56	50	0.90	2004	2003	51	7.00E+06
33B	1	1	71.1	71.2	Mackay/Whitsunday	Nebo – Mackay	5.00	0.27	44	0.64	2004	2003	51	7.00E+06
33B	1	1	71.2	71.3	Mackay/Whitsunday	Nebo – Mackay	5.00	-0.02	68	3.33	2004	2003	51	7.00E+06
33B	1	1	71.3	71.4	Mackay/Whitsunday	Nebo – Mackay	5.30	0.02	48	-0.30	2004	2003	51	7.00E+06
33B	1	1	71.4	71.5	Mackay/Whitsunday	Nebo – Mackay	5.30	0.08	43	0.33	2004	2003	51	7.00E+06
33B	1	1	71.5	71.6	Mackay/Whitsunday	Nebo – Mackay	4.20	-0.11	45	0.31	2004	2003	51	7.00E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Total rutting (mm/year)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Construction year	Surfacing year	Annual maintenance cost ($)	Cumulative traffic (ESA)
33B	1	1	71.6	71.7	Mackay/Whitsunday	Nebo – Mackay	4.20	-0.21	47	0.10	2004	2003	51	7.00E+06
33B	1	1	71.7	71.8	Mackay/Whitsunday	Nebo – Mackay	5.10	0.01	43	0.16	2004	2003	51	7.00E+06
33B	1	1	71.8	71.9	Mackay/Whitsunday	Nebo – Mackay	5.90	0.08	46	0.73	2004	2003	51	7.00E+06
33B	1	1	71.9	72	Mackay/Whitsunday	Nebo – Mackay	7.00	0.25	51	0.45	2004	2003	51	7.00E+06
33B	1	1	72	72.1	Mackay/Whitsunday	Nebo – Mackay	8.50	0.35	41	-0.34	2004	2003	157	7.00E+06
33B	1	1	72.1	72.2	Mackay/Whitsunday	Nebo – Mackay	7.60	0.26	43	1.23	2004	2003	157	7.00E+06
33B	1	1	72.2	72.3	Mackay/Whitsunday	Nebo – Mackay	6.60	0.12	43	0.64	2004	2003	157	7.00E+06
33B	1	1	72.3	72.4	Mackay/Whitsunday	Nebo – Mackay	6.40	0.23	49	0.88	2004	2003	157	7.00E+06
33B	1	1	72.4	72.5	Mackay/Whitsunday	Nebo – Mackay	4.80	0.03	43	1.34	2004	2003	157	7.00E+06
33B	1	1	72.5	72.6	Mackay/Whitsunday	Nebo – Mackay	8.20	0.14	62	3.17	2006	2003	157	7.00E+06
33B	1	1	72.6	72.7	Mackay/Whitsunday	Nebo – Mackay	3.00	-0.34	41	-0.20	2006	2003	157	7.00E+06
33B	1	1	72.7	72.8	Mackay/Whitsunday	Nebo – Mackay	1.90	-0.48	39	-0.87	2005	2003	157	7.00E+06
33B	1	1	72.8	72.9	Mackay/Whitsunday	Nebo – Mackay	5.40	0.14	33	0.55	2004	2003	157	7.00E+06
33B	1	1	72.9	73	Mackay/Whitsunday	Nebo – Mackay	8.30	0.39	45	1.26	2004	2003	157	7.00E+06
33B	1	1	73	73.1	Mackay/Whitsunday	Nebo – Mackay	5.30	0.04	34	-0.54	2004	2003	47	7.00E+06
33B	1	1	73.1	73.2	Mackay/Whitsunday	Nebo – Mackay	7.00	0.17	37	0.34	2004	2003	47	7.00E+06
33B	1	1	73.2	73.3	Mackay/Whitsunday	Nebo – Mackay	5.50	-0.03	39	0.43	2004	2003	47	7.00E+06
33B	1	1	73.3	73.4	Mackay/Whitsunday	Nebo – Mackay	10.30	0.47	42	0.28	2004	2003	47	7.00E+06
33B	1	1	73.4	73.5	Mackay/Whitsunday	Nebo – Mackay	11.80	0.78	62	2.46	2004	2003	47	7.00E+06
33B	1	1	73.5	73.6	Mackay/Whitsunday	Nebo – Mackay	6.60	0.19	36	-0.49	2004	2003	47	7.00E+06
33B	1	1	73.6	73.7	Mackay/Whitsunday	Nebo – Mackay	12.30	0.69	36	-0.13	2004	2003	47	7.00E+06
33B	1	1	73.7	73.8	Mackay/Whitsunday	Nebo – Mackay	12.30	0.80	38	0.73	2004	2003	47	7.00E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Total rutting (mm/year)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Construction year	Surfacing year	Annual maintenance cost ($)	Cumulative traffic (ESA)
33B	1	1	73.8	73.9	Mackay/Whitsunday	Nebo – Mackay	12.30	0.95	44	1.22	2004	2003	47	7.00E+06
33B	1	1	73.9	74	Mackay/Whitsunday	Nebo – Mackay	7.60	0.32	35	-0.06	2004	2003	47	7.00E+06
33B	1	1	74	74.1	Mackay/Whitsunday	Nebo – Mackay	7.80	0.34	36	0.12	2004	2003	47	7.00E+06

Table A 4: Condition details for FBS network selected for further study

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Road name	Total rutting (mm)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Surfacing year	Construction year	Annual maintenance cost ($)	Cumulative traffic (ESA)
208	1	1	3	3.1	DNR	South Coast	Beenleigh Connection Rd	4.0	-0.044	78	-2.8318	1994	1994	760.91	1.70E+07
208	1	1	3.7	3.8	WR	South Coast	Beenleigh Connection Rd	5.7	0.206	74	-1.8206	1995	1995	760.91	1.70E+07
208	1	1	3.1	3.2	WR	South Coast	Beenleigh Connection Rd	4.0	0.189	47	0.6000	2000	2000	760.91	1.70E+07
208	1	1	3.2	3.3	WR	South Coast	Beenleigh Connection Rd	5.4	0.404	52	0.4035	2000	2000	760.91	1.70E+07
208	1	1	3.4	3.5	WR	South Coast	Beenleigh Connection Rd	7.8	0.537	81	1.4716	2000	2000	760.91	1.70E+07
208	1	1	3.5	3.6	WR	South Coast	Beenleigh Connection Rd	10.2	0.707	59	1.1011	2000	2000	760.91	1.70E+07
208	1	1	3.3	3.4	WR	South Coast	Beenleigh Connection Rd	17.5	1.461	73	2.2708	2000	2000	760.91	1.70E+07
208	1	1	3.6	3.7	WR	South Coast	Beenleigh Connection Rd	6.2	0.290	70	1.4202	2000	2000	760.91	1.70E+07
1003	1	1	3.3	3.4	WNR	South Coast	Stapylton-Jacobs Well Rd	3.6	0.320	43	-2.7480	2009	2004	470.02	1.45E+07
1003	1	1	3.4	3.5	WNR	South Coast	Stapylton-Jacobs Well Rd	3.6	0.338	29	-2.3600	2009	2004	470.02	1.45E+07
1003	1	1	3.5	3.6	WNR	South Coast	Stapylton-Jacobs Well Rd	4.2	0.328	21	-3.0360	2009	2004	470.02	1.45E+07
1003	1	1	3.6	3.7	WNR	South Coast	Stapylton-Jacobs Well Rd	3.0	0.175	27	-3.8120	2009	2004	470.02	1.45E+07
1003	1	1	3.7	3.8	WNR	South Coast	Stapylton-Jacobs Well Rd	3.5	-0.113	41	-2.5800	2009	2004	470.02	1.45E+07
1003	1	1	3.8	3.9	WNR	South Coast	Stapylton-Jacobs Well Rd	3.9	0.113	31	-8.5760	2009	2004	470.02	1.45E+07
1003	1	1	3.9	4	WNR	South Coast	Stapylton-Jacobs Well Rd	3.4	0.053	41	-4.9240	2009	2004	470.02	1.45E+07
1003	1	1	4	4.1	WNR	South Coast	Stapylton-Jacobs Well Rd	3.5	0.063	35	-5.0480	2009	2004	378.79	1.45E+07
1003	1	1	4.1	4.2	WNR	South Coast	Stapylton-Jacobs Well Rd	3.3	0.062	28	-5.2640	2009	2004	378.79	1.45E+07
1003	1	1	4.2	4.3	WNR	South Coast	Stapylton-Jacobs Well Rd	2.6	-0.550	33	-4.0880	2009	2004	378.79	1.45E+07
1003	1	1	4.3	4.4	WNR	South Coast	Stapylton-Jacobs Well Rd	3.5	-0.120	20	-5.0920	2009	2004	378.79	1.45E+07
1003	1	1	4.4	4.5	WNR	South Coast	Stapylton-Jacobs Well Rd	3.2	0.087	29	-5.9400	2009	2004	378.79	1.45E+07
1003	1	1	4.5	4.6	WNR	South Coast	Stapylton-Jacobs Well Rd	2.7	0.245	49	-0.8800	2006	2004	378.79	1.45E+07

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Road name	Total rutting (mm)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Surfacing year	Construction year	Annual maintenance cost ($)	Cumulative traffic (ESA)
1003	1	1	4.6	4.7	WNR	South Coast	Stapylton-Jacobs Well Rd	2.2	-0.025	40	-4.1160	2008	2004	378.79	8.47E+06
1003	1	1	4.7	4.8	WNR	South Coast	Stapylton-Jacobs Well Rd	3.7	0.127	29	-5.7120	2009	2004	378.79	8.47E+06
1003	1	1	4.8	4.9	WNR	South Coast	Stapylton-Jacobs Well Rd	3.0	-0.067	29	-5.4080	2009	2004	378.79	8.47E+06
16B	1	1	122	122.1	DR	Fitzroy	Capricorn Hwy	4.5	0.149	67	1.2286	2006	2005	287.31	7.03E+06
16B	1	1	122.1	122.2	DR	Fitzroy	Capricorn Hwy	4.6	-0.032	37	0.5114	2006	2005	287.31	7.03E+06
16B	1	1	122.2	122.3	DR	Fitzroy	Capricorn Hwy	10.3	0.945	48	-0.6971	2006	2005	287.31	7.03E+06
16B	1	1	122.3	122.4	DR	Fitzroy	Capricorn Hwy	5.7	0.129	56	1.2829	2006	2005	287.31	7.03E+06
16B	1	1	122.4	122.5	DR	Fitzroy	Capricorn Hwy	6.0	0.283	51	0.8057	2006	2005	287.31	7.03E+06
16B	1	1	122.5	122.6	DR	Fitzroy	Capricorn Hwy	3.2	0.142	63	1.2400	2006	2005	287.31	7.03E+06
16B	1	1	122.6	122.7	DR	Fitzroy	Capricorn Hwy	3.7	-0.023	62	-10.0257	2006	2005	287.31	7.03E+06
16B	1	1	122.7	122.8	DR	Fitzroy	Capricorn Hwy	2.8	-0.048	44	0.3971	2006	2005	287.31	7.03E+06
16B	1	1	122.8	122.9	DR	Fitzroy	Capricorn Hwy	2.9	0.037	62	0.5743	2006	2005	287.31	7.03E+06
17C	1	1	104.2	104.3	DNR	Darling Downs	Cunningham Hwy	7.4	0.250	48	-0.9200	2004	1995	445.70	9.03E+06
17C	1	1	104.3	104.4	DNR	Darling Downs	Cunningham Hwy	13.1	0.614	86	1.7147	2005	1997	445.70	9.03E+06
17C	1	1	104.4	104.5	DNR	Darling Downs	Cunningham Hwy	25.6	1.296	188	8.7176	2005	1997	445.70	9.03E+06
17C	1	1	104.5	104.6	DNR	Darling Downs	Cunningham Hwy	12.0	0.460	97	2.4476	2005	1997	445.70	9.03E+06
17C	1	1	104.6	104.7	DNR	Darling Downs	Cunningham Hwy	10.8	0.256	79	1.2524	2005	1997	445.70	9.03E+06
17C	1	1	104.7	104.8	DNR	Darling Downs	Cunningham Hwy	10.4	0.288	105	3.1676	2005	1997	445.70	9.03E+06
17C	1	1	104.8	104.9	DNR	Darling Downs	Cunningham Hwy	10.1	0.256	97	2.2694	2005	1997	445.70	9.03E+06
17C	1	1	104.9	105	DNR	Darling Downs	Cunningham Hwy	10.6	0.197	66	-0.4318	2005	1997	445.70	9.03E+06
17C	1	1	105	105.1	DNR	Darling Downs	Cunningham Hwy	9.0	-0.161	62	0.2276	2005	1997	379.29	9.03E+06
17C	1	1	105.1	105.2	DNR	Darling Downs	Cunningham Hwy	12.1	0.353	94	1.9065	2005	1997	379.29	9.03E+06

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17C	1	1	105.2	105.3	DNR	Darling Downs	Cunningham Hwy	9.0	0.210	71	0.2359	2005	1997	379.29	9.03E+06
17C	1	1	105.3	105.4	DNR	Darling Downs	Cunningham Hwy	7.4	0.115	110	1.9094	2005	1997	379.29	9.03E+06
17C	1	1	105.4	105.5	DNR	Darling Downs	Cunningham Hwy	24.7	1.058	106	1.6347	2005	1997	379.29	9.03E+06
17C	1	1	105.5	105.6	DNR	Darling Downs	Cunningham Hwy	3.8	-0.179	56	-0.4259	2007	1997	379.29	9.03E+06
17C	1	1	105.6	105.7	DNR	Darling Downs	Cunningham Hwy	6.2	0.060	67	0.2218	2007	1977	379.29	9.03E+06
2020	1	1	8.8	8.9	WNR	South Coast	Beechmont Rd	0.0	0.300	85	3.3840	2009	2009	643.01	7.91E+05
2020	1	1	8.9	9	WNR	South Coast	Beechmont Rd	0.0	2.880	78	3.9600	2009	2009	643.01	7.91E+05
2020	1	1	9	9.1	WNR	South Coast	Beechmont Rd	0.0	-0.780	81	2.7360	2009	2009	560.16	7.91E+05
2020	1	1	9.1	9.2	WNR	South Coast	Beechmont Rd	7.8	1.160	70	0.7920	2009	2009	560.16	7.91E+05
2020	1	1	9.2	9.3	WNR	South Coast	Beechmont Rd	8.1	-0.480	78	4.9680	2009	2009	560.16	7.91E+05
2020	1	1	9.3	9.4	WNR	South Coast	Beechmont Rd	8.2	0.220	92	4.9440	2009	2009	560.16	7.91E+05
2020	1	1	9.4	9.5	WNR	South Coast	Beechmont Rd	7.3	-3.330	90	1.2240	2009	2009	560.16	7.91E+05
2020	1	1	9.5	9.6	WNR	South Coast	Beechmont Rd	10.4	-1.440	86	3.8880	2009	2009	560.16	7.91E+05
2020	1	1	9.6	9.7	WNR	South Coast	Beechmont Rd	9.4	-1.090	92	-0.1920	2009	2009	560.16	7.91E+05
2020	1	1	9.7	9.8	WNR	South Coast	Beechmont Rd	9.1	-2.270	96	4.8720	2009	2009	560.16	7.91E+05
2020	1	1	9.8	9.9	WNR	South Coast	Beechmont Rd	8.8	-0.250	91	5.4480	2009	2009	560.16	7.91E+05
2020	1	1	9.9	10	WNR	South Coast	Beechmont Rd	6.7	-1.230	82	2.5440	2009	2009	560.16	7.91E+05
2020	1	1	10	10.1	WNR	South Coast	Beechmont Rd	6.6	0.540	94	10.6800	2009	2009	458.66	7.91E+05
2020	1	1	10.1	10.2	WNR	South Coast	Beechmont Rd	6.4	-0.600	86	3.0240	2009	2009	458.66	7.91E+05
2020	1	1	10.2	10.3	WNR	South Coast	Beechmont Rd	8.0	0.570	97	9.4800	2009	2009	458.66	7.91E+05
22B	1	1	36.9	37	DR	Darling Downs	New England Hwy	7.9	0.281	51	0.1694	2011	1998	123.31	6.24E+06
22B	1	1	37	37.1	DR	Darling Downs	New England Hwy	7.9	0.350	46	0.2068	2011	1998	125.12	6.24E+06

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22B	1	1	37.1	37.2	DR	Darling Downs	New England Hwy	6.4	0.274	42	-0.1669	2011	1998	125.12	6.24E+06
22B	1	1	37.2	37.3	DR	Darling Downs	New England Hwy	4.7	0.234	48	0.8711	2011	1998	125.12	6.24E+06
22B	1	1	37.3	37.4	DR	Darling Downs	New England Hwy	5.4	0.338	64	1.4460	2011	1998	125.12	6.24E+06
22B	1	1	37.4	37.5	DR	Darling Downs	New England Hwy	10.0	0.513	51	0.4934	2011	1998	125.12	6.24E+06
22B	1	1	37.5	37.6	DR	Darling Downs	New England Hwy	8.0	0.469	52	0.8682	2011	1998	125.12	6.24E+06
22B	1	1	37.6	37.7	DR	Darling Downs	New England Hwy	5.4	0.269	57	0.0212	2011	1998	125.12	6.24E+06
22B	1	1	37.7	37.8	DR	Darling Downs	New England Hwy	4.7	0.073	51	-2.6188	2011	1998	125.12	6.24E+06
22B	1	1	37.8	37.9	DR	Darling Downs	New England Hwy	4.8	0.188	41	-0.4324	2011	1998	125.12	6.24E+06
22B	1	1	37.9	38	DR	Darling Downs	New England Hwy	4.9	0.178	44	0.0042	2011	1998	125.12	6.24E+06
22B	1	1	38	38.1	DR	Darling Downs	New England Hwy	4.1	0.156	43	-0.0826	2011	1998	125.75	6.24E+06
22B	1	1	38.1	38.2	DR	Darling Downs	New England Hwy	3.8	0.200	50	1.0147	2011	1998	125.75	6.24E+06
22B	1	1	38.2	38.3	DR	Darling Downs	New England Hwy	5.0	0.144	46	0.8545	2011	1998	125.75	6.24E+06
22B	1	1	38.3	38.4	DR	Darling Downs	New England Hwy	6.2	0.177	42	1.0789	2011	1998	125.75	6.24E+06
22B	1	1	38.4	38.5	DR	Darling Downs	New England Hwy	4.3	0.153	38	0.1249	2011	1998	125.75	6.24E+06
22B	1	1	38.5	38.6	DR	Darling Downs	New England Hwy	4.1	0.185	37	-0.2231	2011	1998	125.75	6.24E+06
22B	1	1	38.6	38.7	DR	Darling Downs	New England Hwy	4.8	0.175	39	-0.3321	2011	1998	125.75	6.24E+06
22B	1	1	38.7	38.8	DR	Darling Downs	New England Hwy	5.7	0.229	51	0.2368	2011	1998	125.75	6.24E+06
22B	1	1	38.8	38.9	DR	Darling Downs	New England Hwy	4.6	0.200	60	0.7147	2011	1998	125.75	6.24E+06
22B	1	1	38.9	39	DR	Darling Downs	New England Hwy	7.2	0.295	46	-0.0526	2011	1998	125.75	6.24E+06
22B	1	1	39	39.1	DR	Darling Downs	New England Hwy	10.1	0.510	52	0.2111	2011	1998	126.94	6.24E+06
22B	1	1	39.1	39.2	DR	Darling Downs	New England Hwy	8.3	0.394	48	0.4489	2011	1998	126.94	6.24E+06
22B	1	1	39.2	39.3	DR	Darling Downs	New England Hwy	5.4	0.262	51	0.2167	2011	1998	126.94	6.24E+06

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22B	1	1	39.3	39.4	DR	Darling Downs	New England Hwy	9.1	0.574	61	0.7133	2011	1998	126.94	6.24E+06
22B	1	1	39.4	39.5	DR	Darling Downs	New England Hwy	5.4	0.330	70	0.4415	2011	1998	126.94	6.24E+06
22B	1	1	39.5	39.6	DR	Darling Downs	New England Hwy	4.9	0.238	62	0.0762	2011	1998	126.94	6.24E+06
22B	1	1	39.6	39.7	DR	Darling Downs	New England Hwy	5.2	0.233	43	-0.1768	2011	1998	126.94	6.24E+06
22B	1	1	39.7	39.8	DR	Darling Downs	New England Hwy	4.7	0.182	58	0.6275	2011	1998	126.94	6.24E+06
22B	1	1	39.8	39.9	DR	Darling Downs	New England Hwy	4.0	0.192	53	-0.7828	2011	1998	126.94	6.24E+06
22B	1	1	39.9	40	DR	Darling Downs	New England Hwy	5.7	0.291	58	-0.3484	2011	1998	126.94	6.24E+06
22B	1	1	40	40.1	DR	Darling Downs	New England Hwy	5.4	0.246	74	-0.3356	2011	1998	155.58	6.24E+06
22B	1	1	40.1	40.2	DR	Darling Downs	New England Hwy	5.4	0.268	50	-0.3155	2011	1998	155.58	6.24E+06
22B	1	1	40.2	40.3	DR	Darling Downs	New England Hwy	12.6	0.838	45	0.0360	2011	1998	155.58	6.24E+06
22B	1	1	40.3	40.4	DR	Darling Downs	New England Hwy	7.2	0.470	63	0.6794	2011	1998	155.58	6.24E+06
22B	1	1	40.4	40.5	DR	Darling Downs	New England Hwy	4.4	0.187	58	-0.0547	2011	1998	155.58	6.24E+06
22B	1	1	40.5	40.6	DR	Darling Downs	New England Hwy	4.5	0.199	54	-0.3551	2011	1998	155.58	6.24E+06
22B	1	1	40.6	40.7	DR	Darling Downs	New England Hwy	5.1	0.251	69	0.4387	2011	1998	155.58	6.24E+06
22B	1	1	40.7	40.8	DR	Darling Downs	New England Hwy	4.5	0.241	54	-1.3338	2011	1998	155.58	6.24E+06
22B	1	1	40.8	40.9	DR	Darling Downs	New England Hwy	5.0	0.278	46	-0.9692	2011	1998	155.58	6.24E+06
22B	1	1	40.9	41	DR	Darling Downs	New England Hwy	6.9	0.339	61	-0.2188	2011	1998	155.58	6.24E+06
22B	1	1	41	41.1	DR	Darling Downs	New England Hwy	6.1	0.285	58	-0.4465	2011	1998	145.14	6.24E+06
22B	1	1	41.1	41.2	DR	Darling Downs	New England Hwy	4.8	0.195	42	-0.1419	2011	1998	145.14	6.24E+06
22B	1	1	41.2	41.3	DR	Darling Downs	New England Hwy	4.7	0.215	57	0.4638	2011	1998	145.14	6.24E+06
22B	1	1	41.3	41.4	DR	Darling Downs	New England Hwy	6.0	0.424	46	-0.4549	2011	1998	145.14	6.24E+06
22B	1	1	41.4	41.5	DR	Darling Downs	New England Hwy	6.5	0.429	47	-0.1062	2011	1998	145.14	6.24E+06

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22B	1	1	41.5	41.6	DR	Darling Downs	New England Hwy	4.0	0.152	55	0.4285	2011	1998	145.14	6.24E+06
22B	1	1	41.6	41.7	DR	Darling Downs	New England Hwy	3.7	0.176	47	-0.1246	2011	1998	145.14	6.24E+06
22B	1	1	41.7	41.8	DR	Darling Downs	New England Hwy	4.5	0.237	44	-0.0081	2011	1998	145.14	6.24E+06
22B	1	1	41.8	41.9	DR	Darling Downs	New England Hwy	4.4	0.226	58	0.5488	2011	1998	145.14	6.24E+06
22B	1	1	41.9	42	DR	Darling Downs	New England Hwy	4.4	0.223	49	-0.5862	2011	1998	145.14	6.24E+06
22B	1	1	42	42.1	DR	Darling Downs	New England Hwy	4.9	0.278	51	-0.4108	2011	1998	145.31	6.24E+06
22B	1	1	42.1	42.2	DR	Darling Downs	New England Hwy	9.2	0.607	62	0.1588	2011	1998	145.31	6.24E+06
22B	1	1	42.2	42.3	DR	Darling Downs	New England Hwy	6.2	0.400	50	-0.3974	2011	1998	145.31	6.24E+06
22B	1	1	42.3	42.4	DR	Darling Downs	New England Hwy	4.7	0.251	50	0.0364	2011	1998	145.31	6.24E+06
22B	1	1	42.4	42.5	DR	Darling Downs	New England Hwy	5.9	0.330	62	0.9025	2011	1998	145.31	6.24E+06
22B	1	1	42.5	42.6	DR	Darling Downs	New England Hwy	8.1	0.539	64	0.1768	2011	1998	145.31	6.24E+06
22B	1	1	42.6	42.7	DR	Darling Downs	New England Hwy	9.5	0.611	54	0.7811	2011	1998	145.31	6.24E+06
22B	1	1	42.7	42.8	DR	Darling Downs	New England Hwy	7.5	0.499	50	0.7394	2011	1998	145.31	6.24E+06
22B	1	1	42.8	42.9	DR	Darling Downs	New England Hwy	5.9	0.364	66	0.3300	2011	1998	145.31	6.24E+06
22B	1	1	42.9	43	DR	Darling Downs	New England Hwy	9.5	0.561	69	1.3496	2011	1998	145.31	6.24E+06
22B	1	1	43	43.1	DR	Darling Downs	New England Hwy	7.0	0.468	62	0.8788	2011	1998	128.05	6.24E+06
22B	1	1	43.1	43.2	DR	Darling Downs	New England Hwy	9.0	0.649	58	0.8968	2011	1998	128.05	6.24E+06
22B	1	1	43.2	43.3	DR	Darling Downs	New England Hwy	11.8	0.785	55	1.2593	2011	1998	128.05	6.24E+06
22B	1	1	43.3	43.4	DR	Darling Downs	New England Hwy	7.8	0.484	59	1.0542	2011	1998	128.05	6.24E+06
22B	1	1	43.4	43.5	DR	Darling Downs	New England Hwy	5.2	0.245	78	1.6782	2011	1998	128.05	6.24E+06
22B	1	1	43.5	43.6	DR	Darling Downs	New England Hwy	5.8	0.318	71	1.2279	2011	1998	128.05	6.24E+06
22B	1	1	43.6	43.7	DR	Darling Downs	New England Hwy	7.5	0.460	77	0.6568	2011	1998	128.05	6.24E+06

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22B	1	1	43.7	43.8	DR	Darling Downs	New England Hwy	5.6	0.302	57	-0.1366	2011	1998	128.05	6.52E+06
22B	1	1	43.8	43.9	DR	Darling Downs	New England Hwy	7.4	0.393	58	0.0558	2011	1998	128.05	6.52E+06
22B	1	1	43.9	44	DR	Darling Downs	New England Hwy	6.9	0.390	53	-0.6318	2011	1998	128.05	6.52E+06
22B	1	1	44	44.1	DR	Darling Downs	New England Hwy	5.2	0.278	51	0.0702	2011	1998	128.24	6.52E+06
22B	1	1	44.1	44.2	DR	Darling Downs	New England Hwy	5.2	0.266	69	0.8382	2011	1998	128.24	6.52E+06
22B	1	1	44.2	44.3	DR	Darling Downs	New England Hwy	4.3	0.228	62	1.1104	2011	1998	128.24	6.52E+06
22B	1	1	44.3	44.4	DR	Darling Downs	New England Hwy	4.7	0.250	51	0.4175	2011	1998	128.24	6.52E+06
22B	1	1	44.4	44.5	DR	Darling Downs	New England Hwy	4.3	0.224	47	-0.0769	2011	1998	128.24	6.52E+06
22B	1	1	44.5	44.6	DR	Darling Downs	New England Hwy	4.4	0.225	63	0.3759	2011	1998	128.24	6.52E+06
22B	1	1	44.6	44.7	DR	Darling Downs	New England Hwy	4.2	0.152	58	0.5114	2011	1998	128.24	6.52E+06
22B	1	1	44.7	44.8	DR	Darling Downs	New England Hwy	5.2	0.294	59	0.5280	2011	1998	128.24	6.52E+06
22B	1	1	44.8	44.9	DR	Darling Downs	New England Hwy	4.2	0.238	48	0.3233	2011	1998	128.24	6.52E+06
22B	1	1	44.9	45	DR	Darling Downs	New England Hwy	4.2	0.208	56	0.2908	2011	1998	128.24	6.52E+06
22B	1	1	45	45.1	DR	Darling Downs	New England Hwy	5.2	0.280	49	-0.2891	2011	1998	168.51	6.52E+06
22B	1	1	45.1	45.2	DR	Darling Downs	New England Hwy	8.7	0.556	45	0.0007	2011	1998	168.51	6.52E+06
22B	1	1	45.2	45.3	DR	Darling Downs	New England Hwy	6.6	0.394	47	-0.4394	2011	1998	168.51	6.52E+06
22B	1	1	45.3	45.4	DR	Darling Downs	New England Hwy	5.1	0.270	52	0.6145	2011	1998	168.51	6.52E+06
22B	1	1	45.4	45.5	DR	Darling Downs	New England Hwy	5.3	0.293	49	0.5104	2011	1998	168.51	6.52E+06
22B	1	1	45.5	45.6	DR	Darling Downs	New England Hwy	4.8	0.191	40	-0.0113	2011	1998	168.51	6.52E+06
22B	1	1	45.6	45.7	DR	Darling Downs	New England Hwy	4.7	0.222	52	0.3519	2011	1998	168.51	6.52E+06
22B	1	1	45.7	45.8	DR	Darling Downs	New England Hwy	4.1	0.211	55	0.0858	2011	1998	168.51	6.52E+06
22B	1	1	45.8	45.9	DR	Darling Downs	New England Hwy	4.3	0.223	45	0.4041	2011	1998	168.51	6.52E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Road name	Total rutting (mm)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Surfacing year	Construction year	Annual maintenance cost ($)	Cumulative traffic (ESA)
22B	1	1	45.9	46	DR	Darling Downs	New England Hwy	4.9	0.279	44	0.0776	2011	1998	168.51	6.52E+06
22B	1	1	46	46.1	DR	Darling Downs	New England Hwy	4.3	0.237	40	-0.2566	2011	1998	510.87	6.52E+06
22B	1	1	46.1	46.2	DR	Darling Downs	New England Hwy	4.4	0.248	38	0.4129	2011	1998	510.87	6.52E+06
22B	1	1	46.2	46.3	DR	Darling Downs	New England Hwy	9.9	0.608	38	0.2947	2011	1998	510.87	6.52E+06
22B	1	1	46.3	46.4	DR	Darling Downs	New England Hwy	6.8	0.413	56	0.0734	2011	1998	510.87	6.52E+06
22B	1	1	46.4	46.5	DR	Darling Downs	New England Hwy	4.7	0.258	61	-0.1899	2011	1998	510.87	6.52E+06
22B	1	1	46.5	46.6	DR	Darling Downs	New England Hwy	6.7	0.438	54	-0.0660	2011	1998	510.87	6.52E+06
22B	1	1	46.6	46.7	DR	Darling Downs	New England Hwy	5.9	0.371	57	-0.0169	2011	1998	510.87	6.52E+06
22B	1	1	46.7	46.8	DR	Darling Downs	New England Hwy	5.1	0.282	55	-0.1712	2011	1998	510.87	6.52E+06
22B	1	1	46.8	46.9	DR	Darling Downs	New England Hwy	4.3	0.160	68	0.5725	2011	1998	510.87	6.52E+06
22B	1	1	46.9	47	DR	Darling Downs	New England Hwy	4.0	0.158	50	-0.4560	2011	1998	510.87	6.52E+06
22B	1	1	47	47.1	DR	Darling Downs	New England Hwy	3.5	0.139	49	0.4260	2011	1998	549.27	6.52E+06
22B	1	1	47.1	47.2	DR	Darling Downs	New England Hwy	4.0	0.152	93	-0.0918	2011	1990	549.27	6.52E+06
22B	1	1	47.2	47.3	DR	Darling Downs	New England Hwy	5.4	0.244	69	1.5051	2011	1999	549.27	6.52E+06
22B	1	1	47.3	47.4	DR	Darling Downs	New England Hwy	8.9	0.572	39	-0.3703	2011	1999	549.27	6.52E+06
22B	1	1	47.4	47.5	DR	Darling Downs	New England Hwy	9.7	0.677	36	0.3326	2011	1999	549.27	6.52E+06
22B	1	1	47.5	47.6	DR	Darling Downs	New England Hwy	10.4	0.741	38	0.5537	2011	1999	549.27	6.52E+06
22B	1	1	47.6	47.7	DR	Darling Downs	New England Hwy	9.7	0.647	68	2.0074	2011	1999	549.27	6.52E+06
22B	1	1	47.7	47.8	DR	Darling Downs	New England Hwy	7.4	0.465	83	2.7692	2011	1998	549.27	6.52E+06
22B	1	1	47.8	47.9	DR	Darling Downs	New England Hwy	6.8	0.402	74	1.1306	2011	1990	549.27	6.52E+06
22B	1	1	47.9	48	DR	Darling Downs	New England Hwy	7.3	0.387	65	0.1324	2011	1998	549.27	6.52E+06
22B	1	1	48	48.1	DR	Darling Downs	New England Hwy	14.1	0.802	62	0.9081	2011	1998	947.42	6.52E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Road name	Total rutting (mm)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Surfacing year	Construction year	Annual maintenance cost ($)	Cumulative traffic (ESA)
22B	1	1	48.1	48.2	DR	Darling Downs	New England Hwy	9.2	0.458	46	0.1094	2011	1998	947.42	6.52E+06
22B	1	1	48.2	48.3	DR	Darling Downs	New England Hwy	5.8	0.271	56	-0.0568	2011	1998	947.42	6.52E+06
22B	1	1	48.3	48.4	DR	Darling Downs	New England Hwy	4.8	0.199	49	0.3925	2011	1998	947.42	6.52E+06
22B	1	1	48.4	48.5	DR	Darling Downs	New England Hwy	7.0	0.387	46	0.3882	2011	1998	947.42	6.52E+06
22B	1	1	48.5	48.6	DR	Darling Downs	New England Hwy	5.4	0.270	71	0.7539	2011	1998	947.42	6.52E+06
22B	1	1	48.6	48.7	DR	Darling Downs	New England Hwy	4.0	0.176	49	0.1814	2011	1998	947.42	6.52E+06
22B	1	1	48.7	48.8	DR	Darling Downs	New England Hwy	3.6	0.150	42	-0.0812	2011	1998	947.42	6.52E+06
22B	1	1	48.8	48.9	DR	Darling Downs	New England Hwy	4.3	0.180	60	0.8901	2011	1998	947.42	6.52E+06
22B	1	1	48.9	49	DR	Darling Downs	New England Hwy	5.6	0.257	51	-0.1429	2003	1990	947.42	6.52E+06
22B	1	1	52.7	52.8	DR	Darling Downs	New England Hwy	4.8	0.131	51	0.0100	2007	1987	641.45	6.52E+06
22B	1	1	52.8	52.9	DR	Darling Downs	New England Hwy	5.1	0.306	43	0.3833	2011	1998	641.45	6.52E+06
22B	1	1	52.9	53	DR	Darling Downs	New England Hwy	4.8	0.337	35	-0.0187	2011	1998	641.45	6.52E+06
22B	1	1	53	53.1	DR	Darling Downs	New England Hwy	4.9	0.321	49	0.3953	2011	1998	872.35	6.52E+06
22B	1	1	53.1	53.2	DR	Darling Downs	New England Hwy	5.1	0.326	64	0.3406	2011	1998	872.35	6.52E+06
22B	1	1	53.2	53.3	DR	Darling Downs	New England Hwy	4.6	0.194	63	0.7475	2011	1998	872.35	6.52E+06
22B	1	1	53.3	53.4	DR	Darling Downs	New England Hwy	10.0	0.835	67	1.4492	2011	1998	872.35	6.52E+06
22B	1	1	53.4	53.5	DR	Darling Downs	New England Hwy	7.7	0.523	70	1.3564	2011	1998	872.35	6.52E+06
22B	1	1	53.5	53.6	DR	Darling Downs	New England Hwy	5.6	0.321	60	0.7814	2011	1998	872.35	6.52E+06
22B	1	1	53.6	53.7	DR	Darling Downs	New England Hwy	7.9	0.481	52	0.4624	2011	1998	872.35	6.52E+06
22B	1	1	53.7	53.8	DR	Darling Downs	New England Hwy	6.5	0.584	55	-0.1546	2011	1998	872.35	6.52E+06
22B	1	1	53.8	53.9	DR	Darling Downs	New England Hwy	5.3	0.502	60	1.0525	2011	1998	872.35	6.52E+06
22B	1	1	53.9	54	DR	Darling Downs	New England Hwy	4.9	0.433	51	-0.0325	2011	1998	872.35	6.52E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Road name	Total rutting (mm)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Surfacing year	Construction year	Annual maintenance cost ($)	Cumulative traffic (ESA)
22B	1	1	54	54.1	DR	Darling Downs	New England Hwy	5.0	0.439	60	-0.1260	2011	1998	683.52	6.52E+06
22B	1	1	54.1	54.2	DR	Darling Downs	New England Hwy	4.9	0.426	51	0.0494	2011	1998	683.52	6.52E+06
22B	1	1	54.2	54.3	DR	Darling Downs	New England Hwy	6.1	0.530	46	-0.0618	2011	1998	683.52	6.52E+06
22B	1	1	54.3	54.4	DR	Darling Downs	New England Hwy	6.6	0.487	51	0.5262	2011	1998	683.52	6.52E+06
22B	1	1	54.4	54.5	DR	Darling Downs	New England Hwy	5.1	0.285	64	-0.3681	2011	1998	683.52	6.52E+06
22B	1	1	54.5	54.6	DR	Darling Downs	New England Hwy	6.7	0.438	68	0.4066	2011	1998	683.52	6.52E+06
22B	1	1	54.6	54.7	DR	Darling Downs	New England Hwy	9.8	0.842	88	2.7314	2011	1998	683.52	6.52E+06
22B	1	1	54.7	54.8	DR	Darling Downs	New England Hwy	11.5	0.785	89	1.8911	2011	1998	683.52	6.52E+06
22B	1	1	54.8	54.9	DR	Darling Downs	New England Hwy	9.3	0.673	63	-0.0014	2011	1998	683.52	6.52E+06
22B	1	1	54.9	55	DR	Darling Downs	New England Hwy	6.3	0.466	51	-0.4112	2011	1998	683.52	6.52E+06
25B	1	1	10.2	10.3	DNR	South Coast	Mount Lindesay Hwy	3.7	-0.759	26	-0.1498	2008	2003	364.69	4.00E+06
25B	1	1	10.3	10.4	DNR	South Coast	Mount Lindesay Hwy	3.4	-0.815	34	-0.0742	2008	2003	364.69	4.00E+06
25B	1	1	10.4	10.5	DNR	South Coast	Mount Lindesay Hwy	2.7	-0.839	37	0.4902	2008	2003	364.69	4.00E+06
25B	1	1	10.5	10.6	DNR	South Coast	Mount Lindesay Hwy	2.8	-0.804	30	-0.7447	2008	2003	364.69	4.00E+06
25B	1	1	10.6	10.7	DNR	South Coast	Mount Lindesay Hwy	2.7	-0.744	48	0.5091	2008	2003	364.69	4.00E+06
25B	1	1	10.7	10.8	DNR	South Coast	Mount Lindesay Hwy	3.0	-0.685	34	-0.0742	2008	2003	364.69	4.00E+06
25B	1	1	10.8	10.9	DNR	South Coast	Mount Lindesay Hwy	4.2	-0.578	46	0.6284	2008	2003	364.69	4.00E+06
25B	1	1	10.9	11	DNR	South Coast	Mount Lindesay Hwy	5.2	-0.536	36	0.6182	2008	2003	364.69	4.00E+06
25B	1	1	11	11.1	DNR	South Coast	Mount Lindesay Hwy	4.8	-0.522	32	0.2356	2008	2003	237.92	4.00E+06
25B	1	1	11.1	11.2	DNR	South Coast	Mount Lindesay Hwy	1.5	-0.928	24	0.1673	2008	2003	237.92	4.00E+06
25B	1	1	11.2	11.3	DNR	South Coast	Mount Lindesay Hwy	2.8	-0.767	26	0.1324	2008	2003	237.92	4.00E+06
25B	1	1	11.3	11.4	DNR	South Coast	Mount Lindesay Hwy	3.6	-0.563	29	0.4742	2008	2003	237.92	4.00E+06

Road ID	Carriageway #	Lane #	Start chainage (km)	End chainage (km)	Environmental zone	Region name	Road name	Total rutting (mm)	Rutting deterioration rate	Total roughness (counts/year)	Roughness deterioration rate	Surfacing year	Construction year	Annual maintenance cost ($)	Cumulative traffic (ESA)
25B	1	1	11.4	11.5	DNR	South Coast	Mount Lindesay Hwy	2.9	-0.659	25	-0.0567	2008	2003	237.92	4.00E+06
25B	1	1	11.5	11.6	DNR	South Coast	Mount Lindesay Hwy	2.5	-0.803	33	0.7098	2008	2003	237.92	4.00E+06
25B	1	1	11.6	11.7	DNR	South Coast	Mount Lindesay Hwy	2.7	-0.798	22	-0.7011	2008	2003	237.92	4.00E+06
25B	1	1	11.7	11.8	DNR	South Coast	Mount Lindesay Hwy	3.3	-0.775	37	0.3927	2008	2003	237.92	4.00E+06
25B	1	1	11.8	11.9	DNR	South Coast	Mount Lindesay Hwy	2.7	-0.788	24	-0.5411	2008	2003	237.92	4.00E+06
25B	1	1	11.9	12	DNR	South Coast	Mount Lindesay Hwy	3.3	-0.725	39	-0.3229	2008	2003	237.92	4.00E+06
25B	1	1	12	12.1	DNR	South Coast	Mount Lindesay Hwy	4.3	-0.585	35	-0.4611	2008	2003	492.13	4.00E+06
25B	1	1	12.1	12.2	DNR	South Coast	Mount Lindesay Hwy	2.8	-0.771	45	0.3069	2008	2003	492.13	4.00E+06
25B	1	1	12.2	12.3	DNR	South Coast	Mount Lindesay Hwy	3.5	-0.666	38	0.2953	2008	2003	492.13	4.00E+06
25B	1	1	12.3	12.4	DNR	South Coast	Mount Lindesay Hwy	3.2	-0.722	81	4.1949	2008	2003	492.13	4.00E+06
25B	1	1	12.4	12.5	DNR	South Coast	Mount Lindesay Hwy	3.3	-0.821	48	0.5076	2008	2003	492.13	4.00E+06
25B	1	1	12.5	12.6	DNR	South Coast	Mount Lindesay Hwy	3.8	-0.867	69	0.9251	2008	2003	492.13	4.00E+06
25B	1	1	12.6	12.7	DNR	South Coast	Mount Lindesay Hwy	6.2	-0.523	66	2.3767	2008	2003	492.13	4.00E+06
25B	1	1	12.7	12.8	DNR	South Coast	Mount Lindesay Hwy	7.2	-0.240	52	0.7084	2008	2003	492.13	4.00E+06
25B	1	1	12.8	12.9	DNR	South Coast	Mount Lindesay Hwy	9.8	0.114	60	1.5680	2008	2003	492.13	4.00E+06
25B	1	1	12.9	13	DNR	South Coast	Mount Lindesay Hwy	16.7	0.756	56	1.3527	2008	2003	492.13	4.00E+06
25B	1	1	13	13.1	DNR	South Coast	Mount Lindesay Hwy	4.4	-0.620	60	2.1542	2008	2003	349.59	4.00E+06
25B	1	1	13.1	13.2	DNR	South Coast	Mount Lindesay Hwy	3.8	-0.759	52	2.5760	2003	2003	349.59	4.00E+06
25B	1	1	13.2	13.3	DNR	South Coast	Mount Lindesay Hwy	4.0	-0.688	39	1.4938	2003	2003	349.59	4.00E+06
25B	1	1	13.3	13.4	DNR	South Coast	Mount Lindesay Hwy	3.7	-0.750	30	0.3171	2003	2003	349.59	4.00E+06

APPENDIX B FWD DATA SUMMARY

Table B 1: FWD deflection summary for PM-CMB pavement sections

Material type	Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection			
				D0 (µm)	D900 (µm)	Curvature (µm)	DR
PM-CMB	10L	36.1	36.2	253	72	58	0.68
PM-CMB	10L	36.2	36.3	261	75	42	0.77
PM-CMB	10L	36.3	36.4	319	84	55	0.76
PM-CMB	10L	36.4	36.5	264	69	45	0.76
PM-CMB	10L	36.5	36.6	224	74	39	0.75
PM-CMB	10L	36.6	36.7	237	79	39	0.77
PM-CMB	10L	36.7	36.8	260	85	36	0.80
PM-CMB	10L	36.8	36.9	173	64	25	0.82
PM-CMB	10L	36.9	37.0	188	66	30	0.78
PM-CMB	10L	37.0	37.1	200	72	25	0.81
PM-CMB	10L	37.1	37.2	210	78	32	0.78
PM-CMB	10L	37.2	37.3	213	78	33	0.78
PM-CMB	10L	37.3	37.4	242	85	36	0.77
PM-CMB	10L	37.4	37.5	190	69	28	0.80
PM-CMB	10L	37.5	37.6	200	76	27	0.82
PM-CMB	10L	37.6	37.7	157	70	21	0.84
PM-CMB	10L	37.7	37.8	533	141	96	0.76
PM-CMB	10M	118.7	118.8	177	70	23	0.87
PM-CMB	10M	118.8	118.9	105	52	9	0.86
PM-CMB	10M	118.9	119.0	191	51	37	0.75
PM-CMB	10M	119.0	119.1	362	62	80	0.68
PM-CMB	10M	119.1	119.2	258	47	60	0.69
PM-CMB	10M	119.2	119.3	333	62	71	0.72
PM-CMB	10M	119.3	119.4	309	57	71	0.66
PM-CMB	10M	119.4	119.5	210	58	37	0.70
PM-CMB	10M	119.5	119.6	178	51	31	0.70
PM-CMB	10M	119.6	119.7	228	66	40	0.76
PM-CMB	10M	119.7	119.8	378	83	88	0.68
PM-CMB	10N	19.1	19.2	275	37	76	0.61
PM-CMB	10N	19.2	19.3	561	69	117	0.63
PM-CMB	10N	19.3	19.4	843	113	165	0.69
PM-CMB	10N	19.4	19.5	709	80	149	0.67
PM-CMB	10N	19.5	19.6	700	67	145	0.67
PM-CMB	10N	19.6	19.7	689	72	146	0.66
PM-CMB	10N	19.7	19.8	741	68	146	0.67
PM-CMB	10N	19.8	19.9	758	72	156	0.67

Material type	Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection			
				D0 (µm)	D900 (µm)	Curvature (µm)	DR
PM-CMB	10N	19.9	20.0	771	81	166	0.66
PM-CMB	10N	20.0	20.1	811	78	168	0.67
PM-CMB	10N	20.1	20.2	811	79	169	0.67
PM-CMB	10N	20.2	20.3	773	79	159	0.67
PM-CMB	10N	20.3	20.4	750	76	146	0.68
PM-CMB	10N	20.4	20.5	346	108	43	0.83
PM-CMB	10N	20.5	20.6	347	97	55	0.79
PM-CMB	10N	20.6	20.7	370	94	58	0.79
PM-CMB	10N	20.7	20.8	264	87	37	0.83
PM-CMB	10N	20.8	20.9	199	59	28	0.81
PM-CMB	10N	20.9	21.0	208	59	36	0.78
PM-CMB	10N	21.0	21.1	175	66	22	0.83
PM-CMB	10N	21.1	21.2	148	54	22	0.81
PM-CMB	10N	21.2	21.3	179	60	25	0.81
PM-CMB	10N	21.3	21.4	171	61	24	0.82
PM-CMB	10N	21.4	21.5	171	66	17	0.85
PM-CMB	33B	69.2	69.3	563	70	124	0.68
PM-CMB	33B	69.3	69.4	280	67	50	0.79
PM-CMB	33B	69.4	69.5	344	59	69	0.74
PM-CMB	33B	69.5	69.6	401	86	57	0.82
PM-CMB	33B	69.6	69.7	552	126	99	0.76
PM-CMB	33B	69.7	69.8	538	88	91	0.78
PM-CMB	33B	69.8	69.9	727	107	171	0.69
PM-CMB	33B	69.9	70.0	463	67	103	0.75
PM-CMB	33B	70.0	70.1	245	45	43	0.76
PM-CMB	33B	70.1	70.2	386	75	73	0.78
PM-CMB	33B	70.2	70.3	213	59	35	0.81
PM-CMB	33B	70.3	70.4	525	71	108	0.73
PM-CMB	33B	70.4	70.5	523	78	100	0.74
PM-CMB	33B	70.5	70.6	458	58	81	0.75
PM-CMB	33B	70.6	70.7	438	61	78	0.75
PM-CMB	33B	70.7	70.8	280	50	50	0.77
PM-CMB	33B	70.8	70.9	486	103	95	0.77
PM-CMB	33B	70.9	71.0	596	132	116	0.76
PM-CMB	33B	71.0	71.1	413	83	82	0.77
PM-CMB	33B	71.1	71.2	239	76	35	0.80
PM-CMB	33B	71.2	71.3	290	64	50	0.78
PM-CMB	33B	71.3	71.4	324	76	55	0.79
PM-CMB	33B	71.4	71.5	680	113	149	0.69

Material type	Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection			
				D0 (µm)	D900 (µm)	Curvature (µm)	DR
PM-CMB	33B	71.5	71.6	216	55	44	0.74
PM-CMB	33B	71.6	71.7	229	59	39	0.75
PM-CMB	33B	71.7	71.8	286	63	67	0.70
PM-CMB	33B	71.8	71.9	172	57	27	0.80
PM-CMB	33B	71.9	72.0	331	81	53	0.79
PM-CMB	33B	72.0	72.1	283	84	45	0.79
PM-CMB	33B	72.1	72.2	372	133	57	0.81
PM-CMB	33B	72.2	72.3	268	69	53	0.75
PM-CMB	33B	72.3	72.4	325	55	65	0.72
PM-CMB	33B	72.4	72.5	536	62	127	0.65
PM-CMB	33B	72.5	72.6	448	61	90	0.68
PM-CMB	33B	72.6	72.7	222	47	37	0.76
PM-CMB	33B	72.7	72.8	201	57	29	0.76
PM-CMB	33B	72.8	72.9	221	57	49	0.70
PM-CMB	33B	72.9	73.0	281	50	71	0.70
PM-CMB	33B	73.0	73.1	329	70	66	0.74
PM-CMB	33B	73.1	73.2	331	68	77	0.71
PM-CMB	33B	73.2	73.3	161	54	25	0.76
PM-CMB	33B	73.3	73.4	396	72	81	0.72
PM-CMB	33B	73.4	73.5	403	59	95	0.68
PM-CMB	33B	73.5	73.6	359	43	101	0.60
PM-CMB	33B	73.6	73.7	214	26	58	0.62
PM-CMB	33B	73.7	73.8	383	65	85	0.68
PM-CMB	33B	73.8	73.9	500	68	103	0.69
PM-CMB	33B	73.9	74.0	410	74	59	0.74
PM-CMB	33B	74.0	74.1	137	59	22	0.78
PM-CMB	530	1.1	1.2	170	65	27	0.79
PM-CMB	530	1.2	1.3	106	58	16	0.80
PM-CMB	530	1.3	1.4	102	61	13	0.83
PM-CMB	530	1.4	1.5	85	47	13	0.80
PM-CMB	530	1.5	1.6	99	27	21	0.70
PM-CMB	530	1.6	1.7	74	30	13	0.75
PM-CMB	530	1.7	1.8	97	49	14	0.80
PM-CMB	530	1.8	1.9	105	43	21	0.74
PM-CMB	530	2.4	2.5	181	91	20	0.82
PM-CMB	530	2.5	2.6	157	89	16	0.86
PM-CMB	530	2.6	2.7	148	76	17	0.83
PM-CMB	530	2.7	2.8	122	58	19	0.79
PM-CMB	530	2.8	2.9	165	65	34	0.74

Material type	Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection			
				D0 (µm)	D900 (µm)	Curvature (µm)	DR
PM-CMB	856	2.6	2.7	523	36	119	0.65
PM-CMB	856	2.7	2.8	492	41	116	0.64
PM-CMB	856	2.8	2.9	532	42	135	0.62
PM-CMB	856	2.9	3.0	616	55	135	0.66
PM-CMB	856	3.0	3.1	549	46	131	0.64
PM-CMB	856	3.1	3.2	504	38	124	0.63
PM-CMB	856	3.2	3.3	365	38	83	0.67
PM-CMB	856	3.3	3.4	480	43	89	0.69
PM-CMB	856	4.4	4.5	486	67	98	0.70
PM-CMB	856	4.5	4.6	315	30	73	0.65
PM-CMB	856	4.6	4.7	107	39	18	0.77
PM-CMB	856	4.7	4.8	76	34	9	0.82
PM-CMB	856	4.8	4.9	54	17	7	0.71
PM-CMB	856	4.9	5.0	66	21	12	0.74
PM-CMB	856	5.0	5.1	93	34	18	0.74
PM-CMB	856	5.1	5.2	101	36	14	0.78
PM-CMB	856	5.2	5.3	59	21	14	0.65
PM-CMB	856	5.3	5.4	65	27	12	0.76
PM-CMB	856	5.4	5.5	96	39	15	0.80
PM-CMB	856	5.5	5.6	77	28	16	0.73
PM-CMB	856	5.6	5.7	74	39	9	0.83
PM-CMB	856	5.7	5.8	78	42	10	0.84
PM-CMB	856	5.8	5.9	94	42	14	0.77
PM-CMB	856	5.9	6.0	109	64	10	0.86
PM-CMB	856	6.0	6.1	67	33	11	0.77
PM-CMB	856	6.1	6.2	33	10	11	0.59
PM-CMB	856	6.2	6.3	95	33	20	0.75

Table B 2: FWD deflection summary for I-FBS pavement sections

Material type	Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection			
				D0 (µm)	D900 (µm)	Curvature (µm)	DR
I-FBS	10N	103.4	103.5	416	80	74	0.75
I-FBS	10N	103.5	103.6	369	63	73	0.73
I-FBS	10N	103.6	103.7	383	64	74	0.73
I-FBS	10N	103.7	103.8	398	67	83	0.72
I-FBS	10N	103.8	103.9	440	75	88	0.73
I-FBS	10N	103.9	104.0	428	87	72	0.76
I-FBS	10N	104.0	104.1	429	85	75	0.76
I-FBS	10N	104.1	104.2	443	78	75	0.76
I-FBS	10N	104.2	104.3	391	81	71	0.75
I-FBS	10N	104.3	104.4	369	71	64	0.76
I-FBS	10N	104.4	104.5	382	73	65	0.75
I-FBS	10N	110.8	110.9	141	64	19	0.82
I-FBS	10N	110.9	111.0	136	54	22	0.80
I-FBS	10N	111.0	111.1	132	42	31	0.73
I-FBS	10N	111.1	111.2	163	45	41	0.71
I-FBS	10N	111.2	111.3	192	62	29	0.80
I-FBS	10N	111.3	111.4	240	90	38	0.81
I-FBS	10N	111.4	111.5	203	76	30	0.82
I-FBS	10N	111.5	111.6	356	94	72	0.75
I-FBS	10N	111.6	111.7	487	82	92	0.74
I-FBS	10N	111.7	111.8	471	82	93	0.72
I-FBS	10N	111.8	111.9	500	85	122	0.69
I-FBS	10N	123.0	123.1	263	79	44	0.80
I-FBS	10N	123.1	123.2	199	70	28	0.81
I-FBS	10N	123.2	123.3	176	73	19	0.86
I-FBS	10N	123.3	123.4	297	103	34	0.84
I-FBS	10N	123.4	123.5	515	151	69	0.82
I-FBS	10N	123.5	123.6	590	155	79	0.81
I-FBS	10N	123.6	123.7	587	132	78	0.81
I-FBS	10N	123.7	123.8	440	108	67	0.80
I-FBS	10N	123.8	123.9	413	106	59	0.81
I-FBS	10N	123.9	124.0	749	144	131	0.77
I-FBS	10N	124.0	124.1	696	174	96	0.82
I-FBS	10P	51.3	51.4	168	64	23	0.83
I-FBS	10P	51.4	51.5	159	68	16	0.86
I-FBS	10P	51.5	51.6	131	54	15	0.85
I-FBS	10P	51.6	51.7	130	52	13	0.86
I-FBS	10P	51.7	51.8	126	51	14	0.84

Material type	Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection			
				D0 (µm)	D900 (µm)	Curvature (µm)	DR
I-FBS	10P	51.8	51.9	126	52	13	0.85
I-FBS	10P	51.9	52.0	145	60	15	0.85
I-FBS	10P	52.0	52.1	176	68	22	0.84
I-FBS	10P	52.1	52.2	160	64	18	0.85
I-FBS	10P	52.2	52.3	161	66	19	0.85
I-FBS	10P	52.3	52.4	112	47	13	0.85
I-FBS	10P	64.1	64.2	247	89	28	0.85
I-FBS	10P	64.2	64.3	242	89	22	0.86
I-FBS	10P	64.3	64.4	279	88	39	0.81
I-FBS	10P	64.4	64.5	182	52	34	0.77
I-FBS	10P	64.5	64.6	235	62	39	0.78
I-FBS	10P	64.6	64.7	229	56	43	0.75
I-FBS	10P	64.7	64.8	242	52	48	0.75
I-FBS	10P	64.8	64.9	285	53	68	0.71
I-FBS	10P	64.9	65.0	335	61	82	0.68
I-FBS	10P	65.0	65.1	295	59	68	0.70
I-FBS	10P	65.1	65.2	293	73	56	0.76
I-FBS	10P	65.2	65.3	277	72	45	0.78
I-FBS	22B	34.4	34.5	190	45	41	0.73
I-FBS	22B	34.5	34.6	298	56	65	0.72
I-FBS	22B	34.6	34.7	212	52	33	0.80
I-FBS	22B	34.7	34.8	257	56	47	0.78
I-FBS	22B	34.8	34.9	206	60	31	0.82
I-FBS	22B	34.9	35.0	206	88	20	0.86
I-FBS	22B	35.0	35.1	367	154	53	0.86
I-FBS	22B	35.1	35.2	649	168	144	0.74
I-FBS	22B	35.2	35.3	324	138	38	0.86
I-FBS	22B	35.3	35.4	212	117	14	0.91
I-FBS	22B	35.4	35.5	283	144	19	0.90
I-FBS	22B	54.7	54.8	396	128	65	0.78
I-FBS	22B	54.8	54.9	311	100	48	0.79
I-FBS	22B	54.9	55.0	316	78	77	0.72
I-FBS	22B	55.0	55.1	226	102	21	0.88
I-FBS	22B	55.1	55.2	275	101	46	0.79
I-FBS	22B	55.2	55.3	247	107	37	0.84
I-FBS	22B	55.3	55.4	311	113	55	0.81
I-FBS	22B	55.4	55.5	249	94	35	0.82
I-FBS	22B	55.5	55.6	181	73	28	0.82
I-FBS	22B	55.6	55.7	344	110	57	0.78

Material type	Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection			
				D0 (µm)	D900 (µm)	Curvature (µm)	DR
I-FBS	22B	55.7	55.8	714	118	206	0.62
I-FBS	25B	11.5	11.6	233	99	29	0.85
I-FBS	25B	11.6	11.7	202	88	26	0.84
I-FBS	25B	11.7	11.8	178	86	23	0.85
I-FBS	25B	11.8	11.9	169	81	21	0.85
I-FBS	25B	11.9	12.0	227	92	33	0.85
I-FBS	25B	12.0	12.1	250	110	29	0.86
I-FBS	25B	12.1	12.2	200	88	27	0.84
I-FBS	25B	12.2	12.3	232	107	26	0.86
I-FBS	25B	12.3	12.4	358	138	42	0.85
I-FBS	25B	12.4	12.5	351	134	50	0.83
I-FBS	25B	12.5	12.6	481	134	73	0.79
I-FBS	25B	12.6	12.7	611	168	100	0.79
I-FBS	25B	12.7	12.8	477	149	69	0.80
I-FBS	25B	12.8	12.9	710	188	149	0.80
I-FBS	25B	12.9	13.0	449	165	46	0.87
I-FBS	25B	13.0	13.1	519	184	66	0.85
I-FBS	25B	13.1	13.2	413	141	48	0.86
I-FBS	25B	13.2	13.3	339	119	49	0.86
I-FBS	25B	13.3	13.4	164	83	13	0.90
I-FBS	25B	13.4	13.5	156	80	11	0.90
I-FBS	25B	13.5	13.6	168	77	16	0.87
I-FBS	208	3.0	3.1	236	33	63	0.66
I-FBS	208	3.1	3.2	137	24	37	0.67
I-FBS	208	3.2	3.3	102	27	22	0.72
I-FBS	208	3.3	3.4	362	60	98	0.69
I-FBS	208	3.4	3.5	113	27	26	0.72
I-FBS	208	3.5	3.6	313	23	92	0.64
I-FBS	208	3.6	3.7	416	39	119	0.64
I-FBS	208	3.7	3.8	367	41	115	0.60
I-FBS	208	3.8	3.9	254	10	95	0.51
I-FBS	212	3.2	3.3	147	54	39	0.70
I-FBS	212	3.3	3.4	129	55	32	0.72
I-FBS	212	3.4	3.5	126	61	23	0.79
I-FBS	212	3.5	3.6	111	52	21	0.78
I-FBS	212	3.6	3.7	117	56	20	0.81
I-FBS	212	3.7	3.8	112	57	20	0.80
I-FBS	212	3.8	3.9	146	66	27	0.79
I-FBS	212	3.9	4.0	138	58	27	0.78

Material type	Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection			
				D0 (µm)	D900 (µm)	Curvature (µm)	DR
I-FBS	212	4.0	4.1	229	62	78	0.63
I-FBS	212	4.1	4.2	502	83	174	0.55
I-FBS	1003	3.2	3.3	320	47	89	0.64
I-FBS	1003	3.3	3.4	346	61	90	0.68
I-FBS	1003	3.4	3.5	176	43	50	0.68
I-FBS	1003	3.5	3.6	237	84	52	0.76
I-FBS	1003	3.6	3.7	244	92	52	0.76
I-FBS	1003	3.7	3.8	258	94	55	0.75
I-FBS	1003	3.8	3.9	218	83	50	0.74
I-FBS	1003	3.9	4.0	239	81	54	0.74
I-FBS	1003	4.0	4.1	185	71	42	0.75
I-FBS	1003	4.1	4.2	190	78	45	0.74
I-FBS	1003	4.2	4.3	313	79	84	0.69
I-FBS	1003	4.3	4.4	336	95	83	0.71
I-FBS	1003	4.4	4.5	243	86	47	0.77
I-FBS	1003	4.5	4.6	255	92	52	0.76
I-FBS	1003	4.6	4.7	227	93	37	0.80
I-FBS	1003	4.7	4.8	216	80	46	0.76
I-FBS	1003	4.8	4.9	257	84	61	0.73
I-FBS	1003	4.9	5.0	449	101	120	0.67
I-FBS	1003	5.0	5.1	685	105	188	0.64

APPENDIX C DETAILS OF STATISTICAL ANALYSIS

C.1 PM-CMB ANOVA

C.1.1 Rutting Results

The rutting results are shown in Table C 1, with differences between the PM-CMB case in the continuous and nominal factors that affected I-FBS rutting.

Table C 1: P-value table for rutting (CMB)

Continuous variables / Nominal variables	Surfacing age	Vertical strain	Normalised deflection curvature (µm)	Cumulative traffic	Average cost	Modulus E1 (MPa)	Modulus E2 (MPa)	Modulus E5 (MPa)
Total pavement thickness	0.02	1.5E-6	4.6E-6	0.006	0.04	0.002	0.018	
Binder type		4.3E-7	6.5E-7		0.017		6.5E-7	
Surface type		5E-9	3.3E-9	6E-5	1.2E-13			
AADT	0.02	2.3E-8	5.8E-8	0.04	0.005204 387		1.4E-5	9.9E-8

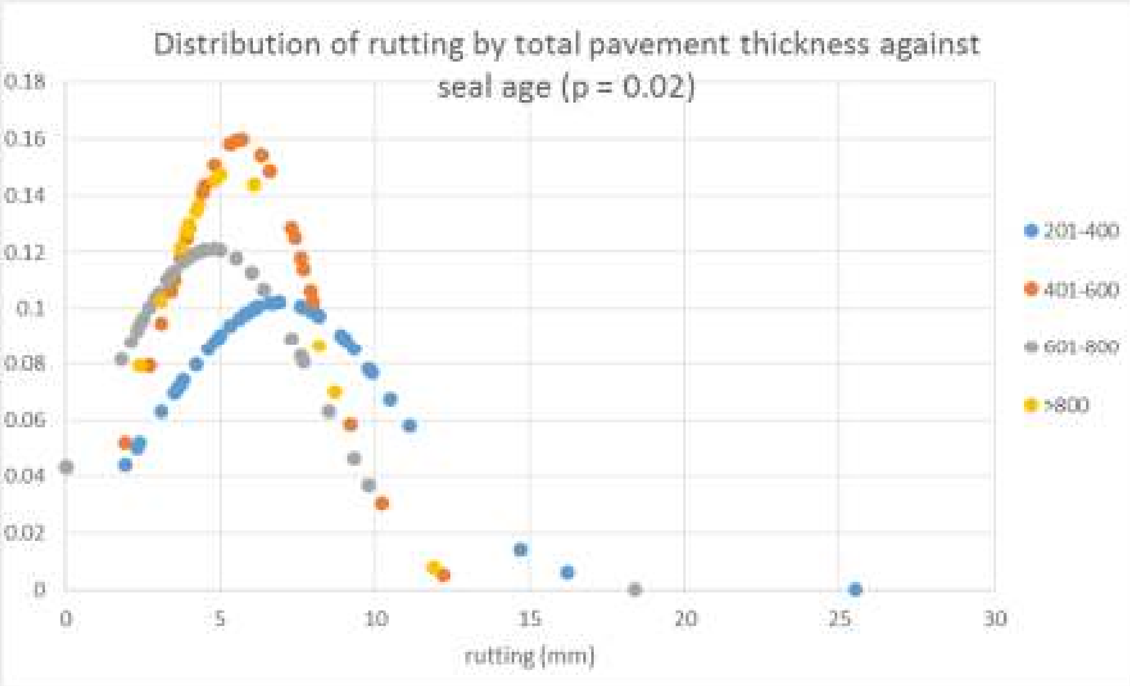
Total pavement thickness

The rutting grouped by total pavement thickness had more relationships for PM-CMB than for I-FBS pavements.

Surfacing age

The mean rutting appears to be lower for the 601–800 mm and > 800 mm bins as presented in Figure C 1. The highest rutting is seen for the 201–400 mm category and also has the highest spread indicating perhaps that a wider range of Surfacing ages were recorded for this pavement tape and possibly a greater number of interactions with other factors such as binder type or AADT.

Figure C 1: Distribution of rutting grouped by total pavement thickness bins against surfacing age

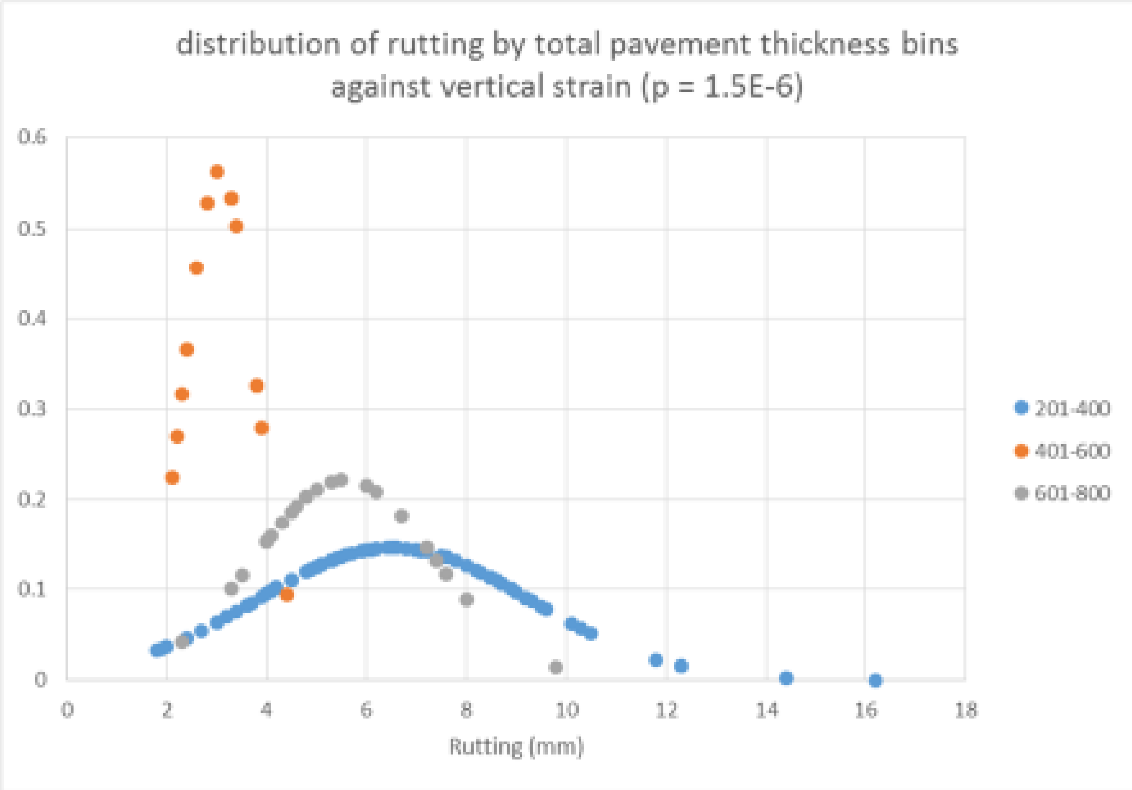


Pavements in the 201–400 mm thickness category had a larger spread of rutting with the highest mean and, as expected, pavements in the > 800 category had a low mean with a narrow spread. There are also fewer pavements that are greater than 800 mm.

Vertical strain

Rutting vs. vertical strain grouped by pavement thickness have vastly different distributions and means. The lowest mean is for the 401–600 m bin which may also indicate a smaller range of vertical strain values for rutting. The 201–400 bin had the widest spread (as presented in Figure C 2) indicating more sample sizes, a greater spread of samples, and the likelihood that this pavement thickness is used in more varied conditions and interacts with other factors to influence rutting.

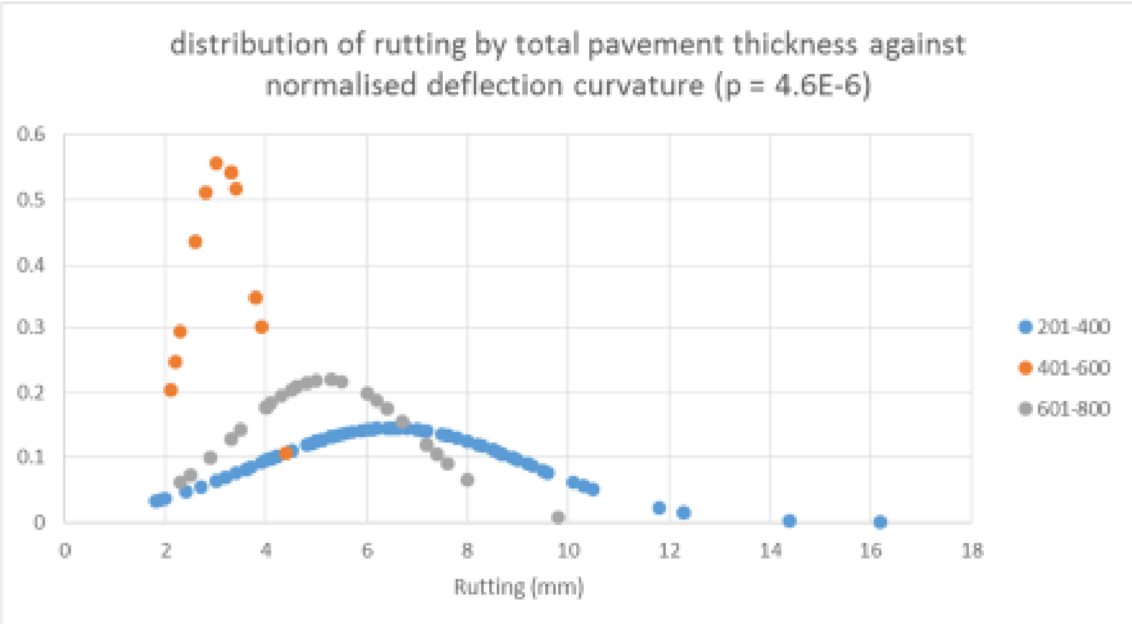
Figure C 2: Distribution of rutting grouped by total pavement thickness bins against vertical strain



Normalised deflection curvature

The relationship shown in Figure C 3 displays a similar pattern of behaviour as seen in Figure C 2.

Figure C 3: Distribution of rutting grouped by total pavement thickness against normalised deflection curvature

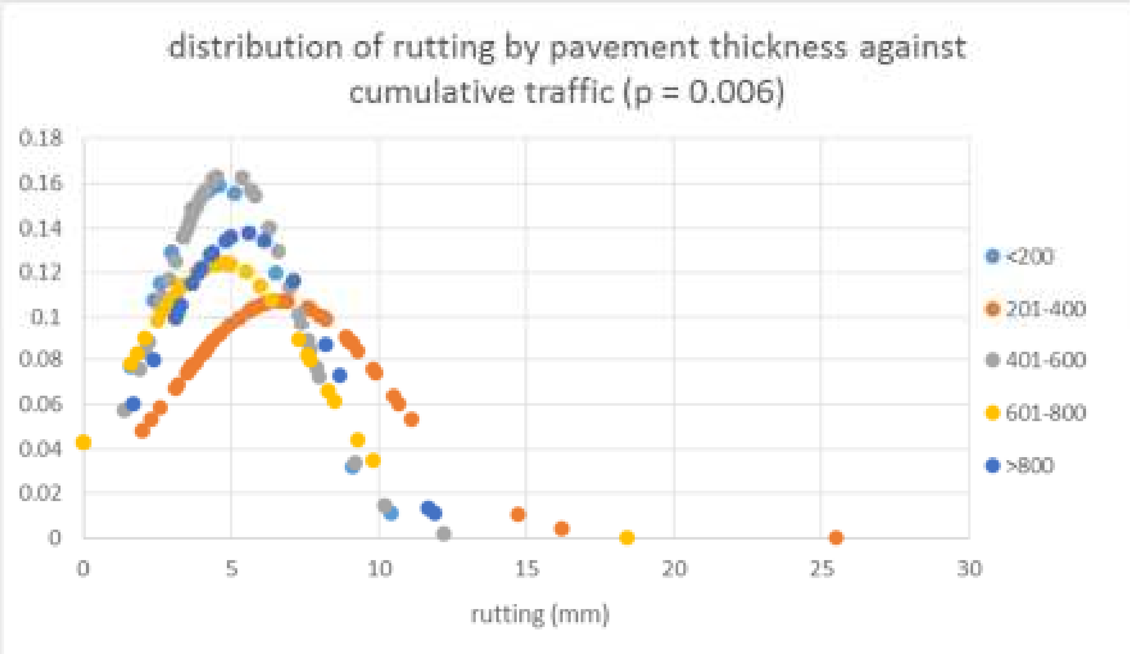


Cumulative traffic

Cumulative traffic and rutting, when grouped by pavement thickness, show more of a statistically significant difference in means between the 201–400 mm bin and the

601–800 mm as shown in Figure C 4. The means do not appear to correlate with increasing pavement thickness, suggesting greater influence on the rutting than just cumulative traffic itself.

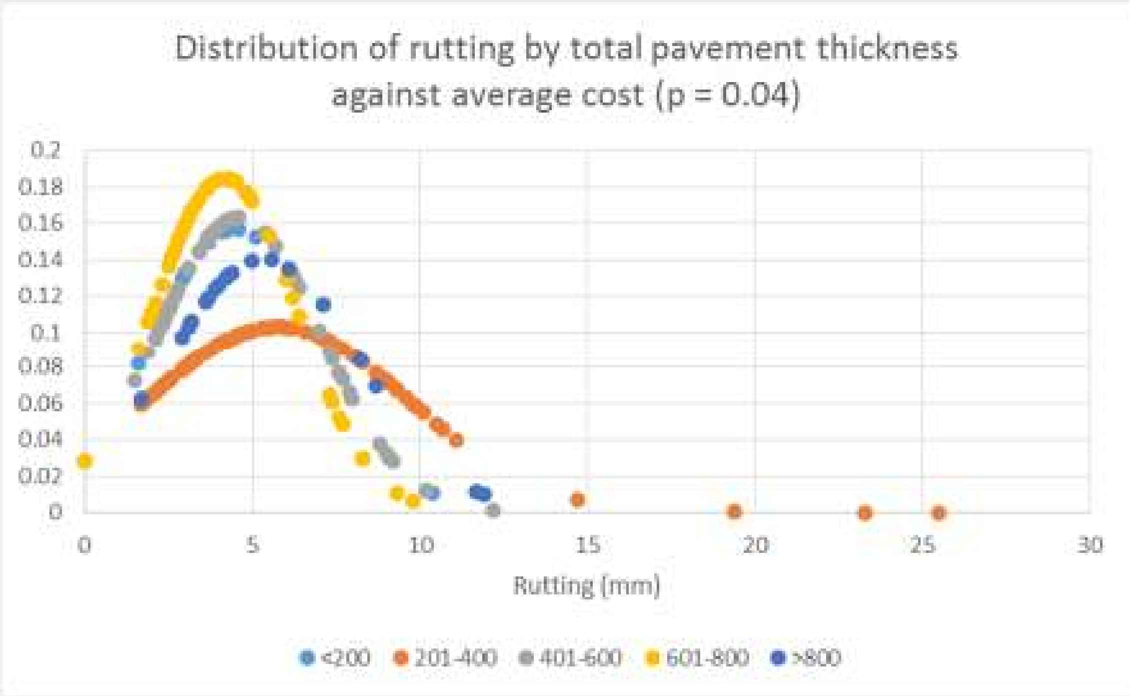
Figure C 4: Distribution of rutting grouped by pavement thickness bins against cumulative traffic



Average cost

The average cost vs. rutting by total pavement thickness displays a similar relationship to the previous, where the mean rutting does not seem to increase with increasing AADT bins. This suggests that other factors than average cost and pavement thickness are influencing the rutting. The consistent pattern thus far is that compared with the other bins, the 201–400 category has the highest mean and the largest spread with greater outliers, as shown in Figure C 5.

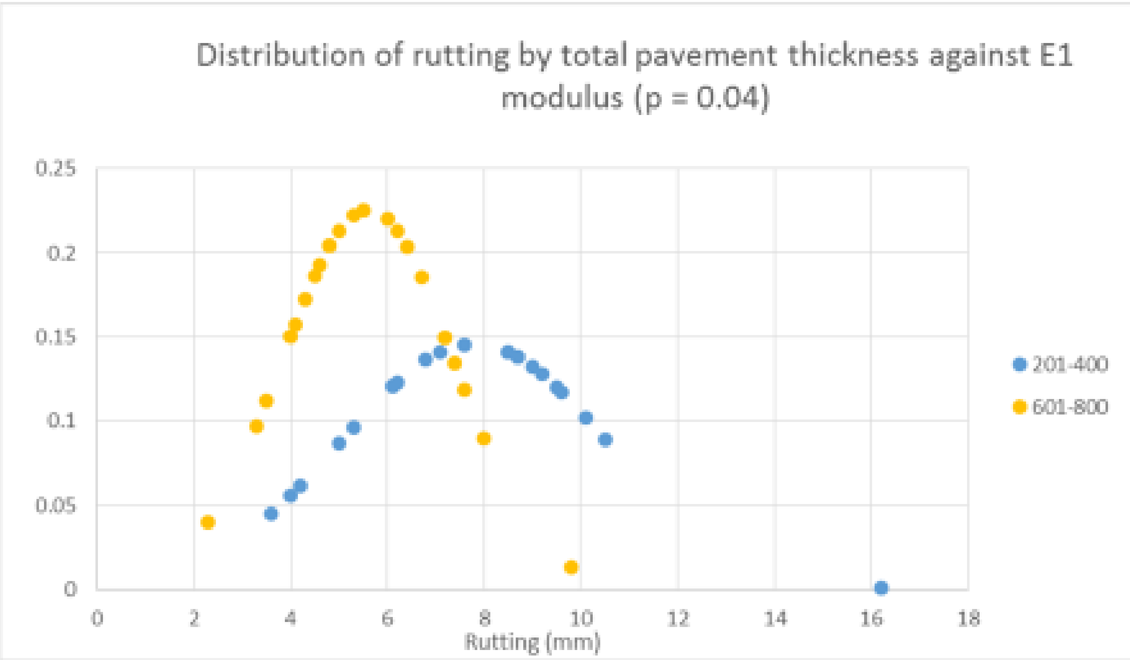
Figure C 5: Distribution of rutting grouped by pavement thickness bins against average cost



Modulus E1

The sample sizes for rutting by pavement thickness against modulus E1 is small, as presented in Figure C 6. As expected, it follows a pattern where the thicker pavement thickness range has lower rutting than the thinner pavement thickness range.

Figure C 6: Distribution of rutting grouped by total pavement thickness against E1 modulus



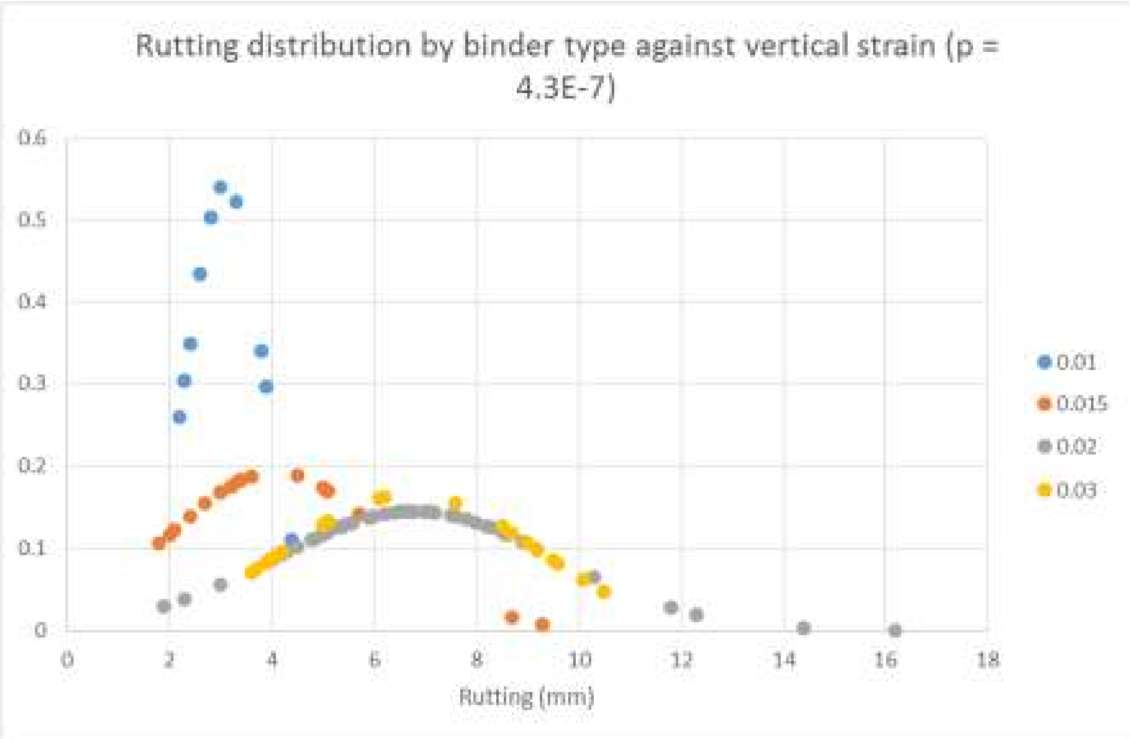
Binder content

This category may be questionable as most of the binder types are largely unknown. Regardless of the nature of the binder content, there appears to be statistical significance between rutting when grouped by binder content as shown in Figure C 7 to Figure C 11.

Vertical strain

The category with the narrowest spread is 0.01 and it also has the lowest rutting indicating that the range of values for vertical strain are probably concentrated on the lower end of the strain spectrum. The other distributions are more populated over a greater range of vertical strain values, indicating a likely relationship, linear or otherwise, with vertical strain.

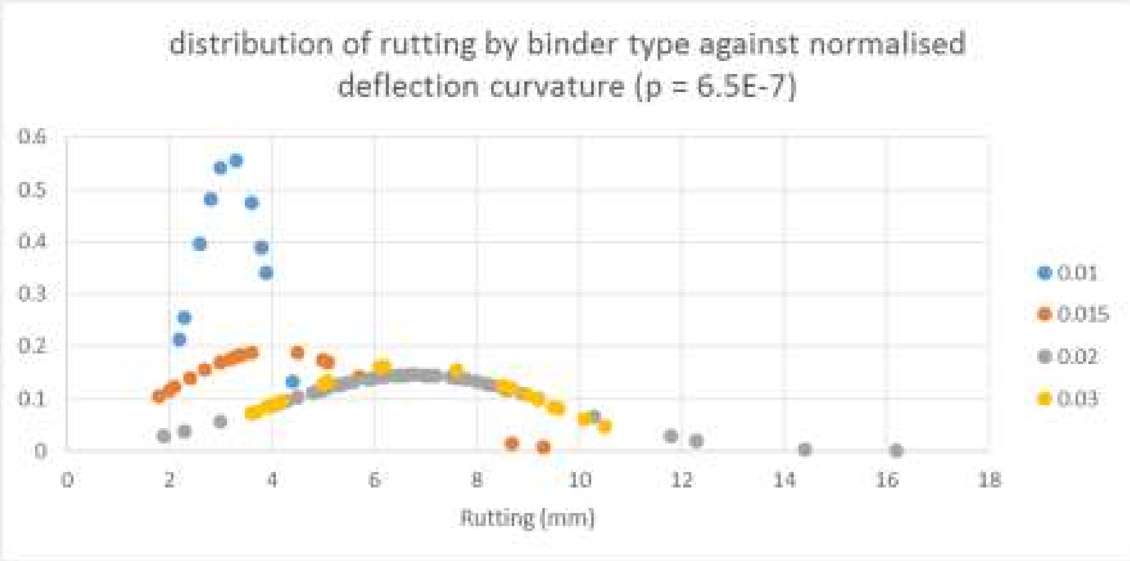
Figure C 7: Distribution of rutting grouped by binder type against vertical strain



Normalised deflection curvature

The relationship for normalised deflection curvature and binder type is similar to vertical strain and shown in Figure C 7.

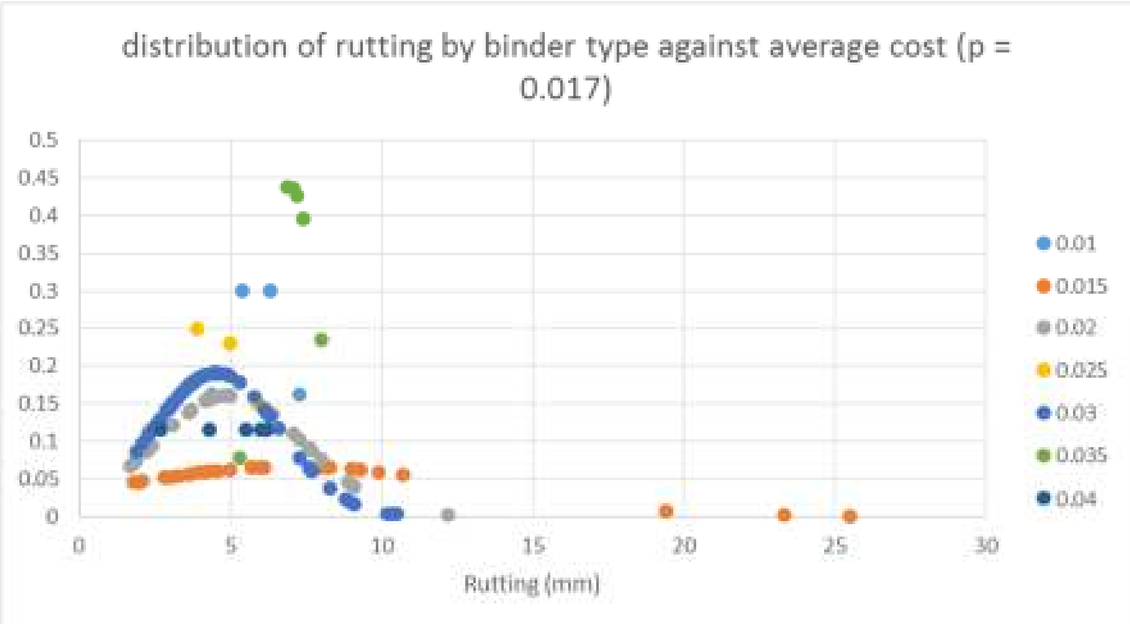
Figure C 8: Distribution of rutting group by binder type against normalised deflection curvature



Average cost

The rutting distributions against average cost vary significantly in spread rather than means. The biggest spread is seen for the 0.015 category and the narrowest for the 0.035 category that has fewer samples in general. Average cost and rutting by binder type is a good candidate for sensitivity analysis.

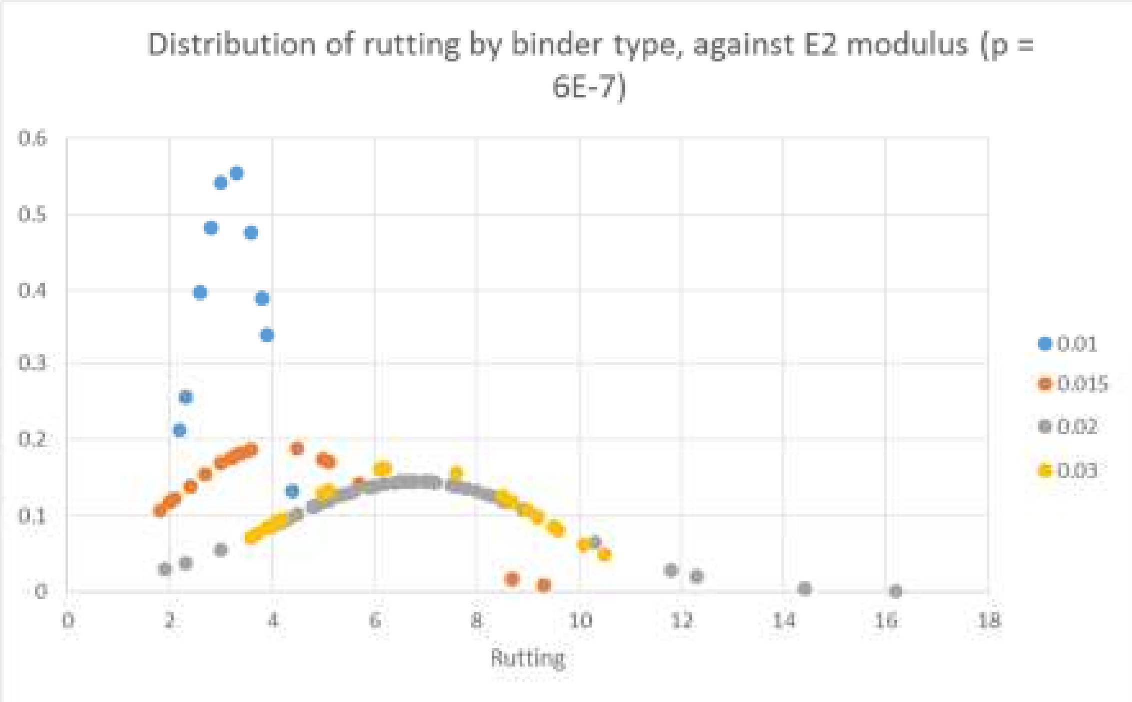
Figure C 9: Distribution of rutting grouped by binder type against average cost of maintenance



E2 modulus

The E2 modulus relationship is similar to the normalised deflection curvature and vertical strain relationships.

Figure C 10: Distribution of rutting grouped by binder type against E2 modulus

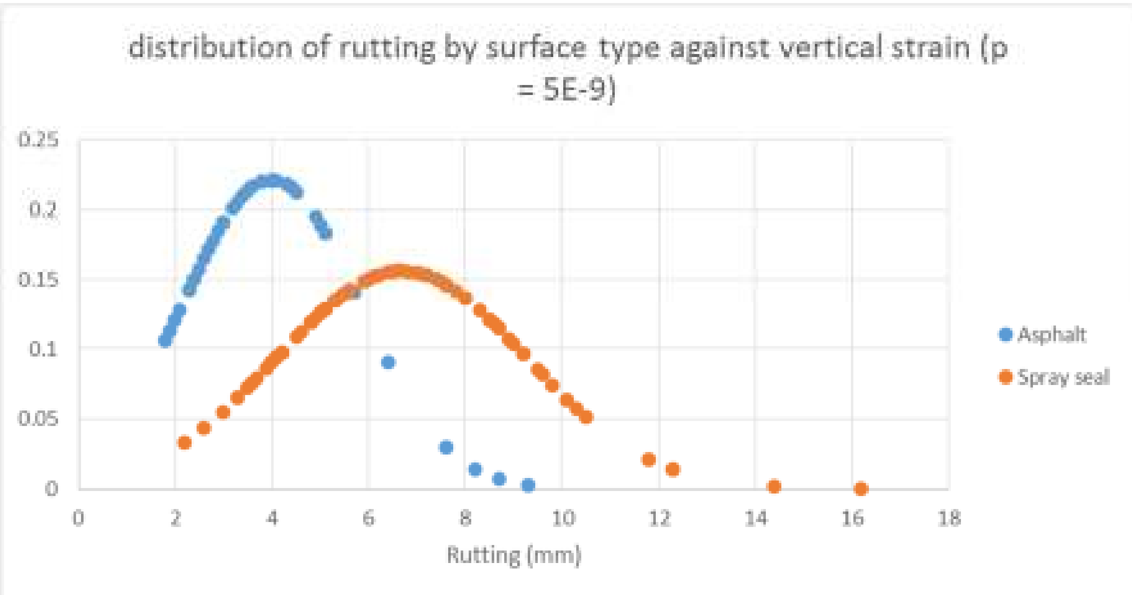


Surface type

Surface type is related to more continuous variables for CMB than for FBS as seen in Figure C 11 to Figure C 14, which have only cumulative traffic in common. The spread for spray seal is wider than for asphalt in every case and the mean of rutting is also greater.

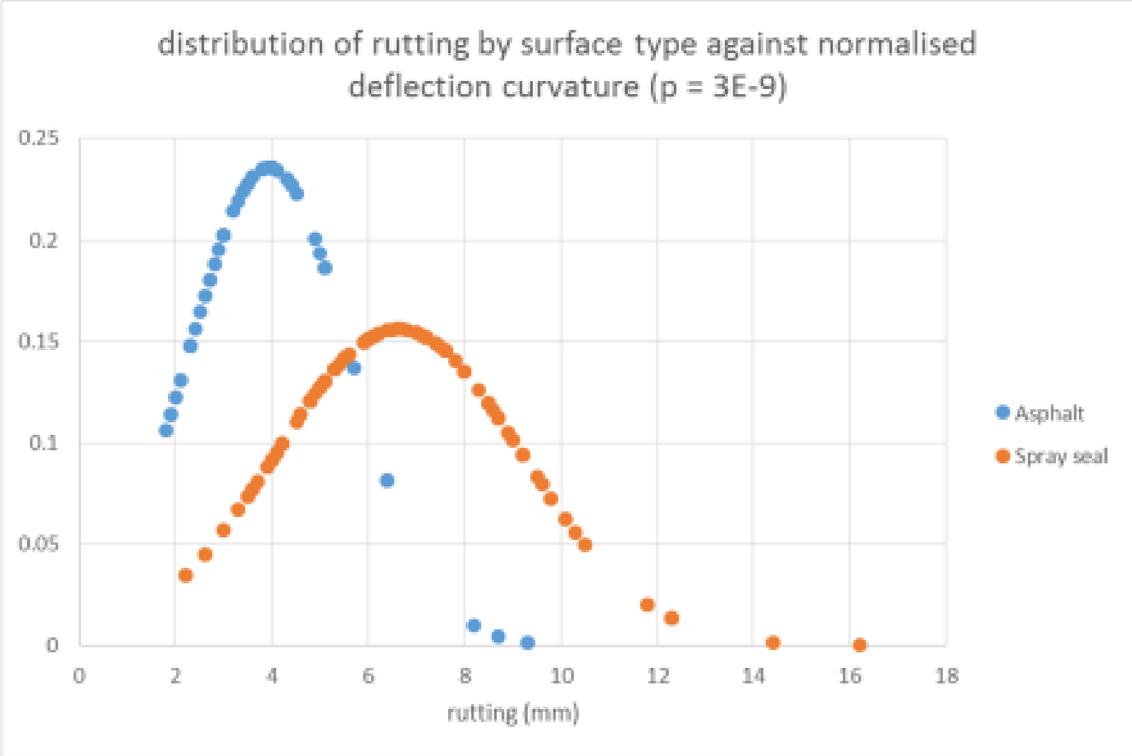
Vertical strain

Figure C 11: Distribution of rutting grouped by binder type against vertical strain



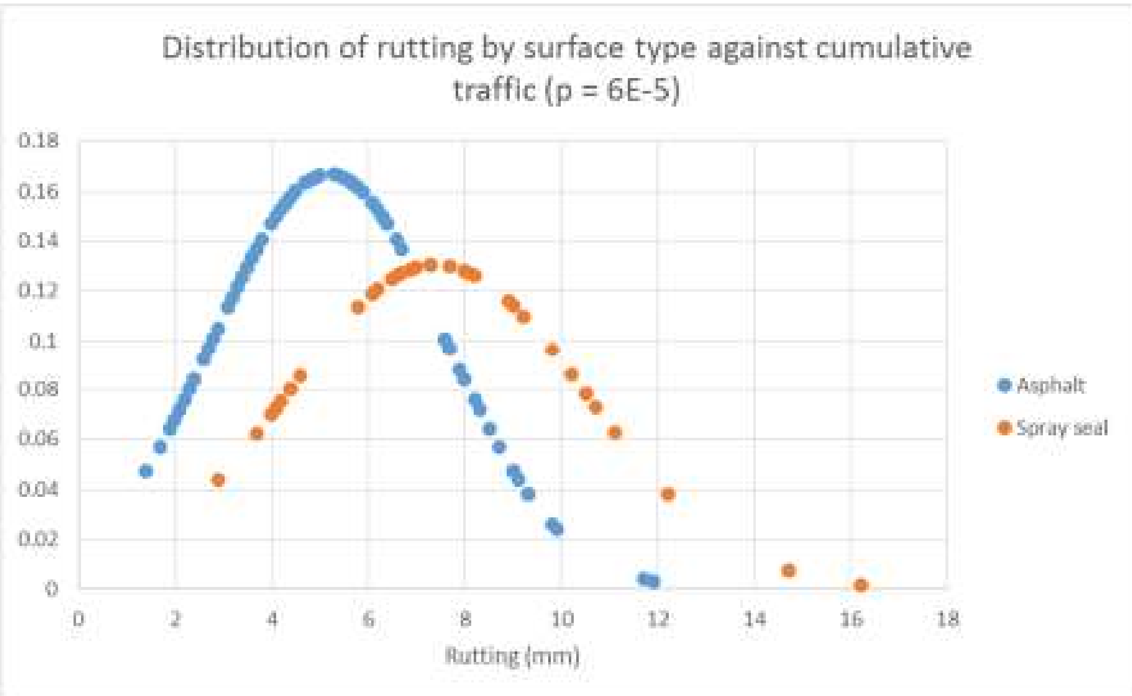
Normalised deflection curvature

Figure C 12: Distribution of rutting grouped by surface type against normalised deflection curvature



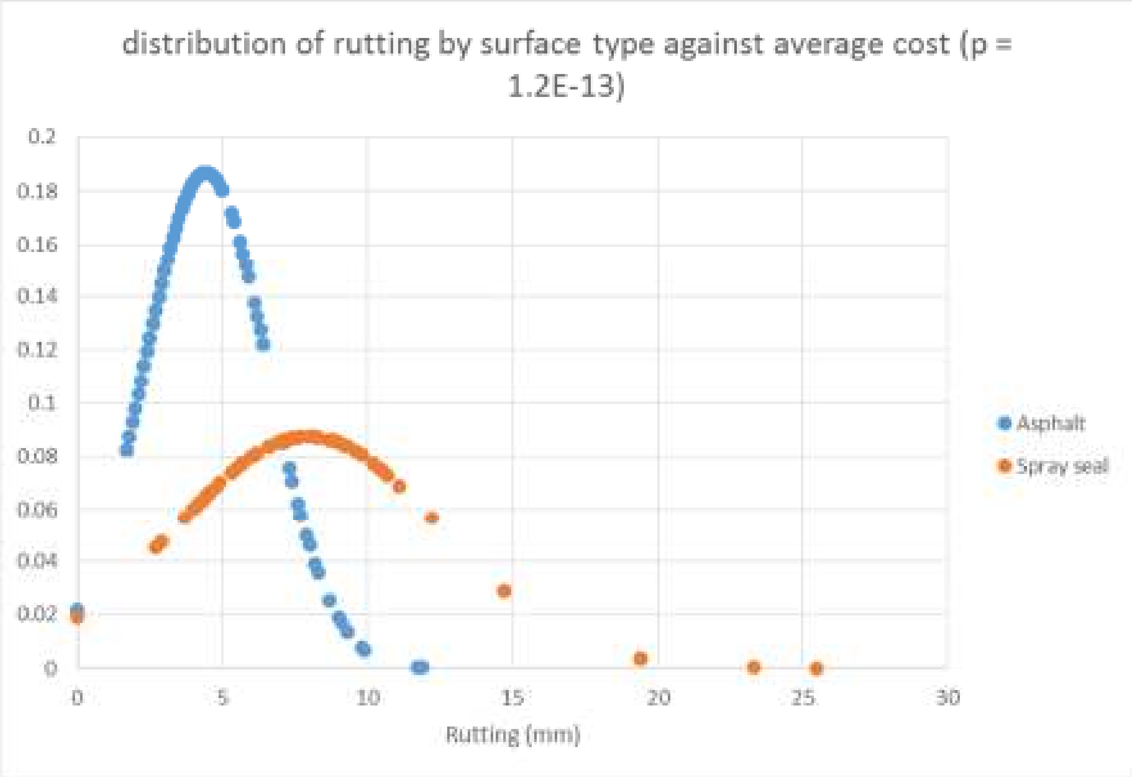
Cumulative traffic

Figure C 13: Distribution of rutting grouped by surface type against cumulative traffic



Average cost

Figure C 14: Distribution of rutting grouped by surface type against average cost



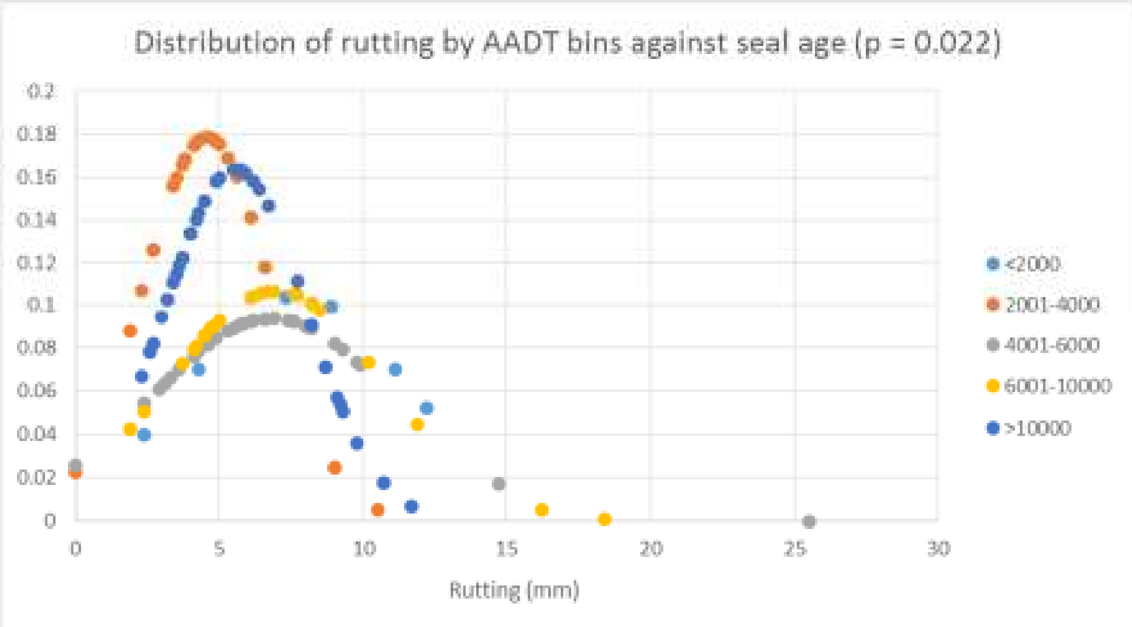
AADT

The AADT bins used for rutting in CMB are more refined than those used for FBS, with more levels for the CMB case, as the samples were distributed too unevenly. There appears to be no correlation between the means of rutting and the AADT categories; however, the statistical difference is significant between the groups for many continuous variables as with total pavement thickness. This makes AADT a good candidate for sensitive analysis and suggests that there may be collinearity between many of the continuous variables.

The average cost, cumulative traffic and Surfacing age relationships look very similar with similar spreads for each category and very similar order of means. The vertical strain, normalised deflection curvature and modulus distributions share similarities as well, suggesting there are two levels of a much bigger category. This could be the result of vastly different sample sizes that exist for the populations.

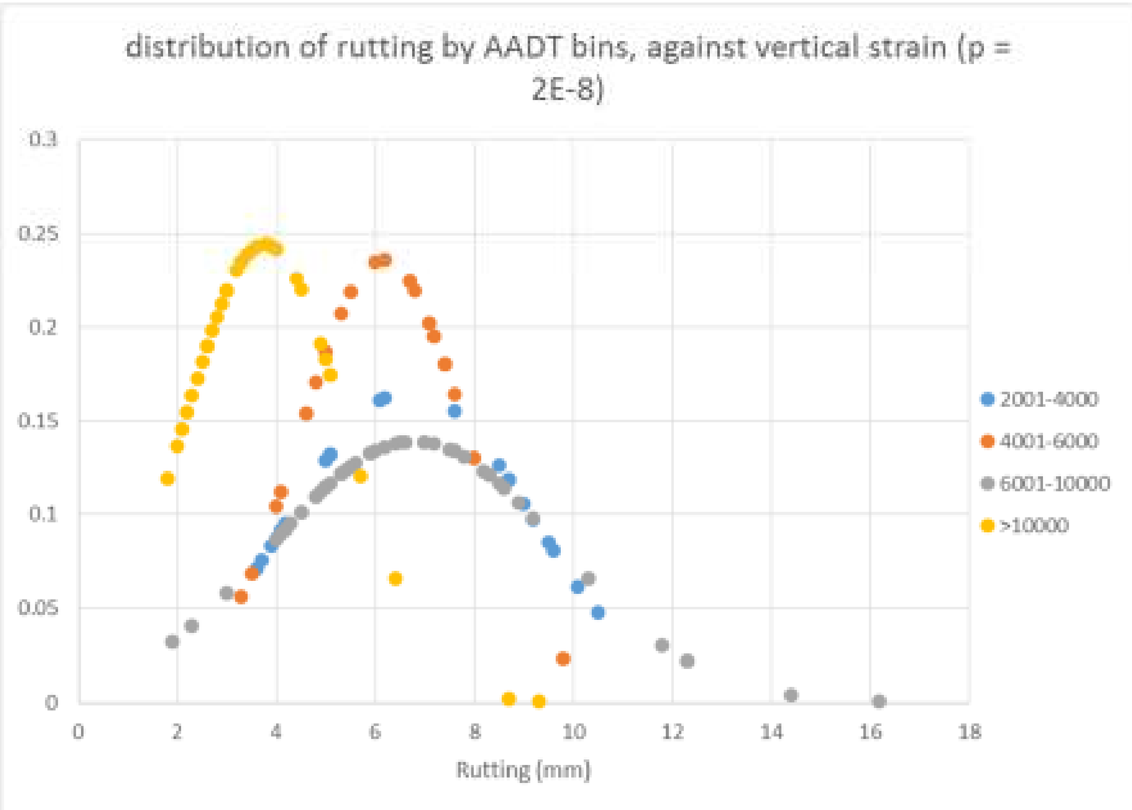
Surfacing age

Figure C 15: Distribution of rutting grouped by AADT bins against surfacing age



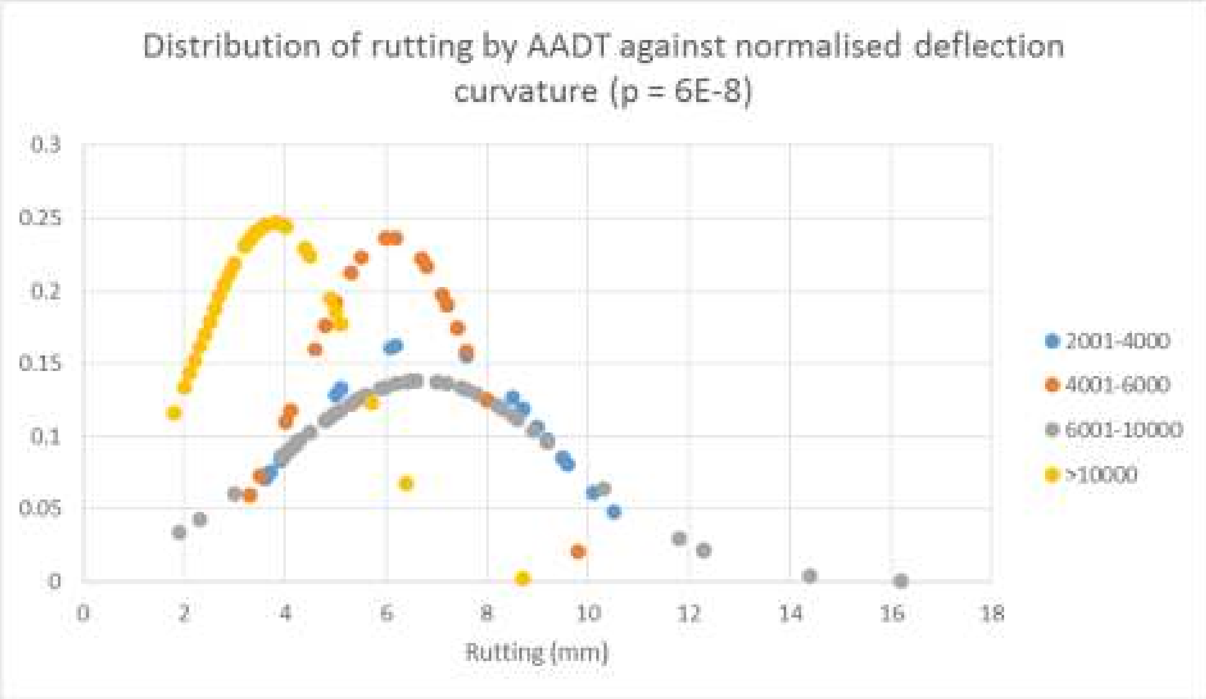
Vertical strain

Figure C 16: Distribution of rutting grouped by AADT bins against vertical strain



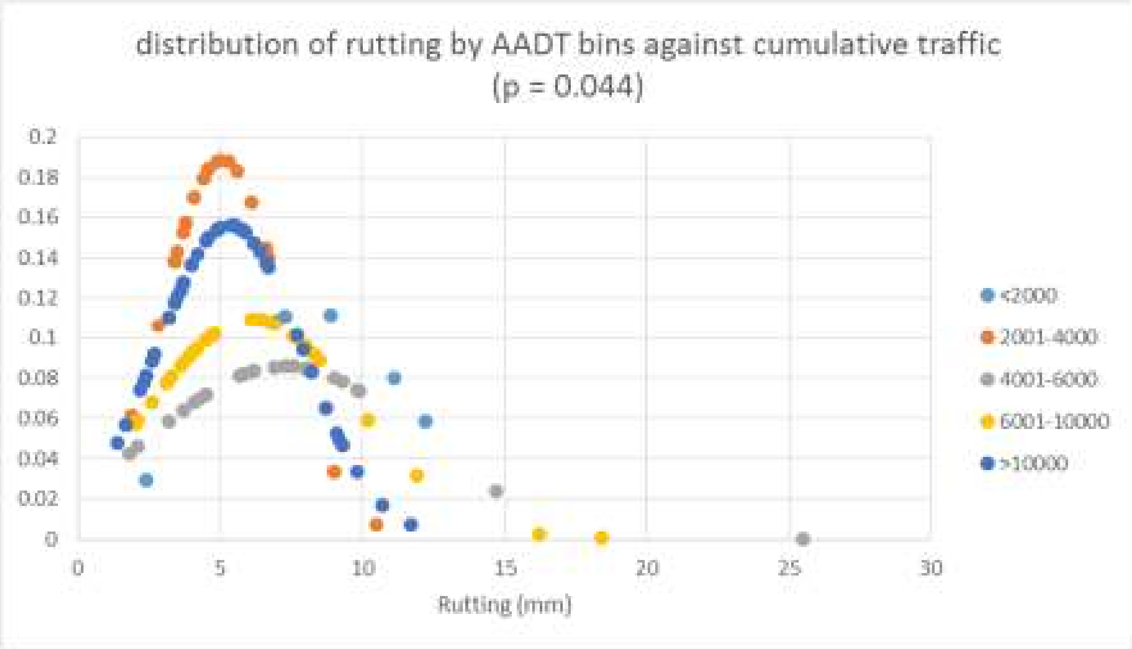
Normalised deflection curvature

Figure C 17: Distribution of rutting grouped by AADT bins against normalised deflection curvature



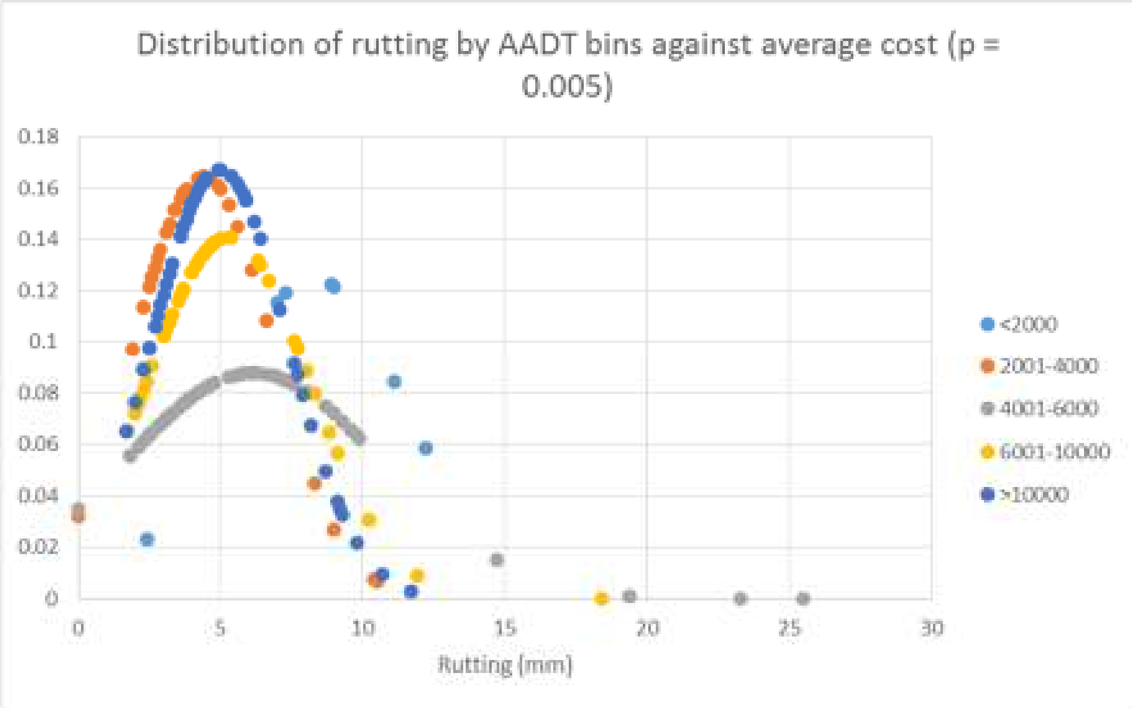
Cumulative traffic

Figure C 18: Distribution of rutting grouped by AADT bins against cumulative traffic



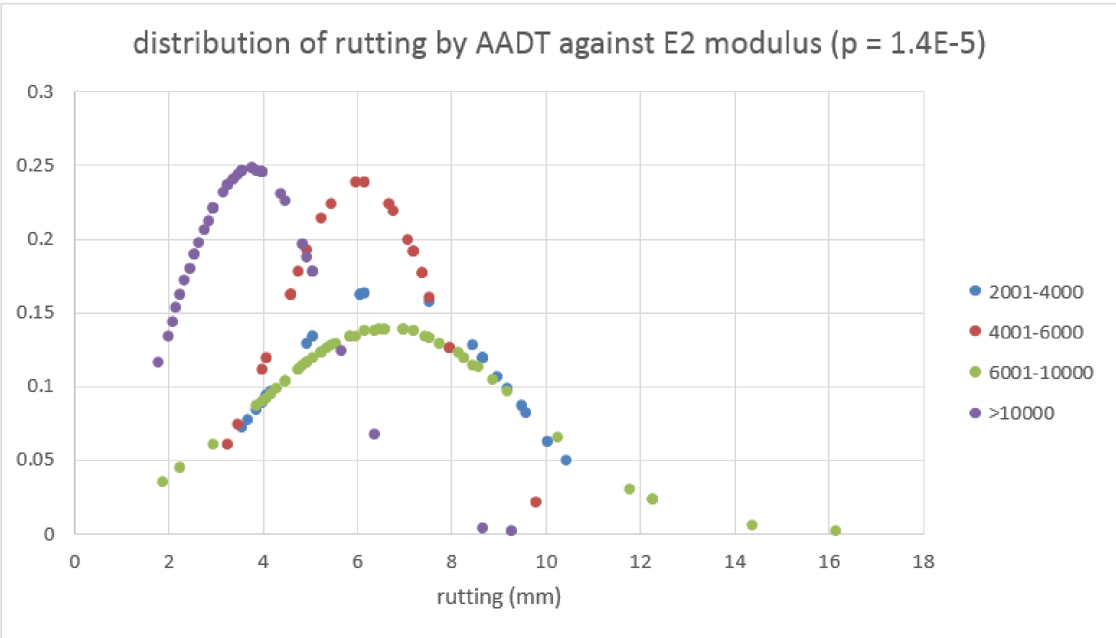
Average cost

Figure C 19: Distribution of rutting grouped by AADT bins against average cost



E2 modulus

Figure C 20: Distribution of rutting grouped by AADT against E2 modulus



Roughness Results

Roughness for PM-CMB has far fewer relationships than for I-FBS as shown in Table C 2. There are fewer nominal variables and fewer continuous variables indicating that, when grouped into the categories shown below, the sample points are reduced considerably.

Table C 2: P-value table for roughness (CMB)

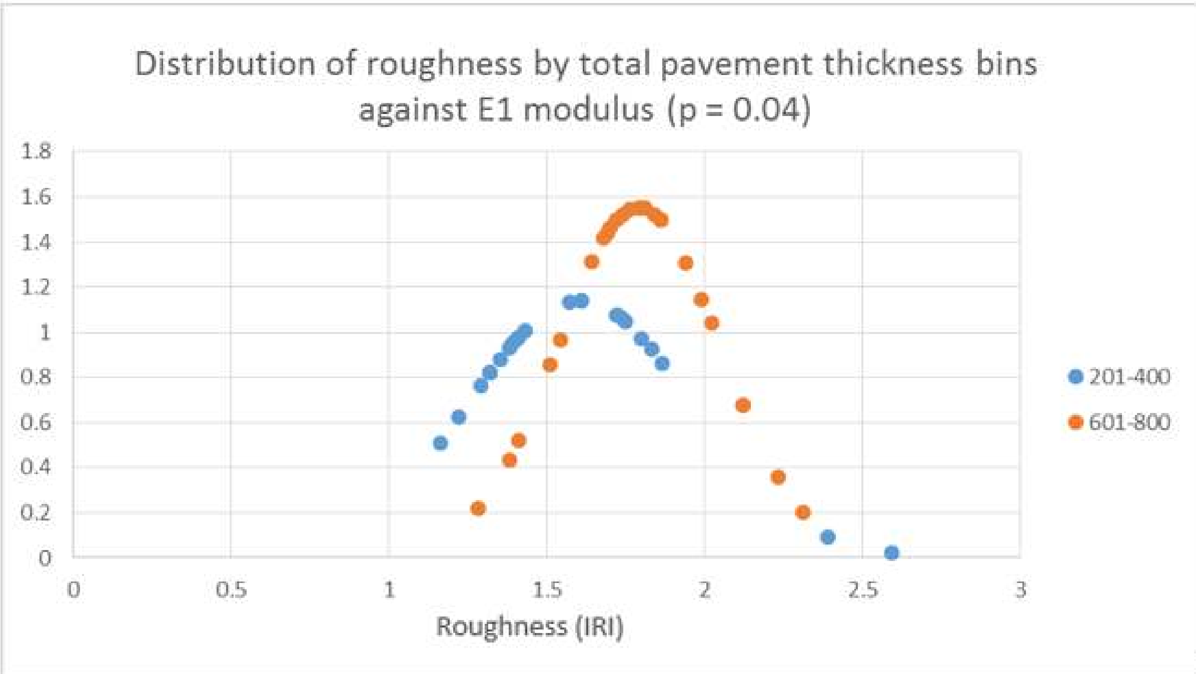
Continuous variables / Nominal variables	Total pavement thickness	Surfacing age	Average cost	Cumulative traffic	Modulus E1 (MPa)	Modulus E4 (MPa)
Binder			4E-6			
Total pavement thickness					0.04	0.047
Surface type	0.026		3E-6			0.036
AADT	0.002	0.037	0.00014	0.036		

Total pavement thickness

When grouped by total pavement thickness and plotted against E1 modulus, only two groups with sufficient data points remain. The greater pavement thickness appears to have the higher mean for roughness but a narrower spread, indicating that the modulus range may be lower than for the 201–400 mm bin.

E1 Modulus

Figure C 21: Distribution of roughness grouped by total pavement thickness bins against E1 modulus

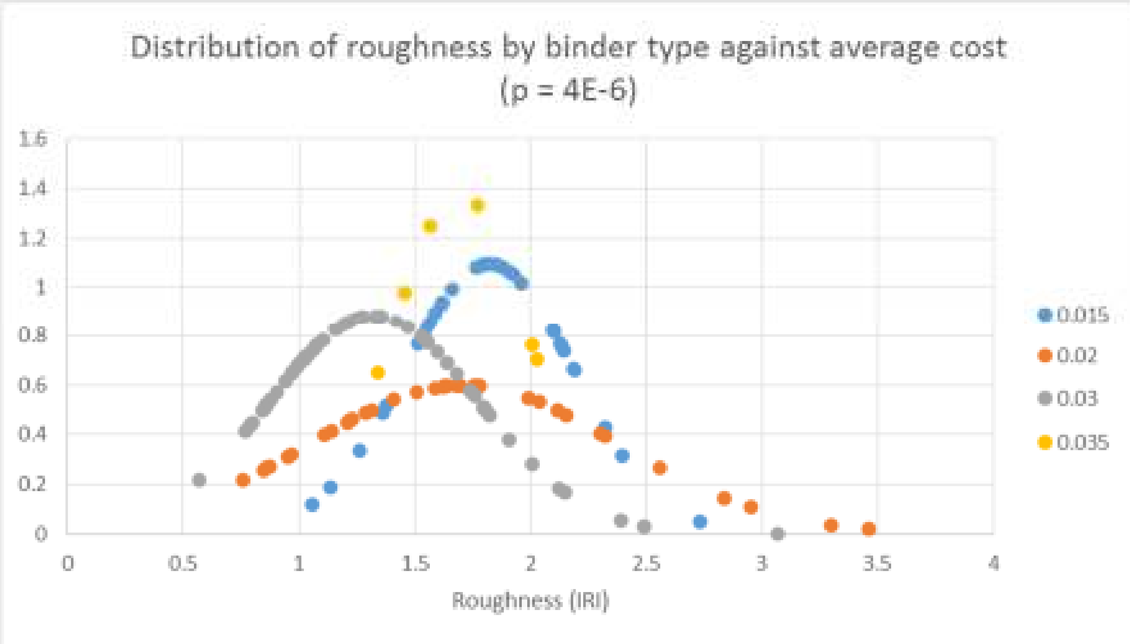


Binder content

Binder content and average cost do not appear to form a linear relationship, indicating a wider range of factors involved in influencing roughness for CMB pavements.

Average cost

Figure C 22: Distribution of roughness grouped by binder type against average cost

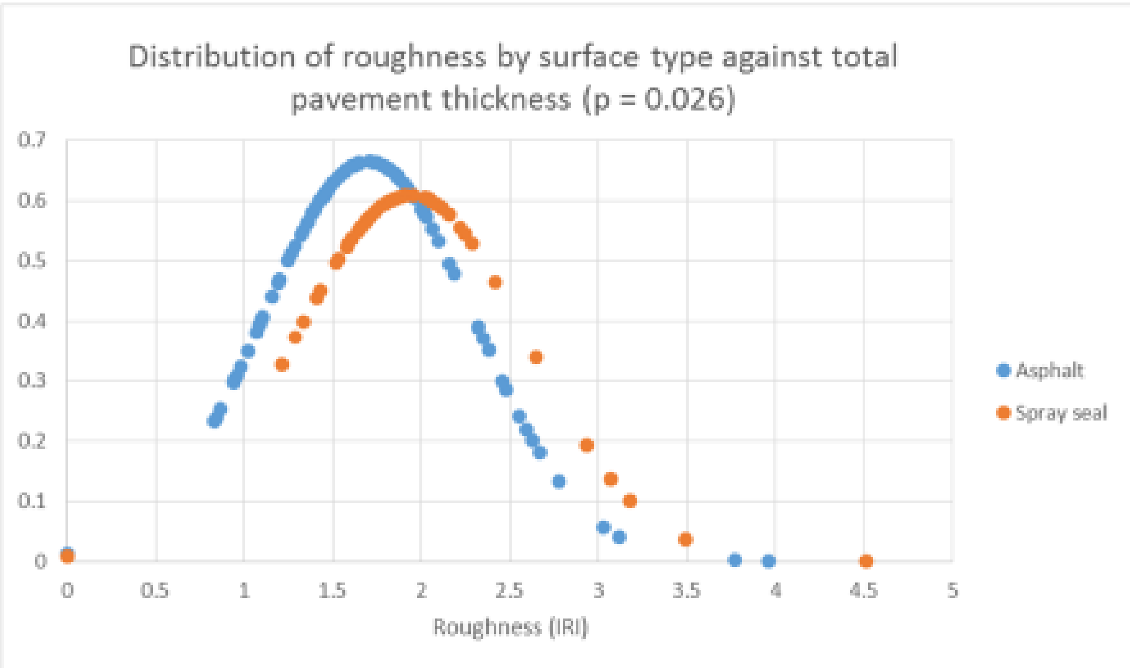


Surface type

The surface type, as with rutting, displays the same relationship where spray seal roughness is higher than asphalt roughness in all cases. The exception is modulus, which has far fewer sample points than average cost or total pavement thickness.

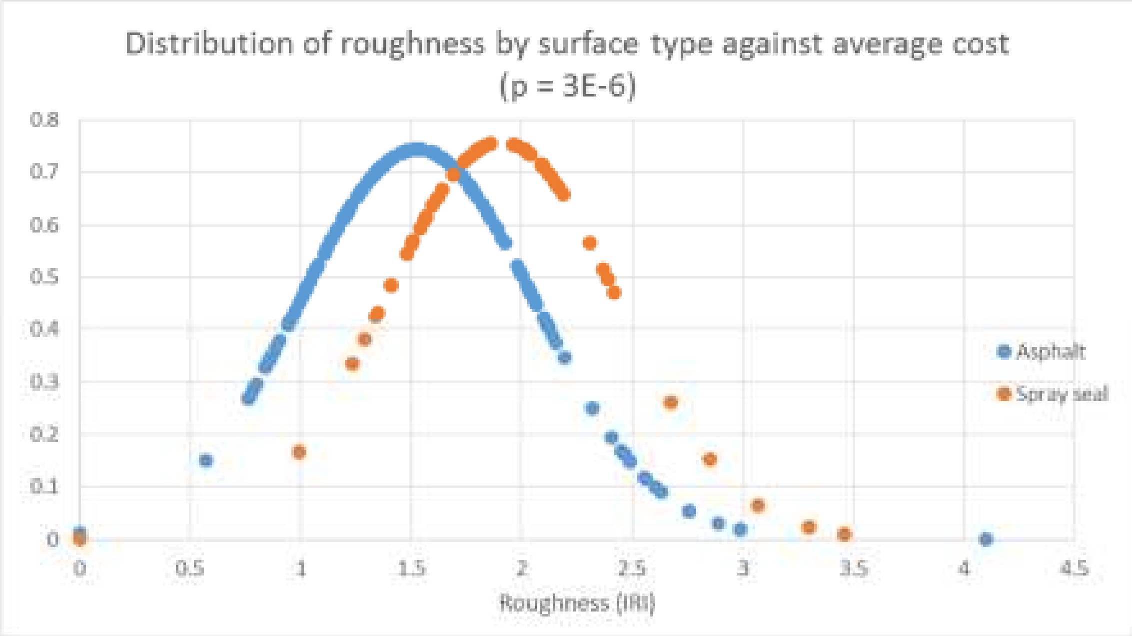
Total pavement thickness

Figure C 23: Distribution of roughness grouped by surface type against total pavement thickness



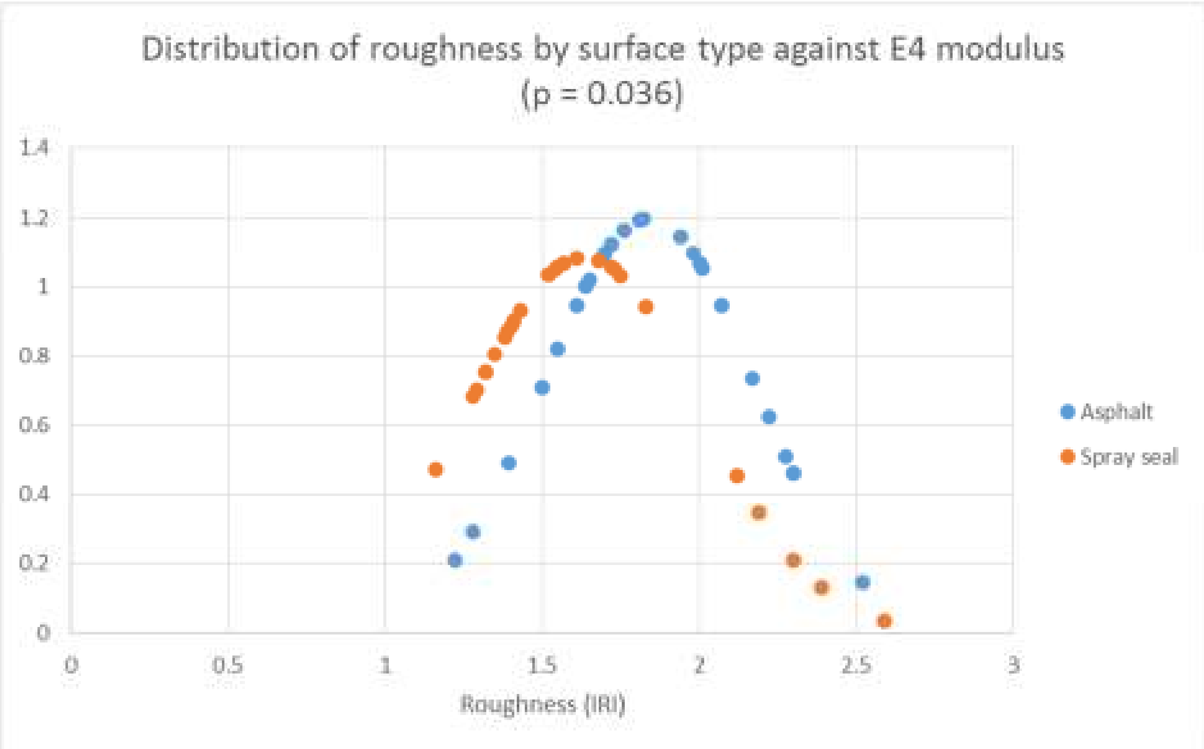
Average cost

Figure C 24: Distribution of roughness grouped by surface type against average cost



E4 Modulus

Figure C 25: Distribution of roughness grouped by surface type against E4 modulus

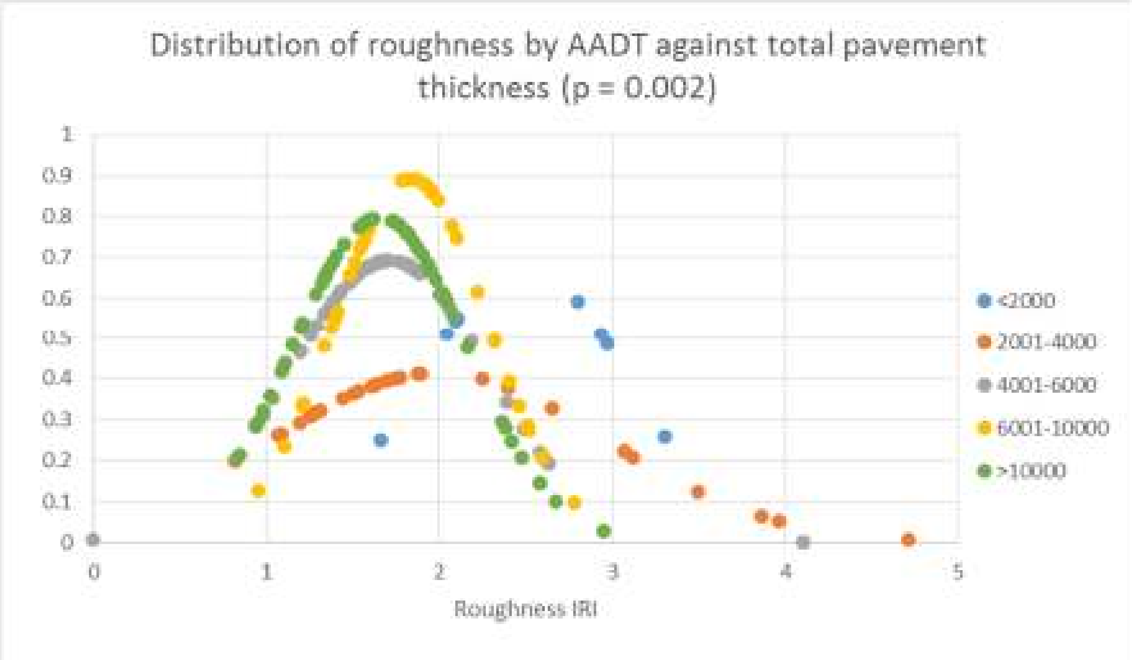


AADT

The distributions for AADT shown in Figure C 26 through to Figure C 29 all have very similar relationships. The lowest roughness mean belongs to the category with the highest AADT values, while the distribution with the greater spread belongs to the mid to low AADT bins.

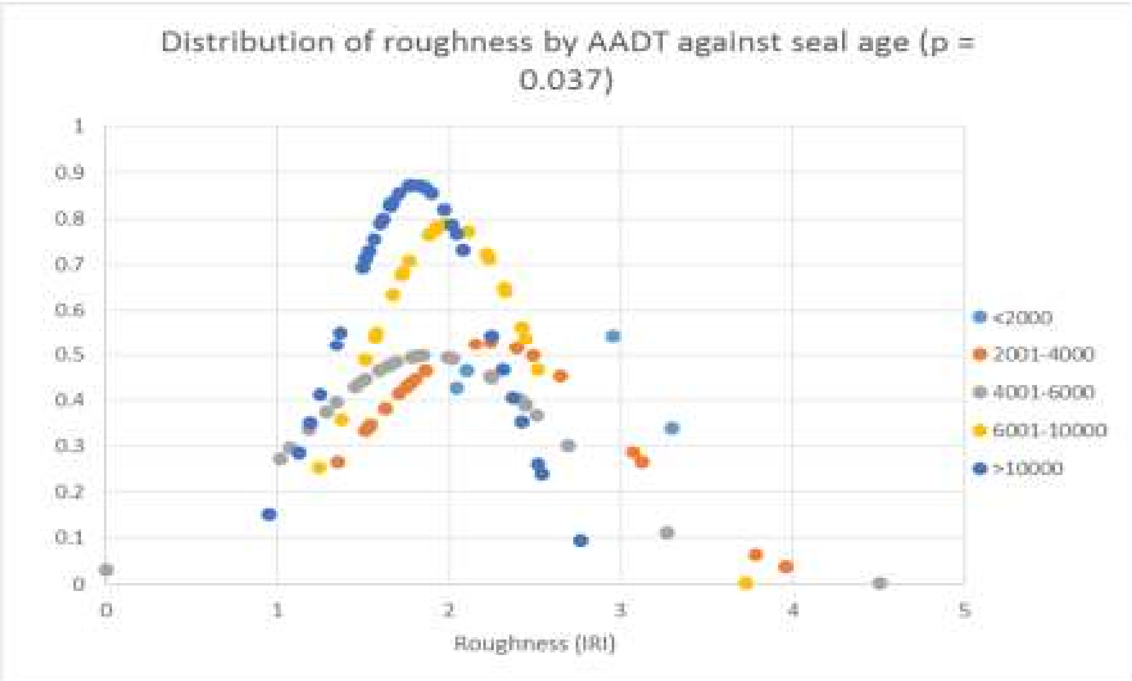
Total pavement thickness

Figure C 26: Distribution of roughness grouped by AADT bins against total pavement thickness



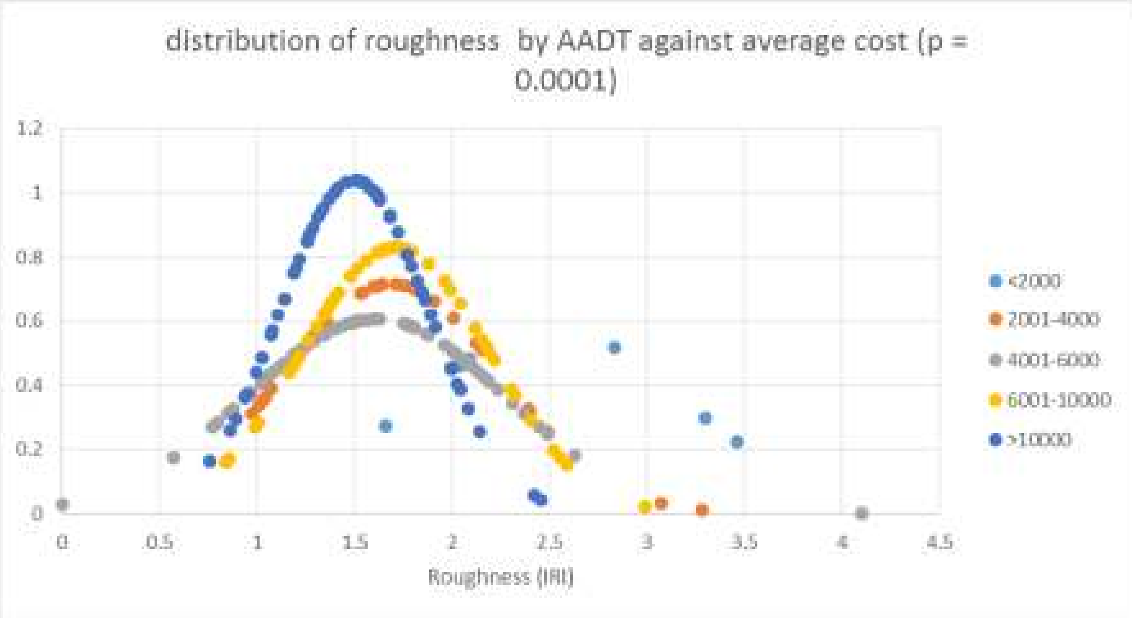
Surfacing age

Figure C 27: Distribution of roughness grouped by AADT against seal age



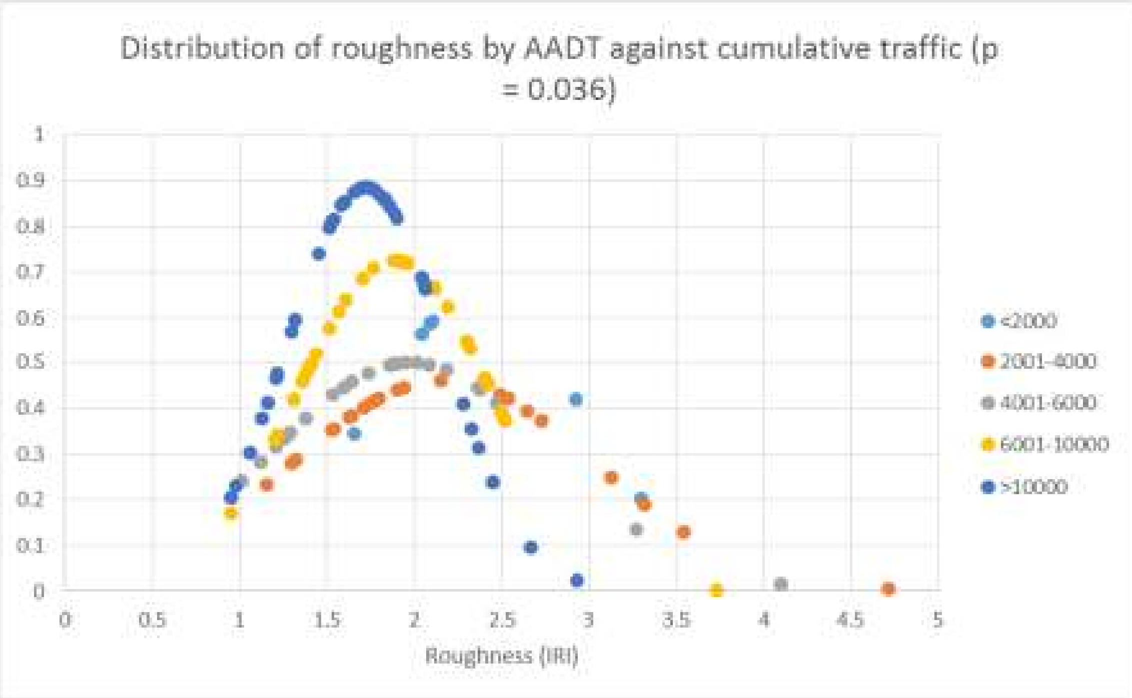
Average cost

Figure C 28: Distribution of roughness grouped by AADT against average cost



Cumulative traffic

Figure C 29: Distribution of roughness grouped by AADT against cumulative traffic



C.2 PM-CMB Sensitivity Analysis

C.2.1 Rutting Results

The influential factors found to affect rutting for PM-CMB were:

- vertical strain
- normalised deflection curvature
- modulus
- average cost
- surfacing age
- cumulative traffic.

These factors interact with the nominal categories in different ways, with the greatest number of influential factors found to interact with total pavement thickness bins and AADT bins, as is the case with FBS rutting. Linear regression was conducted for these two categories and the correlations are outlined in Table C 3 and Table C 4. Extremely weak correlations (where the coefficient is much closer to zero compared to the others) can be considered to have no correlation. The relationships described in the following sections are not considered strong, as most have R2 values much less than 0.7. The levels with the greater number of sample points also contained the weaker correlations; thus when they are resolved into further levels, it is expected that the correlations would be stronger.

Table C 3: Correlations between rutting for total pavement thickness bins and influential continuous variables

Total pavement thickness bins vs.	Influential factors					
	Surfacing age	Vertical strain	Normalised deflection curvature	Cumulative traffic	Average cost	E2 modulus
< 200	N/A	N/A	N/A	N/A	None	N/A
201–400	None	Positive	None	None	None	Negative
401–600	Positive	None(1)	None(3)	None(4)	None	Negative(2)
601–800	Positive	Negative(2)	Negative(2)	Positive	None	Positive(5)
800 +	None	None	N/A	Negative(2)	None	N/A

1 Surfacing ages past 20 years were excluded from dataset.
2 Weak negative.
3 Could be positive: cluster concentrated near the lower end with an outlier.
4 Weak positive with no points below a line.
5 Clustered under 5000 MPa.

The sample sizes for each level did not have consistent sample sizes or consistent range of rutting over the range of the continuous variables against which they were plotted. The average cost relationships all show no correlations for any of the categories, which indicates that the relationship between total thickness bins and average cost of maintenance does not appear to be linear in any way. The rutting for surfacing age correlated as expected for the pavement thickness bins 401–600 and 601–800; however, overall the continuous variables appear to have a tenuous relationship to total pavement thickness where linearity is concerned and may thus be good candidates for sensitive analysis.

Table C 4: Correlations between rutting for binder type and influential continuous variables

Binder type vs.	Influential factors				
	Surfacing age	Vertical strain	Normalised deflection curvature	Average cost	E2 modulus
0.01	Positive(2)	Negative(2)	Positive(1)	Positive(1)	Negative(2)
0.015	Positive(1)	Positive	Positive(1)	Positive	Negative
0.02	Positive(1)	Positive	Positive	NA	Negative
0.025	NA	NA	NA	None	NA
0.03	Negative(2)	Positive	Positive	None	Negative

1 Weak positive.
2 Weak negative.

For Table C 4, the E2 modulus levels at 0.03 have sample points that clustered under 2000 MPa for binder types and display some weak linearity when grouped by binder types. The results are mixed for all the continuous variables except for normalised deflection curvature and E2 modulus where the behaviour of rutting was as expected. For vertical strain, the rutting for binder type 0.01 clustered under 400 με, and for 0.015 they clustered under 100 με.

Table C 5 has relationships for rutting when grouped by surface type similar to that of the binder type.

Table C 5: Correlations between rutting for surface type and influential continuous variables

Surface type vs.	Influential factors			
	Vertical strain	Normalised deflection curvature	Cumulative traffic	Average cost
Asphalt	Positive(1)	Positive(1)	Positive(1)	None
Spray seal	Positive	Positive	Negative	None

1 Weak positive.

Table C 6 shows relationships for rutting when grouped by AADT bins similar to that of the binder type.

Table C 6: Correlations between rutting for AADT bins and influential continuous variables

AADT bins vs.	Influential factors					
	Surfacing age	Vertical strain	Normalised deflection curvature	Cumulative traffic	Average cost	E2 modulus
< 2000	None	NA	NA	Negative	Negative	NA
2001–4000	Positive(1)	Positive	Positive	Positive	Negative(2)	Negative
4001–6000	None	Positive	Positive(1)	Negative(2)	Positive(1)	Positive(1)
6001–10 000	Positive(1)	Positive	Positive	Positive(1)	None	Negative
> 10 000	Positive(1)	None	Positive(1)	None	None	Negative(2)

1 Weak positive.
2 Weak negative.

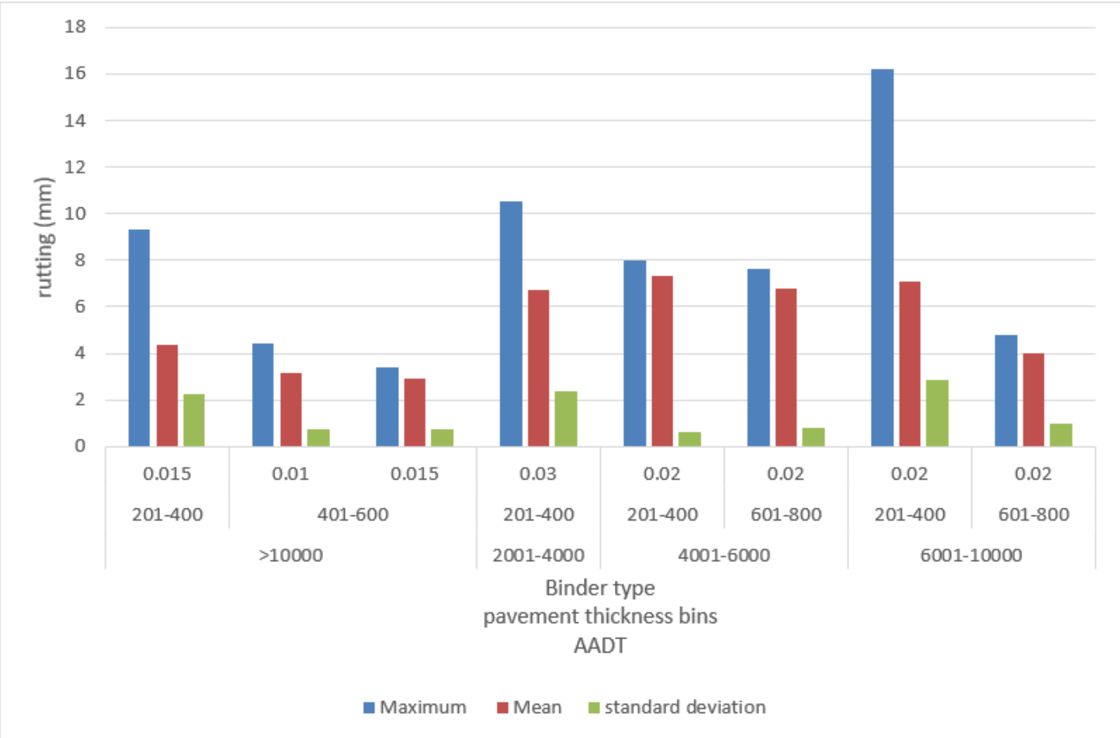
The AADT bins used for the CMB case are more refined than those used for FBS. As such, the sample points are more evenly distributed between the levels. The rutting against vertical strain, normalised deflection curvature, surfacing age and modulus mostly behaves as expected. The cumulative traffic is mixed and the average cost is also mixed; however, there were very few data points in the bins with negative correlations and one of them has an obvious outlier which skewed the regression. As for average cost, the < 2000 bin did not have many data points and they were mostly clustered under 500.

The sensitivity analysis for rutting for CMB will resolve the rutting groups into AADT, binder type, total pavement thickness and surface type against cumulative traffic, vertical strain and average cost.

The statistical description for rutting vs. normalised deflection curvature is shown in Figure C 30. Resolving the data points by AADT and pavements, it is clear that there are only four types of binders: 0.01, 0.015, 0.02 and 0.03. The following was observed:

- For AADT of 2001–4000, there is only one pavement thickness bin (201–400 mm) and one binder used (0.03).
- The maximum rutting is 16 mm for the 0.02 binder type on a 201–400 mm pavement thickness in a high volume traffic area (6001–10000 AADT).
- The lowest rutting is found for a binder type of 0.015, on pavement thicknesses 401–600 mm, and on a very high volume traffic area with AADT > 10 000.
- The thickest pavements are not used for the > 10 000 AADT road segments. The binder type also appears to decrease for the > 10 000 AADT, with only binders 0.015 and 0.01 used.

Figure C 30: Statistical description of rutting vs. normalised deflection curvature resolved in AADT, pavement thickness and binder type



The statistical description for the total population of rutting for CMB pavements is shown in Figure C 31. The following was observed:

- The highest maximum rutting at over 25 mm was found for pavement thickness in the 201–400 mm bin, in the 4001–6000 AADT bin for the 0.015 binder type.

- For pavement thickness in the < 200 level, the highest maximum rutting is over 10 mm for the 2001–4000 AADT bin. There is only one binder type in this level: 0.03.
- For pavement thickness bin 401–600, the highest maximum rutting is 15 mm for AADT < 2000 corresponding to binder type 0.02. There are two binder types for this AADT category, the other one is 0.01.
- For pavement thickness bin 601-800, the highest maximum rutting is around 8 mm for AADT of 6001–10 000 (highest AADT category in this thickness bin) corresponding to binder type 0.015. There are two binder types for this AADT category, the other one is 0.02.
- The highest maximum rutting is around 11 mm in the > 800 pavement thickness bin for an AADT of < 2000. The binder type is 0.02.
- There appears to be little consistency or pattern for the application of a particular binder type based on AADT or pavement thickness, except that:
  - for all pavement thicknesses for AADT < 2000, the 0.02 binder is used
  - for 2001–4000 AADT, the 0.03 binder is used, but for pavement thickness 201–400 mm binder type 0.015 is also used.
- The 4001–6000 AADT bin has the greatest variability of binder types used, particularly for 201–400 and 401–600 pavement thickness bins.

The rutting vs. vertical strain when resolved into binder types, AADT and pavement thickness is shown in Figure C 32. There are five sublevels that have rutting over 10 mm:

- pavement thickness < 200, AADT 2001–4000 and binder type 0.03
- pavement thickness 201–400, AADT 2001–4000 and binder type 0.03
- pavement thickness 201–400, AADT 4001–6000 and binder type 0.015
- pavement thickness 201–400, AADT 6001–10000 and binder type 0.02
- pavement thickness 401–600, AADT < 2000 and binder type 0.02
- pavement thickness 401–600, AADT 6001-10 000 and binder type 0.03.

Figure C 31: Statistics for total population of rutting by binder type, AADT, pavement thickness

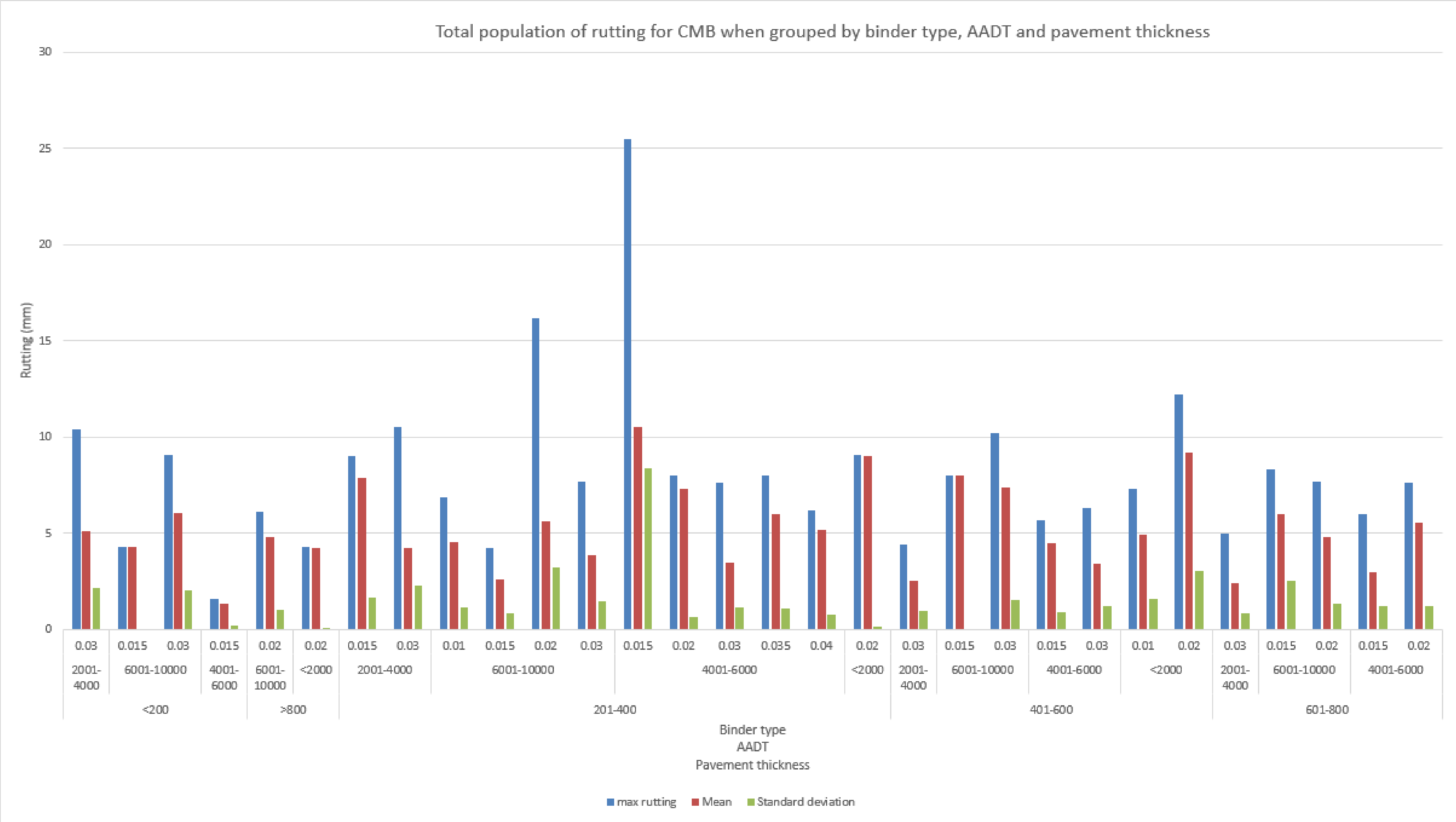


Figure C 32: Statistics for rutting by binder type, AADT, pavement thickness against vertical strain

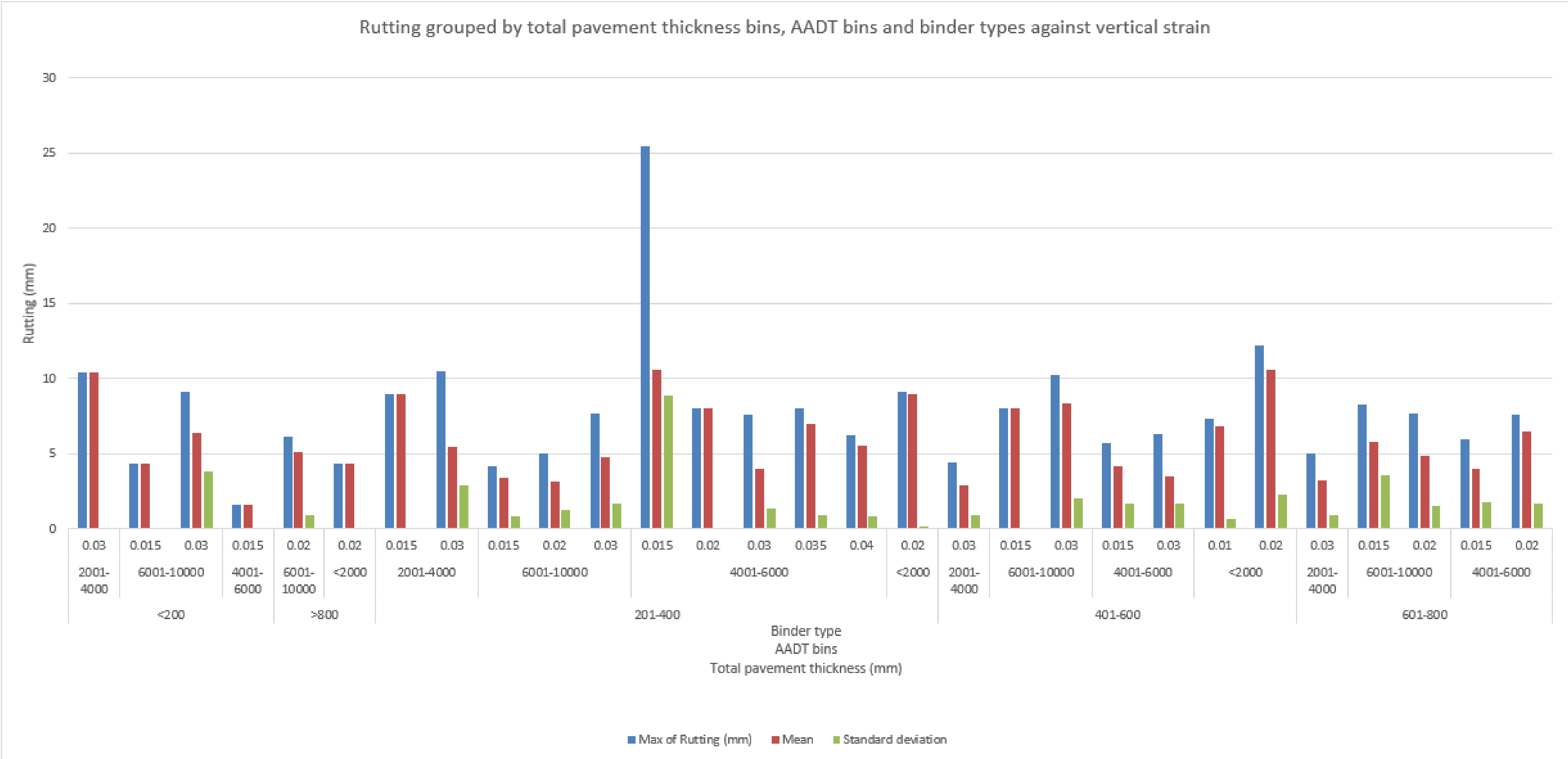


Figure C 33: Statistics for rutting by AADT and pavement thickness against cumulative traffic

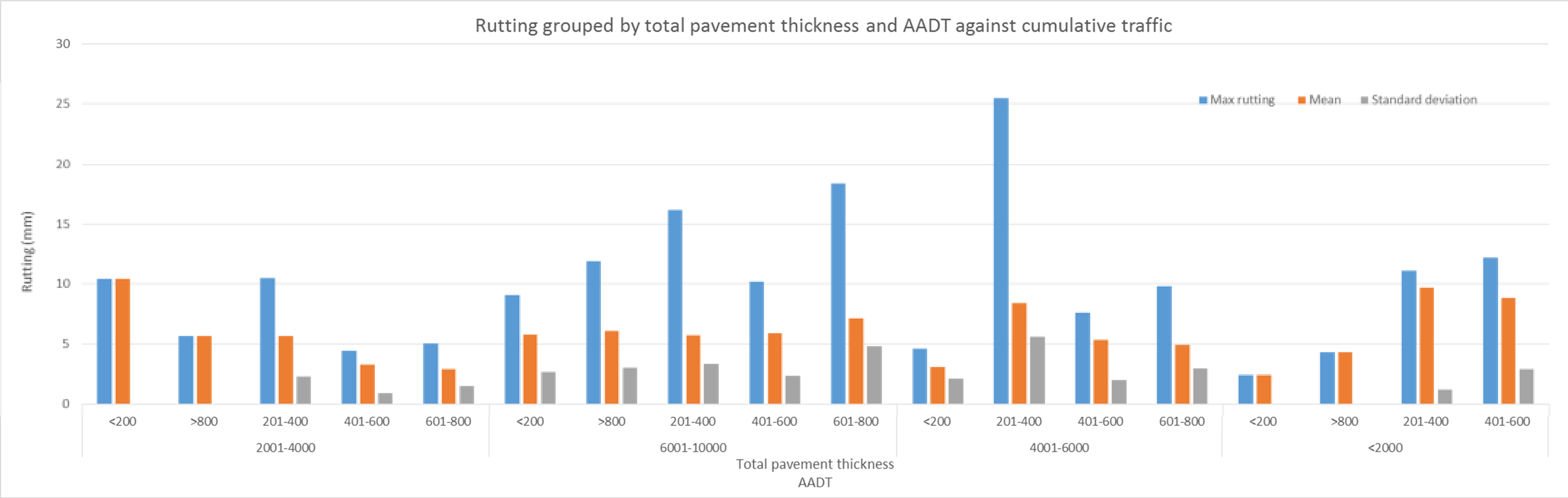


Figure C 34: Statistics for average cost vs. cumulative traffic resolved in AADT and pavement thickness bins

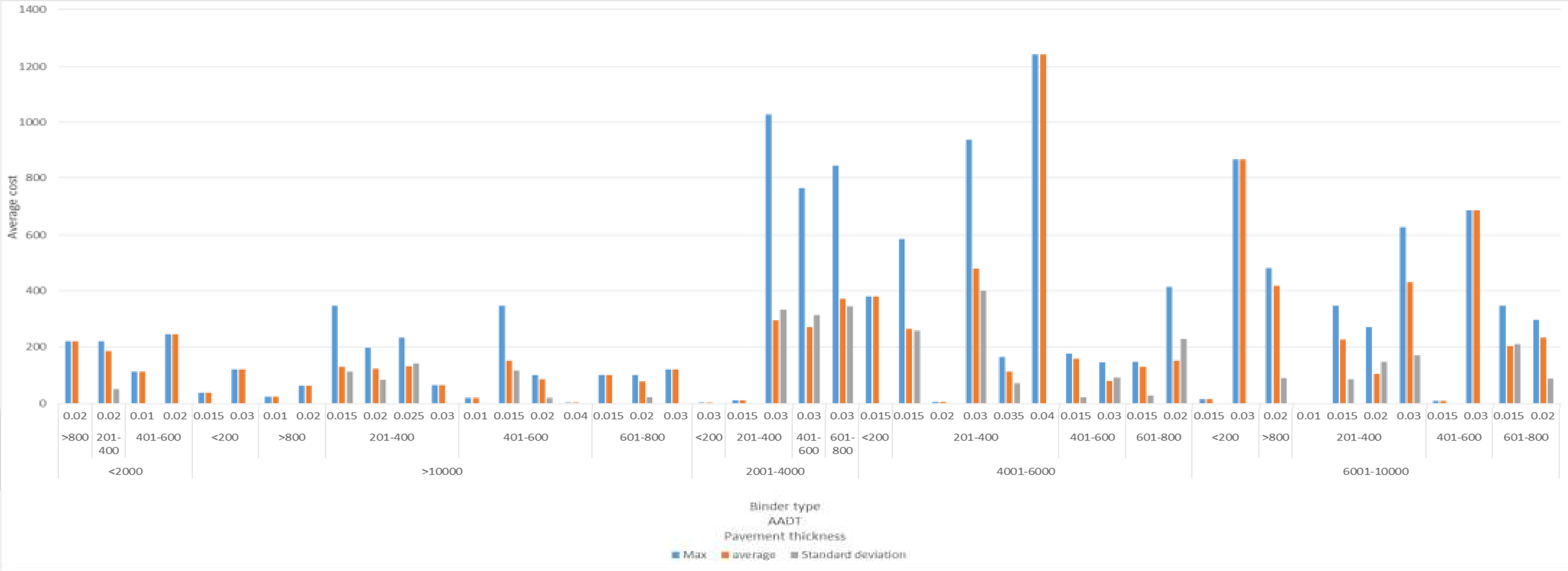
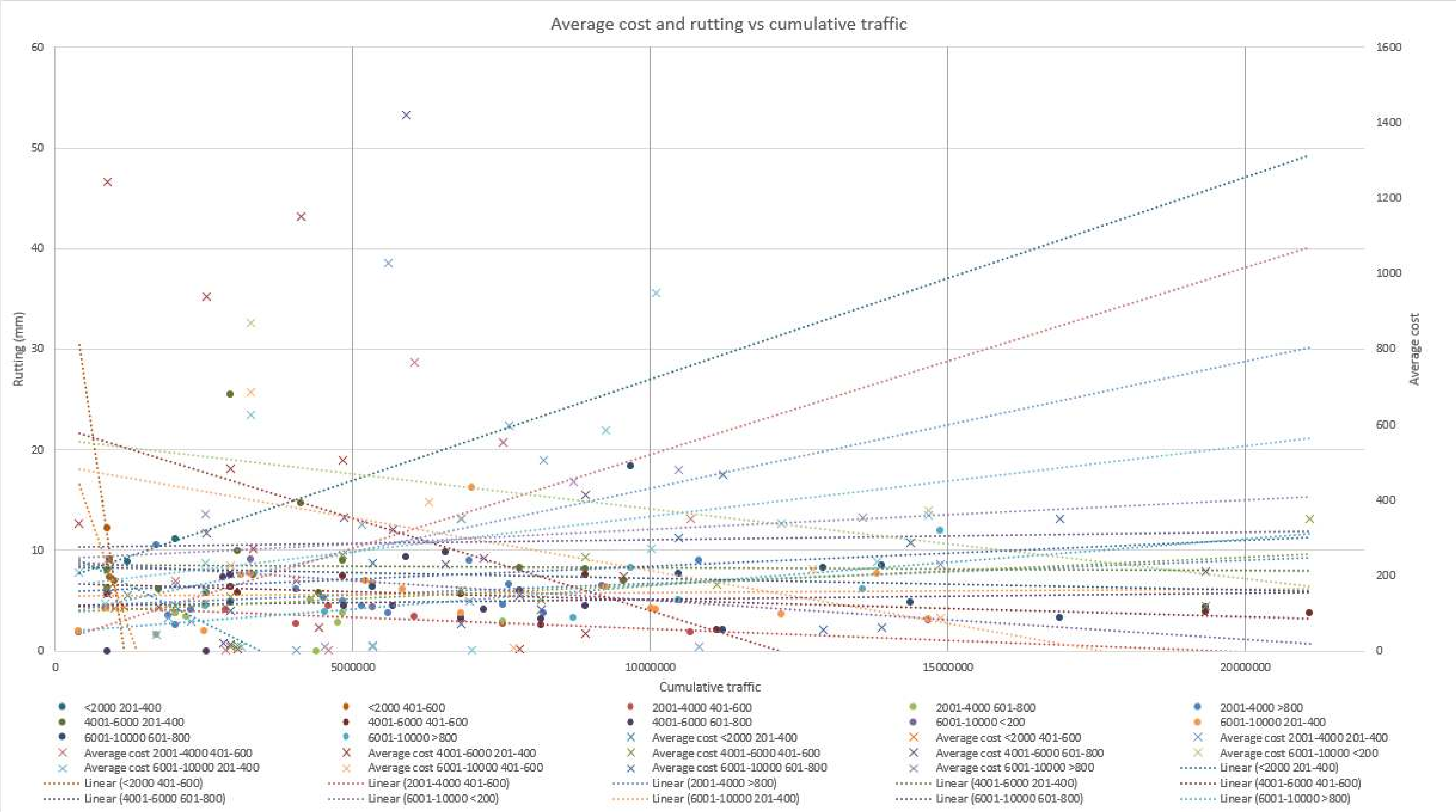


Figure C 35: Average cost and maximum rutting grouped by AADT and pavement thickness against cumulative traffic



C.3 Roughness Results

The influential factors found to affect roughness were:

- modulus
- total pavement thickness
- average cost
- surfacing age
- cumulative traffic.

These factors interact with the nominal categories in different ways, with the greatest number of influential factors found to interact with surface types and AADT. Linear regression was conducted for these two categories and the correlations are outlined in Table C 7 and Table C 8. The relationships described in the following sections are not considered strong, as most have R2 values much less than 0.7. The levels with the greater number of sample points also contained the weaker correlations; thus when they are resolved into further levels, it is expected that the correlations would be stronger.

Table C 7: Correlations between roughness and AADT bins and influential continuous variables

Surface type vs.	Influential factors		
	Total pavement thickness bins	Average cost	E4 modulus
Asphalt	None	None	None
Spray seal	None	None	None

Table C 8: Correlations between roughness and surface type and influential continuous variables

AADT bins vs.	Influential factors			
	Surfacing age	Total pavement thickness	Cumulative traffic	Average cost
< 2000	Positive	Positive	Negative(2)	Positive(1)
2001–4000	Positive	Negative	Positive(1)	None
4001–6000	Positive(1)	None	None	None
6001–10 000	Positive(1)	None	Positive(1)	Negative(2)
> 10 000	Positive	None	Positive(1)	None

1 Weak positive.
2 Weak negative.

The < 2000 AADT level contained very few sample points against all its influential factors. The average cost factor for the < 2000 AADT bin had the sample points clustered under 400. There appeared to be no correlation between roughness and any of the influential factors, when grouped by surface type, which could be due to there being two populations and large sample sizes. To better understand the relationship, the roughness would need to be resolved against AADT or binder types. The statistical descriptions of surface type resolved by AADT against average cost is shown in Figure C 36.

Figure C 36: Statistical description of roughness vs. average cost grouped into surface type, AADT and Binder type

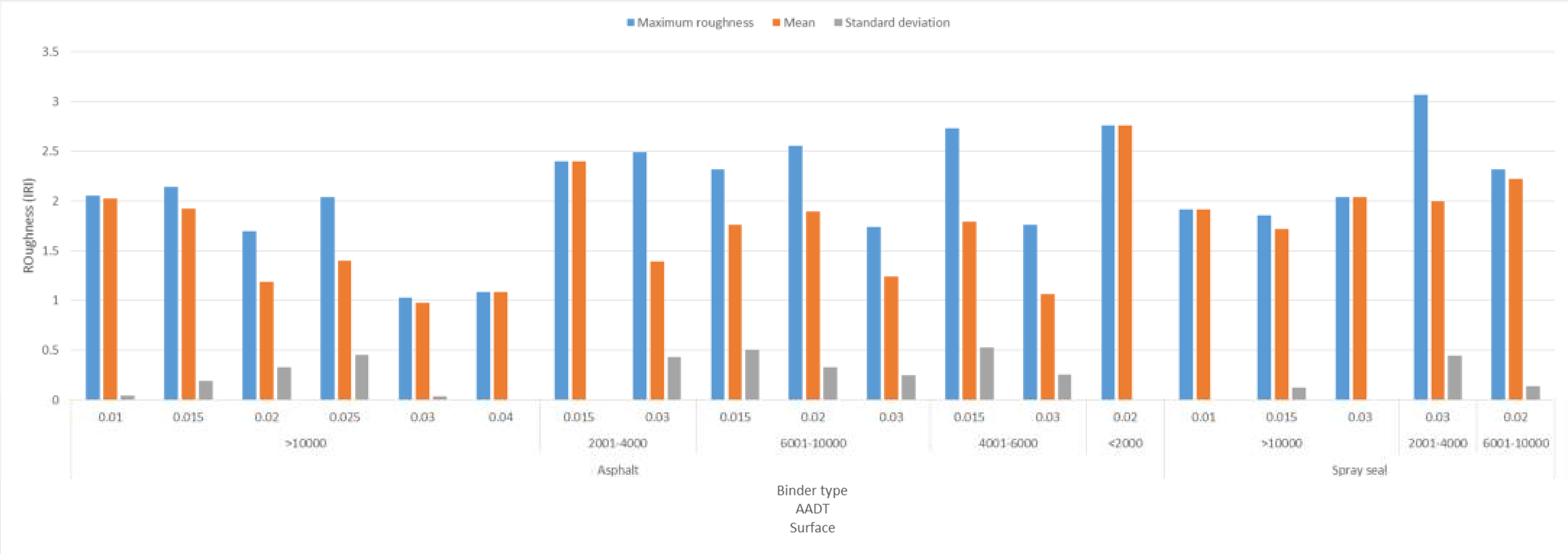
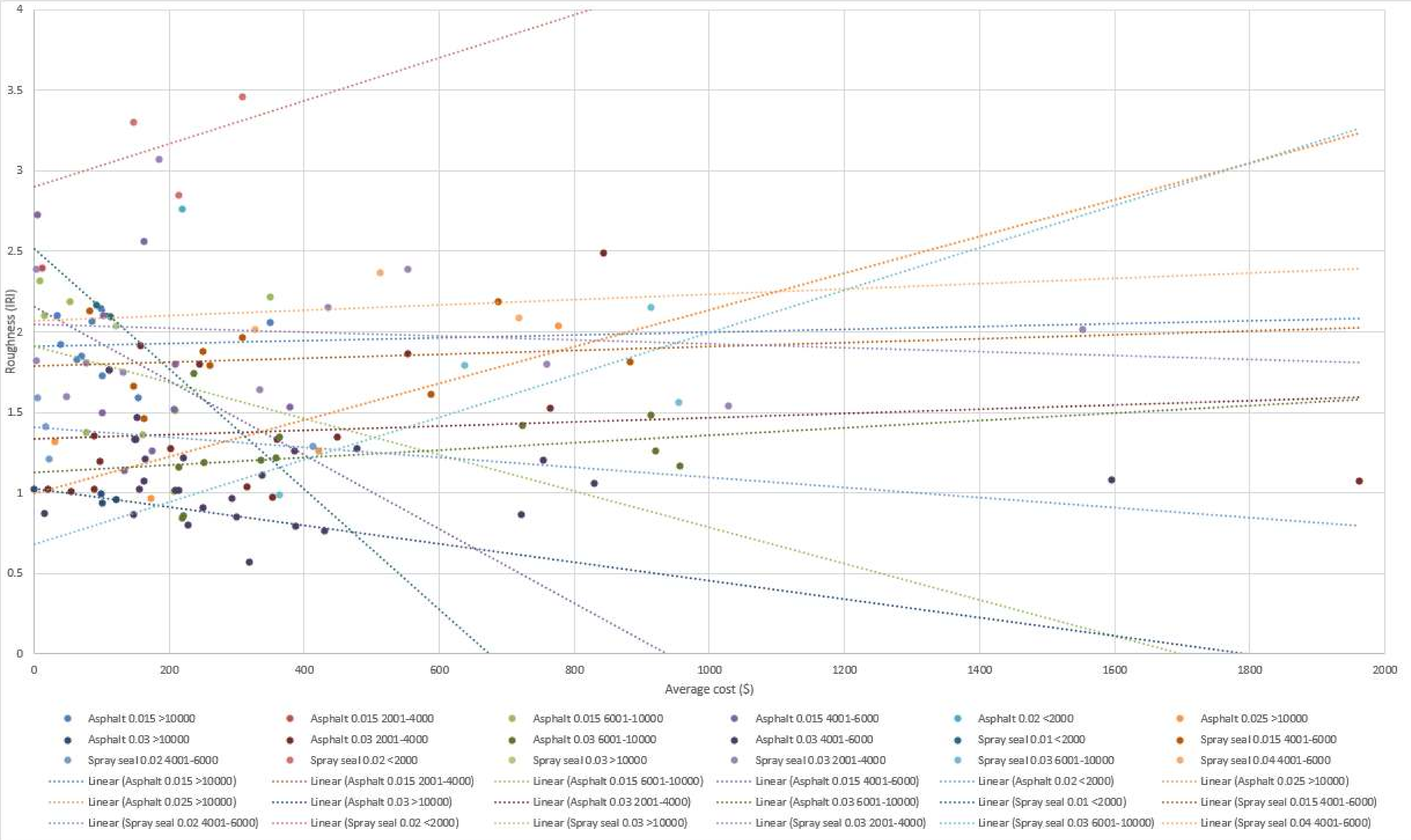


Figure C 37: Average roughness vs. average cost grouped by surface type, AADT and binder type



The roughness for CMB stabilised pavements, when grouped by surface type, AADT and binder type is shown in Figure C 37. The following was noted:

- The spray seal category has fewer sample groups for binder and rutting, with the binder type ranging from 0.01-0.03.
- For both surfaces, the > 10 000 AADT category had the most binder types.
- For asphalt, the mean roughness decreases linearly when the binder type increases; however, no pattern exists for spray seal surfaces.
- The sublevel with the maximum roughness, where there is more than one binder type:
  - asphalt: AADT > 10 000, BT 0.01
  - asphalt: AADT 6001–10 000, BT 0.02
  - asphalt: AADT 4001–6000, BT 0.015
  - asphalt: AADT 2001–4000, BT 0.03
  - spray seal: AADT > 10 000, BT 0.03.

C.4 I-FBS ANOVA

C.4.1 Rutting Results

The ANOVA method has found statistical significance between the levels of the nominal factors for rutting, for the continuous variables outlined in Table C 9.

Table C 9: P-value table for rutting

Nominal variables \ Continuous variables	Surfacing age	Vertical strain	Average normalised deflection curvature (µm)	Cumulative traffic	Modulus E1 (MPa)	Modulus E2 (MPa)	Subgrade modulus E5 (MPa)
Environmental zones			0.022				
Total pavement thickness		1.6E-5	6.5E-4		3.6E-3	0.001	0.002
Binder types			4.0E-4		4.9E-3	3.9E-4	0.0004
Surface type	1.5E-6			4.6E-7			
AADT	0.013	1.3E-4	0.026		0.016	0.005	0.00890

Total pavement thickness bins

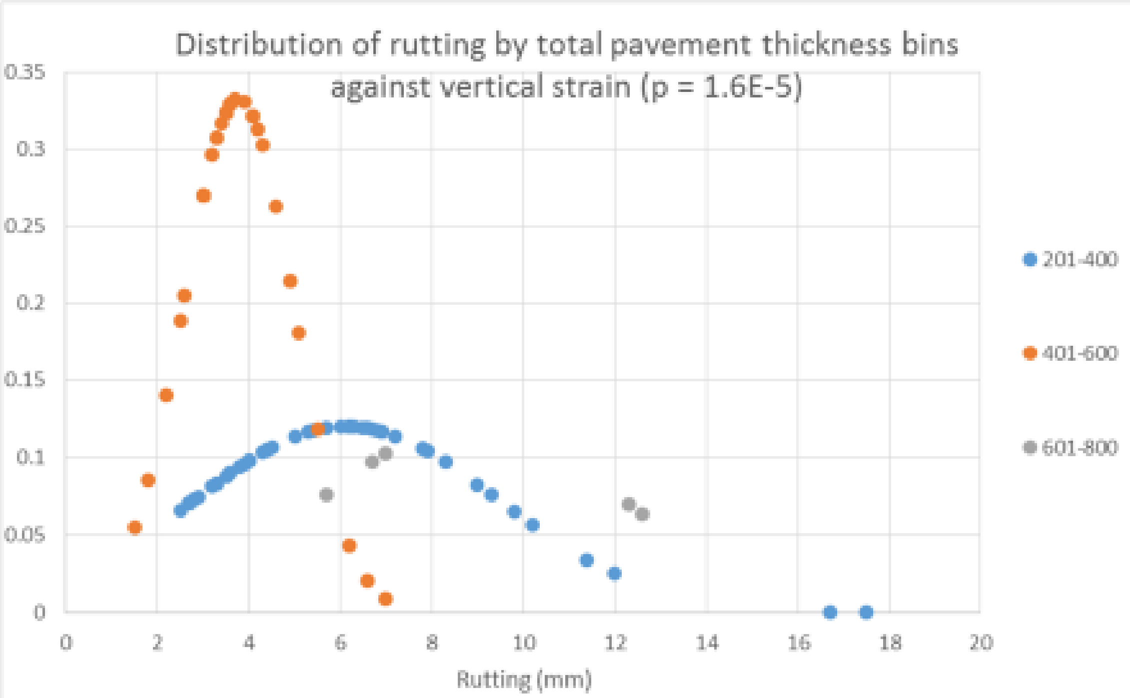
Total pavement thickness can be considered a continuous variable and an ordinal/nominal variable when grouped into bins of equal intervals. The intervals of 0–200, 201–400, 401–600, 601–800 and 800+ were chosen. Some categories were omitted due to insufficient sample sizes and could affect the ANOVA analysis considerably. Depending on the intervals chosen, the p-values may also differ.

The ANOVA analysis found there is a statistical significance between the different pavement thickness bins, for vertical strain, normalised deflection curvature and modulus, which indicates that the material properties are major influencing factors on the different categories of pavement thickness categories. This is expected as pavement thickness would probably have been chosen based on material properties.

Vertical strain

The rutting when grouped by pavement thickness bins and plotted against vertical strain produced statistically significant means. The distributions as shown in Figure C 38 indicate that greatest variance exists for the 201–400 mm and 401–600 mm level and also have the highest means in rutting, suggesting interactions with other factors such as environmental zone, binder types used, etc.

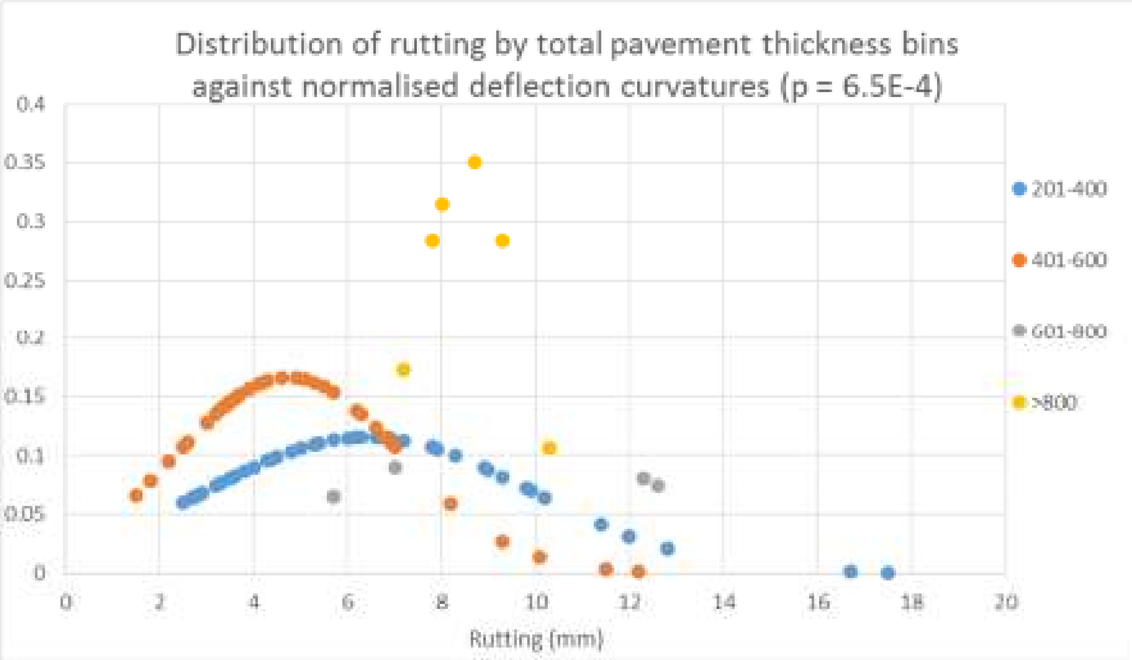
Figure C 38: FBS rutting distribution by total pavement thickness bins for vertical strain



Normalised deflection curvature

The rutting distribution for total pavement thickness against normalised deflection curvature is shown in Figure C 39. The 201–400 mm level appears to have the greatest spread, and a relatively high mean for rutting when compared to the 401–600 mm category. The 601-800 mm level has the fewest sample sizes giving rise to a large spread of rutting. There are maximum rutting of 16 mm and 18 mm are suspected outliers.

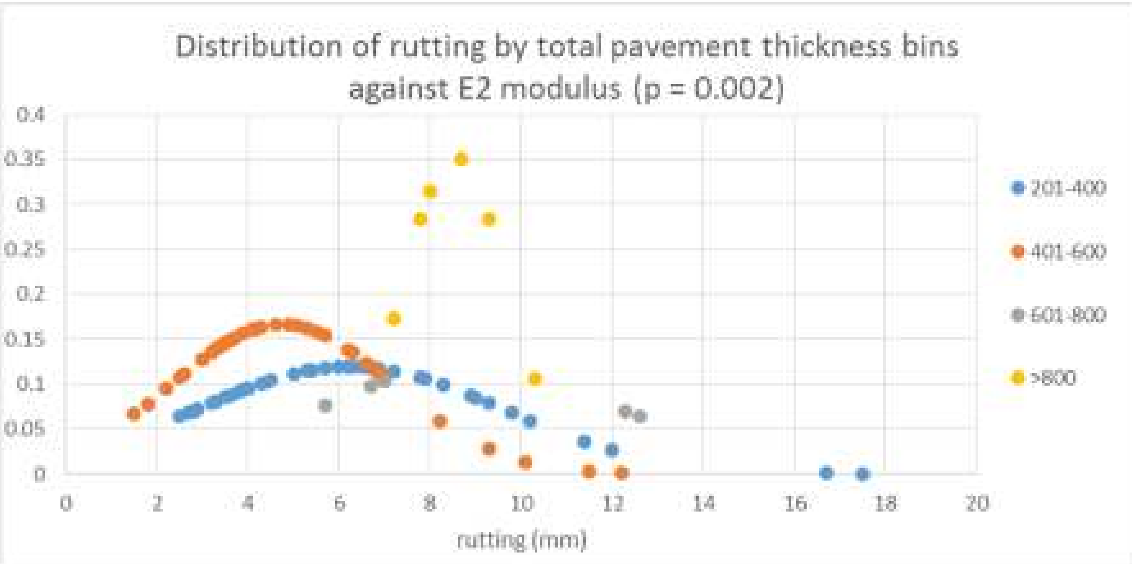
Figure C 39: FBS rutting distribution by total pavement thickness bins for normalised deflection curvature



Modulus

The distribution of rutting against the E2 modulus (see Figure C 40) is very similar to Figure C 39 implying that these variables may not be independent, i.e. the modulus may have been determined using the normalised deflection curvature.

Figure C 40: FBS rutting distribution by total pavement thickness bins for modulus

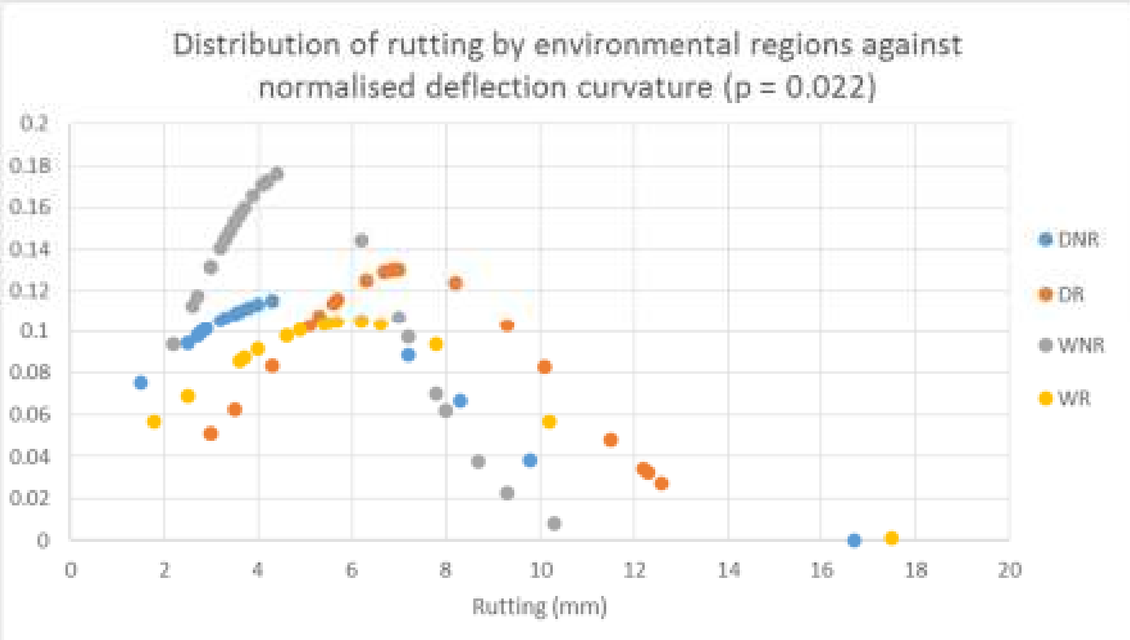


Environmental zones

The rutting, when grouped by environmental zones, seems to have a relationship with normalised deflection curvature. The distribution shows that the nonreactive regions have lower means for maximum rutting compared with the wet regions, and that the spreads are also lower. The distribution is shown in Figure C 41.

Normalised deflection curvature

Figure C 41: FBS rutting distribution by environmental zone for normalised deflection curvature



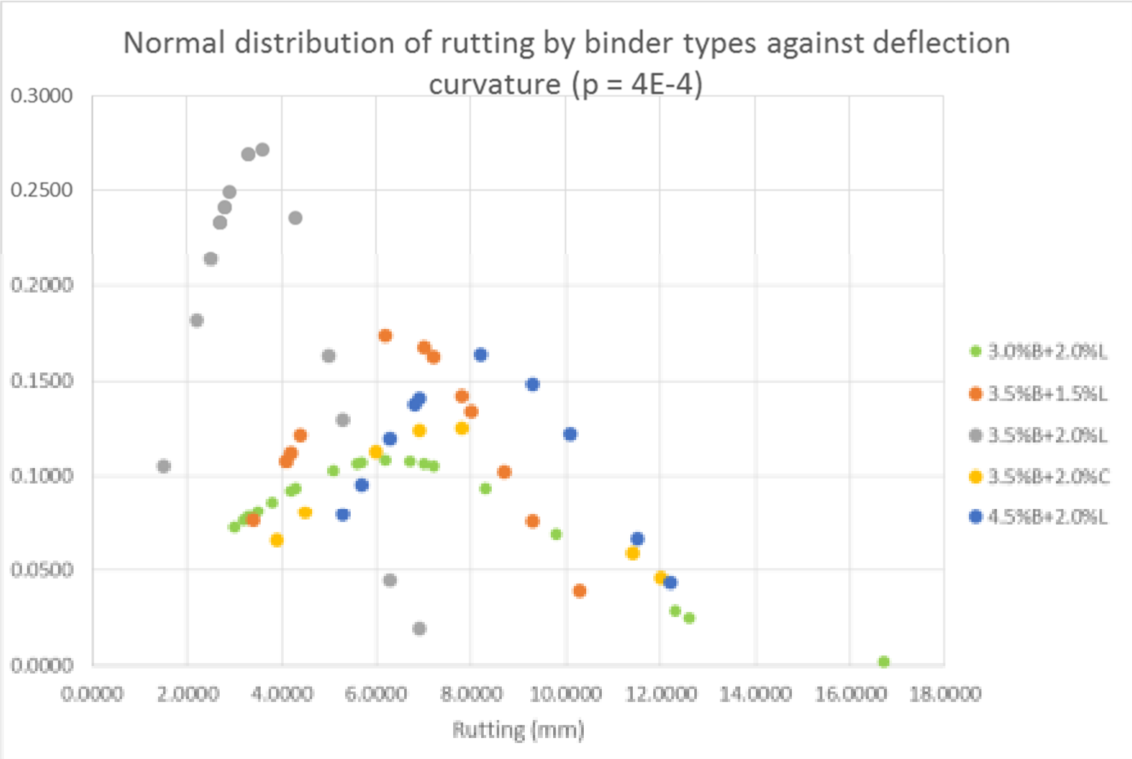
Binder types

The normal distributions of rutting when grouped by binder type can be seen in Figure C 42 and Figure C 43. The spreads of all but one of the distributions are relatively large, which indicates interactions with other factors. There appears to be a relationship between rutting, normalised deflection curvature and E2 modulus when the rutting is grouped by binder types. Binder type is a good candidate for sensitivity analysis to determine what other factors interact with this factor to influence rutting on FBS pavements.

Normalised deflection curvature

The greatest variance is observed for 3.0%B+2.0%L. The highest mean corresponds with the 4.5%B+2.0%L binder type, and also the greatest percentage of bitumen while 3.0%B+2.0%L has the lowest rutting mean (Figure C 42).

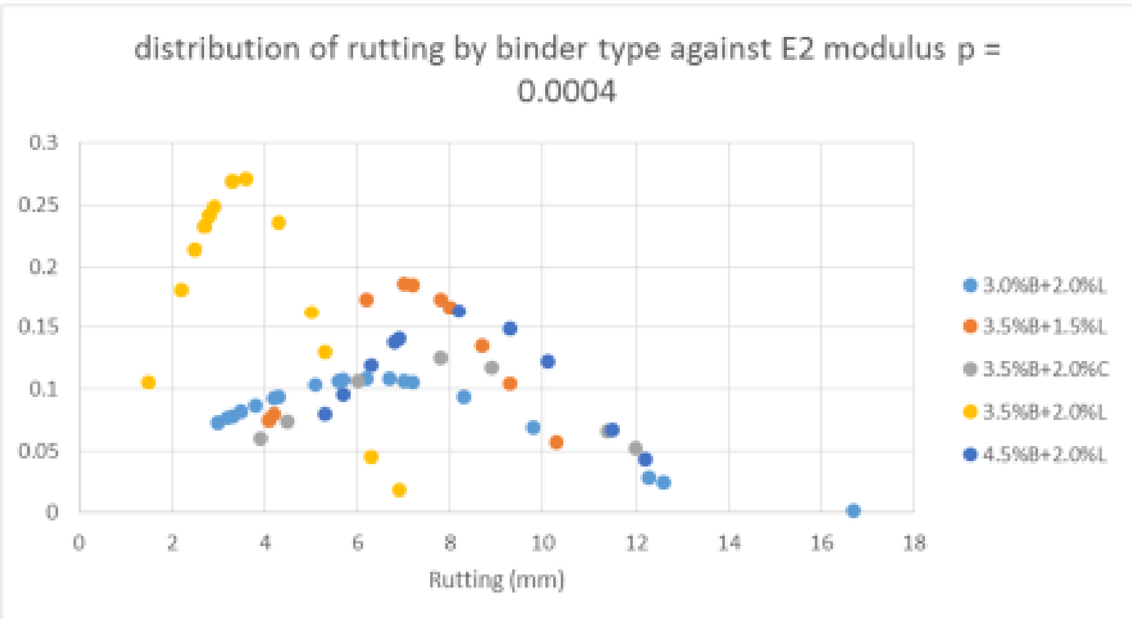
Figure C 42: FBS rutting distribution by binder type for normalised deflection curvature



E2 Modulus

The greatest variance is seen for 3.0%B+2.0%L. As demonstrated in earlier examples, the modulus resembles the deflection curvature to a great extent. As such, sensitivity analysis should be conducted using one or the other, but not both.

Figure C 43: FBS rutting by binder type for E2 modulus



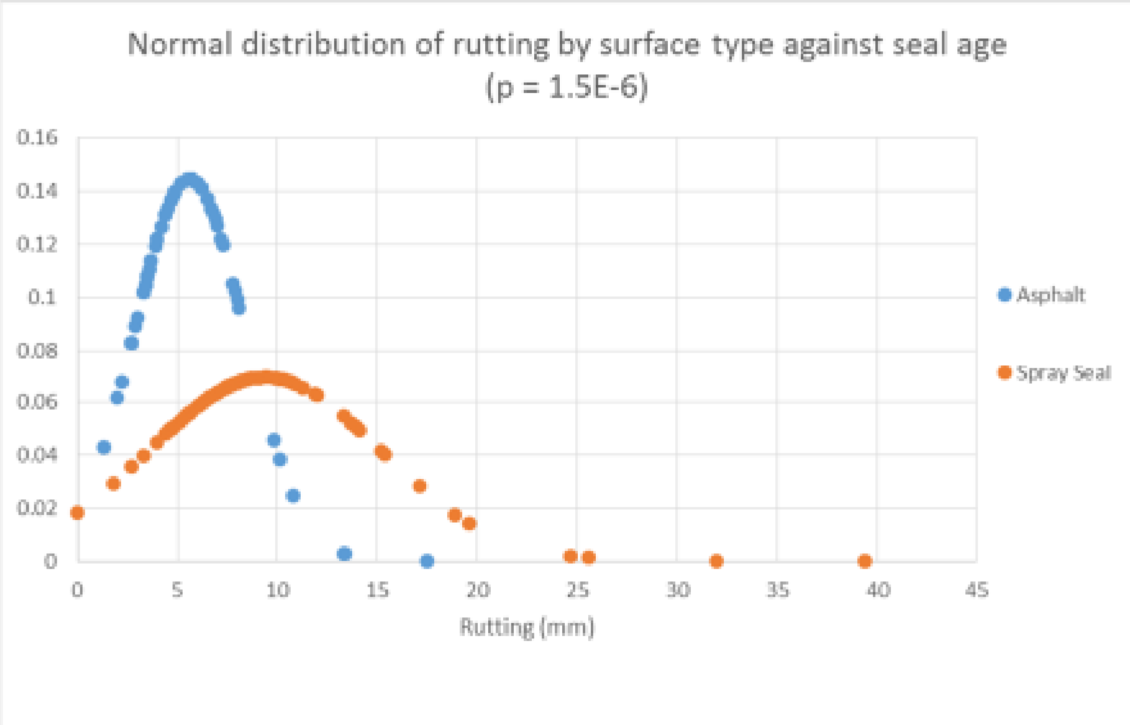
Surface type

Surface types have only two levels: asphalt and spray seal. This is due to very few sample points in the other categories. The main influencing factors on surface type was found to be the surfacing age and cumulative traffic. The distributions are shown in Figure C 44 and Figure C 45.

Surfacing age

The spray seal has greater spread of rutting and a higher mean compared with the asphalt, suggesting multiple interactions with other factors. Spray seals may be used in a wider variety of conditions and as such the rutting varies greatly.

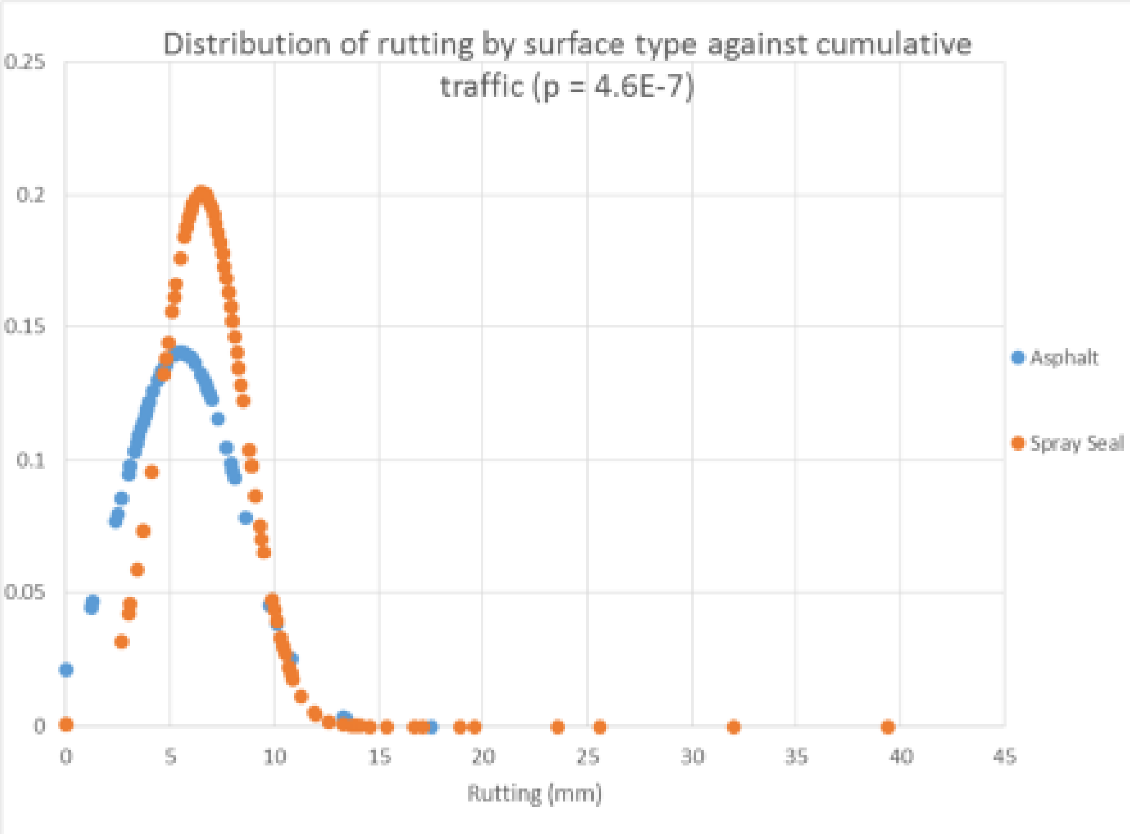
Figure C 44: FBS rutting distribution by surface type for surfacing age



Cumulative traffic

The mean of the rutting for the different surface types against cumulative traffic appears to be higher for spray seals as expected from the previous example. The spreads are very similar; however, the spread for the asphalt is shown to be slightly greater than the previous example suggesting that asphalt pavements may interact with other factors such as AADT.

Figure C 45: FBS rutting distribution by surface type for cumulative traffic



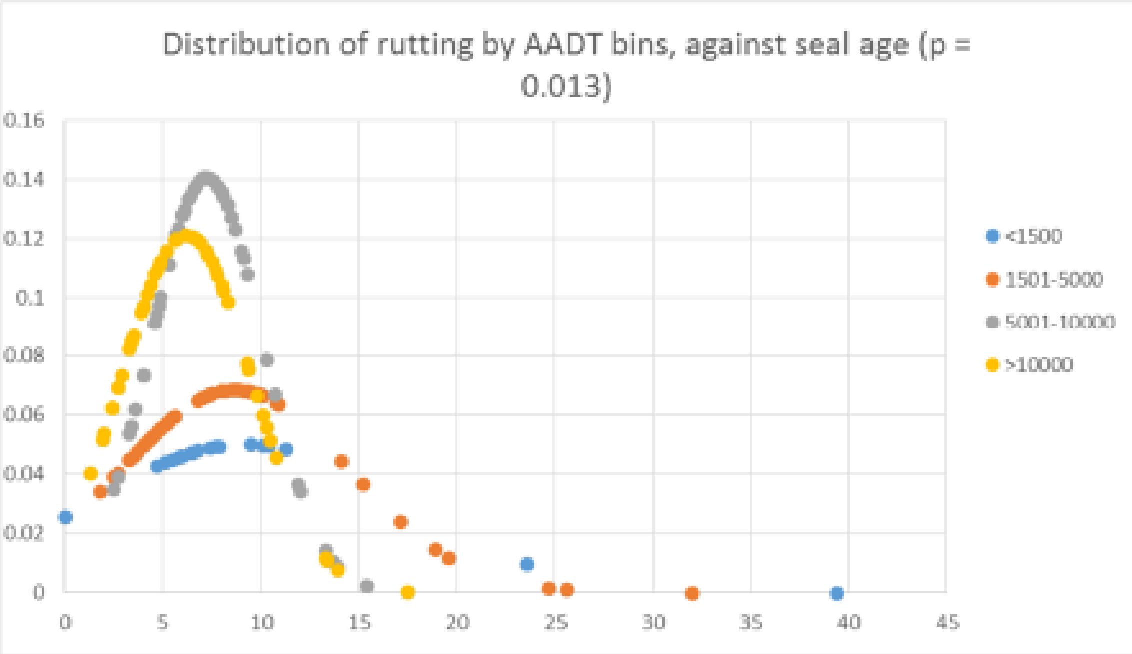
AADT bins

The AADT category could be considered an ordinal as well as continuous category; however, dividing the rutting into discrete AADT bins can provide some insight into the levels of AADT on rutting means, as rutting may not necessarily have a linear relationship with AADT. Figure C 46, Figure C 47, Figure C 48 and Figure C 49 show the distributions of rutting grouped by AADT. The number of sample points in each level appear to be the same as shown in Figure C 46 and compared with the other subsequent charts. This is because the strain, deflection and modulus were obtained from FWD data, which is more restricted.

Surfacing age

The AADT bin with the lowest mean rutting is > 10 000 indicating that pavements may have been designed for higher volume of traffic and may be sealed on a regular basis. Meanwhile, the greatest spread is observed for the < 2000 bin that also appears to have the highest mean rutting and an outlier rutting value close to 40 mm.

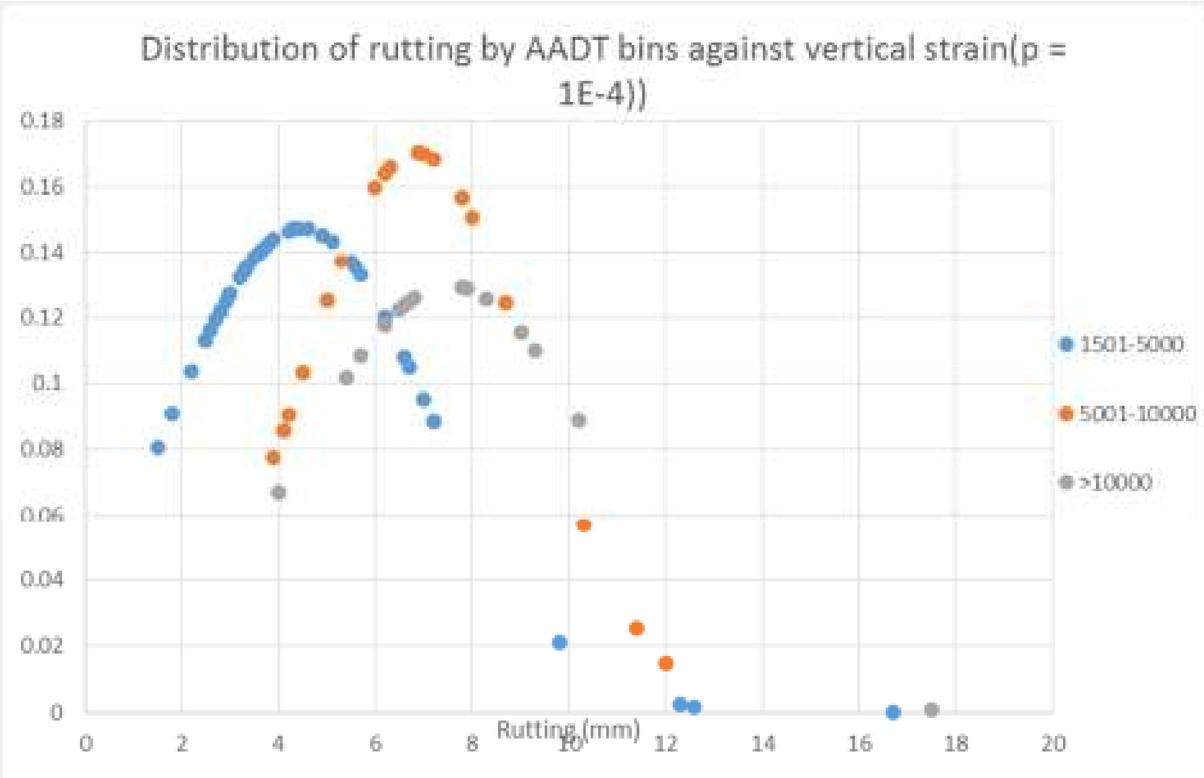
Figure C 46: FBS rutting distribution by AADT bins for surfacing age



Vertical strain

When measured against vertical strain, the mean rutting increases with increasing AADT suggesting that as AADT increases, the rutting increases and is also likely to be positively correlated with vertical strain.

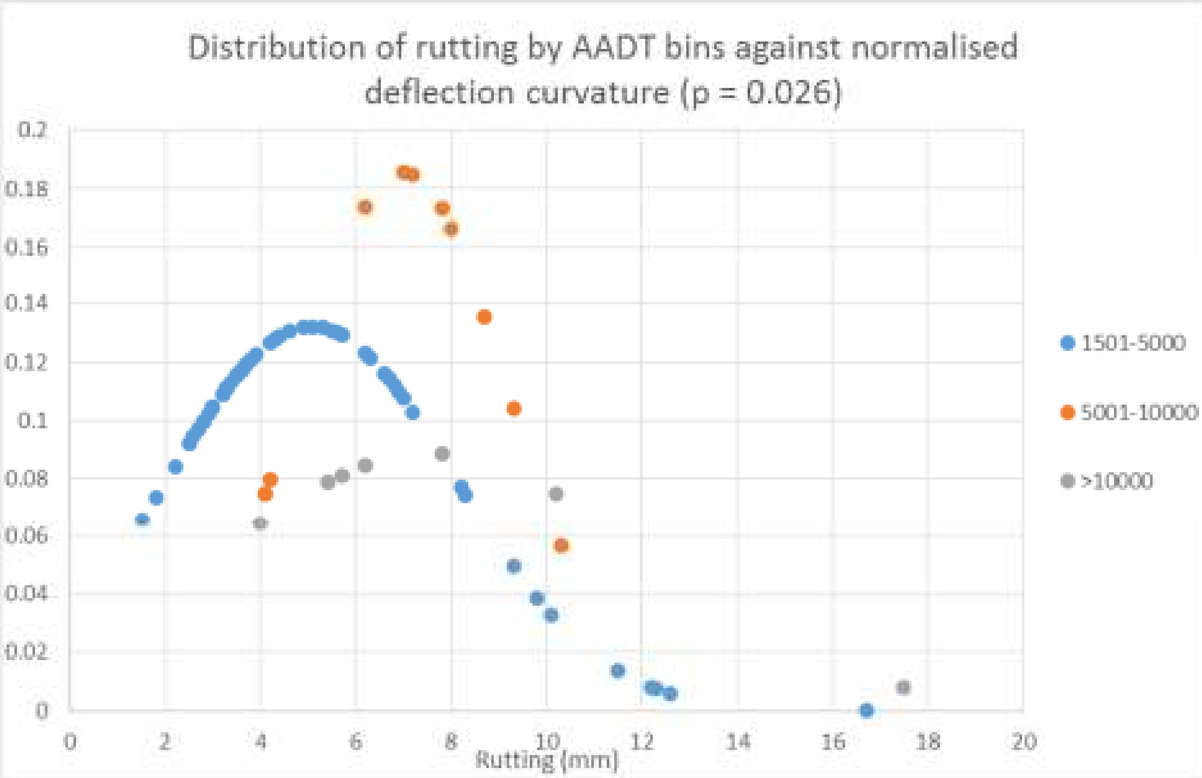
Figure C 47: FBS rutting distribution by AADT bins for vertical strain



Deflection curvature

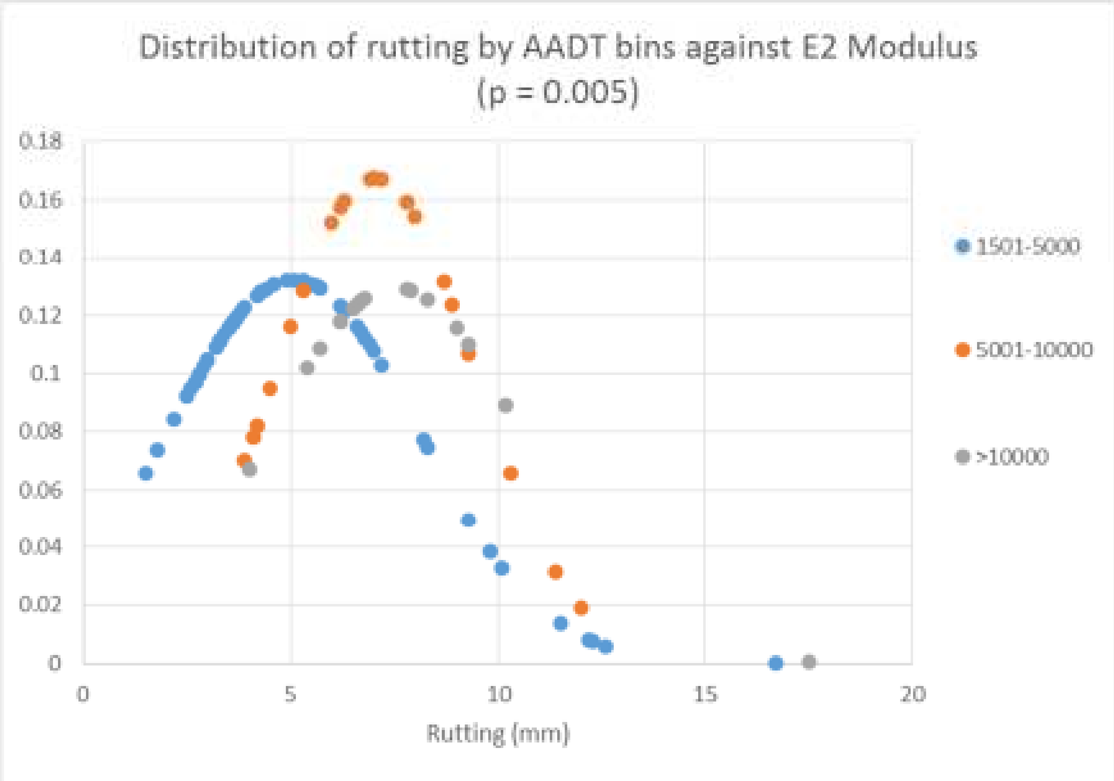
The distribution of rutting grouped by AADT follows a similar pattern to the previous relationship where increasing AADT bins also had an increasing mean rutting; however, the sample sizes vary in each category with fewer samples in the > 10 000 bin.

Figure C 48: FBS rutting distribution by AADT bins for normalised deflection curvature



Modulus

Figure C 49: FBS rutting distribution by AADT bins for modulus



C.4.2 Roughness Results

The ANOVA method has found statistical significance between the levels of the nominal variables for roughness, for the continuous variables outlined in Table C 10. The continuous variables that have an effect on roughness differ from that of rutting. There are only four nominal variables for FBS roughness compared to rutting that had five. Surface type does not seem to influence FBS roughness. Total pavement thickness and AADT greatly influenced rutting that appears to relate to a number of continuous variables but only appear to relate to a one or two for roughness.

Table C 10: P-value table for roughness

Continuous variables / Nominal variables	Total pavement thickness	Surface depth	CBR	Vertical strain	Average normalised deflection curvature	Cumulative traffic	Modulus E1	Modulus E2	Subgrade modulus
Environmental zones				2E-4	7E-4		3.7E-4	2.3E-4	
Total pavement thickness bins			9E-3						
Binder types	0.013	9E-3	5.8E-3		1.1E-6	6E-3	4.3E-6	3E-7	2.8E-7
AADT bins				3E-3	0.011				

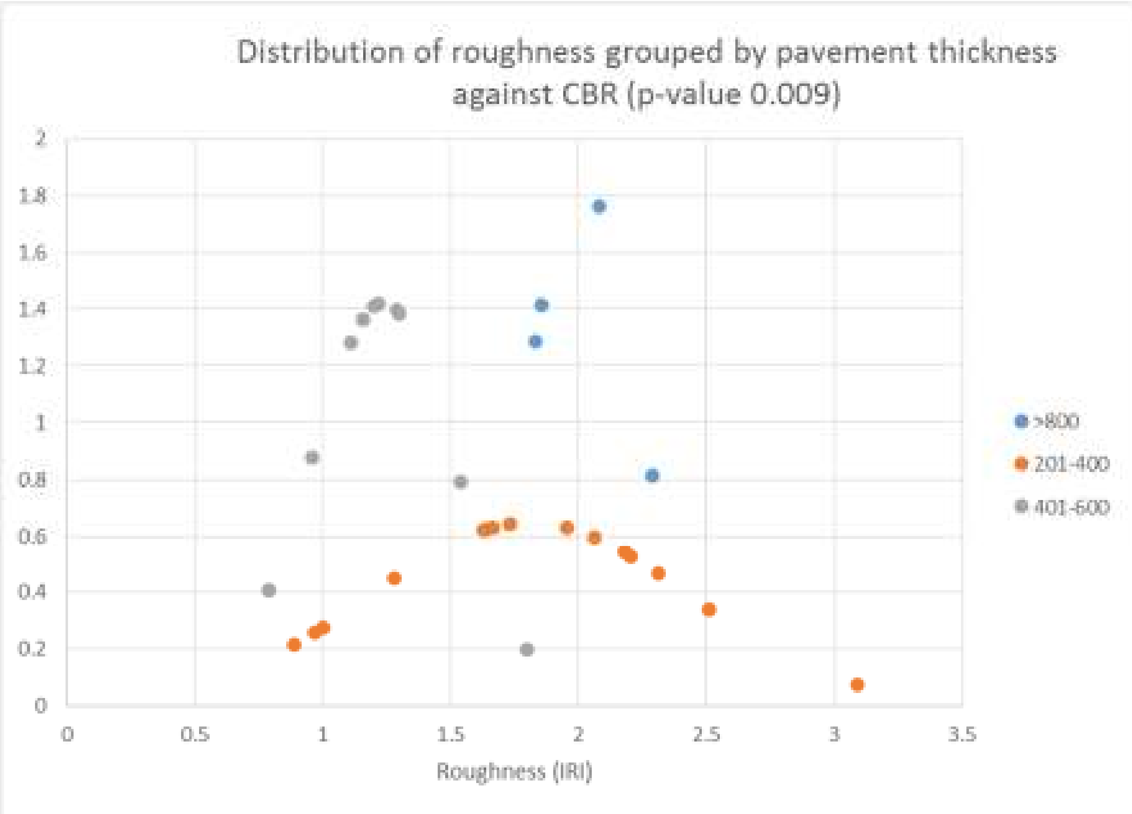
Total pavement thickness bin

Roughness grouped by total thickness bins does not have as many sample points within each level as for rutting, and as such the relationships may be slightly tenuous. Roughness, grouped by pavement thickness, seems to be influenced only by CBR. The distribution is shown in Figure C 50.

CBR

The greatest spread exists for the 201–400 mm pavement thickness bin; however, the mean does not appear to follow a consistent pattern, with the greatest pavement thickness bin (> 800 mm) displaying the greatest mean roughness.

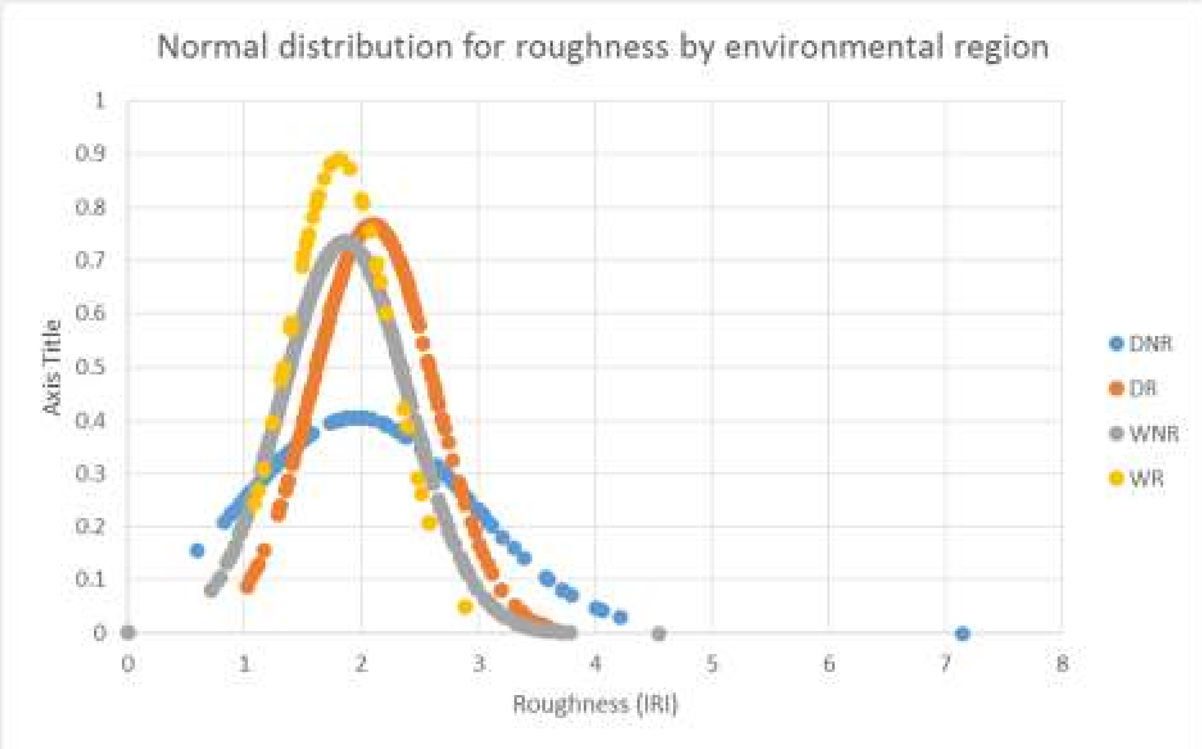
Figure C 50: FBS roughness distribution by total pavement thickness bins for CBR



Environmental zones

The distribution of roughness grouped by environmental zones for the total population of roughness (if attributed to an environment) is shown in Figure C 51. There is no statistical difference between the means in general; however, the spread for roughness associated with regions of DNR zones is much bigger than for other categories. When plotted against continuous variables, three relationships were identified from the ANOVA, namely modulus, normalised deflection curvature and vertical strain, as shown in Figure C 52, Figure C 53, and Figure C 54.

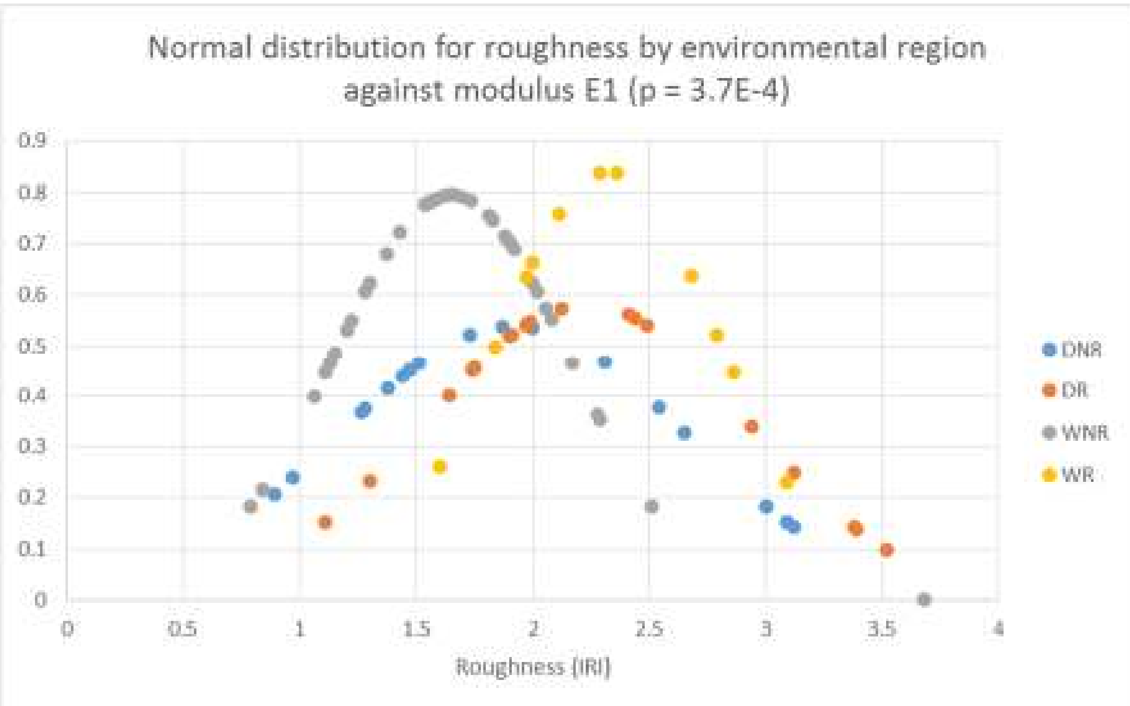
Figure C 51: FBS roughness distribution by environmental zones



Modulus E1

The lowest mean roughness is found in the WNR zone while the highest are equal between WR and DR, the reactive zones. Both the WNR and the DNR have lower means than the WR and DR; however, the DR and DNR distributions have greater spread than the WR and WNR distributions.

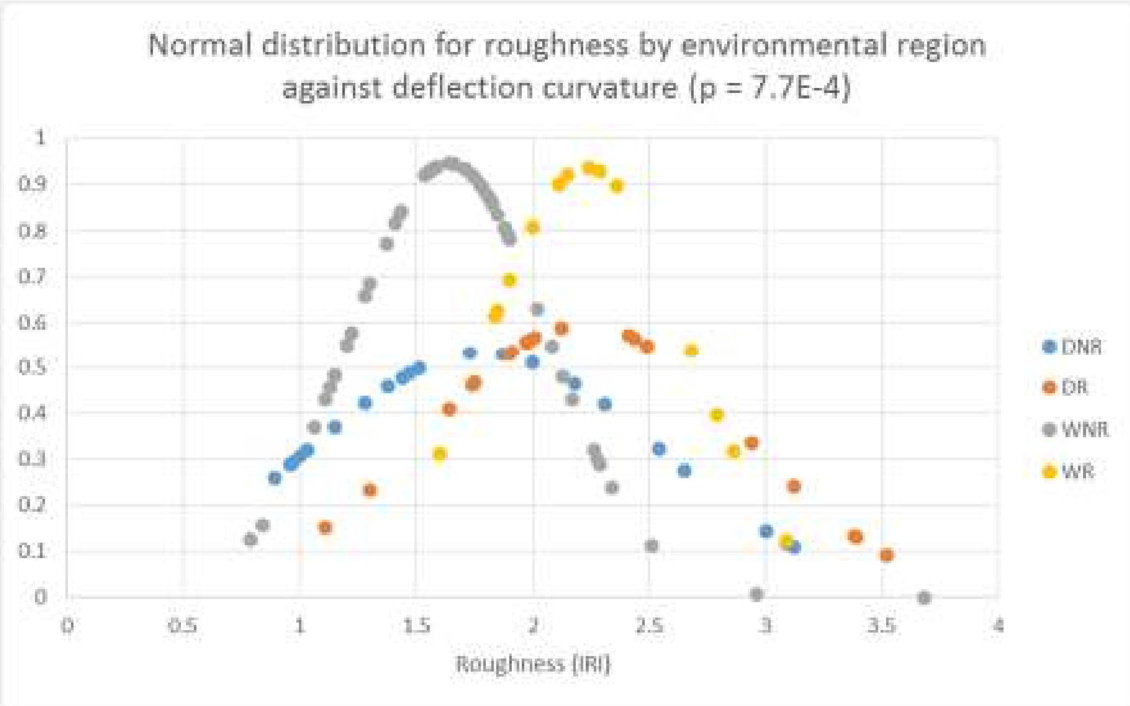
Figure C 52: FBS roughness distribution by environmental zones for modulus



Normalised deflection curvature

The relationship for environmental zones against normalised deflection curvature for roughness is similar to that seen Figure C 52.

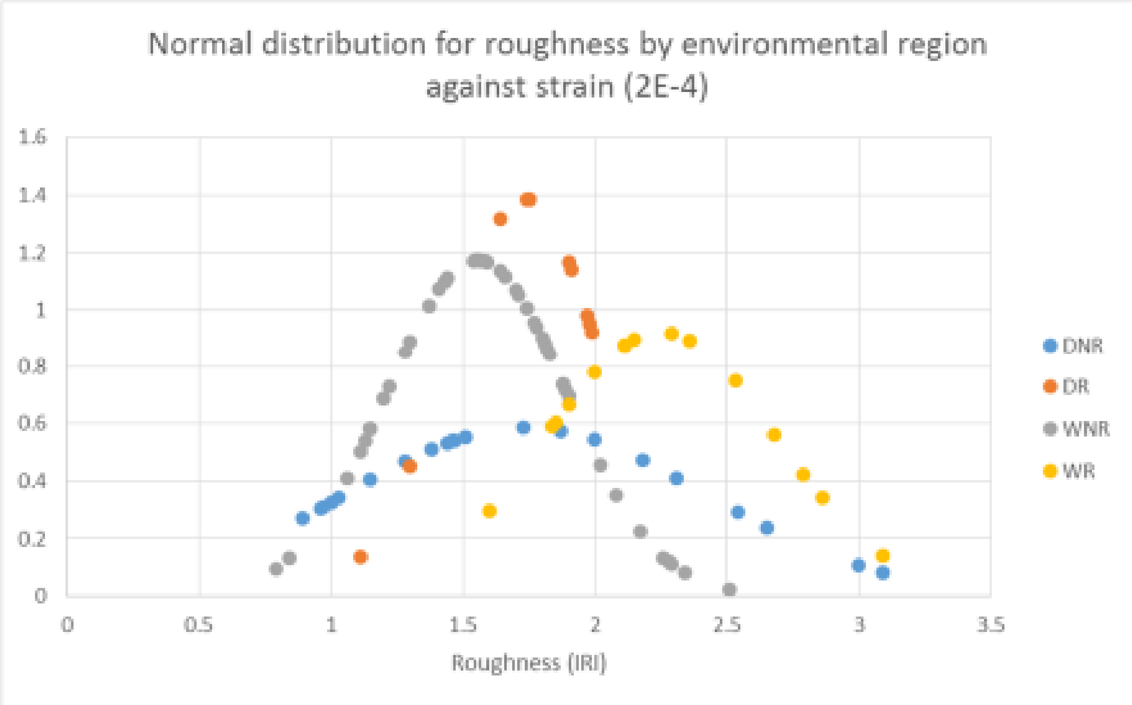
Figure C 53: FBS roughness distribution by environmental zones for deflection curvature



Vertical strain

The relationship differs slightly from the previous two for vertical strain as, this time, the DNR and WR have the larger spreads.

Figure C 54: FBS roughness distribution by environmental zones vertical strain



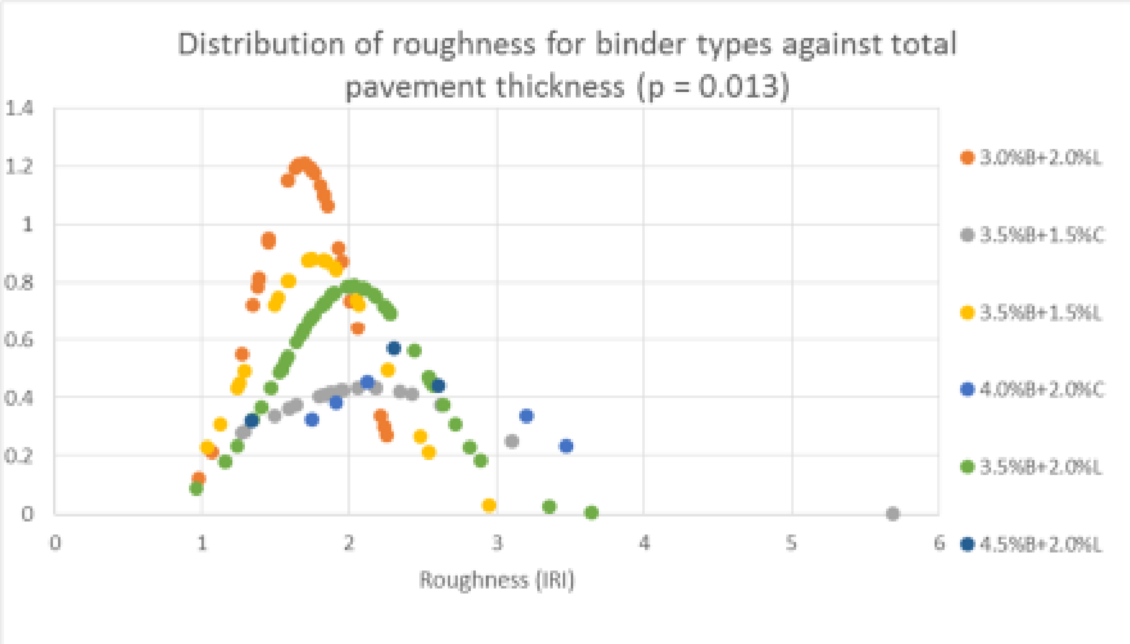
Binder types

There are more continuous variables that relate to rutting when grouped by binder types for roughness than there are for rutting.

Total pavement thickness

The binder type with the greatest spreads for roughness are 3.5%B+1.5%C and 4.0%B+2%C, suggesting that these binder types are used on pavements with a great range of thicknesses. The narrowest distribution and with the lowest mean rutting is 3.0%B+2%L (Figure C 55).

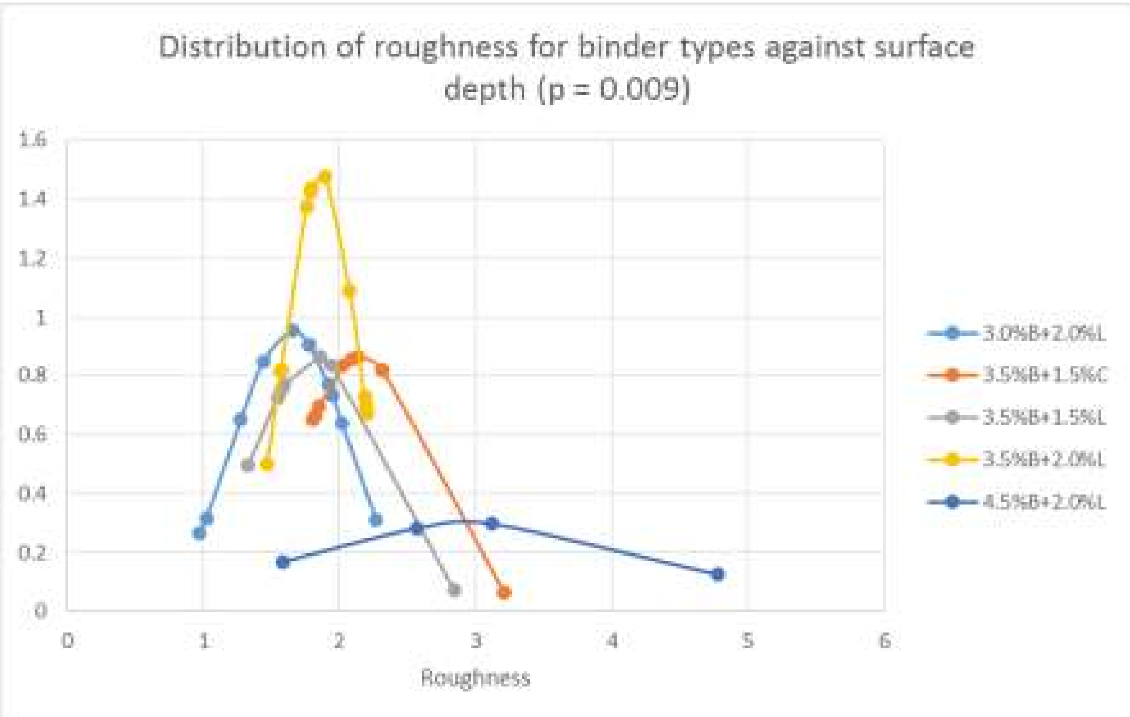
Figure C 55: FBS roughness distribution by binder types for total pavement thickness



Surface depth

Surface depth influences roughness in a similar manner to total pavement thickness. The greatest spread this time is 4.5%B+2.0%L as there were no sample points in the 4.0%B+2.0%C category that correlated to a surface depth. The distribution with the greatest spread and highest mean is 4.5%B+2%L which has the highest bitumen content (Figure C 56).

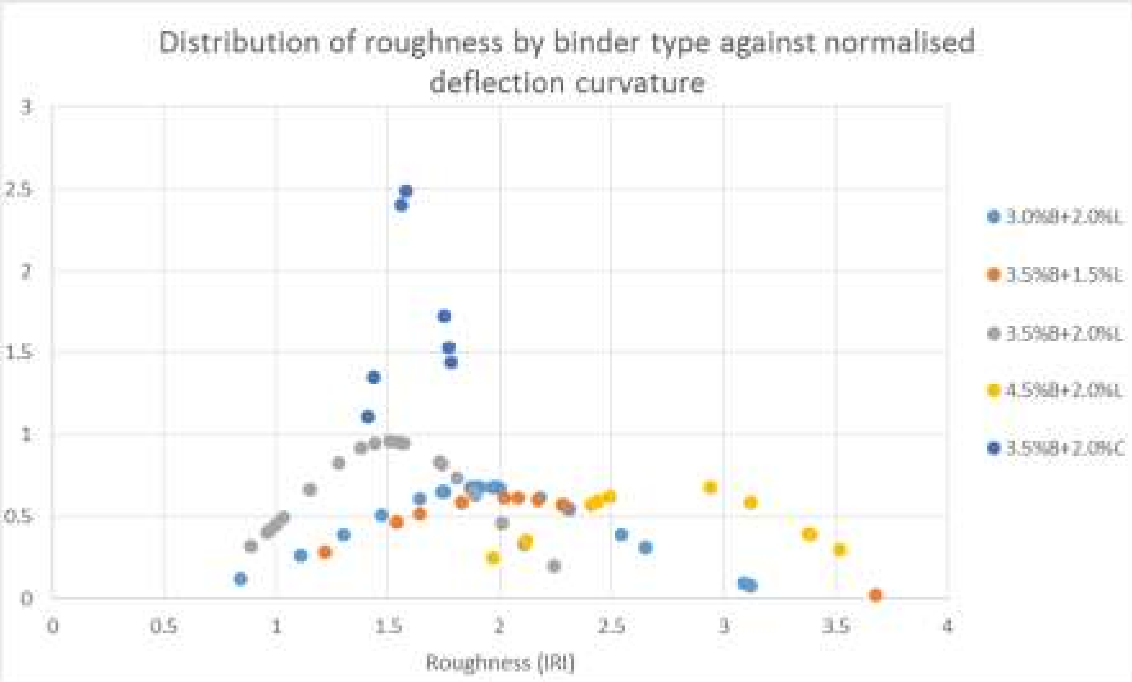
Figure C 56: FBS roughness distribution by binder types for surface depth



Normalised deflection curvature

Almost all the categories have large spreads when plotted against normalised deflection curvature when compared with the graphs in Figure C 55 and Figure C 56, suggesting that there are interactions with many other factors other than binder type against normalised deflection curvature (Figure C 57).

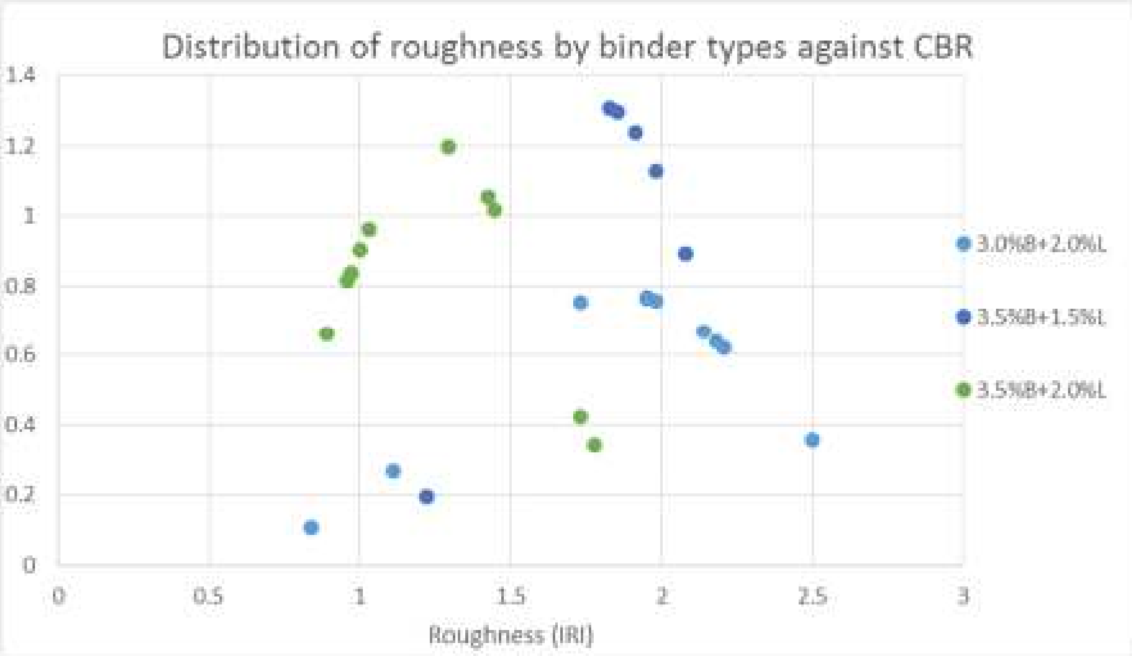
Figure C 57: FBS roughness distribution by binder types for normalised deflection curvature



CBR

The sample sizes were significantly reduced for roughness when plotted against CBR and grouped by binder types. For this case, it is difficult to draw any inferences on the general population for CBR, roughness and binder type (Figure C 58).

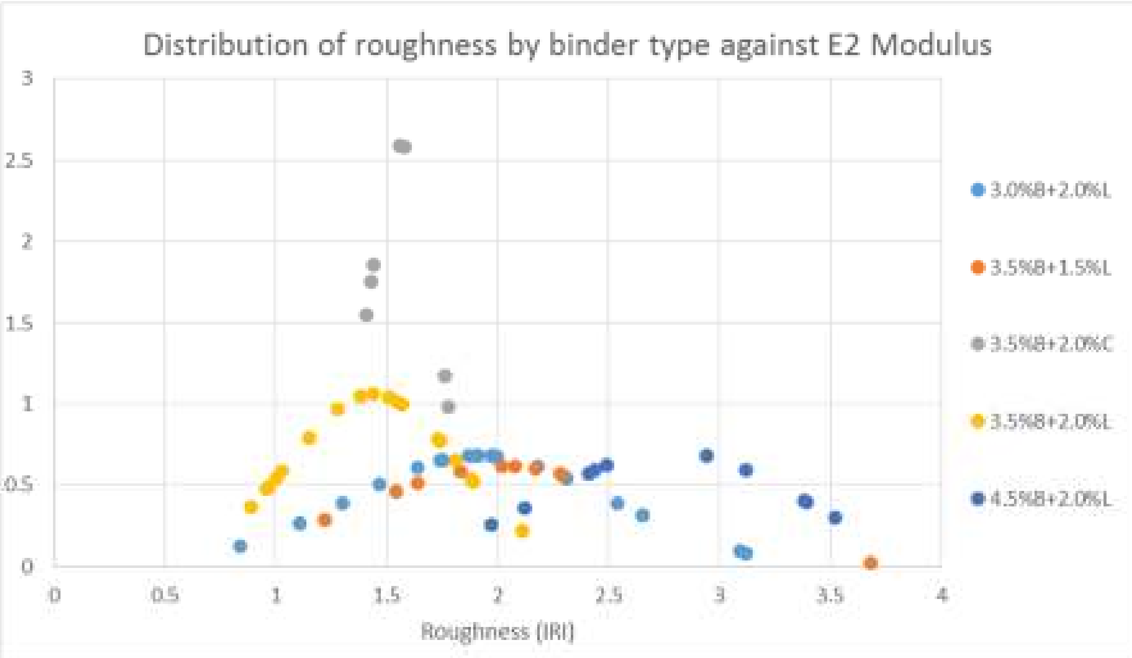
Figure C 58: FBS roughness distribution by binder types for CBR



Modulus

The relationship in the distribution in Figure C 59 is similar to the normalised deflection curvature.

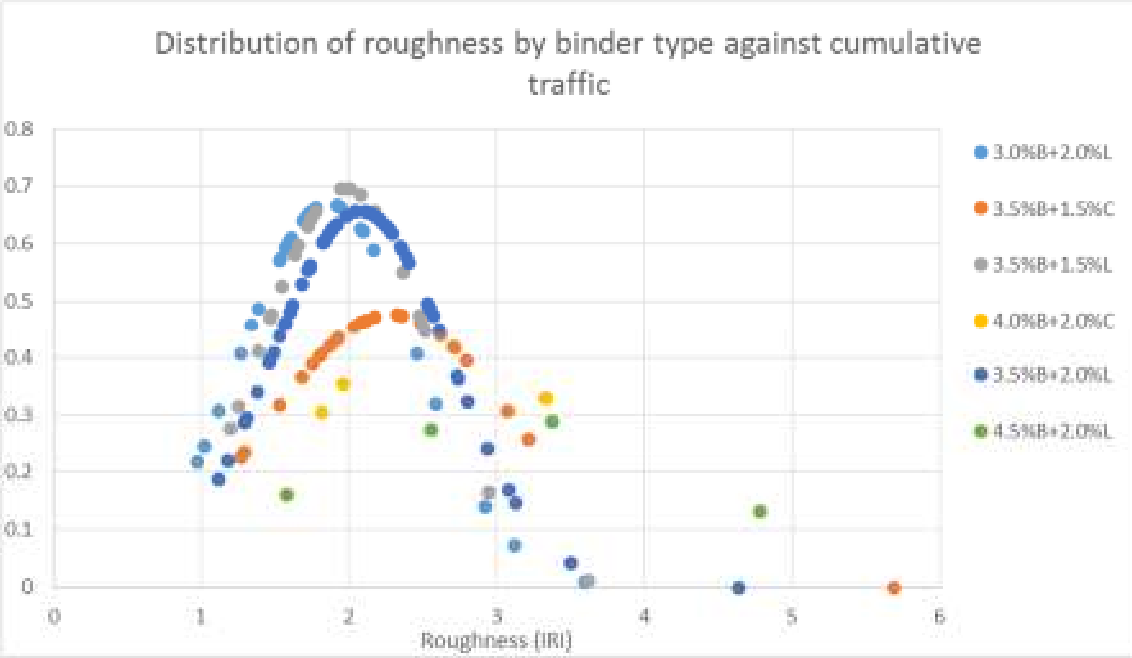
Figure C 59: FBS roughness distribution by binder types for modulus



Cumulative traffic

When plotted against cumulative traffic and grouped by binder type, the roughness and mean seem to both increase as the bitumen content increases (Figure C 60).

Figure C 60: FBS roughness distribution by binder types for cumulative traffic



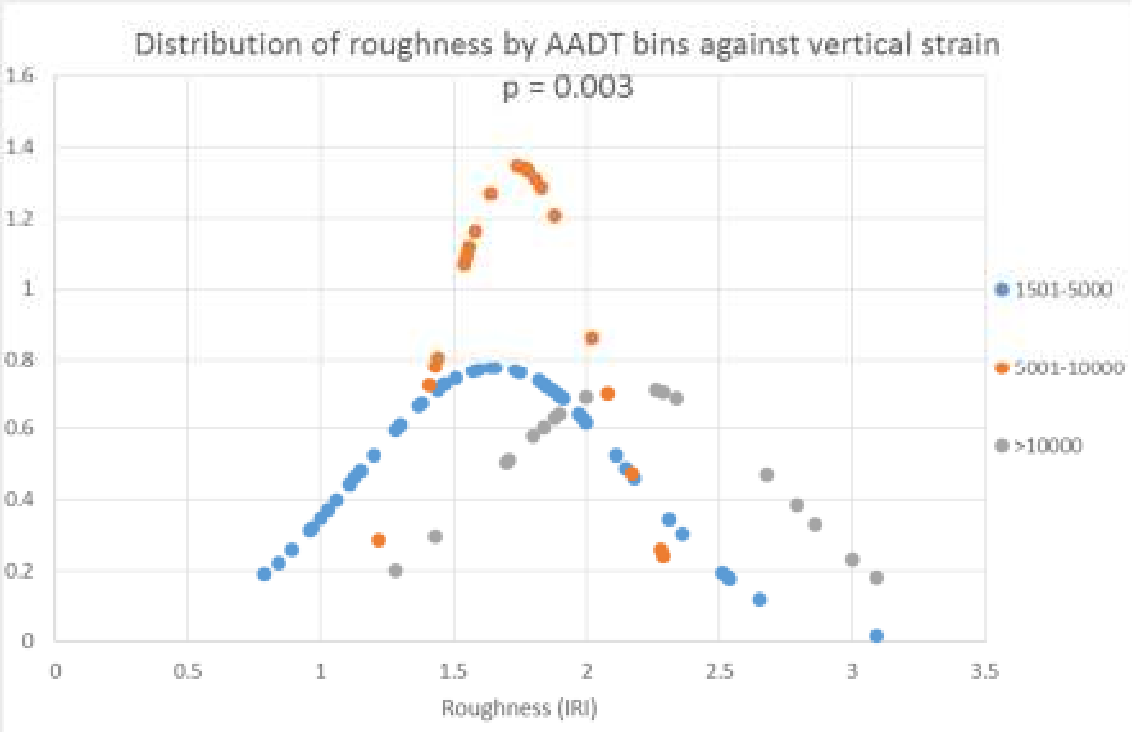
AADT

The AADT bin categories have less influence on roughness than on rutting. The only two relationships found to produce statistically significant differences in the mean between AADT groups are vertical strain and normalised deflection curvature, as shown in Figure C 61 and Figure C 62 respectively.

Vertical strain

The mean roughness increases with increasing AADT, suggesting a linear positive correlation. The spread is narrower for the 5001–10000 category than for the others, which suggests that the range of values for vertical strain was limited compared with the other categories, or there were more influences from other factors for the other two.

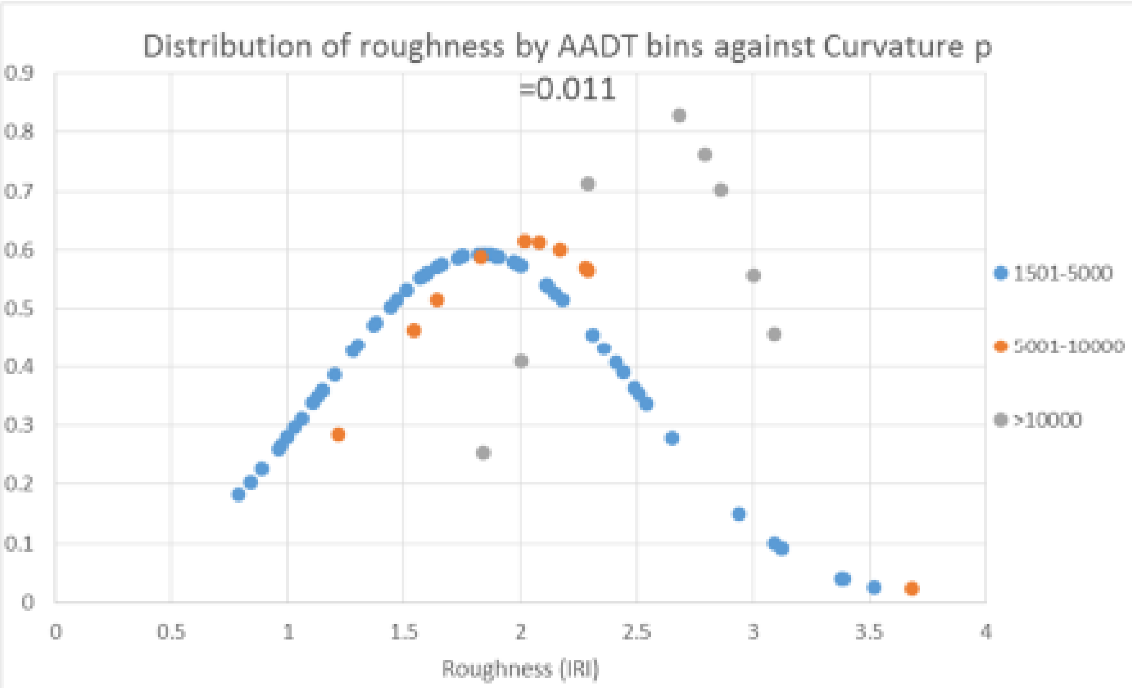
Figure C 61: FBS roughness distribution by AADT bins for vertical strain



Normalised deflection curvature

The mean roughness increased with each increase of AADT against normalised deflection curvature. The spreads also became narrower with increasing AADT, suggesting that fewer sample points existed for the full deflection curvature range in each category.

Figure C 62: FBS roughness distribution by AADT bins for normalised deflection curvature



C.5 I-FBS Sensitivity Analysis

The distributions from the previous sections indicated that there may be multiple interactions between the factors that have an influence on the performance indicators. The distributions that have a high spread suggest either a high residual, thus more sample points over a greater range, therefore linear or otherwise relationship, may exist between independent and dependent variables. The sensitivity analysis, as described in Section 10.2.2, involves resolving the performance indicators into further levels for the nominal variables before plotting them against the continuous variables to identify possible trends or correlations.

For example, the trends for rutting against the average normalised deflection curvature, when grouped by binder type, is shown in Figure C 42. Initial inspections of the trends show no correlation between maximum rutting and the normalised deflection curvature; however, when grouped by binder type types as shown in Figure C 63, some weak trends emerge. There are positive correlations (rutting increases as normalised deflection curvature increases, indicating a weaker pavement), a negative trend that may be due to the outliers in the sample points. Little to no correlation indicates that factors other than binder type influence the rutting.

The following sections show examples of the analysis undertaken to identify the relationship and interaction between factors to develop a suitable model for FBS pavement deterioration. The same techniques are applied to PM-CMB.

C.5.1 Rutting Results

The influential factors found to affect rutting were:

- vertical strain
- normalised deflection curvature
- modulus (mostly E2 and E4)
- surfacing age
- cumulative traffic.

These factors interact with the nominal categories in different ways, with the greatest number of influential factors found to interact with total pavement thickness bins and AADT bins. Linear regression was conducted for these two categories and the correlations are outlined in Table C 11 and Table C 12 below. Extremely weak correlations (where the coefficient is much closer to zero compared to the others) can be considered to have no correlation. The relationships described in the following sections are not considered strong, as most have R2 values much less than 0.7. The levels with the greater number of sample points also contained the weaker correlations; thus when they are resolved into further levels, it is expected that the correlations would be stronger.

AADT bins

In Table C 11 the < 2000 level did not have sufficient sample points against vertical strain, normalised deflection curvature or E2 modulus to draw a relationship; however, the surfacing age category was large enough. It is expected that as surfacing age increases, the rutting should also increase, and that the degree of rutting should increase with AADT; however, that was not found to be the case with rutting decreasing with age for the lower AADT levels until the > 10 000 level where it increases. It was found that the relationship was not strictly linear, rather, the rutting would increase, reaching a peak around the 7–10 years point, and then drop off rapidly. For the > 10 000 category the peak is reached around the 10–15 year mark.

Table C 11: Correlations between rutting for AADT bins and influential continuous variables

AADT bins vs.	Continuous independent variables			
	Surfacing age	Vertical strain	Normalised deflection curvature	E2 modulus
< 2000	None	N/A	N/A	N/A
2001–5000	Negative	Positive	Negative	Positive
5001–10000	Negative	Positive	Positive	Negative
> 10 000	Positive	Positive	Positive	None

For the AADT bin category of 2001–5000, the trends for normalised deflection curvature and E2 modulus are not as expected. It is assumed that as normalised deflection curvature increases, rutting should increase, and that as the modulus increases, the rutting should decrease. This may be due to compaction under traffic for roads that do not experience high AADT, hence the material is still settling, which may account for the positive correlation between modulus and rutting. Deflection curvature often gives an indication of cracking and distress within the base layer, but due to the low traffic volumes the positive correlation may indicate a strong base layer.

Total pavement thickness bins

In Table C 12 the < 200 mm category did not contain enough sample points to determine any relationship. The expected relationship for rutting against vertical strain is positive, as seen for the AADT case; however, this is only true for the 401–600 mm and 800+ mm pavement thickness bins. This could be due to the mid thicknesses being more densely and unevenly populated. The vertical strain values for 201–400 ranges from 100–550 με while 401–600 spans from 60–400 με. The range for the 601–800 and 800+ categories are smaller and sparsely populated.

When plotted against the modulus, the 201–400 bin has a greatest range from under 50 to over 300 MPa, while 401–600 is concentrated in the 50–150 MPa band for E5 moduli. The 601–800 group has fewer points and they concentrate around the 60–80 MPa band. The 800+ group has very few sample points, with modulus clustered around the 50 MPa band.

Table C 12: Correlations between rutting for total pavement thickness bins and influential continuous variables

Total pavement thickness bins vs.	Influential factors			
	Vertical strain	Normalised deflection curvature	E2 modulus	E5 modulus
< 200	N/A	N/A	N/A	N/A
201–400	None	None	Positive	Positive
401–600	Positive	None	Positive	Negative
601–800	Negative	Negative	Positive	Positive
800+	Positive	Positive	Negative	Negative

Binder type

Binder types are different to the AADT and pavement thickness bin as the latter are ordinal variables, which means they can be ordered. Binder types do not fall into any specific order, and attempting to find a relationship between the binders is not a simple process, as seen in Figure C 63 and Figure C 64. The sample sizes are not consistent across the categories, neither is the range of the continuous variables; as a result they are good candidates for sensitivity analysis.

Figure C 63: Rutting grouped by binder type against average modulus E2

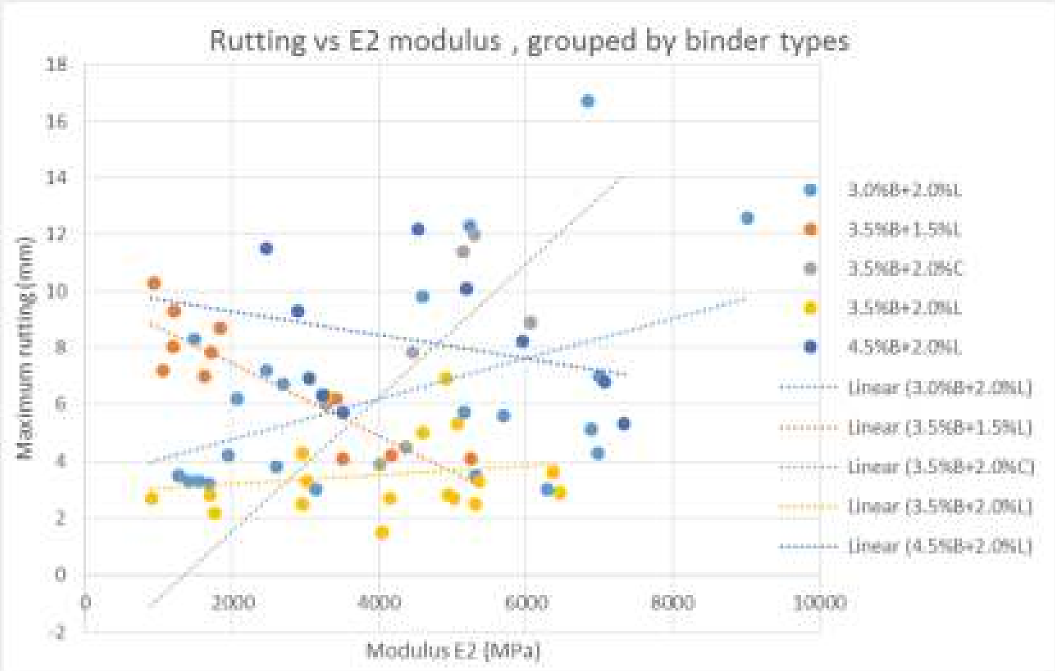
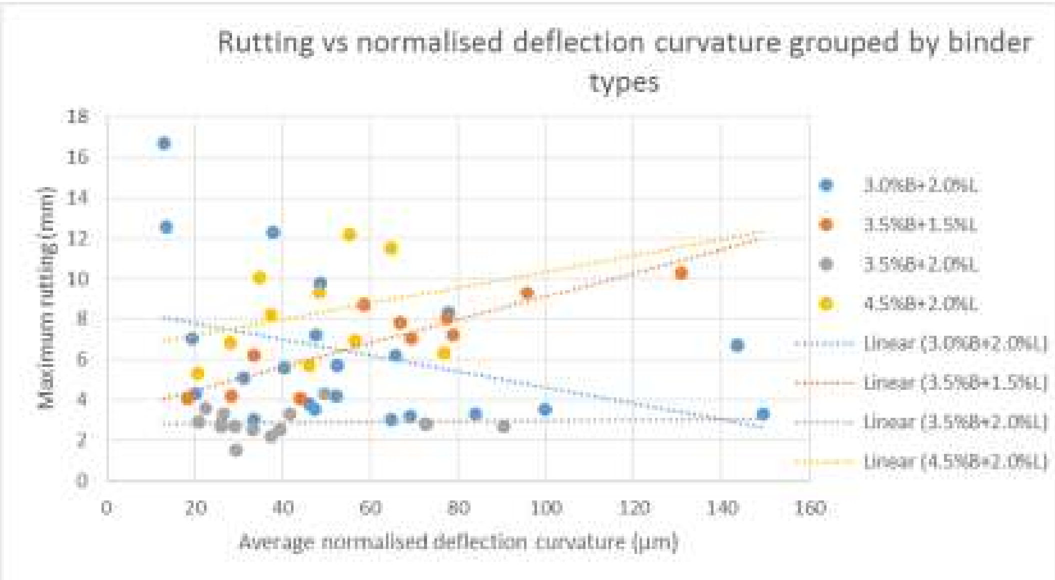
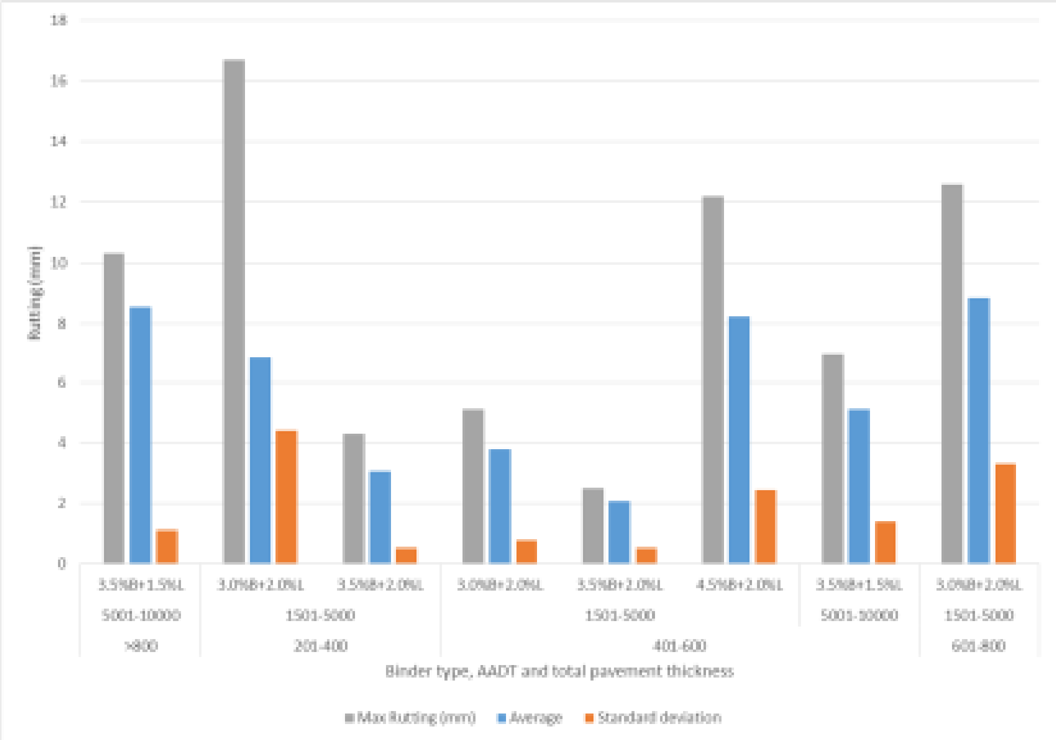


Figure C 64: Rutting grouped by binder type against normalised deflection curvature



The sensitivity analysis was undertaken for rutting in binder type bins resolved into AADT bins, and total pavement thickness bins. A summary of the mean and standard deviation for the rutting grouped by binder types resolved into AADT and pavement thickness bins, and plotted against average normalised curvature are shown in Figure C 65. Notice that when the rutting is resolved into the three layers, there are not as many sublevels as one would expect, i.e. each AADT bin does not have the full range of pavement bins and the pavement bins do not each contain all five binder types.

Figure C 65: Statistics for rutting by binder type, total pavement thickness and AADT against normalised deflection curvature

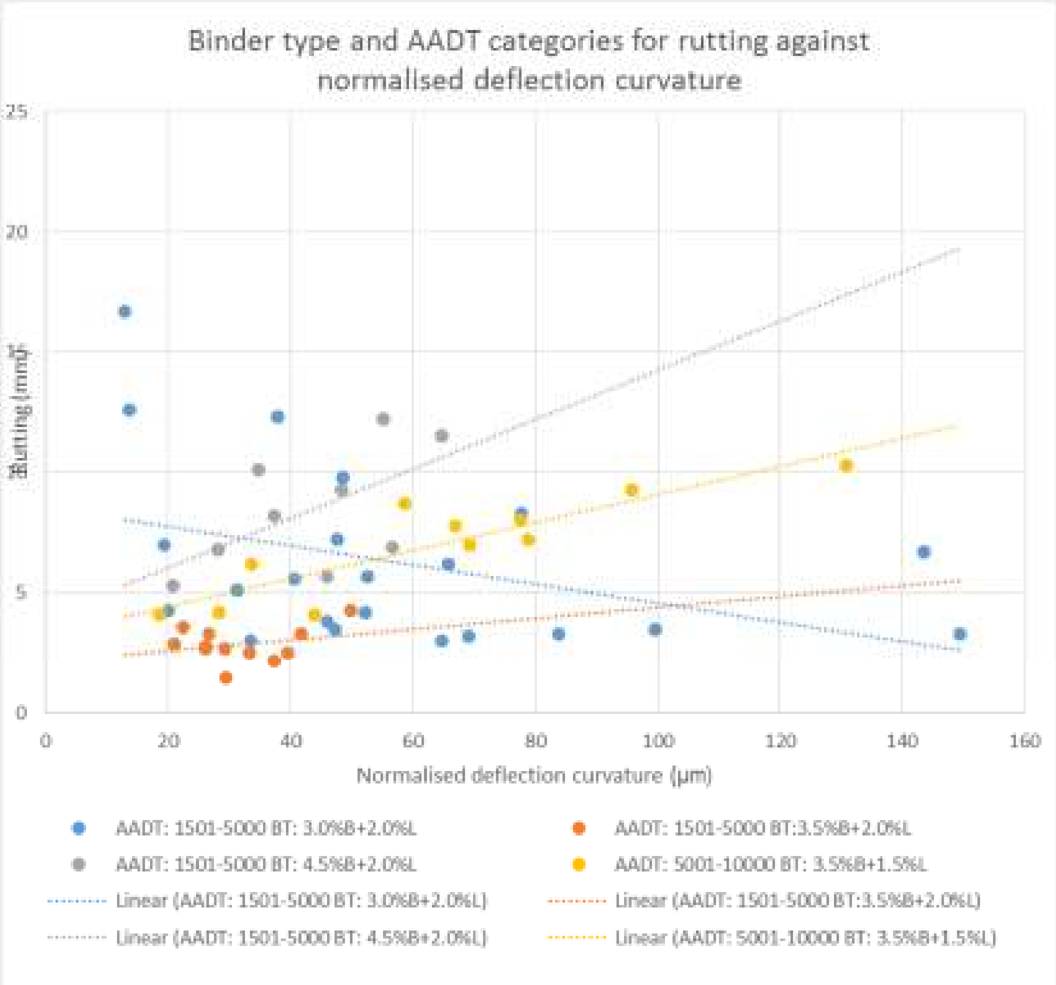


This representation of the dataset reveals the following where normalised deflection curvature and rutting is concerned:

- The greatest variation of binder types and AADT levels exists for the 401–600 pavement thickness bin, which has a greater number of sample points.
- The most prevalent AADT bin is 1501–5000, which is found in 201–400, 401–600 and 601–800 pavement thickness bins.
- The pavement thickness bin > 800 only does not appear to have samples for rutting AADT under 5001.
- The lowest AADT bin 1501–500 exists in the lowest total pavement thickness bin of 201–400 (noting that there were not enough data points for < 1500 AADT and < 200 pavement thickness).
- The maximum rutting of over 16 mm occurred in the lowest pavement thickness and lowest AADT bin category for material 3.0%+2.0%L.
- The most prevalent binder type is 3.0%+2.0%L.
- The lowest mean rutting and lowest standard deviation is for binder type 3.5%B+2.0%L with AADT 1501–500 and in the pavement thickness bin of 401–600, indicating very few sample points in this sublevel.
- In the lowest AADT and pavement thickness category, 3.0%B+2.0%L has much higher maximum rutting, mean and standard deviation than the 3.5%B+2.0%L.

When the points are plotted against normalised deflection curvature, the correlations can be seen in Figure C 66. The relationships between the binder types are easier to observe in this figure than in the unresolved dataset in Figure C 64.

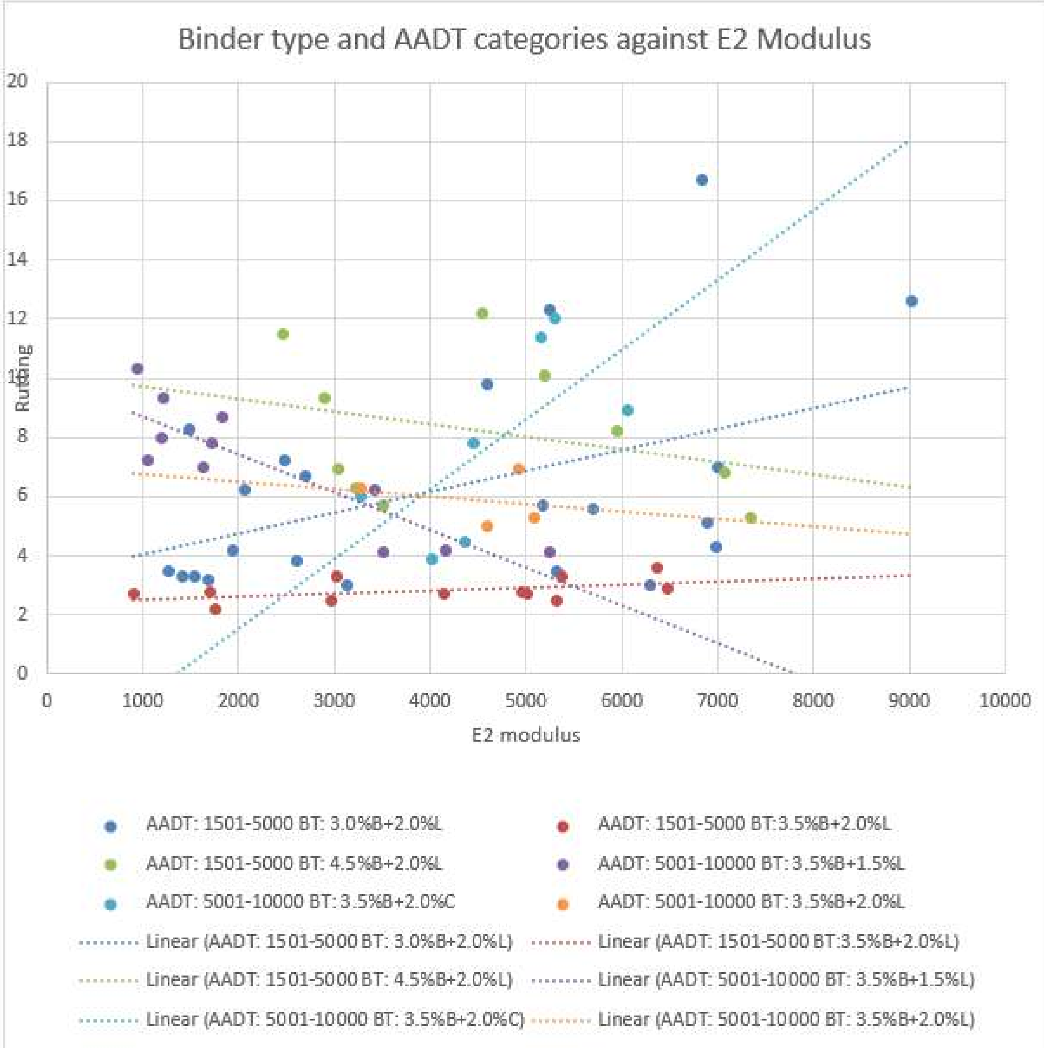
Figure C 66: Linear regression for rutting by binder type and AADT against normalised deflection curvature



The range of values for normalised deflection curvature is small for the AADT 1501–5000 BT 3.5%B+2.0%L sublevel. The 3%B+2%L binder type displays a negative correlation with rutting, which is unexpected; however, this binder type is often used in areas with low AADT (1501-5000). The steepest trend observed, where rutting increases with increasing normalised deflection curvature, is for the 4.5%B+2.0%L binder type (the highest bitumen percentage of all the categories despite being used in an area with very low AADT).

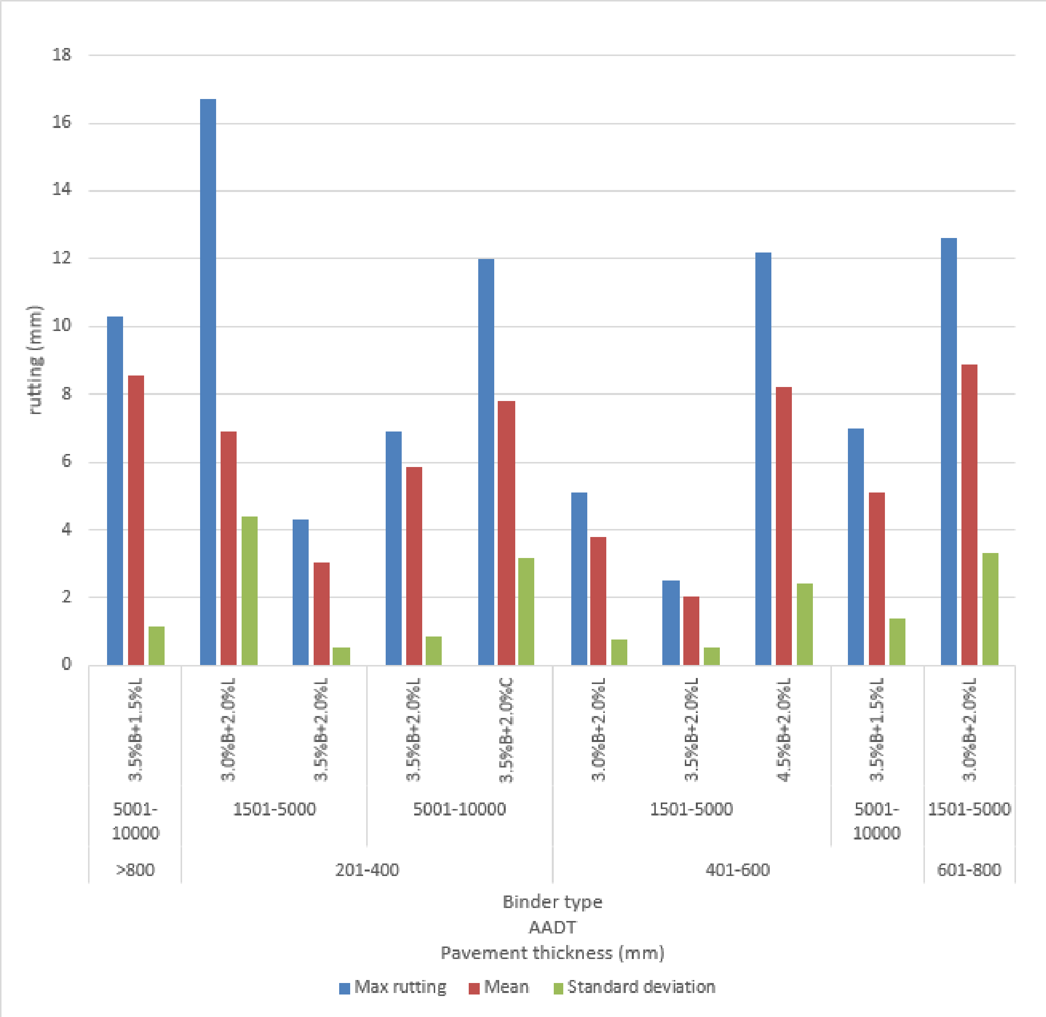
Linear regression for rutting, grouped by binder type and AADT against E2 modulus, is shown in Figure C 67, and the relationships differ from the normalised deflection curvature. While rutting is expected to increase with normalised deflection curvature, it is meant to decrease with modulus.

Figure C 67: Linear regression for rutting by binder type and AADT against E2 Modulus



For the rutting vs. normalised deflection curvature case, the statistics for the sublevels for rutting vs. E2 modulus are shown in Figure C 68. There are two more sublevels for the E2 modulus than for the normalised deflection curvature case.

Figure C 68: Statistics for rutting by binder type, total pavement thickness and AADT against E2 Modulus



The following observations are made for rutting vs. E2 modulus by binder type, AADT and pavement thickness:

- As shown in the previous analysis the maximum rutting is observed in the 201–400 mm pavement thickness and 1501–5000 AADT.
- For sublevels where the maximum rutting exceeded 10 mm other than the above:
  - 201–400 mm, 5001–10000, binder type: 3.5%B+2.0%C
  - 401–600 mm, 1501–5000, binder type: 4.5%B+2.0%L
  - 601–800 mm, 1501–5000, binder type: 3.0%B+2.0%L
  - > 800 mm, 5001–10 000, binder type: 3.5%B+1.5%L.

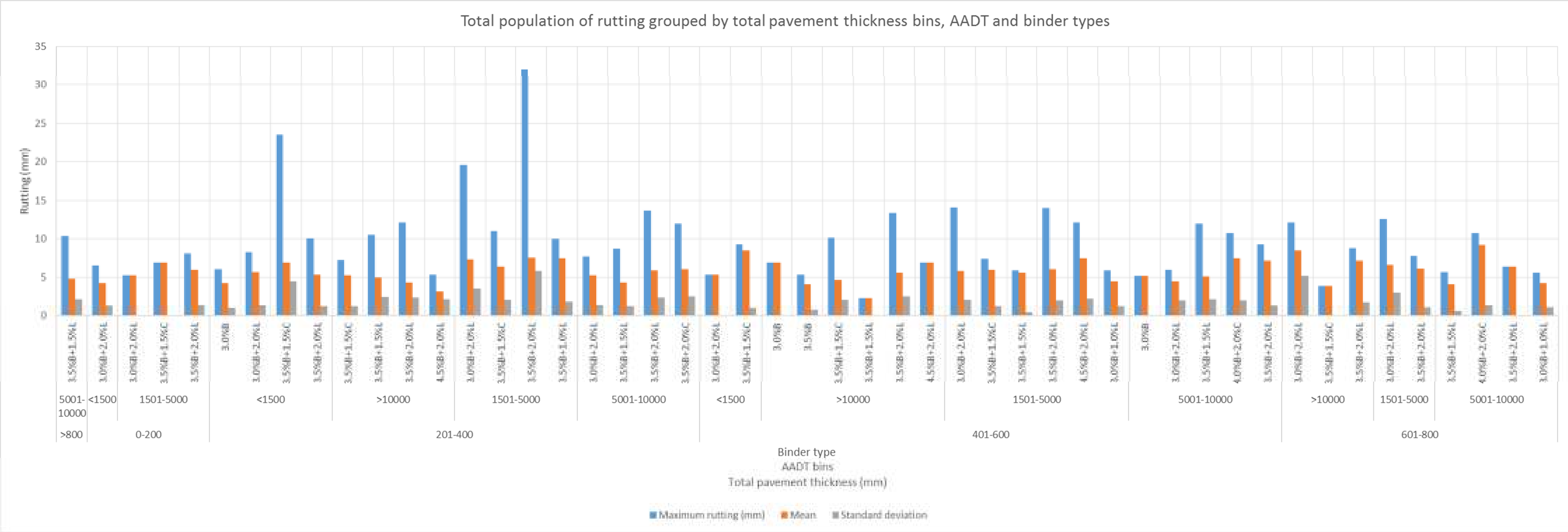
The last two pavement thickness categories did not contain any other AADT bins or binder types.

The total population of rutting by binder type and resolved against AADT and pavement thickness is shown in Figure C 69. The following is observed for the total population:

- Total pavement thickness of all ranged between 200 and 800 have road segments with > 10 000 AADT; however, they do not necessarily have the greatest maximum rutting.

- The binder types, for pavement bins with more than one AADT bins, containing the maximum rutting are:
  - — 0–200 pavement thickness, 1501–500 AADT: 3.5%B+2.0%L
  - — 201–400 pavement thickness, < 1500 AADT: 3.5%B+1.5%C
  - — 201–400 pavement thickness, 1501–5000 AADT: 3.5%B+2.0%L
  - — 201–400 pavement thickness, 5001–10 000 AADT: 3.5%B+2.0%L
  - — 201–400 pavement thickness, > 10 000 AADT: 3.5%B+2.0%L
  - — 401–600 pavement thickness, < 1500 AADT: 3.5%B+1.0%C
  - — 401–600 pavement thickness, 1501–5000 AADT: 3.0%B+2.0%L
  - — 401–600 pavement thickness, 5001–10 000 AADT: 3.5%B+1.5%L
  - — 401–600 pavement thickness, > 10 000 AADT: 3.5%B+2.0%L
  - — 601–800 pavement thickness, 1501–5000 AADT: 3.0%B+2.0%L
  - — 601–800 pavement thickness, 5001–10 000 AADT: 4.0%B+2.0%C
  - — 601–800 pavement thickness, > 10 000 AADT: 3.5%B+2.0%L.

Figure C 69: Total population of maximum rutting when grouped by binder types, AADT bins, and total pavement thickness



C.5.2 Roughness Results

The influential factors found to affect roughness were:

- vertical strain
- normalised deflection curvature
- modulus
- CBR
- total pavement thickness
- surface depth
- cumulative traffic.

These factors interact with the nominal categories in different ways, with the greatest number of influential factors found to interact with binder types. Linear regression was conducted for these two categories and the correlations are outlined in Table C 13. Extremely weak correlations (where the coefficient is much closer to zero compared to the others) can be considered to have no correlation. The relationships described in the following sections are not considered strong, as most have R2 values much less than 0.7. The levels with the greater number of sample points also contained the weaker correlations; thus when they are resolved into further levels, it is expected that the correlations would be stronger.

Binder types

As with rutting, binder types do not have a specific order and are purely nominal variables, and attempting to find a relationship between the binders is not clear, as shown in Table C 13. The sample sizes are not consistent across the categories and neither is the range of the continuous variables; as a result these are a good candidate for sensitivity analysis.

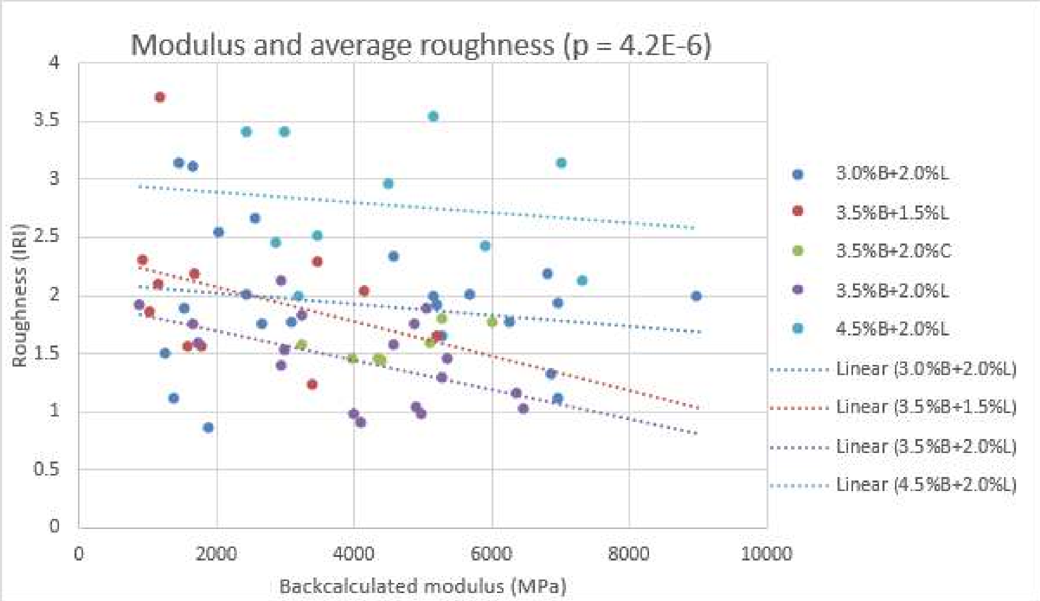
Table C 13: Correlations between roughness for binder types and influential continuous variables

Binder type vs.	Total pavement thickness	Surface depth	Average normalised deflection curvature	CBR(2)	E2 modulus	Cumulative traffic
3.5%B+1.5%C	Positive	None	N/A	N/A	N/A	Positive
3.5%B+2.0%C	N/A	N/A	Negative	N/A	Positive	N/A
3.0%B+2.0%L	None	Positive	None	Negative	Negative	Positive
3.5%B+2.0%L	None	None	Positive	None	Negative	Negative
3.5%B+1.5%L	None	Positive	Positive	None	Negative	Positive
4.5%B+2.0%L	Positive(1)	Negative	None	N/A	None	Positive

1 Three sample points only.
2 CBR sample sizes are not very big.

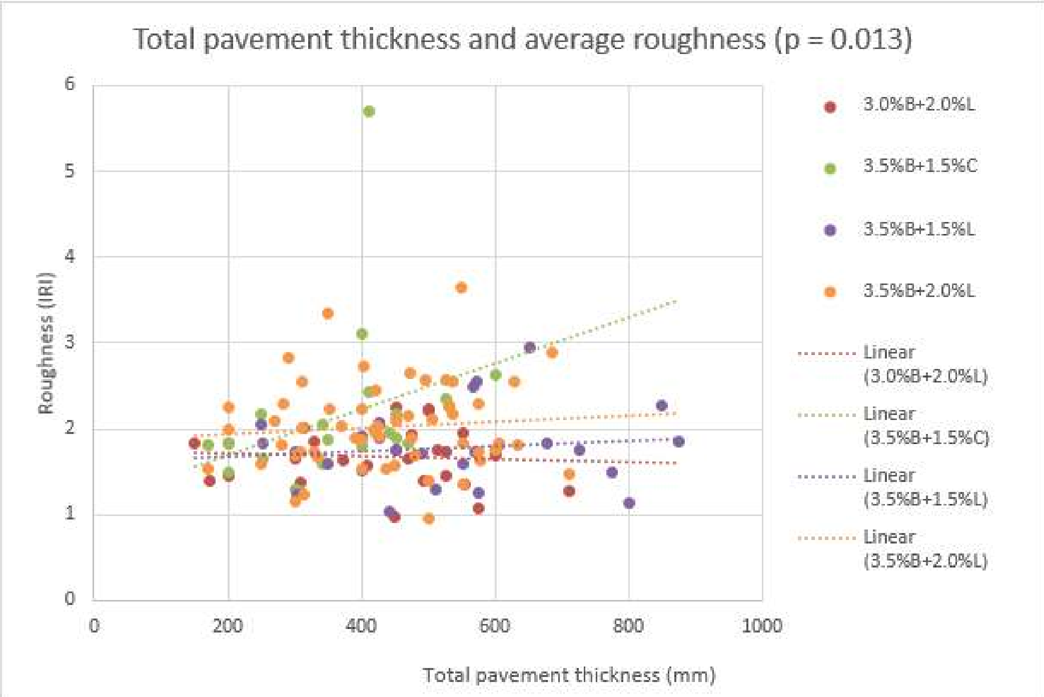
The linear regression was undertaken for binder types to determine if the roughness correlated with the continuous variables in the same manner as rutting. The outcome is inconclusive for all relationships except E2 modulus. This may be due to the fact that absolute roughness value may not be a good indicator for deterioration as there is assumed to be inbuilt roughness at construction and absolute roughness may decrease over time due to different measurements. The roughness vs. E2 modulus relation by binder types is shown in Figure C 70 below. The majority of the binder types show a negative correlation as expected.

Figure C 70: Roughness grouped by binder type against modulus



An example of weak correlations for roughness despite being an influencing factor is roughness vs. total pavement thickness when grouped by binders. The sample points are significantly reduced to only four binder types due to the lack of sample sizes, as seen in Figure C 71.

Figure C 71: Roughness grouped by binder type vs. total pavement thickness

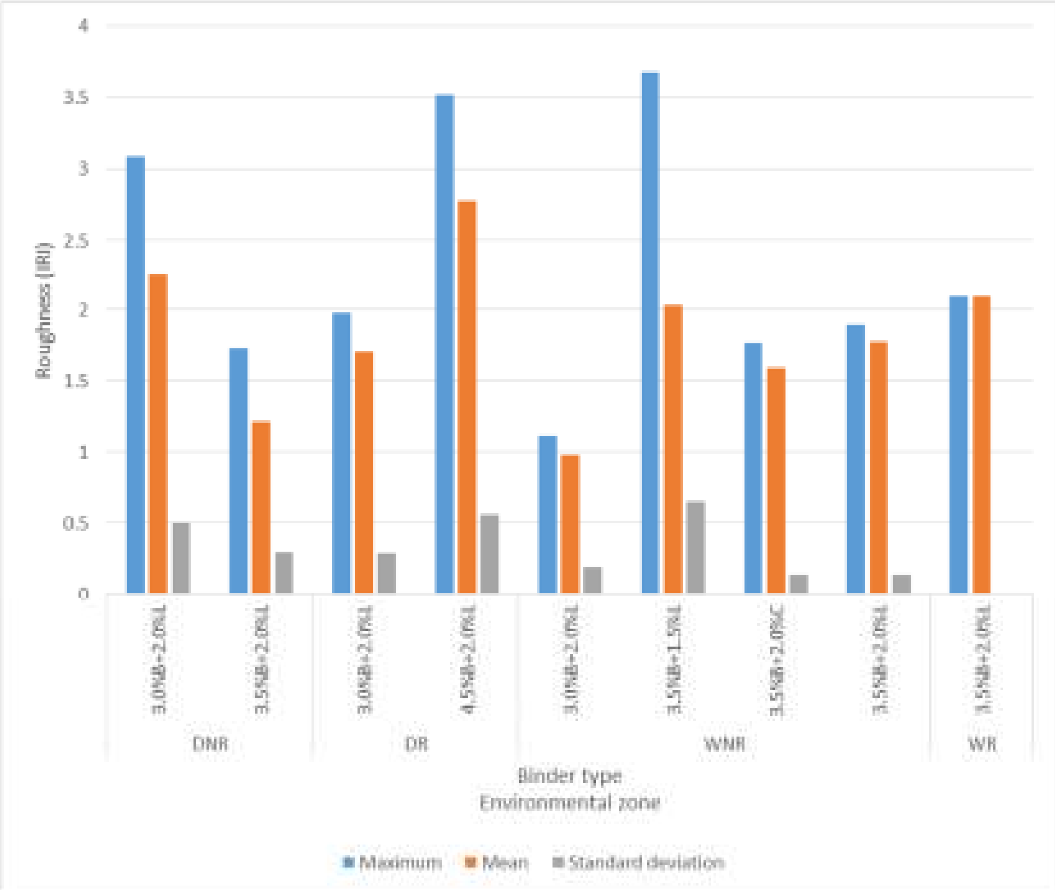


The linear statistical analysis reveals weak correlations for all roughness and all the continuous variables despite statistically significant differences in the mean of the sample populations, as determined by ANOVA. A sensitivity analysis breaks the levels into further sublevels to find underlying trends, if they exist. The following sections analyse the relationship between roughness and modulus when categorised first by environmental zone and then binder type.

The modulus of a road segment may not appear to correlate with roughness at first, producing linear R2 value of 0.0018. When the samples are grouped by environmental zone followed by binder types, correlations appear, as seen in Figure C 72. There appears to be a stronger negative correlation between DR and modulus, there appears to be no relationship with wet/nonreactive. Another factor that could influence this relationship is the binder type.

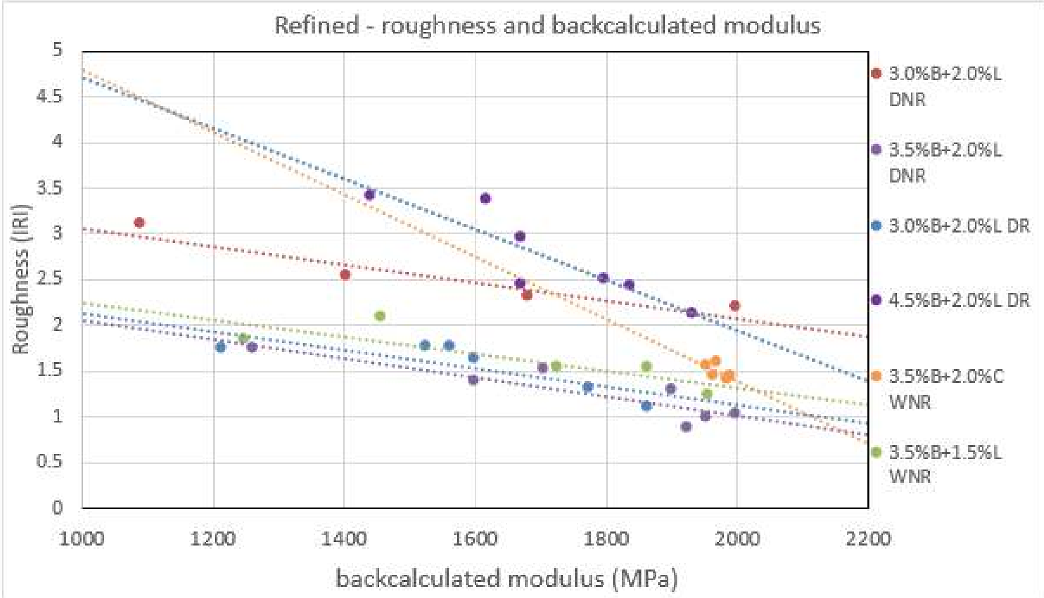
The mean and standard deviations for roughness are shown in Figure C 72, which groups the roughness by environmental zone and then binder types. As seen in the figure, the WR region only has one binder type, and one sample point (hence no standard deviation), while the WNR region has four binder types. The maximum average roughness is observed for the 3.5%B+1.5%L binder type in the WNR region while the lowest is the 3.0%B+2%L in the same region. The linear regression is shown in Figure C 63.

Figure C 72: Roughness vs. E1 modulus grouped by binder types within each environmental zone



The coefficients and intercepts of the trend lines are given in Figure C 73.

Figure C 73: Roughness vs. modulus grouped by environmental zone and binder type



The following was observed and is summarised in Table C 14:

- WR: There were not enough sample points in this category.
- WNR: When controlling for environmental zone, WNR regions appear to have little correlation with the modulus due to the near 0 coefficient; however, the mean roughness does vary with binder type. The binder types found in WNR regions were 3.5% bitumen with 1.5% lime or 2.0% cement. The cement had a lower mean IRI of 1.5 compared with 2.01 for 1.5% lime. This would suggest that cement, in a WNR region will probably produce lower roughness in a WNR region, and increasing modulus has relatively little effect.
- DNR: In DNR regions, the binder type was predominantly 3.5% bitumen and 2.0% lime with some pavements of unknown composition. There is a negative correlation between this binder type and modulus with a coefficient -0.0009 and an average IRI of 2.95. Of all the binder types this had the highest correlation with modulus producing an R2 value of 0.57.
- DR: Two types of binder were present in these regions namely 3.0% bitumen and 2% lime, and 4.5% bitumen and 2% lime. Maintaining the lime content constant, the roughness appears to increase with increasing bitumen content. The correlation with modulus is weak to nonexistent for the 3.0% bitumen case, but has an average IRI of 1.6157 compared with the 4.5% bitumen content, which has a negative correlation, a coefficient of -0.0002, and an average IRI of 3.09.

Table C 14: Linear regression for roughness against modulus when grouped by region and binder type

Region		Bitumen (%)	Lime (%)	Cement (%)	Coefficient	Intercept	R2
Wet	Nonreactive	3.5	1.5		-0.0009	3.17	0.6945
				2.0	-0.0034	8.2	0.4145
Dry	Reactive	3.0	2.0		-0.001	3.14	0.6726
		4.5			-0.0028	7.47	0.7826
	Nonreactive	3.0	2.0		-0.001	4.05	0.9046
		3.5			-0.001	3.09	0.7934

The statistical description by environmental zone, AADT bin and binder type is shown in Figure C 74 and the relations for linear regression against normalised deflection curvature are shown in Figure C 75.

Figure C 74: Statistical description of roughness vs. average normalised deflection curvature grouped by environmental zone, AADT and binder type

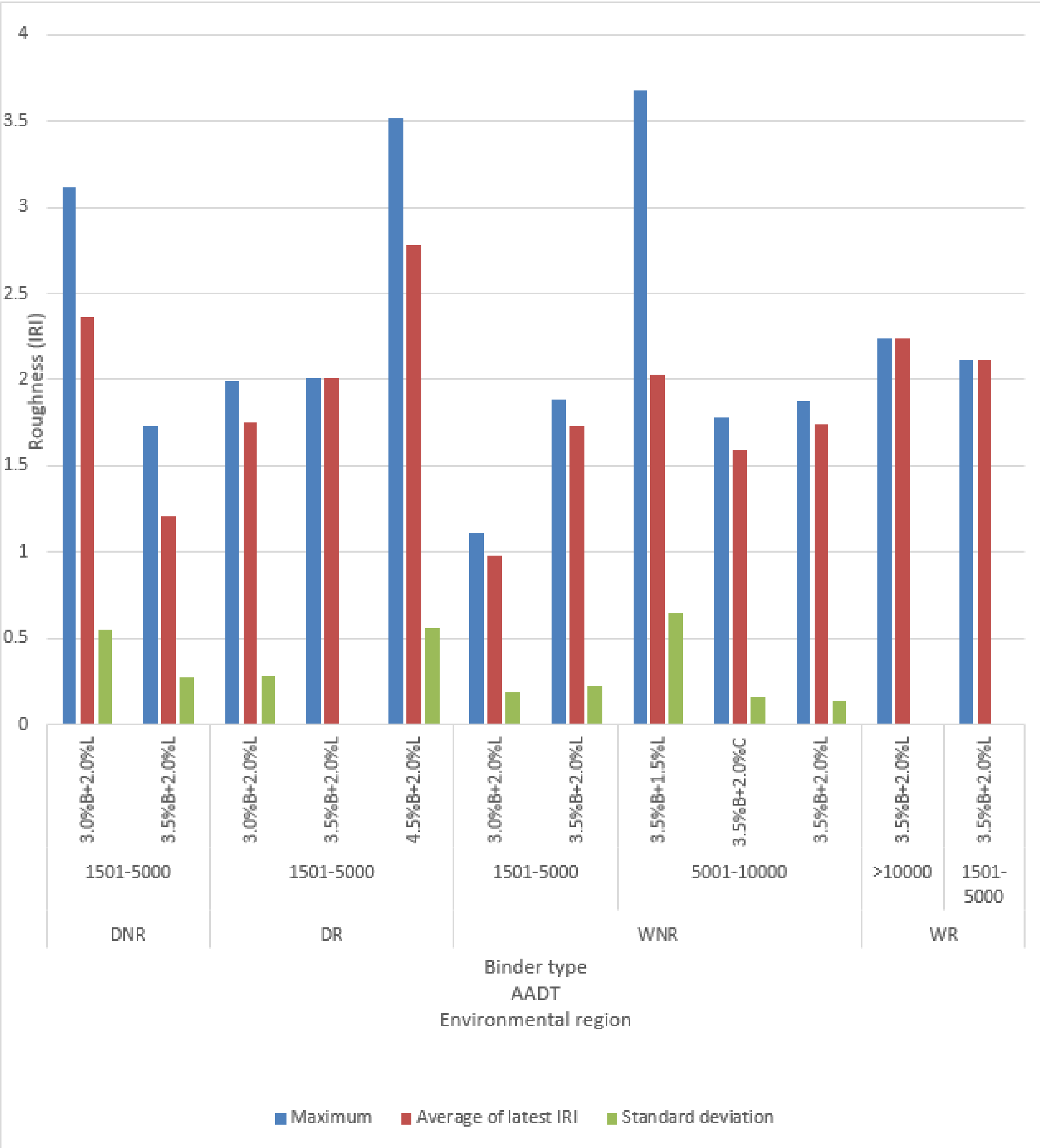
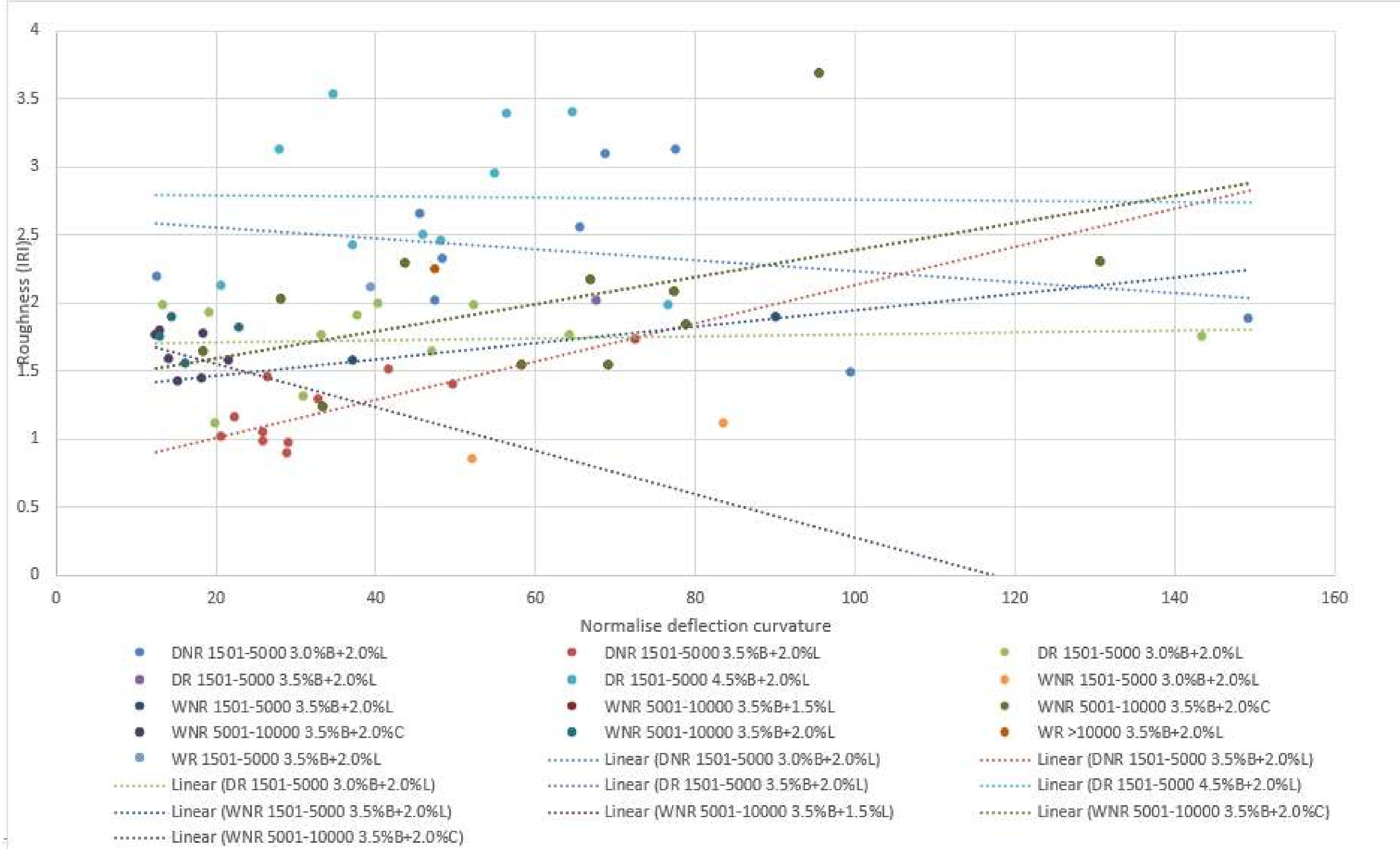


Figure C 75: Roughness vs. normalised deflection curvature against normalised deflection curvature when grouped by AADT, and binder type



The following can be concluded from the statistical analysis:

- The AADT range is only from 1500–5000 for the DR and DNR regions.
- WNR has AADT ranging from 1500–10 000 and has the greatest variety of binders used, which are:
  - 3.0%B+2.0%L
  - 3.5%B+2.0%L
  - 3.5%B+1.5%L
  - 3.5%B+2%C
- The maximum average roughness is approximately 3.7 for 3.5%B+1.5%L binder and is on a segment of road that experiences an AADT of 5001–10 000 in a WNR.
- The DNR, DR and WNR levels all have 3.0%B+2.0%L at the same AADT of 1501–5000; however, IRI is much higher for the DNR at approximately 3.1 compared to the DR value of 2, while WNR has the lowest at just over 1.1.
- The WR region only has two binder types and one sample point per binder and they are both 3.5%B+2.0%L.