FINAL REPORT

Project Title: P2: Stabilisation Practices in Queensland (In Situ Cement/Cementitious Stabilised Materials) (2017-2018)

ARRB Project No: PRP16024

Author/s: Joe Grobler, Joseph Coomer & Lincoln Latter

Client: Queensland Department of Transport and Main Roads

Date: June 2018

AN INITIATIVE BY:

SUMMARY

In situ stabilisation technology using cementitious binders has been applied to the structural rehabilitation of existing road pavements in Queensland on the basis of:

- utilising existing pavement materials, thereby reducing the need to import new materials
- improving the properties of in situ marginal materials
- providing an expedient and economic construction process
- providing pavement materials with a higher tolerance to damage from flooding.

As a systematic approach for evaluating alternative pavement rehabilitation treatments, in situ stabilisation has been traditionally based on historical local practice and experience rather than a holistic approach to an optimal 'triple bottom line' solution considering:

- economic costs reflected in lower construction costs and expedient processes
- environmental benefits reflected in reduced consumption of new quarried materials, use of industrial wastes as binders and additives and reduced greenhouse gas emissions from expedient construction processes
- social impacts reflected in shorter traffic delays, flood resistant pavements with shorter periods of road closures due to flooding.

This report presents the preliminary work conducted by the Australian Road Research Board (ARRB) as part of the National Asset Centre for Excellence (NACOE) project titled *Stabilisation Practices in Queensland*. The purpose of the project is to provide technical guidance on the ideal climatic, environmental and operational conditions to maximise the benefit-cost ratios of in situ cement/cementitious modified base (I-CMB) stabilisation technologies in Queensland.

The work undertaken as part of the project included:

- a review of available literature to determine current best practice
- a summary of I-CMB pavement inventory and condition data
- selection of pavement sections that represent standard practice in Queensland
- a desktop visual assessment from video data
- a structural capacity assessment
- a statistical analysis to investigate possible relationships between inventory, condition and performance data
- an evaluation of the capital costs associated with I-CMB stabilisation technologies.

Conclusions from the investigation include:

- The literature review typically showed that Queensland practices are generally aligned with national and international best practice when considering construction techniques. However, minor differences in stabilisation binder content, spread rate, 7-day unconfined compressive strength (UCS) values and compaction were observed.
- The controlling failure mode considered by TMR in the design of cement/cementitious modified pavements (i.e. permanent deformation of the subgrade reflecting through the overall pavement structure) differs with some current practices internationally. The South African approach considers the fatigue, permanent deformation and crushing potential of lightly bound materials with UCS values similar to TMR specifications. The current approach in the USA considers permanent deformation in the cement/cementitious modified material.
- I-CMB pavement sections are utilised extensively across Queensland, representing approximately 18.5% of the state-controlled road network. The technology is most commonly utilised in the North-West, South-West and Fitzroy districts. A total of 32 road sections were selected for detailed visual and structural capacity assessment.
- Approximately 64% of I-CMB pavements in the network are less than 4-years old and 94% are less than 12-years old.
- Condition categorisation showed that 87% of the I-CMB pavements along the state-controlled road network were in a good condition at the time of this investigation.
- Results from the structural assessment on the representative pavement sections showed that many of these sections have significant (> 10^6 ESA) structural capacity remaining.
- The statistical analysis undertaken found that the environment in which the I-CMB pavements operate has the biggest influence on performance, followed by cement content and total pavement thickness.
- The approximate initial costs of constructing I-CMB pavements ranged from \$14/m^3 to \$28/m^3 where the in situ granular material is of sufficient quality, which is significantly less than importing high quality granular materials.

The following recommendations are made for consideration by TMR:

- A large proportion of the I-CMB network assessed (based on the data in ARMIS) showed evidence of fatigue cracking, i.e. while the design intent would have been to construct a modified pavement, in fact a lightly bound or bound pavement was the result. There may therefore be a need to control the tensile strength developed in cement modified layers (through the mix design process) to ensure that the material behaves as a modified material as per the current design assumption in Queensland.
- The durability (i.e. erosion potential) of stabilised materials is an important property that impacts on the long-term performance of stabilised pavements. TMR currently does not specify any erosion requirements or test methods. There are a number of test methods discussed in the report that could potentially be considered for adoption by TMR to assess the durability of stabilised materials.

This page is intentionally left blank

CONTENTS

TABLES

FIGURES

1 INTRODUCTION

The Queensland state-controlled road network includes approximately 33 300 km of roads connecting an area of roughly 1 850 000 km^2. The Queensland Department of Transport and Main Roads (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 with widely varying prevalent source materials, environmental conditions and historical approaches across the state. These parameters have purportedly given rise to regional bias towards particular stabilisation practices. As a result, stabilisation technologies with a history of satisfactory performance are often selected instead of unfamiliar or previously unsuccessful technologies. Development of a systematic approach for the 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 technologies in Australia. Significant 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 (e.g. foam bitumen replacing hot mix asphalt layers). The effectiveness and costs and benefits of different stabilisation techniques, including cementitious modification are 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 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 current study continues on from the National Asset Centre of Excellence report into plant-mixed cement modified base (PM-CMB) and in situ mixed foam bitumen stabilised base (I-FBS) technologies.

This report focussed on the assessment of in situ lightly bound (also known in Queensland as ‘modified’) cement or cementitious (cement/cementitious) stabilised pavements across the network.

1.1 Purpose

The purpose of this project is to provide technical guidance on the ideal environmental and operational conditions to maximise the benefit-cost ratios of in situ cement/cementitious stabilisation technologies utilised on the state-controlled road network. This includes the preparation of a report reviewing stabilisation practices in Queensland, in addition to the development of TMR technical notes to transition best practice engineering knowledge into practice. This report documents the findings of a review undertaken of current in situ cement/cementitious modified (I-CMB) practices in Queensland undertaken during 2015/16 and 2016/17.

1.2 Objectives

The main outcome of the multiyear project includes technical guidance on:

- appropriate stabilisation technologies for a given budget, material characteristics, subgrade, environment, traffic, resilience and performance requirements
- identifying factors associated with stabilising non-conforming materials as a cost-effective alternative to unbound granular overlays or layer replacement and/or hot mix asphalt layers in various pavement configurations considering traffic and environmental conditions in Queensland
- benchmarking TMR stabilisation practices with national and international practices
- reviewing the performance of I-CMB pavements on TMR’s road network.

1.3 Approach

The development of technical guidance for selecting the best stabilisation technology based on costs and benefits for utilisation in typical Queensland conditions was accomplished through:

- reviewing available literature to determine current best practice
- obtaining and summarising I-CMB pavement inventory and condition data
- selection of pavement sections that represent standard practice in Queensland
- undertaking a desktop visual assessment from video data
- conduct of a structural capacity assessment
- conduct of statistical analysis to investigate possible relationships between inventory, condition and performance data
- undertaking an evaluation of the capital costs associated with I-CMB stabilisation technologies.

2 STABILISATION TECHNOLOGIES

Stabilisation in pavement engineering is a process that alters the engineering properties of soil or aggregate by adding a fixed quantity of stabilisation binder. The significance of stabilisation is that marginal road materials may be used for construction with the addition of relatively small amounts of stabilisation binder, thus increasing the environmental sustainability and cost-effectiveness of road construction projects (Paige-Green 2008). The engineering properties that can be improved through stabilisation include particle size distribution, plasticity, bearing capacity, moisture resistance, workability and permeability. Additionally, in environments with excessive moisture, stabilisation may be used to dry pavement materials (AustStab 2012).

The environmental and economic benefits of stabilisation are associated with the use of marginal road materials that would otherwise be ripped up, transported and discarded for replacement with high quality, quarried virgin aggregate (Smith 2005). Austroads (2006) states that stabilisation may also reduce the whole-of-life costs of heavily trafficked pavements. As the stabilisation binder typically represents approximately half of the total cost of stabilisation, the direct and whole-of-life costs may be reduced by ensuring the design and construction is optimised for its application (Austroads 2002a).

Australia is recognised as a world leader in stabilisation but the increasing social, environmental and economic pressures on government require the continuous development of stabilisation technology and practice to meet these increased demands (Wilmot 1996). The vast length of the Australian road network, shortage of high quality materials and low taxpayer base, positions stabilisation to remain a key technology in pavement construction. However, Australian state and territory road agency specifications vary considerably (Wilmot 2006). Variations include the treatment selection, mixture proportioning, structural design and construction techniques adopted. These variations are typically due to the differences in prevalent materials, climates and historical approaches and experiences.

2.1 Types of Stabilisation

Stabilisation binders commonly used by national and international road agencies include granular materials, Portland cement and cementitious blends, lime, bitumen and polymers. Cementitious blends include lime and/or cement together with pozzolanic additives such as fly ash, ground granulated blast furnace slag (GGBFS) or silica fume. Austroads (2006) categorises stabilisation by the stabilised material's performance characteristics and type of material stabilised, which include subgrade, granular, modified, lightly bound and bound stabilisation.

2.1.1 Subgrade Stabilisation

Subgrade stabilisation can be undertaken when the natural subgrade material is deemed to be unsuitable, either in terms of weak bearing strength or reactivity (i.e. swelling and shrinkage in reactive clays). Subgrades are often treated with cement, lime or other cementitious binders (Putra 2014). Stabilisation impacts the properties of the material and can reduce plasticity and swell potential while improving strength, stiffness, workability and durability (Riaz et al. 2014). Subgrade stabilisation has the biggest benefits for weak materials, designated by having a California Bearing Ratio (CBR) less than 8% (Wilmot 2006).

2.1.2 Granular Stabilisation

Granular stabilisation is the improvement in either the particle size distribution (PSD) and/or plasticity and/or the aggregate hardness of a granular pavement material by blending it with a secondary material of different PSD, higher quality aggregate, sand or clay (TMR 2012). However, granular stabilisation does not add tensile strength or increased particle cohesion to the parent material. If granular stabilisation cannot sufficiently improve the properties of the parent material, then modification using stabilising agents should be conducted.

2.1.3 Modified Stabilisation

Modified stabilisation is the addition of small amounts of stabilisation binder to the parent material to improve the engineering properties, notably the moisture susceptibility, strength, cohesion and bearing capacity while maintaining the performance characteristics of an unbound granular material (Austroads 2006). It is essential that little or no tensile strength is developed in modified pavement layers, which could lead to tensile fatigue mechanisms (failure through pavement flexure) being introduced into their performance. For this reason, an upper limit on strength is adopted in determining stabilisation mix proportions.

Modified stabilisation is a commonly adopted stabilisation technique in Australia due to its benefits, both economic and environmental. These benefits are mainly derived from the use of relatively small quantities of binder to improve the bearing capacity of pavement materials, as well as utilising sub-standard in situ materials without the need to replace it.

2.1.4 Bound Stabilisation

Bound stabilisation is achieved by the addition of larger amounts of cementitious stabilisation binder to granular materials. Bound materials support traffic loads in flexure and therefore have significant tensile strength. The pavement thickness is therefore designed to resist tensile fatigue. However, with higher quantities of stabilisation binder these materials are susceptible to binder shrinkage (plastic) cracking as well as temperature-related cracking. Their traffic-induced failure mechanism is by tensile fatigue cracking at the bottom of the layer reflecting through to the surface. Bound pavements are typically considered in floodways or soft subgrade situations as well as high-trafficked conditions when associated with thick asphalt layers.

2.2 Characterisation of Stabilised Materials

Characterisation of stabilised materials is achieved through laboratory testing to determine the site-specific engineering properties indicative of performance. These include compatibility of the stabilisation binder with the parent material, appropriate binder content and strength parameters to meet the conditions of a modified, lightly bound or bound material such as the unconfined compressive strength (UCS), indirect tensile strength (ITS) and the California Bearing Ratio (CBR) (Little & Nair 2009).

Stabilisation is most commonly utilised in the structural pavement layers (base and subbase) of sealed roads and may be either modified or bound depending on the required performance.

The modification of granular materials using small quantities of cementitious materials is desirable as it improves the performance attributes of the material while maintaining the properties of an unbound granular material. Modified layers typically contain a stabilisation binder content of between 1% and 3% by mass and a 28-day UCS of between 0.7 and 1.5 MPa (Austroads 2006). Modified materials maintain the performance characteristics of an unbound granular material and therefore should be characterised and modelled accordingly.

Bound materials contain stabilisation binder contents typically greater than 3% and 28-day UCS greater than 1.5 MPa (Austroads 2006). However, bound materials exhibit significantly higher stiffness than modified materials and are more prone to shrinkage and fatigue cracking. The resilient modulus of the material may also be used for the mixture design process, where the interface between modified and bound materials is at approximately 1500 MPa (Gray et al. 2011).

The methods currently adopted for characterising stabilised materials are not based on in situ pavement data. Typically, testing includes the particle size distribution (PSD), Atterberg limits, compaction characteristics and strength parameters such as UCS, ITS and CBR. However, laboratory testing under ideal conditions may not adequately model the in situ construction conditions (Paige-Green 2008). Materials incorrectly characterised as bound may undergo rapid structural deterioration if sufficient strength and durability is not achieved. Similarly, materials incorrectly characterised as modified may develop excessive tensile capacity and stiffness leading to fatigue cracking. Therefore, the benefits of stabilisation are inconsequential if the stabilised material does not meet the design intent and may require expensive maintenance or rehabilitation (Gray et al. 2011).

2.3 Construction of Stabilises Layers

Stabilised pavement materials are produced through utilisation of either centralised mixing plants (plant-mixed) or mobile mixing plants (in situ) (Austroads 2006). Plant-mixed stabilisation is conducted near the source of the parent material and is generally used when the material is sourced from a single supplier, a high degree of uniformity is required and the haul distance to the construction site is relatively short (Austroads 2009a).

In situ stabilisation, however, is undertaken at the construction site and typically involves surface preparation, incorporation of stabilisation binder, compaction, curing and quality conformance testing. The powder binder distribution is carefully controlled over the stabilisation area using specialised powder binder spreaders (Figure 2.1). Sufficient quality conformance tests must be conducted following stabilisation to ensure the desired level of modification and consistency has been achieved. These tests typically include binder spread rates, compacted densities, moisture contents and in some cases the UCS of the stabilised material post-construction.

Figure 2.1: Modern powder binder spreader and recycler/reclaimer

2.4 Performance of Stabilised Materials

The long-term performance of pavements is dependent on the stability and strength of the pavement layer's subgrade materials). Stabilising the parent materials may provide greater strength, stiffness and durability than unbound granular materials (Little & Nair 2009). However, stabilised materials (more specifically fully bound materials) may also exhibit shrinkage cracking due to the inherent stiffness increases associated with modification (Lay & Metcalf 1983).

Materials stabilised with pozzolanic materials may experience a considerable increase in strength over time depending on the reactivity of the soil minerals with the stabilisation binder (Little & Nair 2009).

The performance improvements associated with stabilisation may degrade over time due to traffic loading and cracking such that these materials have similar properties to unbound granular materials (Dunlop 1980). The stiffness of cement/cementitious stabilised materials is often characterised through the resilient modulus of the modified material (Saxena et al. 2010). The degradation in structural capacity is indicated by decreases in the resilient modulus under traffic loading (Lay & Metcalf 1983).

The primary failure modes associated with stabilised pavements vary according to the degree of stabilisation. Modified pavements primarily fail due to permanent deformation of the subgrade reflecting through the pavement, characterised by rutting. Where shear strength of the pavement material is compromised (e.g. moisture ingress), shoving failures can occur but are not a design condition. Bound materials primarily fail in tensile fatigue, often manifesting in the form of surface cracking and erosion leading to permanent deformation.

2.5 Stabilisation Practices in Queensland

Historically, TMR has adopted the following in situ stabilisation practices (TMR 2012):

- granular (mechanical) stabilisation
- stabilisation (with cementitious binders, lime or foamed bitumen) to achieve a bound material
- modification (with cementitious binders or slow setting additives) to achieve a modified or lightly bound material.

The preferred stabilisation process will depend on the intended application within the pavement structure and is influenced by factors such as the quality of the parent material, traffic loading, climate, pavement drainage and configuration.

TMR classifies cement treated materials into two categories, i.e. bound or modified. Bound materials are characterised by a UCS value of greater than 2.5 MPa, whereas cement modified materials have a UCS of between 1 and 2 MPa at 28 days (TMR 2013).

In contrast, Austroads limits the UCS of a cement modified material to 1 MPa at 28 days (Austroads 2012), which is lower than the upper limit adopted by TMR for modified materials. It is understood that TMR has good experience with cement modified bases with a 28-day UCS of between 1 and 2 MPa as a result of (TMR 2013):

- reduced moisture sensitivity, permeability and erodibility
- higher strength and stiffness
- reduced sensitivity to variations in the properties of the parent material
- higher binder contents being easier to achieve and control during the construction process.

Guidance for the design and construction of in situ cement stabilised or modified granular materials in Queensland is provided in the following TMR documents:

- *Pavement Rehabilitation Manual* (TMR 2012)
- *Pavement Design Supplement* (TMR 2013)
- *Technical Note 149: Testing of Materials for Cement or Cementitious Blend Stabilisation* (TMR 2017a)
- *Insitu Stabilised Pavements using Cement or Cementitious Blends* (TMR 2017b)

In situ stabilised pavement layers are typically constructed in Queensland where (TMR 2012):

- it is economical to do so based on the quality of the in situ materials to be stabilised, and the dosage rate required
- where traffic management arrangements allow for the construction and curing of the stabilised layers

- where the reuse of existing materials is beneficial and high quality imported materials are not readily available or economical
- improvement of the structural capacity of the pavement is required
- a reduction in the moisture sensitivity of the pavement is required.

3 IN SITU STABILISATION USING CEMENT/CEMENTITIOUS BINDERS

In situ stabilisation techniques have been widely used by both Australian and international road authorities for pavement construction and rehabilitation. It is the most effective technique to improve the stiffness of pavements layers, thus reducing stress in the underlying layers and subgrade (Saxena et al. 2010). The use of in situ stabilisation has become more common in recent times due to economic, environmental and social considerations. In situ cementitious stabilised pavement layers, when used for rehabilitation of existing roads, commonly produces savings of up to 30% when compared to reconstruction alternatives (Smith 2005).

The aim of stabilisation using cementitious modifiers is to improve the load bearing capacity and/or stability of inadequate materials to achieve the desired performance for pavement applications (Austroads 2009b). Typical improvements resulting from cementitious stabilisation include a reduction in plasticity and an increase in shear and bending strength, thus increasing the overall performance of the material (Adamson 2012). Cementitious modification typically involves the treatment of soil or aggregate with small amounts of Portland cement and/or other supplementary cementitious material (SCM) to improve the engineering properties of an unsuitable construction material (American Concrete Institute 2009).

Although the structural capacity of cementitious modified materials is not as high as cementitious bound materials, the potential for shrinkage cracking is reduced (Garber et al. 2011). Low binder contents minimise the potential for internal cracking while maintaining the flexibility of an unbound pavement layer (Dunlop 1980). Improvements developed through cementitious modification are permanent in certain silt/clay soil materials, thus making modification an effective method of producing strong, durable and sustainable pavements (Halsted 2011).

In situ cement stabilisation offers a more sustainable global rehabilitation option as it reduces waste, preserves virgin aggregate sources, minimises material transportation and increases the stability of construction platforms while reducing the volume required for overlaying materials (Garber et al. 2011). Dunlop (1980) recommends that the use of cement stabilised material should increase in future to preserve high-quality virgin aggregates and reduce transportation requirements for construction.

3.1 Applications

Where possible, in situ stabilisation is the universally preferred pavement recycling method due to the economic advantages compared to plant-mixed methods (Wirtgen 2012). However, there are cases where the in situ material is transported and processed through a pugmill plant and returned to site for placement (e.g. bitumen foaming plants). In situ stabilisation is commonly utilised in the base, subbase or subgrade courses of a pavement structure (Saxena et al. 2010). Stabilising the basecourse of pavement layers using local materials is a sustainable alternative to depleting and transporting high quality quarried aggregates to the construction site (Dunlop 1980).

In situ stabilisation can be used where the materials to be stabilised are suitable for stabilisation, the binder application rate is economical, the existing pavements have relatively thin asphalt or sprayed seal surfacing (less than 80 mm) and the existing traffic can be readily managed to accommodate the stabilisation process during construction (TMR 2012). In cases where thick bituminous surfacings exist a proportion may be profiled off and sent to recycling depots for other recycling opportunities.

However, in situ cement/cementitious stabilisation should generally be avoided where the in situ materials are not suitable for stabilisation (i.e. require uneconomical quantities of binder or are highly variable), the underlying support layers are weak or inadequate, shallow underground services are present, or where the stabilised materials may impede pavement drainage (TMR 2012).

3.2 Materials

The performance of an in situ cement/cementitious stabilised pavement layer is significantly influenced by the type, quality and proportions of the different components. The materials used in cement/cementitious stabilised layers are comprised of three main constituents, i.e. soil or aggregate, cement/cementitious binder and water. Chemical admixtures may also be used to improve handling or working time of the treated materials.

3.2.1 Soil or Aggregate

The parent material for cement/cementitious stabilisation can include coarse and/or fine aggregates, as well as industrial by-products such as foundry sand, bottom ash or boiler slag.

Coarse aggregates include natural and some industrial wastes (e.g. blast furnace slag) with a nominal maximum aggregate size (NMAS) greater than 2.36 mm. Fine aggregates include natural and manufactured rocks with NMAS less than 2.36 mm. Industrial by-products such as fly ash and silica fume are classified as fines (American Concrete Institute 2009).

Cementitious stabilisation is ideally suited for well graded soils that contain sufficient quantities of fines to ensure the voids in the base are filled post-stabilisation. Although, for soils containing medium, moderately fine and fine-grained soils, lime has been found to successfully decrease plasticity and swell while increasing workability and strength properties (Little & Nair 2009).

Well graded aggregates consisting of sand and gravel blends typically require small amounts of stabilisation binder, whereas silty, clayey or poorly graded sandy materials require larger quantities in comparison to achieve similar levels of quality (Garber et al. 2011). Research conducted by Symons and Poli (1999) found that well graded materials exhibited the best properties of those tested post-stabilisation and that as the fines content increased, the physical properties of the stabilised soils decreased. The stabilisation of silt-clay (> 35% passing a 0.075 mm sieve) soils and granular (< 35% passing a 0.075 mm sieve) soil differs by the stabilised performance targets.

Soils containing industrial by-products should be closely monitored during stabilisation as the chemistry of the materials may significantly impact the cementitious hydration processes, altering the design properties of the material (American Concrete Institute 2009).

The presence of deleterious compounds, such as sulphates, organics or ferrous oxide in the material to be stabilised can also inhibit the stabilisation reactions (TMR 2017a).

3.2.2 Cementitious Binder

The most common cementitious stabilisation binders used by TMR are:

- GB cement
- Slagment (75% cement:25% slag)
- Pozzoment (75% cement:25% fly ash)
- Stabilment (85% granulated slag:15% hydrated lime)
- Triple blends (lime/slag/fly ash, 30:50:20 or 33:33:33).

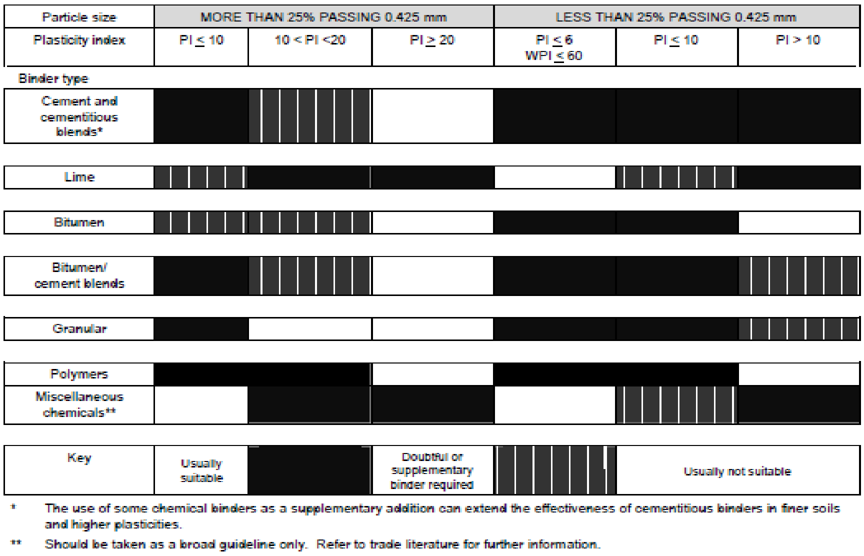
The type and quality of the cementitious binder used for stabilisation has a significant impact on the properties of the granular material. Cementitious stabilisation binder can contain a combination of pozzolanic material and Portland cement and/or lime. Typical agents include general purpose

(GP) and blended (GB) Portland cement, hydrated lime, quicklime, lime slurry, fly ash, ground granulated blast furnace slag and double and triple blends of these agents (Austroads 2006). However, not all the stabilisation binders are appropriate for use with all types of parent material and the material-binder combinations should be taken into consideration during the design process (White 2006).

Blends of GB cement and blends of hydrated lime with pozzolanic materials (fly ash and slag primarily) are preferable to GP cement for stabilisation, as they have longer setting times, thus increasing the allowable working time of the stabilised material. GB cement also utilises waste products such as fly ash, providing an environmentally sustainable and cheaper option than using GP cement (AustStab 2012). However, lime should be considered as the primary stabilisation binder when stabilising fine soils with significant plasticity, as the amount of binder required to stabilise fine soils with cement can be substantially higher than the amount required to stabilise well graded soils (Little & Nair 2009).

A guide to selecting an appropriate stabilisation binder type (based on the properties of the parent materials is provided in Austroads and shown in Figure 3.1.

Figure 3.1: Guide to selecting appropriate stabilising binders



Source: Austroads (2006).

3.2.3 Water

The water content is an essential component of cement treated materials and must be carefully controlled to ensure that the strength is achieved. Not enough water will not sufficiently activate the cement, causing an incomplete reaction that could result in reduced strength, whereas providing too much water also reduces the strength properties of the cement treated material (Yeo et al. 2011). Water quality must also be monitored and should be free of organic materials, sulphates and salinity, as this can impact the quality of the stabilised material (AustStab 2012).

3.2.4 Chemical Admixtures

The treatment of a soil with chemical admixtures involves using a chemical compound to modify or enhance the physical and/or engineering properties of a soil. This may result in an alteration of properties such as improved volume stability and strength (Huat et al. 2005). Chemical admixtures however, are not typically utilised in cement/cementitious stabilisation.

3.3 Cement Stabilised Material Properties

The properties of a cementitious-treated pavement layer are dependent on the parent material, stabilisation binder type and quantity, moisture content, degree of compaction, mixing uniformity, curing conditions and the mixture age (American Concrete Institute 2009). Stabilisation using cementitious binders alters the properties permanently, and may include increased strength, improved durability, decreased moisture susceptibility and an increase in workability (Committee of State Road Authorities 1986).

However, the permanency of the strength gain can be comprised if the stabilised material undergoes carbonation in the field. Carbonation is the process whereby cementitious binders or hydration products react with carbon dioxide in the atmosphere or soil, causing the stabilisation process to be reversed (Gautrans 2004). This reversal process could result in loss of strength, an increase in the plasticity index of the parent material, blistering of the stabilised layer or movement of the surfacing on a basecourse.

The true benefits of cementitious modification in pavement layer applications are difficult to assess due to the uncertainty regarding structural properties (Dunlop 1980). However, the improvements to the material properties can be quantified by measuring the CBR, compressive strength, tensile strength, modulus, moisture sensitivity, and durability and permeability of the stabilised material. The particle size distribution, unit weight and stabilisation binder content of the cement stabilised material also influence the performance (Little & Nair 2009).

3.3.1 Compressive Strength

The unconfined compressive strength (UCS) test is a relatively simple test that is most commonly used in the mix design of stabilised materials (Lim & Zollinger 2003). UCS values can be used to provide an indication of the modulus of the material for structural design purposes, as well as a quality control measure during construction (Austroads 2002b). High UCS values are directly proportional to the durability of a stabilised material, whereas the durability is inversely proportional to the shrinkage potential of the stabilised material (Scullion et al. 2005). It is important to note that variations in the curing conditions in the laboratory and the field may cause UCS values to differ significantly and this should be considered during testing (Austroads 2002b).

The UCS of cement treated materials can be determined in accordance with Test Method Q115: *Unconfined Compressive Strength of Stabilised Materials* (TMR 2016a).

Stabilisation may be used for a large range of host materials and binder types of varying quantities. Therefore, providing unique strength requirements is not necessarily possible based on UCS testing. For modification, specifying a maximum strength rather than minimum strength gains may also be considered depending on the desired application (Austroads 2002a).

3.3.2 Tensile Strength

The tensile strength of cement treated materials is measured by the resistance a material exhibits to breaking or deformation when subjected to bending stresses. Tensile strength is commonly measured using the flexural beam or indirect tension tests.

The tensile strength of pavement materials increases with increasing binder content, where unstabilised granular materials typically have no tensile strength and bound materials have significant tensile strength (Austroads 2006). Kennedy (1983) found that cement treated bases in the USA have a tensile strength of between 570 kPa to 820 kPa. Dunlop (1980) recommends that the tensile strength of a material should be less than 80 kPa (when cured for seven days at 20 °C) to maintain its unbound material mechanical characteristics.

Extensive testing of subgrade and unbound granular materials has indicated that cement/cementitious modified materials should have a maximum flexural strength of 100 kPa, maximum indirect tensile strength (ITS) of 150 kPa and maximum UCS of 1.0 to 1.2 MPa (Austroads 2013). Considering that TMR targets a UCS of 1.5 MPa for cement/cementitious modification, there is a possibility that cement/cementitious modified pavement layers in Queensland may behave as a bound material, rather than an unbound granular material currently being assumed in TMR's design system.

Finally, Doshi and Guirguis (1983) found a quantitative correlation between UCS and the indirect tensile strength of stabilised soils, generally deeming it unnecessary to conduct all strength tests for a pavement, especially considering the relative ease of the UCS test.

Flexural beam testing

The flexural strength of cement treated materials can be determined in accordance with the test method provided in *Cemented Materials Characterisation,* which uses a four-point bending apparatus (Austroads 2014). Cement treated beams are loaded at third-points of a clear span and subjected to an increasing vertical load until failure occurs. The flexural beam method is preferred given that the nature of the stresses and strains induced provides a better indication of the actual stresses and strains that a pavement is subjected to compared to other test methods (White 2006). For stabilised materials, preparation of suitable flexure beam test specimens is difficult (i.e. they are cut from slabs) and coupled with the limited availability of test apparatus the test is not commonly adopted in laboratory mix design methods.

Research has found that the ratio of flexural to compressive strength is approximately 33% higher for low-strength mixtures and approximately 20% lower for high-strength mixtures (American Concrete Institute 2009). Matanovic (2012) also found that the ratio of flexural strength to UCS ranged from 30% to 35% for cement stabilised materials incorporating TMR Type 2.1 aggregates.

Indirect tension testing

The ITS of cement treated materials can be determined in accordance with British Standards Institution 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* (2003). In this method, a cylindrical specimen is subjected to an increasing compressive load along the circumference of the specimen until failure.

The indirect tension test is not often used to characterise cement stabilised mixes in Australia due to the instability of the test specimens. Furthermore, Jameson (1995) identified that the indirect tension test underestimates the fatigue life of cement treated materials compared to other laboratory methods.

3.3.3 Resilient Flexural Modulus

Resilient flexural modulus is a measure of the stiffness of a material and increases with increasing stabilisation binder content and density, while decreasing as moisture content increases (Austroads 2002b). The modulus of cement treated materials is a crucial property for the mechanistic modelling in pavement design and the Austroads pavement design procedure adopts the flexural modulus for cement stabilised materials (Austroads 2012).

The flexural modulus of cement stabilised materials can be determined from laboratory flexural beam testing or estimated from presumptive values or correlations with other laboratory tests (Austroads 2012). The recommended test method to determine the flexural modulus of cement stabilised materials in Australia is provided in Austroads (2014) *which* uses a four-point bending apparatus. However, for stabilised materials, preparation of suitable flexure beam test specimens is difficult (they are cut from slabs) and coupled with the limited availability of test apparatus the test is not commonly adopted in laboratory mix design methods. As a consequence, estimates of flexural modulus are derived from UCS as shown in Equation 1 (Austroads 2012):

$$E = k\ x\ UCS \tag{1}$$

where

E	=	flexural modulus of field beams at 28-days moist curing (MPa)
k	=	values of 1000 to 1250 are typically used for general purpose cements
UCS	=	unconfined compressive strength of laboratory specimens at 28 days (MPa)

The modulus of cementitious stabilised materials will also increase with time and may cause increases in tensile stress at the bottom of the stabilised layer, often leading to tensile cracking (Dunlop 1980). It is for this reason that field specimens typically exhibit higher modulus results compared to laboratory prepared specimens (Austroads 2002b).

In Australia, the resilient modulus of cement modified materials can be characterised for pavement design purposes in a similar manner as for unbound granular material. TMR's preferred method is to determine the resilient modulus by means of repeat load triaxial testing using Test Method Q137: *Permanent Deformation and Resilient Modulus of Unbound Material* (TMR 2016a). Alternatively, presumptive values can be used with a maximum modulus value of 600 MPa (TMR 2012). It is also worth noting that the modulus of cement modified materials is moderated based on the underlying supporting conditions through a process called sub-layering, similar to the process used for unbound granular materials.

3.3.4 Drying Shrinkage

The hydration process of cement stabilised materials induces expansion, but once the hydration process is complete, water is lost through drying, and the material shrinks. The risk of shrinkage cracking rises with increases in stabilisation binder content and initial moisture content (water/cement ratio). Limiting these factors therefore generally reduces the risk of shrinkage cracking occurring (White 2006).

Drying shrinkage is reduced when stabilising non-reactive parent materials with a low plasticity index (PI), linear shrinkage and fines content. The use of efficient mixing, compacting slightly dry of optimum moisture content (OMC), using water-reducing or set-retarding chemical admixtures and slow-setting cements can also reduce drying shrinkage and the risk of shrinkage cracking (Austroads 2002b).

Austroads provides a procedure for determining the drying shrinkage of stabilised materials, which is similar to the Australian Standard *Test Method AS1012, Part 130-1970, Method for the Determination of Drying Shrinkage of Concrete* (Austroads 2002a).

3.3.5 Durability and Moisture Affinity

Durability is the ability of the material to resist deleterious effects from climatic conditions such as wetting, drying and temperature variations. Saxena et al. (2010) found that durability can be as significant to the performance of stabilised materials as stiffness. Long-term durability is determined by construction practices (curing and compaction) and the mixture proportioning used in the pavement layer. Burns and Tillman (2006) also found that an increase in fines content has a protective effect on the durability of the material when subjected to temperature variations. UCS and moisture affinity are commonly used to indicate long-term durability. Higher UCS values may improve durability, but by themselves do not ensure adequate durability is provided (Scullion et al. 2005). Austroads recommends that durability should be assessed for materials containing low quantities of stabilisation binder (Austroads 2002b).

TMR does not currently specify any durability requirements for cement/cementitious stabilised pavement materials. Furthermore, there is currently not a national standard test method available for determining the durability of cement/cementitious stabilised materials. However, regional and international durability test methods do exist, including:

- New South Wales Roads and Maritime Services (RMS) Test Method T133: *Durability of Road Materials Modified or Stabilised by the Addition of Cement* (RMS 2012)
- SANS 3001-GR55: *Determination of the Wet-Dry Durability of Compacted and Cured Specimens of Cementitiously Stabilized Materials by Hand Brushing* (2012)

The moisture affinity of a stabilised material can be determined in accordance with Test Method Q125D: *Capillary Rise of Stabilised Material* (TMR 2016a). In this method, the moisture affinity of compacted specimens measuring 100 mm in diameter and 115 mm in height 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. It should, however, be noted that there does not appear to be any guidelines on the relationship between this test and durability in the field.

Considering that durability can have a significant impact on the long-term performance of cement/cementitious stabilised pavement layers, it is recommended that TMR considers including durability requirements in its specifications.

3.3.6 Permeability

Permeability is the ability of a material to permit the flow of gases or liquids through its pore spaces. The permeability of pavement layers has a significant impact on the performance of the overall pavement structure, as many premature road failures occur due to moisture infiltration.

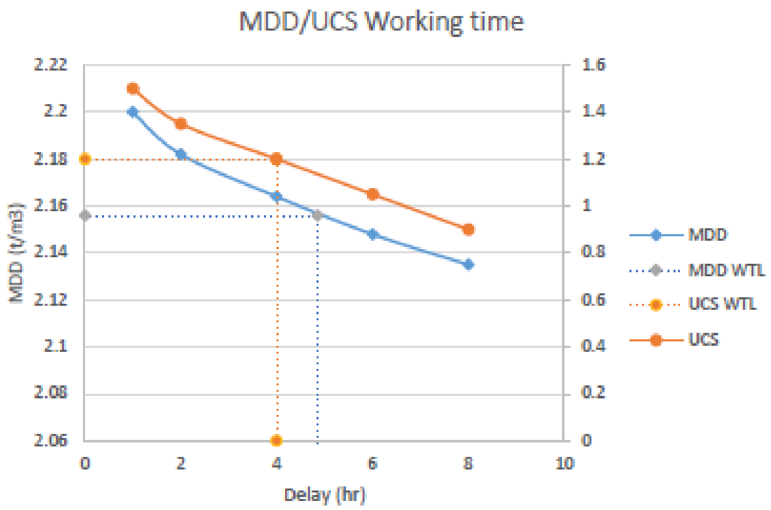
Permeability is a function of particle size distribution, air void content and compaction levels (Gerke 1981). The permeability of cementitious modified materials can be determined in accordance with Australian Standard 1289.6.7.3:2016: *Determination of Permeability of Soil: Constant Head Method Using Flexible Wall Permeameter* (2016). In this method, the permeability of the compacted specimen is determined by applying a constant 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.

3.3.7 Working Time

The working time of cement treated materials is defined as the time available to compact the material before the bonding between particles reaches a stage that prevents adequate compaction density being achieved (Austroads 2006).

Constructing cement treated layers within the allowable working time is critical to the long-term performance of the pavement. Working times for cement and cementitious binders range from 2 hours for general purpose (GP) cement and up to 24 hours for slow setting binders (e.g. lime/fly ash) (TMR 2017b). The working time for cement treated materials can be determined in accordance with Test Method Q136: *Working Time of Stabilised Material* (TMR 2016a). In this method, the working time of compacted specimens is determined by measuring the maximum dry density (MDD) and UCS at delayed intervals of compaction (Figure 3.2). The maximum working time is the delay time in compaction at which 3% of the MDD or 20% of the UCS is lost.

Figure 3.2: Working time example



Source: TMR (2017a).

3.3.8 Particle Size Distribution

The particle size distribution of the material to be stabilised has a significant impact on the development of structural strength in cement/cementitious stabilised pavement layers (Dunlop 1980). The cement/cementitious binder must form strong bonds with the parent material to reach the desired design strength and long-term performance. A study conducted by Symons and Poli (1999) found that well-graded materials develop greater UCS values when all other factors are equal. This research also established that an increase in fines content (passing a 0.0425 mm sieve) caused an increase in erodibility and permanent strain and a decrease in the UCS and resilient modulus of the stabilised material.

The particle size distribution of the parent material can be determined in accordance with Test Method Q103B: *Particle Size Distribution of Aggregate (Dry Sieving)* (TMR 2016a).

3.3.9 Density

The in situ density of cement/cementitious stabilised materials influences the long-term performance of the pavement and is impacted by the construction and curing methods adopted. Density has an impact on the modulus, permeability and durability of the stabilised materials (Austroads 2002b).

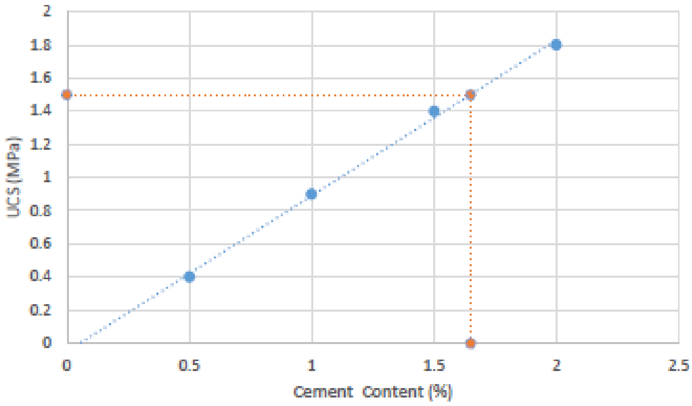
The maximum dry density and optimum moisture content of cement/cementitious stabilised materials can be determined in accordance with Test Method Q142A: *Dry Density-moisture Relationship of Soils and Crushed Rock (Standard)* (TMR 2016a). This method determines the maximum mass of dry material per unit volume and its associated moisture content by compacting the material using standard compactive effort.

The in situ density of the stabilised material can be determined by means of a nuclear gauge or the sand replacement method in accordance with Test Method Q141A: *Compacted Density of Soils and Crushed Rock – Nuclear Gauge* (TMR 2016a) and Test Method Q141B: *Compacted Density of Soils and Crushed Rock – Sand Replacement* (TMR 2016a).

3.3.10 Stabilisation Binder Content

The stabilisation binder content in a cement treated material has a significant impact on the long-term performance of the pavement and should be optimised for the intended use within the pavement configuration. Increasing the stabilisation binder content increases the strength of the material, but also increases susceptibility to cracking, which may shorten the fatigue life of the pavement (White 2006). TMR recommends that for determining the stabilisation binder content of cement treated material, Test Method Q134: *Stabilisation Binder Content (Heat of Neutralisation)* should be used (TMR 2016a). This method determines the stabiliser content by utilising an acetic acid solution to produce an exothermic reaction of free alkalis in the stabilisation binder. The rise in temperature of the mix at 60-second intervals is measured and used in a plot to determine the amount of stabilisation binder in the material. An example of UCS increase with increasing binder content is shown in Figure 3.3.

Figure 3.3: UCS vs cement content



Source: TMR (2017a).

3.4 Technical Specifications

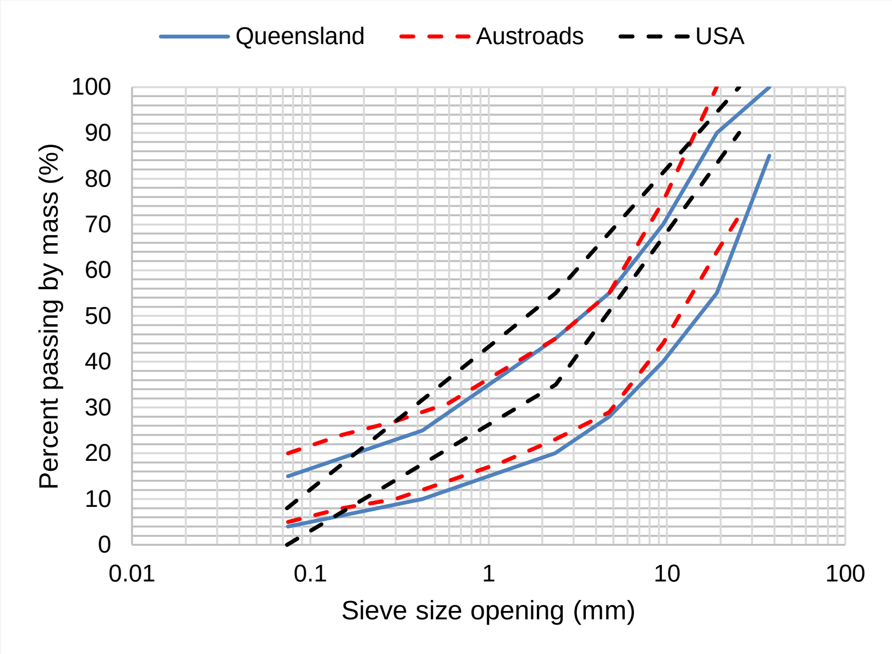
Technical specifications for cement/cementitious stabilised materials generally include requirements for the parent material, stabilising binder requirements, strength requirements, end product requirements, as well as construction requirements. TMR has developed and published technical specification MRTS07B *Insitu Stabilised Pavements Using Cement or Cementitious Blends* (TMR 2017b) specifically for the construction of in situ cement treated pavement layers. Some of the more important specification requirements are discussed below.

3.4.1 Parent Material

Cementitious stabilisation is best suited for well-graded materials with low levels of plasticity and, therefore, road agencies typically specify particle size distributions, fines' ratios and Atterberg limits. Well-graded soils are preferable to maximise the strength development gained through stabilisation where sand (4.75–0.425 mm) content should generally be limited to 70% by mass. The fines (< 0.075 mm) content should be limited to 12% but should have sufficient quantities as it increases cohesion, modulus and shear strength, while reducing permeability (Symons & Poli 1999).

A comparison of the particle size distribution specification limits between Queensland (TMR 2016b), Austroads (2006) and a representative road agency from the USA (Arizona Department of Transportation 2008) is presented in Figure 3.4.

Figure 3.4: Comparison of particle size distribution envelopes for parent material



Particle size distribution specifications for cement/cementitious stabilisation may vary significantly between road agencies, but common requirements include 100% passing the 37.5 mm sieve and a minimum of 30% passing the 4.75 mm sieve (Halsted 2011). Materials meeting the Arizona Department of Transportation (2008) specifications are finer than Austroads (2006) and Queensland (TMR 2016b) which have very similar particle size distribution requirements. The wide variations between the national and international specifications may be attributed to the differences in climatic conditions, available materials and usage. Australian cement/cementitious stabilised basecourses are generally used to support thin sprayed bituminous sealed roads. This requires sufficient coarse graded material to withstand traffic loads, as well as adequate fines to provide a smooth finish to assist spray sealing adhesion.

I-CMB layers are often constructed in areas where moisture is prevalent and therefore measures of moisture sensitivity are also commonly specified. These include properties such as linear shrinkage (LS) and soaked CBR. Commonly specified parent material properties of the national and international road agencies reviewed are presented in Table 3.1.

Table 3.1: Common parent material specifications for national and international road agencies

Road authority	Maximum liquid limit (%)	Plasticity index (%)	Linear shrinkage (%)	Fines ratio	Minimum soaked CBR (%)
TMR[1] (Queensland)	25	2 - 6	1.5 – 3.5	0.30 – 0.55	80
RMS[2] (New South Wales)	–	0 – 10	–	–	–
VicRoads[3] (Victoria)	–	0 – 10	–	–	–
MRWA[4] (Western Australia)	30	0 – 10	0 – 4	–	30
New Zealand[5]	–	0 – 5	–	–	80
South African National Roads Agency[6] (South Africa)	25	0 – 6	3 – 5	–	80
USA[7]	–	0 – 3	–	–	–

1 Source: TMR (2016b).
2 Source: Road and Maritime Services (2013).
3 Source: VicRoads (2008).
4 Source: Main Roads Western Australia (2012).
5 Source: Transit New Zealand (2006).
6 Source: South African National Roads Agency (2014).
7 Source: Arizona Department of Transportation (2008).

The national and international parent material specifications reviewed showed similar requirements, where provided. Queensland, Western Australia and South Africa specified a maximum liquid limit of between 25%–30%. The maximum plasticity index ranged between 3%–10%, while maximum linear shrinkage ranged between 3.5%–5.0%. Minimum soaked CBR values may be specified where soils are likely to be saturated. The road agencies reviewed showed general conformance with a value of 80% where a minimum was provided, the exception being Western Australia with a specified minimum of 30%.

In addition, chemical tests on parent material are undertaken to identify deleterious compounds which affect cementation e.g. organics, ferrous oxide and sulphates (TMR 2017a).

3.4.2 Type of Cement/Cementitious Stabilisation Binder

TMR allows for the use of cement, blended cement, cementitious blends and lime/fly ash blends to be used as cement/cementitious stabilisation binder. These stabilisation binders must, however, comply with the relevant standards given in Table 3.2.

Table 3.2: Stabilisation binder standards

Stabilisation binder	Standard
Cement (GP and LH)	AS 3972
Blended cement (GB)	AS 3972
Fine grade fly ash	AS 3582.1
Ground granulated blast furnace slag	AS 3582.2
Hydrated lime	MRTS23[1]

1 Source: TMR (2017c).

3.4.3 Mixing Water

The water used for cement/cementitious stabilisation must be potable and free from matter that may inhibit the hydration processes or degrade the overall quality of the cement/cementitious stabilised materials. This includes oil, acids, organic matter and any other deleterious substances (TMR 2017b).

3.4.4 Modified End Product

The properties of the modified end product typically specified by national and international road agencies reviewed included the stabilisation binder content and a minimum and/or maximum UCS value. Typical national and international stabilisation binder content and UCS requirements for cement/cementitious modified materials are presented in Table 3.3.

Table 3.3: Cementitious modified material specification limits for national and international road agencies

Road authority	Material type	Stabilisation binder content (%)	7-day UCS values (MPa)
TMR[1] (Queensland)	Modified	1.0 – 2.0	1.0 – 2.0
Austroads[2]	Modified	–	1.0 – 2.0
RMS[3] (New South Wales)	Modified	–	1.0 (max.)
DOI[4] (Northern Territory)	Modified	–	1.5 – 2.0
DOSG[5] (Tasmania)	Modified	1.0 – 3.5	1.0 – 2.0
VicRoads[6] (Victoria)	Modified	1.0 – 3.5	1.0 – 2.0
MRWA[7] (Western Australia)	Modified	1.0 – 2.0	0.6 – 1.0
New Zealand	Modified	1.0 – 3.0[8]	0.70 (max.)[9]
South African National Roads Agency[10] (South Africa)	Lightly bound	–	0.75 – 1.5
USA[11]	Cement treated base	2.0 (min.)	1.0 – 2.75

1 Source: TMR (2016b).
2 Source: Austroads (2012).
3 Source: Road and Maritime Services (2013).
4 Source: Department of Infrastructure (2015).
5 Source: Department of State Growth (2016).
6 Source: VicRoads (2008).
7 Source: Main Roads Western Australia (2012).
8 Source: Transit New Zealand (2008).
9 Source: Transit New Zealand (2007).
10 Source: South African National Roads Agency (2014).
11 Source: Arizona Department of Transportation (2008).

Very low stabilisation binder contents are difficult to achieve in practice, and the stabilisation binder content is therefore generally restricted to a minimum of 1% (or spread rate of more than 5 kg/m^2) to ensure that the binder can be evenly distributed throughout the layer (Gray et al. 2011). This is generally reflected in the stabilisation binder limits of the specifications reviewed, where the minimum stabilisation binder content ranged between 1.0–2.0%. Ensuring the stabilisation binder content is kept below bound limits also reduces the reliance on layer bonding in deep lifts and maintains flexibility in the pavement that reduces fatigue cracking susceptibility (Adamson 2012).

The UCS limits of the road agencies reviewed generally included both upper and lower bounds to ensure design properties are met and that the material does not behave like a bound material.

Typical Australian UCS values range from 1.0–2.0 MPa but are determined by the properties of the parent material and the type and content of the cementitious stabiliser added (Austroads 2012). The minimum specified UCS values for the road agencies reviewed ranged from 0.6–1.5 MPa, with maximum values ranging from 1.0–2.75 MPa.

3.5 Mix Design

The mix design of a structural cement stabilised layer should be optimised to its intended application to meet workability, durability and strength requirements, while minimising shrinkage. Stabilisation binders typically contribute almost half of the total cost of stabilisation; therefore, it is vital that both the binder type and quantity used are optimised for the application (Austroads 2006). Determination of the optimal binder content involves ensuring the material has sufficient strength and durability without the development of excessive tensile capacity (Scullion et al. 2005). Under-stabilising the material may not sufficiently improve the strength, durability or stability of the layer, whereas over-stabilising may cause the material to become bound and rigid, thus increasing the shrinkage and fatigue susceptibility of the stabilised layers.

The constituent proportions are typically selected through an iterative laboratory testing program whereby the initial stabilisation binder content is selected based on soil classification, empirical evidence or engineering judgement. This includes identifying the parent material properties and refining the binder content until the stabilised material meets the strength specifications defined by the applicable road agency (Little & Nair 2009). UCS is a commonly specified criterion for stabilisation, both nationally and internationally, due to its relative speed and ease (Lim & Zollinger 2003). However, selection of binder content should also consider tensile strength, modulus, moisture sensitivity, durability, permeability, workability, particle size distribution and density requirements. Although, to facilitate the speed of the design process only the essential material properties are determined (Dunlop 1980).

TMR's current mix design procedure is documented in *Technical Note 149: Testing of Materials for Cement or Cementitious Blend Stabilisation* (TMR 2017a). The optimum binder content is determined by assessing the UCS values over a range of binder contents. The design binder content is selected to achieve the target strength. Following this, the working time of the cement treated material is determined in accordance with Test Method Q136 (refer to Section 3.3.7) using the design binder content.

Austroads (2006) also provides a mix design procedure. The procedure is similar to TMR's, except that reference is also made to a number of optional tests (including capillary rise, swell, erodibility, indirect tensile strength, flexural modulus and compressive resilient modulus tests).

3.6 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). Austroads (2012) separates pavement materials into five categories according to their fundamental behaviour under traffic loading, including unbound granular, modified granular, cemented, asphalt and concrete materials. The design of cement/cementitious modified materials should be carried out in accordance with Austroads unbound granular methodology but should include a check of the tensile stresses within the layer (Gray et al. 2011). The structural design methods that were reviewed as part of this project are discussed in the following sections.

3.6.1 Austroads

The method used in Australia for the structural design of flexible pavements is presented in Austroads (2012). This method models modified materials as unbound granular materials due to the assumption that modified materials have negligible tensile strength.

The elastic parameters of the selected pavement material are ideally determined through laboratory testing but may also be calculated using presumptive values in the absence of test data. The modified materials are characterised by their modulus and Poisson's ratio when modelled using linear elastic methods. The modified materials are considered cross-anisotropic (degree of anisotropy of 2) with a Poisson's ratio of 0.35, 28-day UCS between 1.0 MPa and 2.0 MPa, ITS less than 80 kPa and a maximum resilient modulus of between 600 and 1000 MPa when sub-layered (Austroads 2012, Dunlop 1980, TMR 2012).

Typical failure modes for unbound granular and modified materials are rutting and shoving caused by inadequate resistance to shear and densification, leading to pavement disintegration. The structural design of unbound granular materials using the Austroads (2012) method does not evaluate the shear or densification potential of unbound granular layers or the stresses and strains within these layers. The key failure criterion, however, is the maximum vertical compressive strain measured at the top of the subgrade layer resulting from the repeated traffic loadings. The number of standard axle repetitions allowable before an unacceptable level of total permanent deformation develops is calculated using the Austroads (2012) subgrade limiting strain criteria as presented in Equation 2.

$$SAR_{Allow} = \left(\frac{9300}{\mu\varepsilon}\right)^7 \tag{2}$$

where

SAR_{Allow} = allowable number of standard axle repetitions

$\mu\varepsilon$ = vertical compressive strain at the top of the subgrade (micro strain)

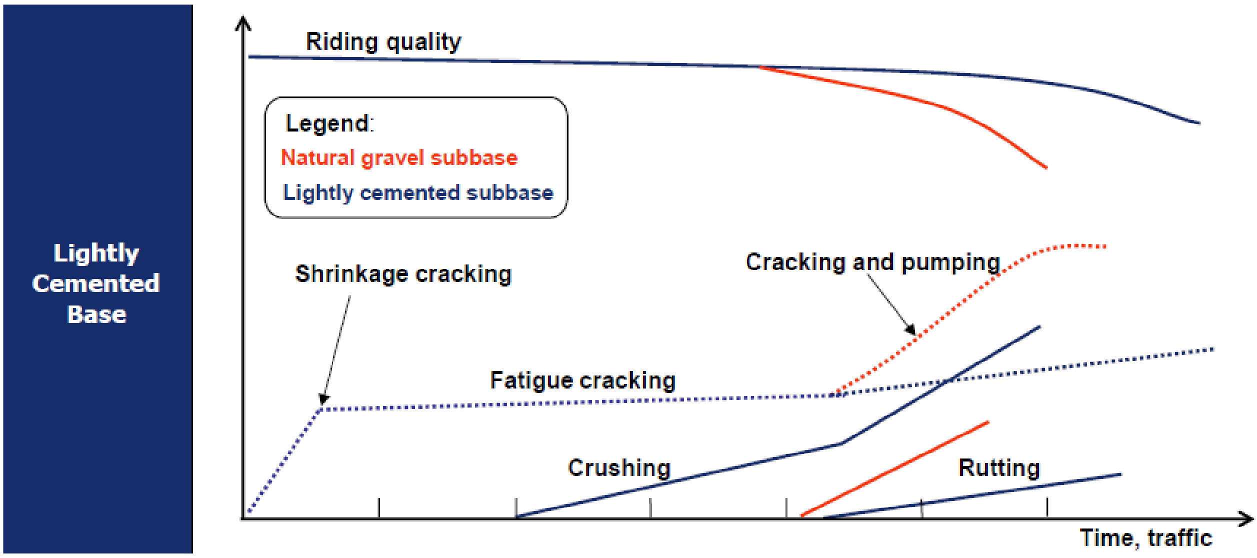
3.6.2 New Zealand Transport Agency

New Zealand adopts the structural design method as outlined in Austroads (2012) for multi-layer elastic pavement design (Transit New Zealand 2007), which is similar to the approach adopted in Australia.

3.6.3 South African Pavement Design Approach

The South African flexible pavement design method is based on a mechanistic-empirical approach, as outlined in the *South African Pavement Engineering Manual* (South African National Roads Agency 2014). This method combines the traditionally used South African mechanistic design method (SAMDM) from 1996 with updates and revisions as appropriate. Cement/cementitious modified materials with a UCS of between 0.75 MPa and 1.5 MPa are considered to be lightly bound layers in South Africa. These layers are analysed for fatigue, crushing of the material at the top of the layer and permanent deformation once the material has reached an 'equivalent granular' state. A schematic of the long- term structural behaviour of lightly bound materials in the South African context is shown in Figure 3.5.

Figure 3.5: Behaviour of lightly bound materials



Source: South African National Roads Agency (2014).

The structural design of stabilised granular layers using the South African mechanistic-empirical method includes:

- selecting a trial pavement configuration
- characterising the cement stabilised material based on the UCS values
- determination of the maximum horizontal strain at the bottom of the stabilised layer to determine the 'effective fatigue' life of the layer
- determination of the principal stresses in the middle of the stabilised layer once it has entered an 'equivalent granular' state
- calculation of the pavement structural capacity in allowable ESA loading by combining the effective fatigue and equivalent granular phases of the stabilised layer (refer to Equation 3 and Equation 4)
- checking the vertical compressive stress at the top of the stabilised layer to determine the risk of a 'crushing' type failure occurring (refer to Equation 6).

$$Neff = SF \times 10^{c(1-\frac{\varepsilon}{d\varepsilon b})} \qquad (3)$$

where

Neff	=	effective fatigue life
e	=	horizontal tensile strain at bottom of layer (micro strain)
eb	=	strain at break (micro strain)
c, d	=	effective fatigue constants based on road category
SF	=	shift factor for crack propagation (based on layer thickness)

$$N = 10^{\alpha F + \beta} \tag{4}$$

where

N	=	number of equivalent standard axles to safeguard against shear failure
α, β	=	constants (based on design reliability level)
F	=	stress ratio (refer to Equation 5)

$$F = \frac{\sigma_3 \phi term + Cterm}{(\sigma_1 - \sigma_3)} \tag{5}$$

where

F	=	stress ratio
σ_1, σ_3	=	major and minor principal stresses in the middle of the granular layer (kPa)
ϕ-term	=	value based on type of material
C-term	=	value based on type of material

$$Nci/ca = 10^{a(1-\frac{\sigma v}{bUCS})} \tag{6}$$

where

Nci/ca	=	standard axles to crush initiation or advanced crushing
σv	=	vertical compressive stress at top of layer (kPa)
UCS	=	unconfined compressive strength (kPa)
a, b	=	constants (based on design reliability)
SF	=	shift factor for crack propagation (based on layer thickness)

3.6.4 USA – National Cooperative Highway Research Program (NCHRP)

The structural design of pavements in the USA is carried out in accordance with the Guide *for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* (National Cooperative Highway Research Program 2004). The NCHRP design guide recommends that cement/cementitious modified materials used in pavement basecourse applications are designed and modelled as unbound granular layers and combined with other unbound layers for evaluation. Unbound granular materials designed using the NCHRP method include:

- selecting a trial pavement configuration

- characterising the material characteristics for each layer of the pavement structure
- determining the equivalent single, tandem, tri and quad-axle loading throughout the design period
- estimating the climatic temperature variation and moisture condition effects on the pavement layers
- determining of the critical compressive strain for each combination of equivalent axle load, temperature variation and equilibrium moisture content
- calculating the accumulated rutting in each layer based on the compressive strain values.

The NCHRP approach specifies that the Jacob Uzan Linear Elastic Analysis (JULEA) multilayer model be used to determine the compressive stresses and strains within the pavement layers. This model uses superposition of the trial pavement configuration, equivalent axle loading and the environmental conditions to predict the critical strains in each layer. The NCHRP approach does not provide recommended elastic properties for cement/cementitious modified basecourse layers. However, low-quality soil in the NCHRP design guide conforms to the Austroads (2012) classification of cement modified materials, including a Poisson's ratio ranging from 0.15 to 0.35 and elastic modulus ranging from 50 000 to 150 000 psi (350–1050 MPa) (NCHRP 2004). Permanent deformation in each sublayer resulting from the design traffic and environmental conditions can be estimated using Equation 7.

$$\delta_i = \frac{\left[\beta_1 \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)_2^{\beta}} \varepsilon_{max} h\right]}{N} \qquad 7$$

where

δ_i = permanent deformation in layer i (inch)

β_1 = layer calibration factor

ε_0, ε_r = $10[0.15(e^{\rho\beta2})+20(e^{(\rho/109)\beta2})]/2$

ρ = $10^9[-4.8929/1-(10^9)\beta_2]^{1/\beta2}$

β_2 = $10[-0.61119-(0.017638M_c)]$

M_c = equilibrium moisture content (%)

ε_{max} = maximum compressive strain

h = design layer thickness (inch)

N = number of traffic repetitions

The layer calibration factor, β_1, is approximately 1.673 for unbound granular base and subbase layers while it is approximately 1.350 for subgrade layers. The equilibrium moisture content is calculated and varied for each month of the design period using the enhanced integrated climate model (EICM). EICM is a database of historical climatic data that predicts temperature and moisture variations within each sub-season according to a normal distribution. Equivalent axle loading for each sub-season is then calculated using the estimated equivalent single, tandem, tri, and quad axles for each month of the design period and is modelled according to a normal distribution.

The primary failure mechanism for unbound granular pavements as specified by the NCHRP approach is permanent deformation. Permanent deformation at the surface of the design model is estimated by combining the relative deformation (RD) of each of the granular layers and subgrade. The RD is determined using the formula presented in Equation 8.

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

where

RD = permanent deformation at the midpoint in each layer and subgrade (inch)

n = number of sublayers

δ_i = permanent deformation in layer i (inch)

h_i = design thickness of sublayer

The trial pavement configuration selected at the start of the design process is adjusted iteratively until the estimated pavement surface deformation resulting from the design traffic loading and environmental conditions is within acceptable serviceability limits.

3.7 Construction of Stabilised Layers

In situ cement stabilisation can have significant cost, social and environmental benefits compared to alternative methods of improving pavement materials (Smith 2005). The construction of in situ cement stabilised layers follows the general process of surface preparation, stabilisation, compaction, curing and quality control. This section presents the current construction practices in Queensland for the in situ stabilisation of pavement materials using cement/cementitious binders. These practices are provided in Technical Specification MRTS07B *Insitu Stabilised Pavements using Cement or Cementitious Blends* (TMR 2017b).

In addition, AustStab has produced an *Insitu Stabilisation Construction Guide* (AustStab 2006) and an AustStab Contractor Accreditation System (with ARRB as the independent assessor).

3.7.1 Surface Preparation

Prior to the stabilisation phase of construction, it is vital that the surface is adequately prepared to ensure the new pavement does not fail early in the new life cycle (Crase 1998). This may include the addition of granular material should the existing pavement level need to be raised or require the removal of thick bituminous surfaces and cement treated patches which cannot be adequately mixed (Austroads 2009a).

In Queensland, preliminary pulverisation of the material to be stabilised is undertaken to a depth of 50 mm less than the target depth of the stabilisation layer by means of a dedicated machine known as a reclaimer/stabiliser. Any material that is not suitable for stabilisation must be removed and replaced. In addition to preliminary pulverisation, the existing surface must be shaped, compacted and trimmed to an extent that facilitates stabilisation (TMR 2017b).

3.7.2 Stabilisation

The rate at which the stabilisation binder is spread is important as it may have an impact on the consistency of the stabilised layer. To ensure a high level of consistency, the stabilisation binder is spread at a maximum uniform controlled single pass rate of 20 kg/m^2 using a purpose-built machine known as a spreader. Spread rates in excess of 20 kg/m^2 require additional spread passes.

Spreaders in Queensland must be calibrated with load cells and have the capability of spreading at varying widths (TMR 2017b). Furthermore, it is essential to check the spread rate regularly during each pass of the spreader. This ensures the stabilisation binder of the compacted material is within specified tolerances. In Queensland, testing is commonly carried out by weighing the stabilisation binder deposited into trays (or mats) laid in the path of the spreader or by utilising the previously mentioned load cells in the spreader. Consequently, the rate of spread can then be regularly adjusted, and any inconsistencies can be identified (Crase 1998).

In situ incorporation of the stabilisation binder into the material is carried out using a dedicated machine known as a recycler or reclaimer. The reclaimer uses a specialised rotor to pulverise the existing pavement while simultaneously mixing the stabilisation binder with water in the mixing chamber to produce a uniform mix (Austroads 2009a).

In Queensland, the process of mixing the stabilisation binder occurs in a minimum of two passes, one pass to incorporate the dry stabilisation binder and a second pass to incorporate the required moisture (TMR 2017b).

3.7.3 Compaction and Curing

To obtain the optimum strength and performance from stabilised materials it is essential to achieve adequate compaction (AustStab 2012). The common types of plant used for compaction are vibrating padfoot rollers (18–21 tonnes) and vibrating smooth drum rollers (18–21 tonnes) and multi-tyred finishing rollers. Compaction and trimming should commence as soon as practicable after mixing and should be completed within the allowable working time of the stabilisation binder, especially for cementitious stabilisation. This is because as cementitious materials set, increasing resistance to compaction could cause reduced densities and as a result, reduced strength (AustStab 2012). The working time of cementitious additives such as cement and lime range from 2 hours (GP cement) up to 24 hours (lime/fly ash) (TMR 2017b).

The thickness of the modified layer should be between 200 mm and 300 mm and there must be only one modified layer in a pavement. Layers below the modified layer shall have a minimum design thickness and California Bearing Ratio (CBR) of 300 mm and 3% respectively, unless a capping layer is provided (TMR 2012).

Curing is a vital process in the construction of cementitious stabilised layers as it is necessary to ensure the design strength in the stabilised material is achieved and that sufficient water is available for the hydration reactions to proceed (AustStab 2012).

In Queensland, cement/cementitious stabilised layers are often cured by using a water cart to regularly wet the stabilised surface until a pavement layer or sprayed bituminous surfacing is applied to prevent shrinkage cracking (TMR 2017b).

3.7.4 Quality Control

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

Trial sections are used for in situ stabilised pavements using the same construction plant, processes and methodology as proposed for the remainder of works to ensure the proposed plan meets all quality assurance requirements and any areas of noncompliance may be rectified (TMR 2017b).

The specification requires both material compliance and construction compliance testing to be undertaken by the contractor. Each of the constituent materials, including stabilisation binder, imported pavement materials, water and curing materials must be tested to demonstrate compliance with the specification.

The construction compliance testing includes the following (TMR 2017b):

- geometrics, including horizontal tolerance, vertical tolerance, deviation from a straight edge, crossfall and surface evenness
- degree of compaction
- stabilisation binder content (either surface spread rate or characteristic value)
- visible deflection of pavement layer by proof rolling.

Sampling of stabilised materials to determine the densities occurs after final mixing but prior to the commencement of compaction while the other requirements are assessed at the appropriate stages of construction.

3.8 Queensland vs. National and International Practice

As mentioned previously, the in situ stabilisation of cement/cementitious modified pavement layers in Queensland is carried out in accordance with TMR (2017b). TMR continues to develop and implement in situ stabilisation practices to achieve satisfactory performance of modified pavements. Although TMR's specifications align closely with those of typical Australian and international practice, the key differences are primarily associated with the quality control aspect of the construction process.

3.8.1 Australian Road Agencies

The various road agencies in Australia have their own specifications for the construction of in situ cement/cementitious stabilised pavement layers. The differences between the Queensland specifications and the other Australian specifications are primarily related to the quality control component of construction. This includes preliminary pulverisation, the stabilisation binder spread rate, stabilisation binder content, characteristic dry density ratios and the target 7-day UCS values. The differences between the Austroads, Queensland, NSW, Northern Territory, South Australian, Tasmanian and Western Australian guidelines are summarised in Table 3.4.

It is worth noting that as the Tasmanian Department of State Growth specifications are based on the VicRoads pavement specifications, the construction process and quality control procedures are very similar, and this is reflected in Table 3.4.

Table 3.4: Quality control differences between Australian road authorities for cementitious stabilisation

Road agency	Preliminary pulverisation	Stabilisation binder content (%)	Stabilisation binder spread rate (kg/m^2)	Minimum relative compaction (%)	7-day UCS values (MPa)
TMR[1] (Queensland)	Compulsory	1.0 – 2.0	20 (max.)	100 (standard compaction)	1.0 – 2.0
RMS[2] (New South Wales)	Situational[2]	–	20 (max. per pass)	100	1.0 (max)

Road agency	Preliminary pulverisation	Stabilisation binder content (%)	Stabilisation binder spread rate (kg/m^2)	Minimum relative compaction (%)	7-day UCS values (MPa)
				– 102 (standard compaction)	
DOI[(3)] (Northern Territory)	Compulsory	–	6 (max.)	97 (modified compaction	1.5 – 2.0
DPTI[(4)] (South Australia)	Situational	–	–	96 (modified compaction)	–
DOSG[(5)] (Tasmania)	Compulsory	1.0 – 3.5	15 (max.)	95 (modified compaction)	1.0 – 2.0
VicRoads[(6)] (Victoria)	Compulsory	1.0 – 3.5	15 (max.)	95 (modified compaction)	1.0 – 2.0
MRWA[(7)] (Western Australia)	Situational	1.0 – 2.0	3 (min.)	96 (modified compaction)	0.6 – 1.0

1 Source: Queensland Department of Transport and Main Roads (2017b).
2 Source: Road and Maritime Services (2015).
3 Source: Department of Infrastructure (2015).
4 Source: Department of Planning, Transport and Infrastructure (2007).
5 Source: Department of State Growth (2016).
6 Source: VicRoads (2008).
7 Source: Main Roads Western Australia (2012).

The preliminary pulverisation of the work area is required in Queensland, the Northern Territory, Tasmania and Victoria to assist in identifying unsuitable stabilisation material. However, Austroads, NSW, South Australia and Western Australia do not require it in every project but instead on a situational basis when the material of the top layer is deemed unsuitable. Austroads recommends minimising the stabilisation binder content as this will lower the moisture required for mixing but also recognises that the limits differ between agencies (Austroads 2012). For the specifications reviewed, the minimum stabilisation binder content is shown as 1% where a limit is specified, and maximum values range from 2.0% to 3.5%.

The maximum spread rate of the stabilisation binder for the specifications reviewed showed that 20 kg/m^2 was the most common requirement although Victoria and Tasmania specify a rate of 15 kg/m^2 while Western Australia specifies the minimum practical spread rate achievable by stabilisation binder spreaders at 3 kg/m^2.

Typical 7-day UCS values in Australia range from 0.6 MPa to 2.0 MPa and may be influenced by the degree of stabilisation, the properties of the original material that is to be stabilised, the road class and the acceptable risk (Austroads 2012).

The characteristic value of relative compaction required to construct stabilised pavements layers in Australia ranges from 95% to 97% using modified compaction and 100% to 102% using standard compaction. TMR and RMS appears to be the only two road agencies that specify a standard compactive effort for in situ cement/cementitious stabilised pavement layers. The degree of relative compaction required is dependent on many factors that may vary between states/territories and includes the quality of the material being stabilised, location, pavement design, and design traffic volume (Austroads 2009a).

3.8.2 International Road Authorities

A review of in situ cement/cementitious stabilisation construction processes utilised by road agencies in New Zealand, South Africa and the USA was conducted to determine how Queensland's practices compare to established practices internationally. The key differences between Queensland and international practice were primarily related to the quality control. This includes preliminary pulverisation, the stabilisation binder spread rate, stabilisation binder content, characteristic dry density ratios and the target 7-day UCS values. The differences between the Queensland, New Zealand, South African and the US guidelines are summarised in Table 3.5.

Although South Africa adopts very similar processes to those used in Queensland it must be noted that in Australia spreading stabilisation binders by hand is only used for minor patching and small remote projects, whereas in South Africa it has increased in recent years due to the enhancement of labour content on projects (South African National Roads Agency 2014).

Table 3.5: Quality control differences between Queensland and international road authorities for cementitious stabilisation

Road authority	Preliminary pulverisation	Stabilisation binder content (%)	Stabilisation binder spread rate (kg/m^2)	Relative compaction (%)	7-day UCS values (MPa)
TMR[1] (Queensland)	Compulsory	1.0 – 2.0	20 (max.)	100 (standard compaction)	1.0 – 2.0
New Zealand	–	1.0 – 3.0[2]	–	95 (min)[3] (vibratory compaction)	0.70 (max.)[4]
South African National Roads Agency (South Africa)[5]	–	–	–	97 – 98 (modified compaction)	0.75 – 1.5
Arizona	Compulsory[6]	2.0 (min.)[7]	–	100 (standard compaction)[7]	1.0 – 2.75[7]

1 Source: Queensland Department of Transport and Main Roads (2017b).
2 Source: Gray et al. (2011).
3 Source: Transit New Zealand (2008).
4 Source: Transit New Zealand (2007).
5 Source: South African National Roads Agency (2014).
6 Source: Kandhal and Mallick (1997).
7 Source: Arizona Department of Transportation (2008).

The preliminary pulverisation of the material to be stabilised is compulsory in the USA (generally because of the thicker asphalt layers present) similar to Queensland, but the New Zealand and South African guidelines reviewed did not mention preliminary pulverisation in their surface preparation procedures. The stabilisation binder content varied between the international guidelines reviewed, ranging from a minimum value of 1.0%, to a maximum value of 3.0% in New Zealand. The *South African Pavement Engineering Manual* (South African National Roads Agency 2014) however, does not specify the ranges of stabilisation binder content but states optimisation through laboratory testing should be conducted for each project. Maximum spread rates were not mentioned in any of the international guidelines reviewed, but all the guidelines specified the stabilisation binder should be spread uniformly across the work area.

The relative compaction levels required to ensure the stabilised pavement meets the desired quality ranges from a minimum of 95% in New Zealand (using vibratory compaction), to a maximum of 100% in Arizona (using standard compaction). It is, however, worth noting that South Africa uses a modified compaction standard, which may result in higher densities compared to current TMR practice.

For the guidelines reviewed, minimum target UCS values range from 0.70 MPa to 1.0 MPa and maximum values range from 1.5 MPa to 2.75 MPa.

4 IDENTIFICATION OF PAVEMENT SECTIONS

A review of the material selection, mixture proportioning, structural design, and construction and maintenance processes utilised throughout the state was required for benchmarking the stabilisation practices across Queensland. However, before beginning the review, the general condition of pavements incorporating I-CMB layers 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. Such a vast network covers a number of varying environmental and climatic zones and caters for a wide range of traffic loadings. Significant portions of the network are composed of stabilised structural layers, including I-CMB base layers. The *A Road Management Information System* (ARMIS) database was referenced to ascertain the inventory, condition and maintenance data for the entire Queensland road network. This allowed for a state-wide review of technology selection, design practice and maintenance programming.

4.1 Network Inventory

The extent, composition and historical performance of I-CMB 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.

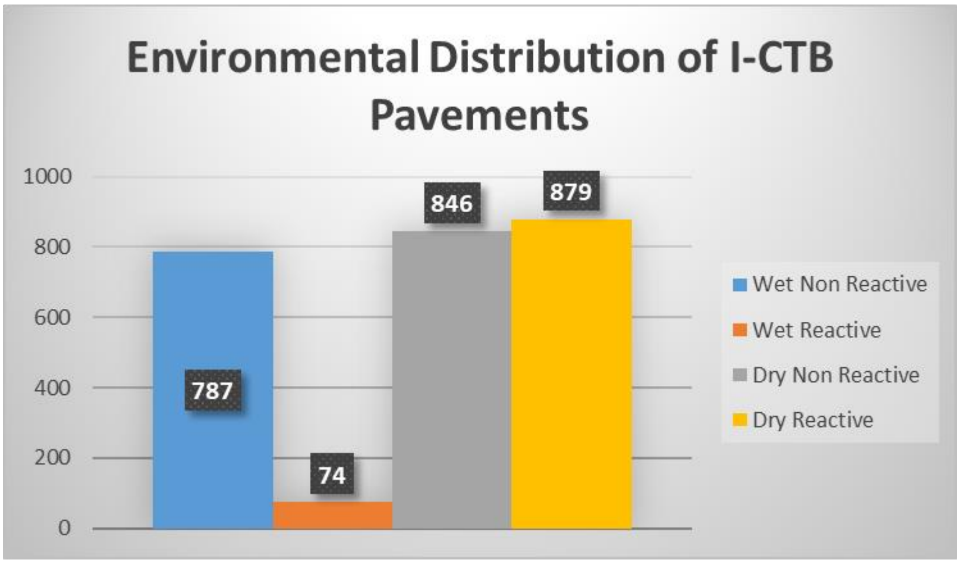
This valuable information was provided to the ARRB Group project team by the Pavement Rehabilitation Section of the TMR Engineering and Technology Branch. The data was extracted from the ARMIS database in May 2015 and all data presented in this report reflects the state of the database midway through the 2015 calendar year. The ARMIS information was instrumental to the investigation, as the I-CMB sections of the road network were identified and categorised according to region, environmental zone, age, stabilised layer thickness, traffic volume and design stabilisation binder content. It should be noted that only pavement layers designated as '*B3 – Granular modified (granular materials with additives to improve stiffness. UCS < 1.5 MPa)*' and '*C4 – Cement stabilised (cement stabilised granular pavement. Can have combination with fly ash. Modulus 1500–2000 MPa. UCS 1.5–2 MPa)*' in the ARMIS database were included in this assessment.

I-CMB pavement sections identified from the ARMIS database and included in this assessment are presented in Appendix A of this report.

The typical I-CMB layer thickness appears to be between 200 and 250 mm. There are a number of sections with incomplete stabilisation binder 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 2012).

As shown in Figure 4.1, approximately 2586 km of the TMR road network comprises pavements with I-CMB layers. These pavements are distributed relatively evenly throughout dry non-reactive environments (32.7%), dry reactive environments (34%) and wet non-reactive environments (30.4%). By comparison, very few pavements in wet reactive environments (2.9%) are constructed with I-CMB layers.

Figure 4.1: Environmental distribution of I-CMB pavements



Pavements with I-CMB layers are also used throughout most regions along the Queensland state-controlled road network. The highest proportion of I-CMB pavements was located in the North-West Region (17.9 %), followed by the Fitzroy (15.2 %) and then South-West (14.5 %) as shown in Figure 4.2.

Figure 4.2: Regional distribution of I-CMB pavements

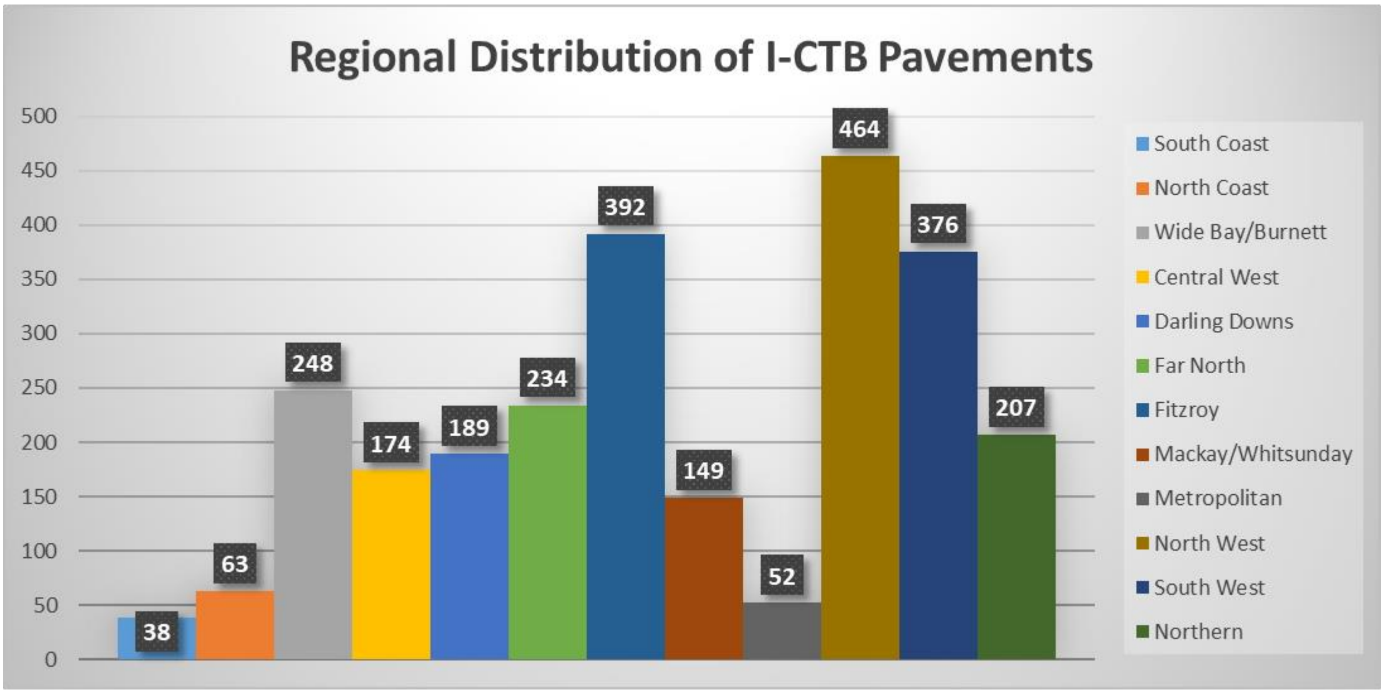
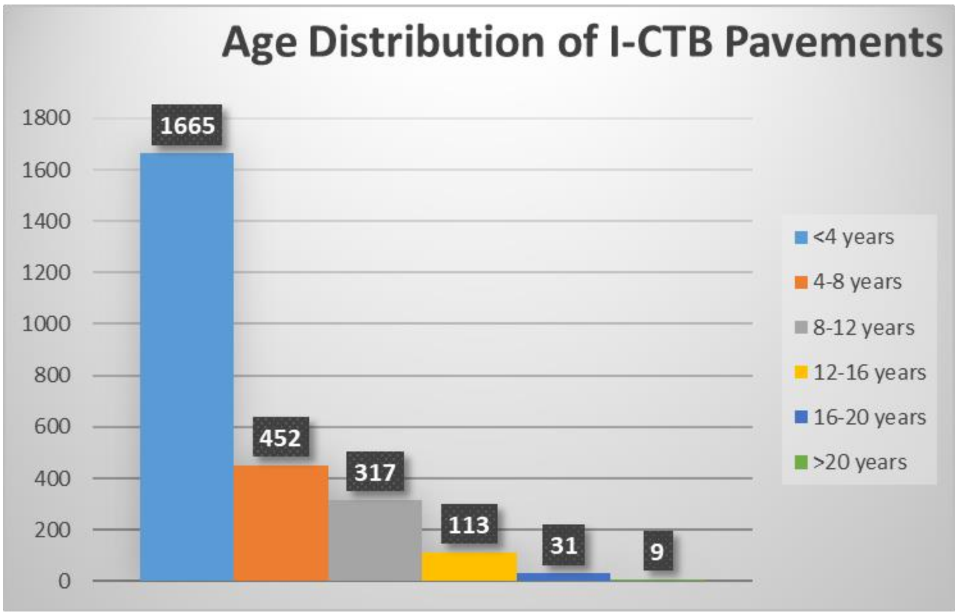


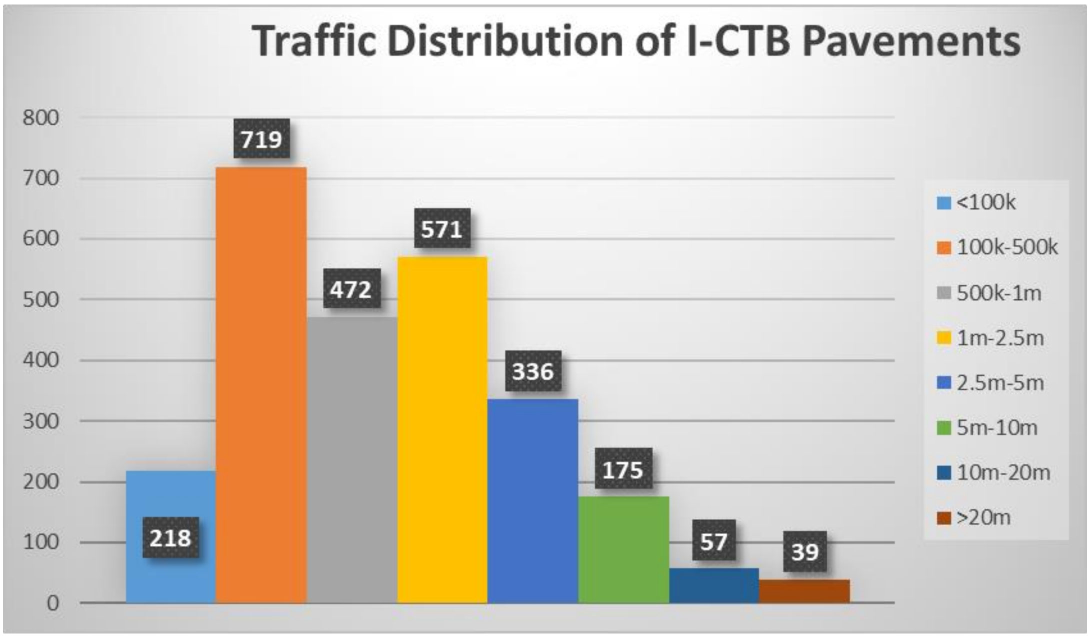
Figure 4.3 shows that the age distribution of I-CMB pavements along the state-controlled road network ranges from less than 4 years to greater than 20 years, with the vast majority of pavements less than 4-years old (64.4%). As such, the I-CMB pavements are relatively young, with only 5.8% of the pavements greater than 12-years old.

Figure 4.3: Age distribution of I-CMB pavements



It can also be seen that I-CMB pavements are selected for a range of traffic conditions, as represented by cumulative traffic loading in Figure 4.4. I-CMB layers are utilised in pavements with traffic counts ranging from fewer than 100 000 ESAs to greater than 20 000 000 ESAs. However, the largest proportion (≈ 68%) of I-CMB pavement structures are used for low to moderately trafficked pavements with cumulative traffic values ranging from 100 000 ESAs to 2 500 000 ESAs.

Figure 4.4: Traffic distribution of I-CMB pavements



4.2 Network Condition

The condition of the I-CMB pavement network throughout Queensland was determined by assessing the pavement condition data available in ARMIS. Standard TMR condition criteria was applied to data obtained from the ARMIS database and the results were categorised as excellent, good, mediocre or poor. The condition data included historical roughness, rutting and macrotexture measurements from laser profiler surveys and cracking assessments from automatic crack detection technology.

4.2.1 Condition Data

Roughness, rutting, texture and cracking data stored in the ARMIS database is often collected using a high-speed network survey vehicle (NSV) fitted with accelerometers, displacement transducers, laser profilometers and cameras that continually monitor the pavement surface.

Roughness counts provide a measure of a pavement's serviceability and is also indicative of the structural condition. Additionally, excessive roughness can accelerate pavement deterioration due to increased dynamic loading. In terms of the International Roughness Index (IRI), roughness is determined according to the quarter-car model and indicates suspension displacement accumulation in m/km.

Rutting is particularly useful in condition analysis as it directly reflects the structural condition of the pavement. Rutting data is collected through the measurement of the distance between a fixed horizontal datum and the pavement surface, usually presented in millimetres. The measurement will typically include the maximum rut depth in each wheelpath in addition to the lane maximum.

The cracking data used for this network condition analysis was collected using automatic crack detection technology and is presented as a percentage of the pavement surface. Fatigue cracking was the key component in the analysis as it is indicative of structural failure caused by inadequate strength or unstable supporting layers. However, transverse and longitudinal cracking were also included in the condition analysis.

4.2.2 Limitations of ARMIS Data

The data set obtained from the ARMIS database was aggregated in 100 m lengths, 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. 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.

Approximately 61% of the I-CMB pavement sections throughout the TMR network did not have any cracking data available. It is also possible that some sites could have been subjected to resurfacing works, potentially masking crack defects and temporarily improving the cracking assessment. Roughness and rutting were therefore used as the primary condition criteria and fatigue cracking was only used as a complementary evaluation parameter.

In addition, a further 25 sections did not have any rutting, roughness or cracking data available and were excluded from the condition analysis.

4.2.3 Condition Evaluation Criteria

The evaluation criteria and performance limits used to assess the condition of the I-CMB pavements are shown in Table 4.1. These criteria were selected in accordance with the Pavement *Rehabilitation Manual* (TMR 2012) and the Guide *to Asset Management Part 5H: Performance Modelling* (Austroads 2009c).

Table 4.1: Initial 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 (2012), Austroads (2009c).

Rutting, roughness and fatigue cracking were used to 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 may be required.

The 2015 performance data was obtained from the ARMIS database and used to assess the condition of the I-CMB sections. The results of the condition assessment are presented in Table 4.2.

Table 4.2: I-CMB network condition distribution with initial evaluation criteria

Evaluation criteria	Roughness (%)(1)	Rutting (%)(1)	Fatigue cracking (%)(2)
Excellent	79.1	97.8	88.1
Good	20.8	2.1	7.4
Mediocre	0.1	0.1	3.3
Poor	0.0	0.0	1.2

1 Roughness and rutting data was available for all I-CMB sections considered in the analysis.
2 Cracking data was not available for 61% of the I-CMB sections considered in the analysis. Results presented are based on sections with available data.

From the results of the analysis presented in Table 4.2, the majority of I-CMB sections were categorised as being in an excellent condition. The results indicate that the I-CMB pavement sections are currently performing well. The typically good condition of the sections is particularly impressive when considering the regional distribution of the sections across most of the state. However, it is worthwhile noting that the majority of the pavement sections assessed are less than 4-years old and it would not be appropriate at this stage to comment on the long-term performance of I-CMB pavements across Queensland. It is also important to note that while this analysis gives a general overview of the condition of the network, a more robust analysis would consider stabiliser content, pavement configuration, traffic loading, maintenance expenditure and environmental conditions.

4.2.4 Revised Categorisation Criteria

The condition data presented in the previous section suggests that the in situ cement stabilisation technology is performing well. However, categorisation of the sections based on the initial evaluation criteria in Table 4.1 does not provide enough sections in the mediocre or poor category to determine meaningful conclusions. Revised and more stringent evaluation criteria were developed to provide a greater distribution of pavement sections between different condition categories. These revised criteria are presented in Table 4.3 and are the same as used in previous work on plant-mixed cementitious modified base pavement sections and in situ foam bitumen stabilised pavement sections.

The revised roughness, rutting and fatigue cracking condition criteria were developed solely for the purpose of identifying representative pavement sections in this study and should not be applied to general network condition assessments.

Table 4.3: 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 results of the condition analysis performed on the I-CMB pavement sections utilising the revised condition criteria are presented in Table 4.4.

Table 4.4: I-CMB network condition distribution with revised evaluation criteria

Evaluation criteria	Roughness (%)(1)	Rutting (%)(1)	Fatigue cracking (%)(2)
Excellent	79.1	53.0	20.6
Good	17.4	34.0	67.0
Mediocre	3.1	10.8	7.8
Poor	0.4	2.2	4.4

1 Roughness and rutting data was available for all I-CMB sections considered in the analysis.
2 Cracking data was not available for 61% of the I-CMB sections considered in the analysis. Results presented are based on sections with available data.

With the application of a more stringent categorisation criteria, the results of the analysis show that the majority of I-CMB sections were categorised as being in either an excellent or good condition.

However, the number of sections that were categorised as mediocre or poor increased considerably for rutting and fatigue cracking. The most notable change was in fatigue cracking, with the number of sections categorised as excellent reducing from 88.1% under the initial criteria to 20.6% under the revised criteria. The number of sections categorised as excellent for rutting also changed significantly, reducing from 97.8% under the initial criteria to 53.0% under the revised criteria.

This indicates that approximately 80% of the network (for which cracking data was available) has some form of fatigue cracking in the surface. From the analysis, the results do not provide a strong indicator of the principal distress mechanism, however it appears that it could be fatigue cracking with 12.2% of the network displaying cracking in more than 5% of the surface.

4.3 Representative Sections

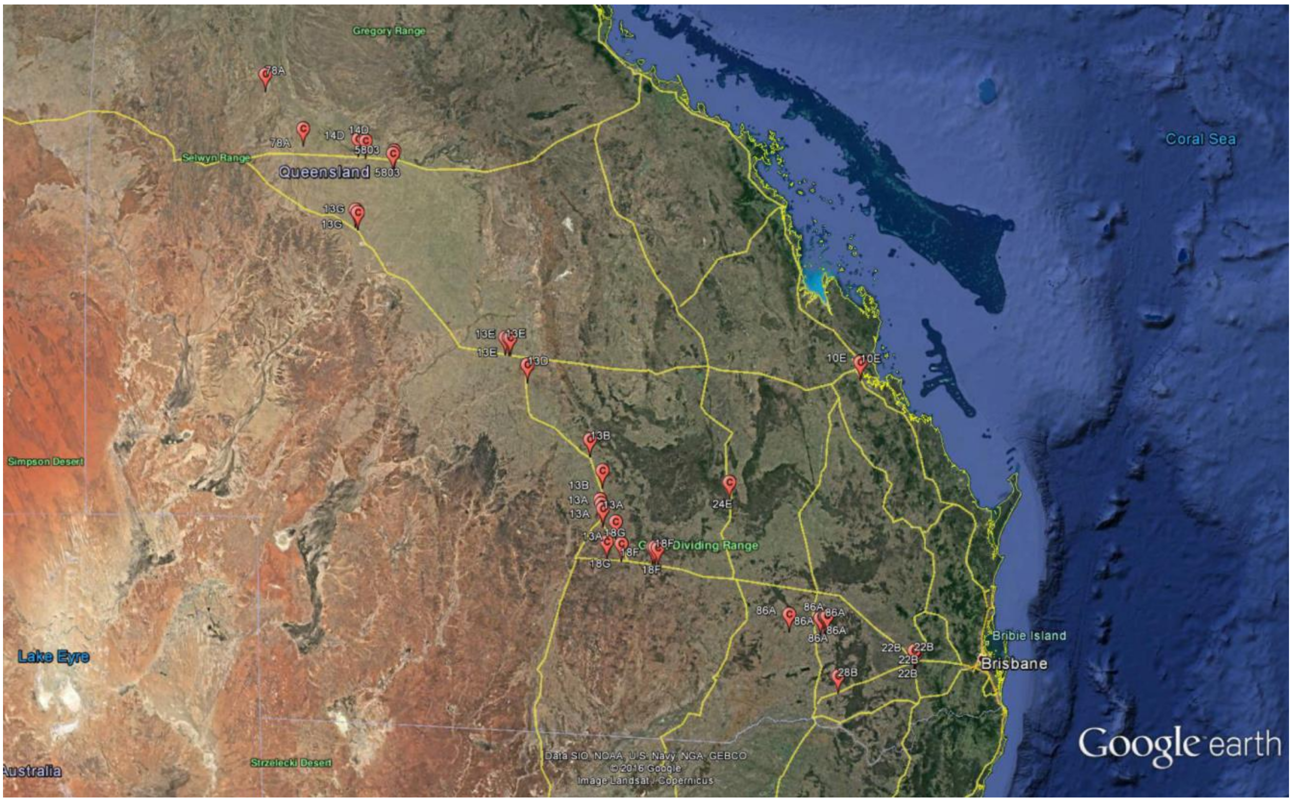
Thirty-eight I-CMB pavement sections were selected for further investigation and are presented in Table 4.5. The selected road sections are representative of the range of I-CMB pavements in Queensland covering five TMR regions with variations in project length, stabilised layer design thickness (150 mm to 580 mm), target stabilisation binder content (1.0% to 3.0%) and pavement age (2 years to 5 years). The pavements selected are typically in dry environments with only two sites represented in a 'wet non-reactive' environment. The geographic locations of the representative sections are shown in Figure 4.5.

Table 4.5: Representative I-CMB Sections

Region	Road ID	Start chainage	End chainage	Environmental zone	Stabilised layer build date	Stabilised layer thickness (mm)	Stabiliser content (%)	Cumulative traffic (ESA)
Central West	13B	100.3	108.0	Dry Reactive	20/12/2013	150	2.0	8.11E+05
Central West	13D	65.2	67.0	Dry Reactive	30/05/2013	200	3.0	7.12E+05
Central West	13E	24.2	26.6	Dry Reactive	10/03/2014	250	3.0	2.66E+05

Central West	13E	26.7	27.6	Dry Reactive	10/03/2014	250	3.0	2.66E+05
Central West	13E	32.4	33.9	Dry Reactive	10/03/2014	250	3.0	2.66E+05
Central West	13G	92.3	97.8	Dry Reactive	29/12/2012	250	2.0	8.16E+05
Central West	13G	98.2	104.2	Dry Reactive	29/12/2012	250	2.0	8.16E+05
Darling Downs	22B	15.3	16.0	Dry Reactive	17/02/2012	125	1.0	5.73E+06
Darling Downs	22B	16.7	17.8	Dry Reactive	14/05/2014	250	11.5 kg/m^2	2.11E+06
Darling Downs	22B	17.8	18.3	Dry Reactive	13/12/2013	320	Not available	4.11E+06
Darling Downs	22B	22.2	22.9	Dry Reactive	14/05/2014	250	Not available	1.35E+06
Darling Downs	28B	70.8	73.1	Dry Non-Reactive	14/08/2012	580	1.0	3.36E+06
Darling Downs	86A	75.4	77.3	Dry Reactive	23/07/2014	250	2.5	1.03E+05
Darling Downs	86A	130.3	131.7	Dry Reactive	23/07/2014	250	2.5	1.40E+05
Darling Downs	86A	137.5	138.7	Dry Reactive	27/02/2013	200	Not available	2.82E+05
Darling Downs	86A	138.7	140.7	Dry Reactive	27/02/2013	250	Not available	2.82E+05
Darling Downs	86A	140.7	141.9	Dry Reactive	27/02/2013	200	Not available	2.82E+05
Darling Downs	86A	141.9	142.5	Dry Reactive	27/02/2013	250	Not available	2.82E+05
Fitzroy	10E	97.7	98.6	Wet Non-Reactive	9/09/2013	310	3.0	8.00E+06
Fitzroy	10E	98.8	99.7	Wet Non-Reactive	9/09/2013	310	3.0	8.00E+06
North West	5803	14.0	16.4	Dry Reactive	18/12/2012	200	1.0 - 1.5	1.91E+05
North West	5803	18.7	20.5	Dry Reactive	22/07/2014	200	Not available	4.66E+04
North West	14D	55.8	60.1	Dry Reactive	28/11/2013	250	1.5	6.72E+05
North West	14D	68.7	69.7	Dry Reactive	28/11/2013	250	1.5	6.72E+05
North West	78A	17.6	19.3	Dry Reactive	31/01/2014	200	1.5	8.96E+03
North West	78A	140.9	142.8	Dry Reactive	31/01/2014	200	1.5	8.96E+03
South West	13A	41.9	46.9	Dry Non-Reactive	18/04/2013	250	2.5	5.73E+05
South West	13A	72.0	72.5	Dry Reactive	18/04/2013	250	2.5	5.89E+05
South West	13A	72.5	75.1	Dry Reactive	18/04/2013	250	2.5	5.89E+05
South West	13A	80.1	81.5	Dry Reactive	18/04/2013	250	2.5	5.89E+05
South West	13A	87.2	87.6	Dry Non-Reactive	18/04/2013	Not available	2.5	9.33E+05
South West	13B	45.9	53.3	Dry Reactive	7/08/2012	250	2.0	1.07E+06
South West	18F	42.0	43.2	Dry Non-Reactive	8/02/2014	250	1.0	5.56E+05
South West	18F	47.8	49.9	Dry Non-Reactive	8/02/2014	250	2.0	5.45E+05
South West	18F	51.0	52.1	Dry Non-Reactive	8/02/2014	250	2.0	5.45E+05
South West	18G	11.1	14.3	Dry Non-Reactive	24/09/2013	200	3.0	5.34E+05
South West	18G	35.1	37.2	Dry Non-Reactive	24/09/2013	200	3.0	5.34E+05
South West	24E	51.7	52.9	Dry Non-Reactive	17/02/2013	200	1.5	1.95E+06

Figure 4.5: Relative location of selected representative pavement sections



Source: Map Data: Google, California, USA, viewed February 2017.

5 IN SITU PAVEMENT CONDITION EVALUATION

A well-maintained asset management system can provide valuable data on road pavement infrastructure including region, environmental zone, age, stabilised layer thickness, traffic volume and condition. These variables are essential for understanding the assumptions, constraints and properties that affect the pavement life cycle. However, differences in the design and as-constructed details routinely vary. The representative I-CMB road sections were subject to further investigation including visual condition inspection using roadway imaging and estimation of structural capacity (refer to Section 6) to validate the inventory and condition data extracted from the ARMIS database.

5.1 Visual Inspection

The purpose of the visual inspection was to assess the current condition of the selected road sections and identify any abnormal features prior to the 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, affecting the reported condition of the entire section.

5.1.1 Landsborough Highway

The Landsborough Highway is a 1050 km major route connecting regional communities. The condition of a number of road sections that incorporate I-CMB pavements are discussed below.

Section 13B: chainage 100.3 km to chainage 108.0 km

This section is located between Augathella and Tambo and in 2014 carried an average annual daily traffic (AADT) of 530 vehicles, of which 34% were heavy vehicles. The section was constructed in December 2013 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm granular layer modified with 2% GB cement over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic to date was estimated at 8.11×10^5 ESAs.

The section investigated was typically found to be in good condition, however minor bleeding was identified in the whee paths along most of the section. Some isolated areas of moderate to severe bleeding were also observed from CH 100.846 to 100.916 km and from CH 100.926 to 101.006 km. Overall the section is in good condition and exhibiting no signs of rutting, cracking, shoving or potholes.

Figure 5.1: Road Section 13B – chainage 100.3 km to chainage 108.0 km (good vs poorer sections)

Section 13D: chainage 65.2 km to chainage 67.0 km

This section is located between Barcaldine and Longreach and in 2014 carried an AADT of 735 vehicles, of which 21% were heavy vehicles. The section was constructed in March 2014 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm standard granular layer, a 250 mm granular layer modified with 3% cement over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic to date was estimated at 2.66×10^5 ESAs.

The section investigated was typically found to be in a very good condition with only minor rutting identified in the wheelpaths across most of the section.

Figure 5.2: Road Section 13D – chainage 65.2 km to chainage 67.0 km

Section 13E: chainage 24.2 km to chainage 26.6 km

This section is located between Barcaldine and Longreach and in 2014 carried an AADT of 735 vehicles, of which 21% were heavy vehicles. The section was constructed in March 2014 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm standard granular layer, a 250 mm granular layer modified with 3% cement over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic to date was estimated at 2.66×10^5 ESAs.

The section investigated was typically found to be in a good condition, however minor bleeding was identified in the wheelpaths along most of the section investigated. Minor longitudinal cracking was also identified from CH 24.197 km to CH 24.237.

Figure 5.3: Road Section 13E – chainage 24.2 km to chainage 26.6 km

Section 13E: chainage 26.7 km to chainage 27.6 km

This section is located between Barcaldine and Longreach and in 2014 carried an AADT of 735 vehicles, of which 21% were heavy vehicles. The section was constructed in March 2014 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm non-standard granular layer, a 250 mm granular layer modified with 3% cement over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic to date was estimated at 2.66 x 10^5 ESAs.

The section investigated was typically found to be in a good condition with only bleeding identified in the wheelpaths across most of the section.

Figure 5.4: Road Section 13E – chainage 26.7 km to chainage 27.6 km

Section 13E: chainage 32.4 km to chainage 33.9 km

This section is located between Barcaldine and Longreach and in 2014 carried an AADT of 735 vehicles, of which 21% were heavy vehicles. The section was constructed in March 2014 and consists of a polymer modified binder sprayed seal surfacing, a 250 mm granular layer modified with 3% cement over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 2.66 x 10^5 ESAs.

The section investigated was typically found to be in very good condition, with only minor bleeding identified in the wheelpaths across most of the section. Minor longitudinal cracking was also identified at CH 32.937 km.

Figure 5.5: Road Section 13E – chainage 32.4 km to chainage 33.9 km

Section 13G: chainage 92.3 km to chainage 97.8 km

This section is located between Winton and Kynuna and in 2014 carried an AADT of 336 vehicles, of which 34% were heavy vehicles. The section was constructed in December 2012 and consists of a polymer modified binder sprayed seal surfacing, a 250 mm granular layer modified with 2% GB cement over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 8.15 x 10^5 ESAs.

The section investigated was typically found to be in a mediocre condition, with an array of minor defects sometimes occurring for some length. Extensive seal and patch repairs have been conducted along most of the road. Minor rutting and bleeding was observed from CH 93.351 km to CH 93.801 km. Further minor bleeding was identified from CH 94.991 km to CH 95.661 km. Some minor to moderate shoving was also observed at CH 93.351 km and CH 95.071 km. Very little cracking was observed, except for minor longitudinal cracking at CH 96.081 km.

Figure 5.6: Road Section 13G – chainage 92.3 km to chainage 97.8 km

Section 13G: chainage 98.2 km to chainage 104.2 km

This section is located between Winton and Kynuna and in 2014 carried an AADT of 336 vehicles, of which 34% were heavy vehicles. The section was constructed in December 2012 and consists of a polymer modified binder sprayed seal surfacing, a 250 mm granular layer modified with 2% GB cement over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 8.15 x 10^5 ESAs.

The section investigated was typically found to be in a good condition, with minor to moderate bleeding present in some areas. Extensive seal and patch repairs have been conducted over long lengths of the road section. Longitudinal (environmental) cracking was also observed in some locations.

Figure 5.7: Road Section 13G – chainage 98.2 km to chainage 104.2 km

Section 13A: chainage 41.9 km to chainage 46.9 km

This section is located between Morven and Augathella and in 2014 carried an AADT of 327 vehicles, of which 37% were heavy vehicles. The section was constructed in January 2014 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm standard granular layer and a 250 mm granular layer modified with 2.5% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated 5.73 x 10^5 ESAs. The section investigated was typically found to be in an excellent condition, with only some minor bleeding observed in the wheelpaths. An isolated longitudinal crack was identified at CH 42.582 km.

Figure 5.8: Road Section 13A – chainage 41.9 km to chainage 46.9 km

Section 13A: chainage 72.0 km to chainage 72.5 km

This section is located between Morven and Augathella and in 2014 carried an AADT of 294 vehicles, of which 49% were heavy vehicles. The section was constructed in April 2013 and consists of a polymer modified binder sprayed seal surfacing, and a 250 mm granular layer modified with 2.5% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section is unknown, however in 2014 the cumulative traffic was estimated 5.89×10^5 ESAs.

The section investigated was typically found to be in a good condition, with minor bleeding observed in the wheelpaths for most of the section, and some isolated instances of moderate bleeding.

Figure 5.9: Road Section 13A – chainage 72.0 km to chainage 72.5 km

Section 13A: chainage 72.5 km to chainage 75.1 km

This section is located between Morven and Augathella and in 2014 carried an AADT of 294 vehicles, of which 49% were heavy vehicles. The section was constructed in April 2013 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm standard granular layer and a 250 mm granular layer modified with 2.5% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 5.73×10^5 ESAs.

The section investigated was typically found to be in an excellent condition, with only minor bleeding observed in the wheelpaths.

Figure 5.10: Road Section 13A – chainage 72.5 km to chainage 75.1 km

Section 13A: chainage 80.1 km to chainage 81.5 km

This section is located between Morven and Augathella and in 2014 carried an AADT of 294 vehicles of which 49% were heavy vehicles. The section was constructed in April 2013 and consists of a polymer modified binder sprayed seal surfacing and a 250 mm granular layer modified with 2.5% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated 5.73 x 10^5 ESAs.

The section investigated was typically found to be in a good condition, with minor to moderate bleeding observed in the wheelpaths along most of the section.

Figure 5.11: Road Section 13A – chainage 80.1 km to chainage 81.5 km

Section 13A: chainage 87.2 km to chainage 87.6 km

This section is located between Morven and Augathella and in 2014 carried an AADT of 632 vehicles of which 38% were heavy vehicles. The section was constructed in April 2013 and consists of a polymer modified binder sprayed seal surfacing and a 250 mm granular layer modified with 2.5% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 9.33 x 10^5 ESAs.

The section investigated was typically found to be in a mediocre to poor condition, with moderate to severe bleeding in the wheelpaths across most of the section. The Augathella-bound lane was typically in worse condition than the Morven-bound lane. Longitudinal cracking (possibly reflective cracking from an expansive subgrade) was identified between CH 87.272 km to CH 87.412 km, typically in the inner wheelpath of the Augathella-bound lane.

Figure 5.12: Road Section 13A – chainage 87.2 km to chainage 87.6 km

Section 13B: chainage 45.9 km to chainage 53.3 km

This section is located between Augathella and Tambo and in 2014 carried an AADT of 401 vehicles, of which 41% were heavy vehicles. The section was constructed in August 2012 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm standard granular layer and a 250 mm granular layer modified with 2.0% cement/slag mix (35/65) over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 1.07×10^6 ESAs.

The section was typically in a mediocre to poor condition, with moderate to severe bleeding present along most of the section. A particularly poor section of pavement was observed from CH 48.326 km to CH 48.615 km, with evidence of depressions in the pavement, longitudinal crack repairs, patch repairs, and moderate to severe bleeding. Seal and patch repairs also occur along most of the section. Some minor, isolated meandering and transverse cracks were identified but were not a common occurrence.

Figure 5.13: Road Section 13B – chainage 45.9 km to chainage 53.3 km

5.1.2 New England Highway

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 for both rural and urban communities. The following sections all incorporate I-CMB pavements and are located between Toowoomba and Warwick. Their condition is discussed below.

Section 22B: chainage 15.3 km to chainage 16.0 km

This section is located between Toowoomba and Warwick and in 2014 carried an AADT of 6630 vehicles, of which 14% were heavy vehicles. The section was constructed in February 2012 and consists of a polymer modified binder sprayed seal surfacing, a 125 mm granular layer modified with 1.0% cement/fly ash mix (60/40), two 125 mm layers of standard granular material, a 225 mm of standard granular material and the natural soil subgrade. The original design traffic for the pavement section was 2.6 x 10^6 ESAs, however in 2014 the cumulative traffic was already estimated at 5.73 x 10^6 ESAs.

The section investigated was typically found to be in a good condition, with only minor bleeding in the wheelpaths observed along most of the section (with the exception of some sections with severe bleeding as shown in Figure 5.14).

Figure 5.14: Road Section 22B – chainage 15.3 km to chainage 16.0 km

Section 22B: chainage 16.0 km to chainage 17.8 km

This section is located between Toowoomba and Warwick and in 2014 carried an AADT of 6630 vehicles, of which 14% were heavy vehicles. The section was constructed in May 2014 and consists of a polymer modified binder sprayed seal surfacing, a 250 mm granular layer modified with a cement/fly ash mix (60/40), two 100 mm layers of unknown granular material, 225 mm of unknown granular material, and natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 2.11 x 10^6 ESAs.

The section investigated was typically found to be in an excellent condition, with no major defects identified. A single pothole and patch repair was observed at CH 17.088 km.

Figure 5.15: Road Section 22B – chainage 16.0 km to chainage 17.8 km

Section 22B: chainage 17.8 km to chainage 18.3 km

This section is located between Toowoomba and Warwick and in 2014 carried an AADT of 6630 vehicles, of which 14% were heavy vehicles. The section was constructed in December 2013 and consists of a geotextile reinforced sprayed seal surfacing, two 160 mm granular layers modified with cement, 300 mm of lime stabilised material and natural soil subgrade. The original design traffic for the pavement section was 1.2 x 10^7 ESAs and in 2014 the estimated cumulative traffic was 4.11 x 10^6 ESAs.

The section investigated was typically found to be in an excellent condition, with no major defects identified.

Figure 5.16: Road Section 22B – chainage 17.8 km to chainage 18.3 km

Section 22B: chainage 22.2 km to chainage 22.9 km

This section is located between Toowoomba and Warwick and in 2014 carried an AADT of 5465 vehicles, of which 11% were heavy vehicles. The section was constructed in May 2014 and consists of a geotextile reinforced sprayed seal surfacing, a 250 mm cement modified layer, 100 mm of standard granular material and natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 1.35 x 10^6 ESAs.

The section investigated was typically found to be in a very good condition, with only minor bleeding in the wheelpaths along most of the section. Minor shoving was also observed at CH 22.868 km.

Figure 5.17: Road Section 22B – chainage 22.2 km to chainage 22.9 km

5.1.3 Gore Highway

Section 28B: chainage 70.8 km to chainage 73.1 km

The Gore Highway is a 202 km stretch of national highway connecting Toowoomba and Goondiwindi. This section is located between Millmerran and Goondiwindi and in 2014 carried an AADT of 1210 vehicles, of which 41% were heavy vehicles. The section was constructed in August 2012 and consists of a bitumen sprayed seal surfacing, a 280 mm granular layer modified with 1% cement/fly ash/lime mix (30/40/30) and a 300 mm granular layer modified with 1% cement/fly ash/lime mix (30/40/30) over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 3.36×10^6 ESAs.

The road section investigated was typically found to be in a mediocre condition, with numerous instances of repaired potholes, shoving and delamination particularly in the Millmerran-bound lane. Minor bleeding was present in the wheelpaths for almost the entire length of the section. Some minor longitudinal cracking was also observed at CH 72.479 km.

Figure 5.18: Road Section 28B – chainage 70.8 km to chainage 73.1 km

5.1.4 Surat Development Road

The Surat Development Road is a state-controlled regional connector route that is approximately 189 km long. The condition of a number of sections that incorporate I-CMB pavements are discussed below.

Section 86A: chainage 75.4 km to chainage 77.3 km

This section is located between Surat and Tara and in 2014 carried an AADT of 209 vehicles, of which 21% were heavy vehicles. The section was constructed in July 2014 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm standard granular layer and a 250 mm granular layer modified with 2.5% cement/fly ash/lime mix (40/30/30) over a natural soil subgrade of unknown quality. The original design traffic for the pavement section was 4.2 x 10^5 ESA and in 2014 the cumulative traffic was estimated at 1.03 x 10^5 ESA.

The section investigated was typically found to be in an excellent condition. Only isolated longitudinal cracking was observed at CH 76.309 km.

Figure 5.19: Road Section 86A – chainage 75.4 km to chainage 77.3 km

Section 86A: chainage130.3 km to chainage 131.7 km

This section is located between Surat and Tara and in 2014 carried an AADT of 264 vehicles, of which 25% were heavy vehicles. The section was constructed in July 2014 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm standard granular layer, a 250 mm granular layer modified with 2.5% cement/fly ash/lime mix (40/30/30) and 40 mm of standard granular material over a natural soil subgrade of unknown quality. The original design traffic for the pavement section was 5.7 x 10^5 ESAs and in 2014 the cumulative traffic was estimated at 1.4 x 10^5 ESAs.

The section investigated was typically found to be in a good condition, with minor bleeding from CH 130.687 to 130.827 and moderate bleeding from CH 131.637 km to CH 131.767 km in both lanes. Isolated instances of minor to moderate longitudinal cracking were observed from CH130.377 km to CH 130.437 km, CH 130.607 km to CH 130.657 km and from CH 131.427 km to CH 131.467 km in the outer wheelpaths of the Tara-bound lane.

Figure 5.20: Road Section 86A – chainage 130.3 km to chainage 131.7 km

Section 86A: chainage 137.5 km to chainage 138.7 km

This section is located between Surat and Tara and in 2014 carried an AADT of 264 vehicles, of which 25% were heavy vehicles. The section was constructed in February 2013 and consists of a polymer modified binder sprayed seal surfacing, a 200 mm granular layer modified with a cement/fly ash/lime mix (30/30/40), a 50 mm standard granular layer and a 50 mm granular layer of unknown quality over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 2.82 x 10^5 ESAs.

The section investigated was typically found to be in a very good condition, with some instances of minor bleeding observed. Isolated instances of minor longitudinal cracking were also observed from CH138.619 km to CH 138.639 km, and at CH 138.857 km in the centre of the Tara bound-lane.

Figure 5.21: Road Section 86A – chainage 137.5 km to chainage 138.7 km

Section 86A: chainage 138.7 km to chainage 140.7 km

This section is located between Surat and Tara and in 2014 carried an AADT of 264 vehicles, of which 25% were heavy vehicles. The section was constructed in February 2013 and consists of a polymer modified binder sprayed seal surfacing, a 250 mm granular layer modified with a cement/fly ash/lime mix (30/30/40), and a 150 mm standard granular layer over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 2.82 x 10^5 ESAs.

The section investigated was typically found to be in an excellent condition, with some instances of very minor bleeding observed. Isolated instances of minor longitudinal cracking were also observed at CH139.217 km and at CH 139.377 km in the centre of the Surat-bound lane.

Figure 5.22: Road Section 86A – chainage 138.7 km to chainage 140.7 km

Section 86A: chainage 140.7 km to chainage 141.9 km

This section is located between Surat and Tara and in 2014 carried an AADT of 264 vehicles, of which 25% were heavy vehicles. The section was constructed in February 2013 and consists of a polymer modified binder sprayed seal surfacing, a 200 mm granular layer modified with a cement/fly ash/lime mix (30/30/40), a 50 mm standard granular layer and a 50 mm granular layer of unknown quality over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 2.82×10^5 ESAs.

The section investigated was typically found to be in an excellent condition, with some instances of minor bleeding observed at an intersection at CH 140.357 km. Isolated instances of pavement repairs also occur in the outer wheelpath of the Surat bound-lane between CH 141.777 km and CH 141.857 km.

Figure 5.23: Road Section 86A – chainage 140.7 km to chainage 141.9 km

Section 86A: chainage 141.9 km to chainage 142.5 km

This section is located between Surat and Tara and in 2014 carried an AADT of 264 vehicles, of which 25% were heavy vehicles. The section was constructed in February 2013 and consists of a polymer modified binder sprayed seal surfacing, a 250 mm granular layer modified with a cement/fly ash/lime mix (30/30/40), and a 150 mm standard granular layer over a natural soil subgrade of unknown quality. The original design traffic for the pavement section is unknown, however in 2014 the cumulative traffic was estimated at 2.82 x 10^5 ESAs.

The section investigated was typically found to be in an excellent condition between CH 141.897 km and CH 142.327 km. However, the pavement was in poor condition between CH 142.327 km to CH 142.507 km, with evidence of pavement repairs and rutting along the outer wheel-path of the Surat-bound lane. Moderate bleeding was present in the wheelpaths particularly at the intersection with Western Road. Some minor longitudinal cracking was also observed.

Figure 5.24: Road Section 86A – chainage 141.9 km to chainage 142.5 km

5.1.5 Bruce Highway

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 interregional route caters for both commuter and commercial traffic. The condition of a number of sections that incorporate I-CMB pavements is discussed below.

Section 10E: chainage 97.7 km to chainage 98.6 km

This section is located between Benaraby and Rockhampton and in 2014 carried an AADT of 6485 vehicles, of which 28% were heavy vehicles. The section was constructed in September 2013 and consists of a spray seal surfacing, 200 mm granular cement stabilised layer, 110 mm layer of unknown material, 150 mm layer of unknown granular material, and 100 mm of subgrade of unknown quality overlying the natural foundation. The original design traffic for the pavement section is 40 x 10^6 ESAs and in 2014 the cumulative traffic was estimated at 8 x 10^6 ESAs.

The section investigated was typically found to be in a mediocre condition, with crocodile cracking and bleeding observed in the wheelpaths from CH 97.707 km to CH 97.987 km. The crocodile cracking extended further to CH 98.137 km.

Figure 5.25: Road Section 10E – chainage 97.7 km to chainage 98.6 km

Section 10E: chainage 98.6 km to chainage 99.7 km

This section is located between Benaraby and Rockhampton and in 2014 carried an AADT of 6485 vehicles, of which 28% were heavy vehicles. The section was constructed in September 2013 and consists of a spray seal surfacing, 200 mm granular cement stabilised layer, 110 mm layer of unknown material, 150 mm layer of unknown granular material, and 100 mm of subgrade of unknown quality overlying the natural foundation. The original design traffic for the pavement section is 40 x 10^6 ESAs and in 2014 the cumulative traffic was estimated at 8 x 10^6 ESAs.

The section investigated was typically found to be in a mediocre condition, with block cracking observed in the wheelpaths from CH 98.797 km to CH 98.867 km and from CH 99.027 km to CH 99.087 km. Minor rutting was also identified from CH 98.797 km to CH 99.087 km.

Figure 5.26: Road Section 10E – chainage 98.6 km to chainage 99.7 km

5.1.6 Richmond to Winton Road

The Richmond to Winton Road is a regional connector in the Central West of Queensland, connecting Richmond to Winton. In 2014, the road carried an AADT of just 73 vehicles, 29% of which were heavy vehicles. The condition of a number of sections that incorporate I-CMB pavements is discussed below.

Section 5803: chainage 14.0 km to chainage 16.4 km

The section investigated was constructed in December of 2012, and consists of a bitumen spray seal, 200 mm of cement modified material (1.0 – 1.5% GB cement), 150 mm of standard granular material and the natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 1.9 x 10^5 ESAs.

The condition of the section investigated varied between good and very poor. The section between CH 14.000 km and CH 14.889 km only showed signs of minor to moderate bleeding in the wheelpaths. More severe defects were observed in the poorer section between CH 14.900 km and CH 15.539 km, including severe bleeding, severe longitudinal cracking, severe rutting and severe crocodile cracking.

Figure 5.27: Road Section 5803 – chainage 14.0 km to chainage 16.4 km


Section 5803: chainage 18.7 km to chainage 20.1 km

The section investigated was constructed in December of 2012, and consists of a bitumen spray seal, 200 mm of cement modified material (1.0 – 1.5% GB cement), 150 mm of standard granular material and the natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 1.9 x 10^5 ESAs.

The section investigated was typically found to be in a good condition, with moderate to severe bleeding observed in the wheelpaths along most of the section.

Figure 5.28: Road Section 5803 – chainage 18.7 km to chainage 20.1 km


5.1.7 Flinders Highway

The Flinders Highways is a major interregional route connecting Townsville to Cloncurry. The sections investigated are located between Richmond and Julia Creek and in 2014 carried an AADT of 385 vehicles, of which 38% were heavy vehicles. The condition of the sections that incorporate I-CMB pavements is discussed below.

Section 14D: chainage 55.8 km to chainage 60.1 km

The section was constructed in November of 2013, and consists of a bitumen spray seal, 250 mm of cement modified base (1.5% GB cement) and a subgrade of unknown quality. The original design traffic loading for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 6.72 x 10^5 ESAs.

The section was typically found to be in a very good condition, with minor bleeding observed in the wheelpaths along most of the section. Isolated instances of minor shoving and stripping were also observed.

Figure 5.29: Road Section 14D – chainage 55.8 km to chainage 60.1 km

Section 14D: chainage 68.7 km to chainage 69.7 km

This section was constructed in November of 2013, and consists of a bitumen spray seal, 250 mm of cement modified base (1.5% GB cement) and a subgrade of unknown quality. The original design traffic loading for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 6.72 x 10^5 ESAs.

The section investigated was typically found to be in an excellent condition, with only minor bleeding observed in the wheelpaths along most of the section. Isolated minor longitudinal cracking (possibly along a construction joint) was observed at CH 68.887 km.

Figure 5.30: Road Section 14D – chainage 68.7 km to chainage 69.7 km

5.1.8 Wills Development Road

The Wills Development Road is a regional connector route connecting Julia Creek to Burketown. In 2014, this section carried an AADT of just 8 vehicles, 50% of which were heavy vehicles. The condition of the sections that incorporate I-CMB pavements is discussed below.

Section 78A: chainage 17.6 km to chainage 19.3 km

This section was constructed in January of 2014, and consists of a bitumen spray seal, 200 mm of cement modified base (1.5% GB cement) and a subgrade of unknown quality. The original design traffic loading for the pavement section is unknown, but in 2014 the cumulative traffic was estimated 8.956 x 10^3 ESAs.

The section investigated was typically found to be in an excellent condition, with only minor stripping of the aggregate observed periodically in between the wheelpaths, mostly at the beginning of the section. Moderate stripping and bleeding of the surface seal was also observed at the intersection with Baroona Road.

Figure 5.31: Road Section 78A – chainage 17.6 km to chainage 19.3 km

Section 78A: chainage 140.9 km to chainage 142.8 km

This section was constructed in January of 2014, and consists of a bitumen spray seal, 200 mm of cement modified base (1.5% GB cement) and a subgrade of unknown quality. The original design traffic loading for the pavement section is unknown, but in 2014 the cumulative traffic was estimated 8.956 x 10^3 ESAs.

The section was typically found to be in an excellent condition, with only minor to moderate stripping of the aggregate at the beginning of the section and periodically throughout the remainder of the section.

Figure 5.32: Road Section 78A – chainage 140.9 km to chainage 142.8 km

5.1.9 Warrego Highway

The Warrego Highway is a 750 km, major interregional route connecting Ipswich to Charleville. The condition of the sections that incorporate I-CMB pavements is discussed below.

Section 18F: chainage 42.0 km to chainage 43.2 km

This section is located between Mitchell and Morven and in 2014 carried an AADT of 732 vehicles, of which 36% were heavy vehicles. The section was constructed in February 2014 and consists of a polymer modified binder sprayed seal surfacing, a 180 mm standard granular layer and a 250 mm granular layer modified with 1.0 % cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section was 5.4 x 10^6 ESAs and in 2014 the cumulative traffic was estimated at 5.56 x 10^5 ESAs.

The section investigated was typically in a good condition, however minor to moderate bleeding was present along most of the section with isolated areas of severe bleeding.

Figure 5.33: Road Section 18F – chainage 42.0 km to chainage 43.2 km

Section 18F: chainage 47.8 km to chainage 49.9 km

This section is located between Mitchell and Morven and in 2014 carried an AADT of 671 vehicles, of which 36% were heavy vehicles. The section was constructed in February 2014 and consists of a polymer modified binder sprayed seal surfacing and a 250 mm granular layer modified with 2.0% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section was 5.4 x 10^6 ESAs and in 2014 the cumulative traffic was estimated at 5.44 x 10^5 ESAs.

The section investigated was typically in a good condition, with minor bleeding observed along parts of the section. Isolated instances of minor to moderate crocodile and longitudinal cracking was observed at CH 48.200 km, CH 48.250 km and CH 48.310 km in the Mitchell-bound lane. The Mitchell-bound lane was in much worse condition than the Morven-bound lane, exhibiting most of the bleeding and cracking defects observed.

Figure 5.34: Road Section 18F – chainage 47.8 km to chainage 49.9 km

Section 18F: chainage 51.0 km to chainage 52.1 km

This section is located between Mitchell and Morven and in 2014 carried an AADT of 671 vehicles, of which 36% were heavy vehicles. The section was constructed in February 2014 and consists of a polymer modified binder sprayed seal surfacing and a 250 mm granular layer modified with 2.0% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section was 5.4 x 10^6 ESAs and in 2014 the cumulative traffic was estimated at 5.44 x 10^5 ESAs.

The section investigated was typically in a good condition, with minor bleeding observed along parts of the section. Isolated areas of moderate bleeding were observed between CH 51.580 km to CH 51.720 km in the Morven-bound lane.

Figure 5.35: Road Section 18F – chainage 51.0 km to chainage 52.1 km

Section 18G: chainage 11.1 km to chainage 14.3 km

This section is located between Morven and Charleville and in 2014 carried an AADT of 384 vehicles, of which 34% were heavy vehicles. The section was constructed in September 2013 and consists of a polymer modified binder sprayed seal surfacing, a 150 mm standard granular layer and a 200 mm granular layer modified with 3.0% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 5.34 x 10^5 ESAs.

The section investigated was typically in an excellent condition, with no major defects observed.

Figure 5.36: Road Section 18G – chainage 11.1 km to chainage 14.3 km

Section 18G: chainage 35.1 km to chainage 37.2 km

This section is located between Morven and Charleville and in 2014 carried an AADT of 384 vehicles, of which 34% were heavy vehicles. The section was constructed in September 2013 and consists of a polymer modified binder sprayed seal surfacing and a 200 mm granular layer modified with 3.0% cement/slag mix (40/60) over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 5.34 x 10^5 ESAs.

The section investigated was typically in an excellent condition, with now major defects observed.

Figure 5.37: Road Section 18G – chainage 35.1 km to chainage 37.2 km

5.1.10 Carnarvon Highway

The Carnarvon Highway is a 696 km section of state-controlled highway that links Moree (NSW) to Rolleston (QLD).

(Section 24E: chainage 51.7 km to chainage 52.9 km)

This section is located between Injune and Rolleston and in 2014 carried an AADT of 651 vehicles, of which 42% were heavy vehicles. The section was constructed in February 2013 and consists of a polymer modified binder sprayed seal surfacing and a 200 mm granular layer modified with 1.5% GGFBS/cement mix over a natural soil subgrade. The original design traffic for the pavement section is unknown, but in 2014 the cumulative traffic was estimated at 1.95 x 10^6 ESAs.

The section investigated was typically in mediocre to poor condition. Minor to moderate bleeding was observed for most of the length in both wheelpaths, particularly in the Injune-bound lane. Instances of moderate to severe longitudinal cracking (environmental) was observed between CH 52.328 km to CH 52.698 km.

Figure 5.38: Road Section 24E – chainage 51.7 km to chainage 52.9 km

5.2 Inspection Outcome

The objective of the site inspections was to validate the current condition of the representative sections that include I-CMB pavements and identify sub-sections for further investigation. The 31 road sections presented in Table 5.1 have been selected for structural capacity assessment using the FWD. The selected sections include mostly early-life pavements covering a range of conditions and exhibiting distresses.

Table 5.1: Roads selected for further investigation

Region	Road ID	Length (km)	Environmental zone	Pavement age (years)	Stabilised layer thickness (mm)	Cumulative traffic (ESA)	Stabiliser content (%)
Central West	13B	15.57	Dry Reactive	3	150	8.11E+05	2.0
Central West	13D	12	Dry Reactive	4	200	7.12E+05	3.0
Central West	13E	2.39	Dry Reactive	3	250	2.66E+05	3.0
Central West	13E	17.63	Dry Reactive	3	250	2.66E+05	3.0
Darling Downs	22B	0.65	Dry Reactive	5	125	5.73E+06	1.0
Darling Downs	22B	1.06	Dry Reactive	3	250	2.11E+06	11.5 kg/m^2
Darling Downs	22B	0.56	Dry Reactive	3	320	4.11E+06	-
Darling Downs	22B	0.73	Dry Reactive	3	250	1.35E+06	-
Darling Downs	28B	2.26	Dry Non-Reactive	4	580	3.36E+06	1.0
Darling Downs	86A	1.9	Dry Reactive	2	250	1.03E+05	2.5
Darling Downs	86A	1.31	Dry Reactive	2	250	1.40E+05	2.5
Darling Downs	86A	1.15	Dry Reactive	4	200	2.82E+05	-
Darling Downs	86A	2	Dry Reactive	4	250	2.82E+05	-
Darling Downs	86A	1.24	Dry Reactive	4	200	2.82E+05	-
Darling Downs	86A	0.56	Dry Reactive	4	250	2.82E+05	-
Fitzroy	10E	0.865	Wet Non-Reactive	3	310	8.00E+06	3.0
Fitzroy	10E	0.827	Wet Non-Reactive	3	310	8.00E+06	3.0
North West	5803	2.28	Dry Reactive	4	200	1.91E+05	1.0 - 1.5
North West	5803	1.72	Dry Reactive	2	200	4.66E+04	-
North West	14D	1.57	Dry Reactive	3	250	6.72E+05	1.5
North West	78A	1.6	Dry Reactive	3	200	8.96E+03	1.5
South West	13A	16.12	Dry Non-Reactive	4	250	5.73E+05	2.5
South West	13A	0.5	Dry Reactive	4	250	5.89E+05	2.5
South West	13A	2.6	Dry Reactive	4	250	5.89E+05	2.5
South West	13A	1.39	Dry Reactive	4	250	5.89E+05	2.5
South West	13B	8	Dry Reactive	4	250	1.07E+06	2.0
South West	18F	1.16	Dry Non-Reactive	3	250	5.56E+05	1.0
South West	18F	4.34	Dry Non-Reactive	3	250	5.45E+05	2.0
South West	18F	4.34	Dry Non-Reactive	3	250	5.45E+05	2.0
South West	18G	3.21	Dry Non-Reactive	3	200	5.34E+05	3.0
South West	18G	16.36	Dry Non-Reactive	3	200	5.34E+05	3.0
South West	24E	1.16	Dry Non-Reactive	4	200	1.95E+06	1.5

6 STRUCTURAL ASSESSMENT

A structural assessment was undertaken of the representative pavement sections presented in Table 5.1 in order to assess the in situ structural condition of the I-CMB pavements and provide an indication of future performance. The selected road sections included 82.5 km of I-CMB pavement sections. The structural assessment comprised falling weight deflectometer (FWD) testing undertaken by ARRB between May 2016 and June 2016. The FWD testing was carried out at 25 m and 50 m intervals along both the outer wheelpath (OWP) and between wheelpaths (BWP) respectively. The surface deflection results were used to determine average characteristic deflection values and estimate the allowable loading in ESAs for each 100 m section.

6.1 Surface Deflection

The FWD delivers an impulse load 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). The resulting surface deflection is measured at the centre and at fixed radial distances, most often 200 mm, 300 mm, 450 mm, 600 mm, 750 mm, 900 mm, 1200 mm and 1500 mm from the loading plate.

The deflection measurements were normalised to a 40 kN load and averaged within each 100 m section. The curvature function (defined as the difference between the deflection at the centre of the loading plate (D0) and the deflection 200 mm from the load (D200)), and the deflection ratio (DR) (defined as the ratio of the deflection 250 mm from the loading plate (D250) and D0), were also calculated. As the D250 was not directly measured, a linear interpolation between the D200 and the deflection 300 mm (D300) away from the loading plate was undertaken. 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 summarised in Appendix B.

The D900 value was also used as an indicator of the subgrade bearing capacity (TMR 2012), as per Equation 9:

$$\text{CBR (\%)} = 0.5883(D_{900})^{-1.479} \qquad 9$$

where

CBR = California Bearing Ratio

D900 = the surface deflection resulting from a 40 kN FWD impulse load measured 900 mm away from the centre of the loading plate.

6.2 Allowable Traffic Loading

The Pavement *Rehabilitation Manual* (TMR 2012) outlines the deflection reduction method for determining the allowable traffic loading of existing pavements. It should be noted that this approach is intended for unbound granular pavements where the principal failure mode is permanent deformation. This method is used where the maximum deflection (D0) and subgrade CBR of a particular pavement section are known. Given that cement modified pavement layers are modelled as unbound granular material in the current TMR design system, the deflection reduction method was considered appropriate to estimate the remaining structural life of the I-CMB sections investigated in this study. The estimated allowable loadings for the representative I-CMB pavement sections are presented in Table 6.1. The representative deflection values were determined by averaging the D0, D900, curvature and DR values across each 100 m segment within the selected pavement section.

Table 6.1: Summary of I-CMB average deflections and allowable loading estimates

Road ID	Start chainage (km)	End chainage (km)	Average normalised deflection				Estimated subgrade CBR (%)	Estimated allowable loading (ESA)
			D0 (µm)	D900 (µm)	Curvature (µm)	DR		
13B	100.3	108.0	555	158	85	0.79	10	7.7E+07
13D	65.2	67.0	592	130	107	0.76	13	6.1E+07
13E	24.2	26.6	526	117	129	0.66	15	7.9E+07
13E	26.7	27.6	462	111	112	0.67	16	9.4E+07
22B	15.3	16.0	476	108	105	0.69	17	7.9E+07
22B	16.7	17.8	534	82	174	0.56	23	4.4E+07
22B	17.8	18.3	355	80	95	0.64	24	9.1E+07
22B	22.2	22.9	402	49	109	0.64	25	9.1E+07
28B	70.8	73.1	925	112	254	0.62	16	1.1E+07
86A	75.4	77.3	298	114	36	0.84	16	>1.0E+08
86A	130.3	131.7	441	125	81	0.76	14	8.1E+07
86A	137.5	138.7	500	100	125	0.66	19	7.4E+07
86A	138.7	140.7	801	93	231	0.31	20	1.5E+07
86A	140.7	141.9	719	102	216	0.59	18	1.0E+07
86A	141.9	142.5	856	105	275	0.57	18	1.8E+07
10E	97.7	98.6	505	108	138	0.68	17	2.1E+07
10E	98.8	99.7	708	113	156	0.68	16	1.7E+07
5803	14.0	16.4	1505	205	449	0.59	7	<1.0E+05
5803	18.7	20.5	856	207	161	0.74	6	2.9E+07
14D	68.7	69.7	1121	198	278	0.66	7	1.9E+05
78A	17.6	19.3	1291	170	370	0.61	8	<1.0E+05
13A	41.9	46.9	244	52	52	0.67	25	>1.0E+08
13A	72.5	75.1	304	77	75	1.65	21	>1.0E+08
13A	80.1	81.5	438	123	80	0.76	14	9.2E+07
13A	87.2	87.6	261	70	58	0.72	24	>1.0+E08
13B	45.9	53.3	557	138	124	0.67	12	7.4E+07
18F	42.0	43.2	628	97	172	0.6	19	2.4E+07
18F	47.8	49.9	587	107	141	0.66	17	4.4E+07
18F	51.0	52.1	530	73	162	0.59	23	4.8E+07
18G	11.1	14.3	265	37	80	0.57	25	>1.0E+08
18G	35.1	37.2	144	24	21	0.79	25	>1.0E+08
24E	51.7	52.9	348	87	74	0.71	19	9.3E+07

From the results calculated by the deflection reduction method, significant structural capacity remains for the majority of the pavement sections investigated in this study. The maximum estimate of allowable traffic loading permitted by the deflection reduction method is 1.0 x 10^8 ESA and the minimum estimate is 1.0 x 10^5 ESA. However, it should be noted that five of these sections have a very stiff subgrade (> 20% CBR).

The Richmond-Winton Road (5803) between CH 14.0 km and CH 16.4 km, as well as Wills Development Road (78A) between CH 17.6 km and CH 19.3 km have very low estimated remaining lifes of less than 1.0×10^5 ESAs. It should however be noted that these sections carry very low traffic volumes (i.e. less than 100 vehicles per day).

The Flinders Highway (14D) between CH 68.7 km and CH 69.7 km only achieved an estimated allowable remaining loading of 1.9×10^5 ESA, indicating that it may require rehabilitation in the short term.

The Warrego Highway (18F) between CH 51.0 km and CH 52.1 km, Surat Development Road (86A) between CH 138.7 and CH 140.7 and sections along the New England Highway (22B) all show relatively high (>300 μm) D0 measurements despite having very high estimated CBR values (> 20% CBR). However, all of these sections have estimated allowable loading of greater than 1×10^7 ESA, indicating sufficient remaining strength.

Eleven sections along the Canarvon Highway (24E), Gore Highway (28B), Surat Development Road (86A), Bruce Highway (10E), Warrego Highway (22B) and Landsborough Highway (13E) all show relatively high (> 300 μm) D0 measurements despite having high estimated CBR values (> 15% CBR). However, similar to the sections that have an estimated CBR of greater than 20%, all of these sections have estimated allowable loading of greater than 1×10^7 ESA, indicating sufficient remaining strength.

For the nine selected road sections where the design traffic loading is known, only the two sections along the Bruce Highway (10E) have a remaining life less that the design life (in terms of ESAs). This is consistent with the crocodile (i.e. fatigue) cracking observed in the visual assessment and indicates that the pavement structure is nearing the end of its design life.

Upon review of the estimates of allowable loading based on the deflection reduction method outlined in the *Pavement Rehabilitation Manual* (TMR 2012), 29 of the 32 selected road sections reviewed should accommodate significant (> 1.0×10^7 ESA) traffic loading before a terminal serviceability condition based on structural distress is reached.

7 PERFORMANCE RELATIONSHIPS

The principal objective of the *Stabilisation Practices in Queensland* project was 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 and structural capacity assessment of I-CMB 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 I-CMB state-controlled road network inventory and condition data, as presented in Section 4 and the structural capacity assessment, as presented in Section 6 were subjected to a statistical analysis in an attempt to highlight factors that play a role in the determination of the performance of an I-CMB pavement. The analysis was undertaken by Smedley's Engineers and are summarised in the sections below (Chong 2017).

An analysis of variance (ANOVA) statistical analysis was undertaken to explore relationships between the performance indicators of roughness, rutting and maximum deflection curvature and the independent operational variables. The operational variables included environmental zone, surfacing age, stabilisation binder content, total pavement thickness and traffic volume in million equivalent standard axle (MESA) units.

It should be noted that the data used in the statistical analysis was resolved in 100 m increments and denoted by chainage. Some gaps were also present in the source data because there were cases where not all operational variables were reported in ARMIS, possibly affecting the results of the analysis. It should also be noted that the data set and how it was treated for this report differs from the PM-CMB and I-FBS report in the following ways:

- Vertical strain was not examined as an operational variable due to a lack of data availability.
- MESA was used in place of AADT and was not binned due to difficulty in determining suitable and meaningful bin sizes.
- CMB content did not appear discretely in the database as with FBS and PM-CMB, hence they were categorised based roughly on sample sizes.
- Average cost was not taken into consideration as an influential factor.
- Sensitivity and linear regression analyses on different performance and operational variables were not conducted.

The full statistical report prepared for this project is included in **Error! Reference source not ound.** .

7.1 Analysis of Variance

The ANOVA method was chosen to examine relationships between the performance indicators (roughness, rutting and deflection curvature) and the nominal, ordinal and interval (categorical) variables presented in Table 7.1, Table 7.2 and Table 7.3. These variables were selected due to the well-documented significance of environmental zone, stabilisation binder content and structural pavement thickness on the long-term performance of stabilised pavements. In most cases, the sample sizes were unevenly distributed in the majority of levels for each factor, hence a one-way ANOVA method was selected over a two-way method with or without replication methods.

The ANOVA methodology examines the means and variances of the performance indicators between different categorical groups and levels for each of the independent variables. To reject the null hypothesis that there is no significant difference between the sample populations (i.e. between environmental zones WR, WNR, DR and DNR), a p-value of less than or equal to 0.05 (5%) has been nominated. When this occurs, it also means the investigated parameter is influential to the I-CMB performance. It is worth noting that while some of the data set can be graphically assessed to be similar (in terms of means and variances/spread), the decision on whether the categorical groups are influential operational variables on the performance indicators will rely on the p-value reported in the ANOVA analysis.

Table 7.1: Nominal variables

Factors	Levels
Environmental zones	Wet non-reactive (WNR) Dry non-reactive (DNR) Wet reactive (WR) Dry reactive (DR)

Table 7.2: Ordinal and interval variables

Factors	Levels
Total pavement thickness (mm)	<200 201-300 301-400 401-500 501-600 >700
CMB (% of cementitious stabilisation binder)	0 (not cement modified) <0.5 0.5-1 1-2 >2

Table 7.3: Continuous variables

Factors
MESA
Surfacing age
Pavement thickness

7.1.1 Assumptions

The data set 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 variances are a concern).
3. Measurements must be independent.
4. Distribution of the variables should be roughly normal.

Assumptions 1 and 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 assumptions 2 and 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.

7.2 Results

The results of the ANOVA analysis are presented in Table 7.4 and identify which operational factors significantly influenced the performance indicators for the I-CMB pavement sections. For example, a marked pair in Table 7.4 corresponds to a p-value of less than 0.05 in the ANOVA analysis. From the set of continuous variables and pavement sections selected in this study, MESA was found to be the most influential variable on the performance indicators as it had a statistically significant relationship ($p < 0.05$) with all nominal variables. From the table, cement content (CMB %) and total pavement thickness were the next most influential, with an equal number of statistically significant relationships with the nominal variables. Finally, surface age was the least influential of the continuous variables, only affecting maximum deflection curvature with a statistically significant relationship with environmental zone (nominal variable) only.

From the set of nominal variables, the 'environmental zone' variable is clearly the most influential on the performance indicators, followed by cement content and 'total pavement thickness' bins which have an equal number of statistically significant relationships with the continuous variables.

Table 7.4: Influential variables summary for I-CMB as determined from ANOVA

Performance indicators	Nominal variables / Continuous variables	Total pavement thickness	Environmental zone	CMB %
Maximum deflection curvature	Total pavement thickness		X	X
	Surface age		X	
	CMB %	X	X	
	MESA	X	X	X
Rutting	Total pavement thickness		X	X
	Surface age			
	CMB %	X	X	
	MESA	X	X	X
Roughness	Total pavement thickness		X	X
	Surface age			
	CMB %	X	X	
	MESA	X	X	X

8 ECONOMIC ASSESSMENT

I-CMB pavements provide numerous technical advantages. In addition to this, I-CMB can 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 structural layers, relative to Queensland traffic and environmental conditions. In order to assess the economic benefits, construction cost information was gathered from the Mackay/Whitsunday and North-West Districts. A number of key factors affect the cost of pavement stabilisation treatments. The most common factors include:

- 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
- stabilisation binder 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.

Economic data was gathered from the TMR regions and broken down into cost items to establish high, medium and low-cost capital budget considerations for I-CMB pavement layers. In order to effectively compare I-CMB base with unbound granular base across a range of traffic design loadings (expressed as ESAs), a representative pavement configuration was developed. The minimum design thickness was determined using a linear-elastic analysis in accordance with Austroads (2012) and an estimate of the capital investment for each alternative was determined by applying the minimum, average and maximum values of the supplied project cost data. 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 I-CMB technology is presented in Section 8.1. It is important to note that the economic benefits of stabilisation technologies extend beyond initial capital investment and that a true assessment would consider the whole-of-life costs. However, due to the relatively low age of a majority of the identified pavement sections and a lack of annual maintenance expenditure data, the economic assessment was constrained to the capital investment data.

8.1 Capital Cost Analysis

When the in situ granular material is of sufficient quality, the cost of I-CMB in the regions varied between $14/m^3 and $28/m^3, with an average unit rate of around $21/m^3. When the in situ granular material is not of sufficient quality and must be supplied, the cost increases dramatically to between $119/m^3 and $228/m^3. These prices included supply of the cementitious stabiliser, mixing of the cementitious stabiliser and aggregate in situ with a stabiliser, transport of the stabiliser to the project site, formation of the base layer using a paver, compaction, trimming and curing. The basic cost items for I-CMB collected as part of this investigation are summarised in Table 8.1.

Table 8.1: Indicative costs of principal I-CMB budget items

Cost item	Low cost	Medium cost	High cost
Cementitious binder ($/tonne)	$315.00	$433.00	$550.00
Parent aggregate (Type 2.2)	$105.00	$153.00	$200.00
In situ mixing, transport and placement	$7.00	$10	$14.00
Water curing	$1.00	$2.00	$3.00

Four design loading conditions were considered to ascertain differences in costs including 1 x 105 ESAs, 1 x 10^6 ESAs, 1 x 10^7 ESAs and 1 x 10^8 ESAs. A summary of the modulus values utilised in the analysis and the minimum design thickness calculated is presented in Table 8.2.

Table 8.2: Representative pavement configuration for I-CMB cost analysis

Design traffic ESA	Unbound granular base		I-CMB		Subbase		Subgrade	
	Modulus	Minimum thickness	Modulus	Minimum thickness	Modulus	Minimum thickness	Modulus	Minimum thickness
1.00E+05	300	125	500	125	150	200	50	∞
1.00E+06	350	225	500	200	150	200	50	∞
1.00E+07	350	325	500	300	150	200	50	∞
1.00E+08	350	425	500	400	150	200	50	∞

Applying the minimum layer thickness values presented in Table 8.2 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 Table 8.3.

Table 8.3: Estimated cost of I-CMB vs unbound granular

Material	Design traffic	Minimum cost (per m^3)	Average cost (per m^3)	Maximum cost (per m^3)
TMR Type 2	1.00E+05	$50	$65	$80
TMR Type 2	1.00E+06	$80	$110	$140
TMR Type 2	1.00E+07	$90	$125	$165
TMR Type 2	1.00E+08	$95	$135	$175
I-CMB with aggregate	1.00E+05	$120	$175	$230
I-CMB with aggregate	1.00E+06	$120	$175	$230
I-CMB with aggregate	1.00E+07	$120	$175	$230
I-CMB with aggregate	1.00E+08	$120	$175	$230
I-CMB without aggregate	1.00E+05	$15	$20	$30
I-CMB without aggregate	1.00E+06	$15	$20	$30

I-CMB without aggregate	1.00E+07	$15	$20	$30
I-CMB without aggregate	1.00E+08	$15	$20	$30

When the in situ granular material is of sufficient quality (refer to Section 3.2), I-CMB pavements can be provided at a much lower cost than replacing the existing material with TMR Type 2 unbound granular material. Due to the investment in parent aggregate already being made, the cost of I-CMB base only includes transport of the cement and utilisation of a dedicated stabilising machine. Other construction activities, including compaction, trimming and curing are typically required for a TMR Type 2 material base. As such, a pavement layer can be modified with cement for approximately 88% less than the cost of TMR Type 2 unbound granular material in a standard pavement design application where high design traffic loadings are present.

However, where the in situ granular material is of insufficient quality and virgin TMR Type 2 material is required, the cost for I-CMB is typically 23% greater than just unbound granular base at traffic levels greater than 1×10^6 ESAs. This is due to the cost of utilising a dedicated stabiliser and purchase and transport of the stabilisation binder.

8.2 Economic Analysis Summary

As shown in Section 4.2, the I-CMB pavement sections throughout the Queensland network are typically performing very well with 95.5% of the network condition categorised as good or excellent. Combining this with the low cost of capital expenditure provided by I-CMB when the in situ granular material is of sufficient quality suggests that I-CMB pavements are a cost-effective solution for structural rehabilitation projects. Given the typically good performance of the I-CMB pavements, it could also be cost-effective for new projects on moderate to high trafficked roads where virgin Type 2 material is used and then in situ stabilised.

In light traffic volume applications (< 3.0×10^5 ESA) the use of I-CMB structural layers may not be an appropriate solution. Current stabilised material construction practice suggests that minimum layer thickness should be limited to 200 mm (TMR 2012).

It is important to note that the overall economic impact of road pavement infrastructure is ideally assessed relative to a whole-of-life cost where initial capital investment, maintenance costs and user costs are considered. Whole-of-life costing of the representative I-CMB pavement sections has not been conducted in this research due to data limitations and the relatively short life of most of the sections, but such an assessment may add value if sufficient surfacing treatment and routine maintenance data is available.

9 CONCLUSIONS AND RECOMMENDATIONS

A review of I-CMB pavement sections was conducted to develop technical guidance on the selection, design and construction of stabilised pavement layers that is consistent with international best practice and Queensland conditions. Establishment of best practice was achieved through a comprehensive literature review that included both national and international specifications, as well as other technical documents. Current practices in Queensland were documented by referencing 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 and structural capacity of the selected pavements. Confirmation of best practice relative to Queensland roadbed conditions was pursued through a statistical analysis of the influence of pavement configuration, traffic, environment and climatic factors on the in-service performance of I-CMB base pavements.

Conclusions resulting from the investigation include:

- The literature review typically showed that Queensland practices are generally aligned with national and international best practice when considering construction techniques. However, minor differences in stabilisation binder content, spread rate, 7-day UCS values and degree of compaction were observed.
- The controlling failure mode considered by TMR in the design of cement/cementitious modified pavements (i.e. permanent deformation in overall pavement structure) differs from some current practices internationally. The South African approach considers the fatigue, permanent deformation and crushing potential of lightly bound materials with UCS values similar to TMR specifications. The current approach in the USA considers permanent deformation in the cement/cementitious modified material.
- I-CMB pavement sections are utilised extensively across Queensland, representing approximately 18.5% of the state-controlled road network. The technology is most commonly utilised in the North-West, South-West and Fitzroy districts.
- Approximately 64% of I-CMB pavements in the network are less than 4- years old and 94% are less than 12-years old.
- Condition categorisation showed that 87% of the I-CMB pavements along the state-controlled road network were in a good condition at the time of this investigation.
- Results from the structural assessment on the representative pavement sections showed that many of these sections have significant (> 10^6 ESA) structural capacity remaining.
- The statistical analysis undertaken found that the environment in which the I-CMB pavements operate has the biggest influence on performance, followed by cement content and total pavement thickness.
- The approximate initial costs of constructing I-CMB pavements ranged from \$14 per m^3 to \$28 per m^3 where the in situ granular material is of sufficient quality. Where Type 2 material must also be provided, the costs ranged from \$119 per m^3 to \$228 per m^3 depending on the location of the project, layer thickness, additive type, location of the additive, total quantities, curing regime and quarry source. This is a potential saving of between \$105 per m^3 and \$200 per m^3.

The following recommendations are made for consideration by TMR:

- A large proportion of the I-CMB network assessed (based on the data in ARMIS) showed evidence of fatigue cracking. There may therefore be a need to limit the tensile strength developed in cement modified layers to ensure that the material behaves similar to unbound granular materials as per the current design assumption in Queensland.

- Durability of the stabilised material is an important property that impacts on the long-term performance of stabilised pavements. TMR currently does not specify any durability requirements or test methods. However, there are durability test methods available, both locally and internationally, that could potentially be considered for adoption by TMR.

REFERENCES

Adamson, L 2012, ‘An investigation into the cement content of stabilised pavement’, MPhil, Curtin University, Perth, WA.

American Concrete Institute 2009, *Report on soil cement*, ACI 230.1R-09, ACI, Farmington Hills, MI, USA.

Arizona Department of Transportation 2008, *Standard specifications for road and bridge construction*, 304b, ADOT, Phoenix, Arizona, USA.

Austroads 2002a, *Mix design for stabilised pavement materials*, AP-T16-02, Austroads, Sydney, NSW.

Austroads 2002b, *Effect of design, construction and environmental factors for long-term performance of stabilised materials*, APRG technical note no. 12, Austroads, Sydney, NSW.

Austroads 2006, *Guide to pavement technology: part 4D: stabilised materials,* AGPT04D-06, Austroads, Sydney, NSW.

Austroads 2009a, *Guide to pavement technology: part 8: pavement construction*, AGPT08-09, Austroads, Sydney, NSW.

Austroads 2009b, *Guide to pavement technology: part 1: introduction to pavement technology,* 2nd edn, AGPT01-09, Austroads, Sydney, NSW.

Austroads 2009c, *Guide to asset management: part 5H: performance modelling,* AGAM05H-09, Austroads, Sydney, NSW.

Austroads 2012, *Guide to pavement technology: part 2: pavement structural design*, 3rd edn, AGPT02-12, Austroads, Sydney, NSW.

Austroads 2013, *Review of definition of modified granular materials and bound materials,* AP-R434-13, Austroads, Sydney, NSW.

Austroads 2014, *Cemented materials characterisation: final report,* AP-R462-14, Austroads, Sydney, NSW.

AustStab 2006, *Insitu stabilisation construction guide,* Australian Stabilisation Industry Association, Sydney, NSW.

AustStab 2012, *Cement stabilisation practice,* AustStab technical note no. 5, Australian Stabilisation Industry Association, Sydney, NSW.

Burns, SE & Tillman, KA 2006, *Evaluation of the strength of cement-treated aggregate for pavement bases,* VTRC 06-CR7, Virginia Department of Transportation, Richmond, VA, USA.

Chong, L 2017, *Statistical analysis of insitu CTB stabilisation,* Smedley’s Engineers, Brisbane, QLD.

Committee of State Road Authorities 1986, *Cementitious stabilizers in road construction,* TRH 13, CSRA, Department of Transport, Pretoria, South Africa.

Crase, N 1998, 'In situ stabilisation: construction issues: getting it right the first time', *In situ stabilisation of local government roads seminar, Brisbane, Qld,* Brisbane City Council & AustStab, Brisbane, Qld.

Department of Infrastructure 2015, *2015/2016 standard specification for roadworks,* NTDOI, Palmerston, NT.

Department of Planning, Transport and Infrastructure 2007, *Insitu stabilisation,* part R25, DPTI, Adelaide, SA.

Department of State Growth 2016, *Section 307: in situ stabilisation of pavements with cementitious binders,* DOSG, Hobart, Tas.

Doshi, SN & Guirguis, HR 1983, 'Statistical relations between compressive and tensile strengths of soil-cement', *Australian Road Research*, vol. 13, no. 3, pp. 195-200.

Dunlop, RJ 1980, 'A review of the design and performance of roads incorporating lime and cement stabilised pavement layers', *Australian Road Research,* vol. 10, no. 3, pp. 12-26.

Garber, S, Rasmussen, RO & Harrington, D 2011, *Guide to cement-based integrated pavement solutions,* Portland Cement Association, Skokie, IL, USA.

Gautrans 2004, *Stabilization manual,* manual L 2/04, Gauteng Department of Public Transport, Roads and Works (Gautrans), Pretoria, South Africa.

Gerke, RJ 1981, *In situ testing of the permeability of roadmaking materials*, AIR 317-3, ARRB Group, Vermont South, Vic.

Gray, W, Frobel, T, Browne, A, Salt, G & Stevens, D 2011, *Characterisation and use of stabilised basecourse materials in transportation projects in New Zealand,* research report 461, NZ Transport Agency, Wellington, NZ.

Halsted, GE 2011, 'Cement-modified soil for long lasting pavements', *Transportation successes: let's build on them: annual conference and exhibition of the Transportation Association of Canada, 2011, Edmonton, Canada,* Transportation Association of Canada, Ottawa, ON, Canada, 13 pp.

Huat, BBK, Maail, S & Mohamed, TA 2005, 'Effect of chemical admixtures on the engineering properties of tropical peat soils', *American Journal of Applied Sciences,* vol. 2, no. 7, pp. 1113-20.

Jameson, GW 1995, *Response of cementitious pavement materials to repeated loading,* University of South Australia, Adelaide, SA.

Kandhal, P & Mallick, RB 1997, *Pavement recycling guidelines for state and local governments*, FHWA-SA-98-042, Federal Highway Administration, Washington, DC, USA.

Kennedy, TW 1983, *Tensile characterization of highway pavement materials,* research report FHWA/TX-84/21+183-15F, University of Texas, Austin, TX, USA.

Lay, MG & Metcalf, JB 1983, 'Soil stabilisation in Australia', *National conference on local government engineering, 2nd, 1983, Brisbane, Qld,* Institution of Engineers, Australia, Barton, ACT, pp. 346-51.

Lim, S & Zollinger, D 2003, 'Estimation of the compressive strength and modulus of elasticity of cement-treated aggregate base materials', *Transportation Research Record,* no. 1837, pp. 30-8.

Little, DN & Nair, S 2009, R*ecommended practice for stabilization of subgrade soils and base materials*, National Cooperative Highway Research Program (NCHRP) web-only document no. 144, Transportation Research Board, Washington, DC, USA.

Main Roads Western Australia 2012, *Specification 501: pavements*, MRWA, Perth, WA.

Matanovic, AM 2012, 'Performance equation for cement modified pavements', technical paper S0184117, Central Queensland University, Rockhampton, Qld.

Milburn, J & Parsons, R 2004, *Performance of soil stabilization agents,* report K-TRAN: KU-01-8, Kansas Department of Transportation, Topeka, KS, USA.

National Cooperative Highway Research Program 2004, *Guide for mechanistic-empirical design of new and rehabilitated pavement structures*, 1-37A, NCHRP, Transport Research Board, Washington, DC, USA.

Paige-Green, P 2008, 'A reassessment of some road material stabilization problems', *Proceedings of the 27th South African transport conference (SATC), 2008, Pretoria, South Africa,* Council for Scientific and Industrial Research (CSIR), Pretoria, South Africa, 10 pp.

Putra, AI 2014, 'Stabilisation of expansive subgrade soils with slag and cement for road construction', MPhil, Curtin University, Perth, WA.

Queensland Department of Transport and Main Roads 2012, *Pavement rehabilitation manual*, TMR, Brisbane, Qld.

Queensland Department of Transport and Main Roads 2013, *Pavement design supplement: supplement to 'Part 2: Pavement Structural Design' of the Austroads Guide to Pavement Technology,* TMR, Brisbane, Qld.

Queensland Department of Transport and Main Roads 2016a, *Materials testing manual*, TMR, Brisbane, Qld.

Queensland Department of Transport and Main Roads 2016b, *Unbound pavements,* Transport and Main Roads specifications MRTS05, TMR, Brisbane, Qld.

Queensland Department of Transport and Main Roads 2017a, *Technical note 149: testing of materials for cement or cementitious blend stabilisation,* TMR, Brisbane, Qld.

Queensland Department of Transport and Main Roads 2017b, *Insitu stabilised pavements using cement or cementitious blends*, Transport and Main Roads specifications MRTS07B, TMR, Brisbane, Qld.

Queensland Department of Transport and Main Roads 2017c, *Supply and delivery of quicklime and hydrated lime for road stabilisation*, Transport and Main Roads specifications MRTS23, TMR, Brisbane, Qld.

Riaz, S, Aadil, N & Waseem, U 2014, 'Stabilization of subgrade soils using cement and lime: a case study of Kala Shah Kaku, Lahore, Pakistan', Pakistan Journal of Science, vol. 66, no. 1, pp. 39-44.

Roads and Maritime Services 2012, *Durability of road materials modified or stabilised by the addition of cement,* test method T133, RMS, Sydney, NSW.

Roads and Maritime Services 2013, *Construction of unbound and modified pavement course*, QA specification R71, RMS, Sydney, NSW.

Roads and Maritime Services 2015, *Insitu pavement stabilisation using slow setting binders*, QA specification R75, RMS, Sydney, NSW.

Saxena, P, Tompkins, D, Khazanovich, L, Tadeu Blabo, J 2010, 'Evaluation of characterization and performance modeling of cementitiously stabilized layers in the mechanistic-empirical pavement design guide', *Transportation Research Record,* no. 2186, pp. 111-9.

Scullion, T, Sebesta, S, Harris, JP & Syed, I 2005, *Evaluating the performance of soil-cement and cement-modified soil for pavements: a laboratory investigation,* Research & Development Bulletin RD120, Portland Cement Association, Skokie, IL, USA.

Smith, W 2005, 'Recognition of environmental and social advantages of using stabilisation in road rehabilitation', *Institute of Public Works Engineering Australia (IPWEA) New South Wales division conference, 5th, 2005, Sydney, New South Wales, Australia,* IPWEA, Sydney, NSW, 13 pp.

South African National Roads Agency 2014, *South African pavement engineering manual,* 2nd edn, SANRAL, Pretoria, South Africa.

Symons, MG & Poli, DC 1999, *Properties of Australian soils stabilised with cementitious binders: volume 1 and 11,* University of South Australia, Adelaide, SA.

Transit New Zealand 2006, *Specification for basecourse aggregate,* TNZ M/4, NZTA, Wellington, New Zealand.

Transit New Zealand 2007, *New Zealand supplement to the document, Pavement Design: A Guide to the Structural Design of Road Pavements (Austroads, 2004)*, NZTA, Wellington, New Zealand.

Transit New Zealand 2008, *Specification for in-situ stabilisation of modified pavement layers*, TNZ B/5, TNZ, Wellington, New Zealand.

VicRoads 2008, *In situ stabilisation of pavements with cementitious binders: section 307*, VicRoads, Kew, Vic.

White, G 2006, 'Laboratory characterisation of cementitiously stabilised pavement materials', MEng, University of New South Wales, Sydney, NSW.

Wilmot, TD 1996, *Fifty years of stabilisation,* Road note 50, Cement and Concrete Association of Australia, Sydney, NSW.

Wilmot, TD 2006, 'The importance of stabilisation techniques for pavement construction', *ARRB conference, 22nd, 2006, Canberra, ACT,* ARRB Group, Vermont South, Vic, 12 pp.

Wirtgen GmbH 2012, *Wirtgen cold recycling technology*, Wirtgen Group, Widhagen, Germany.

Yeo, YS, Jitsangiam, P & Nikraz, H 2011, 'Mix design of cementitious basecourse', *International conference on advances in geotechnical engineering (ICAGE), 2011, Perth, Western Australia,* Curtin University, Perth, WA, pp. 379-85.

Standards

AS 1289.6.7.3:2016, *Methods of testing soils for engineering purposes: method 6.7.3: soil strength and consolidation tests: determination of permeability of soil: constant head method using a flexible wall permeameter*.

AS 3582.1:2016, *Supplementary cementitious materials: part 1: fly ash.*

AS 3582.2:2016 *Supplementary cementitious materials: slag: ground granulated iron blast-furnace.*

AS 3972-2010, *General purpose and blended cements.*

BIS EN 13286-42:2003, *Unbound and hydraulically bound mixtures: part 42: test method for the determination of the indirect tensile strength of hydraulically bound mixtures*.

SANS 2001-GR55:2012, *Determination of the wet-dry durability of compacted and cured specimens of cementitiously stabilized materials by hand brushing.*

APPENDIX A LIST OF I-CMB PAVEMENT SECTIONS

APPENDIX B FWD MEASUREMENTS