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Development and Laboratory Assessment of Two States of the Art Double Coating Techniques for Recycled Concrete Aggregates (RCA) used in Asphalt Mixtures

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Declaration

To the best of my knowledge and belief, this thesis contains no material previously published by any other person except where due acknowledgment has been made.

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

Signature:
Date: 30/01/2020

Abstract

Hot-mix asphalt (HMA) that contains aggregate is the main material used in the construction of roads. Replacement of aggregate with recycled concrete aggregate (RCA) made from construction and demolition waste has the dual benefit of reducing the usage of natural aggregates, which have a limited supply, and diverting building waste from landfill. However, RCA-asphalt mixtures have several issues stemming from the higher bitumen absorption rates and lower strength of RCAs compared to those of natural aggregates. Accordingly, some researchers recommend not using RCAs in HMA, while others have used heating or coating treatments to improve their properties. However, no previously suggested treatments have yet been adopted for HMA production. In Australia, the use of RCAs in HMA is not permitted and only limited information regarding Australian asphalt mixtures containing RCAs is available. This thesis investigates the use of RCAs in Australian HMA by developing two state-of-the-art double-coating techniques (DCTs; named DCT1 and DCT2) that aim to improve the properties of RCAs.

The suitability of the DCTs for HMA production was assessed through a comprehensive experimental program. Testing of the double-coated recycled concrete aggregates (DCRCAs) indicates that the DCTs decrease

water and bitumen absorption by RCAs and increase their strength. Asphalt mixtures made with 0%, 20%, 40% and 60% DCRCA are also evaluated. The results of Marshall tests reveal that asphalt mixtures containing DCRCAs have lower optimum bitumen content (OBC) and greater stability than mixtures containing untreated RCAs. In addition, both DCTs minimise the difference between dry and wet indirect tensile strength, and improve stripping resistance. Asphalt mixtures made with double-coated RCAs showed better resilient modulus than control mix at up to 40% replacement rates at 40 °C in case of DCT1, and at 25 °C and 40 °C in the case of DCT2. Rutting and flow number tests show that the addition of DCRCAs does not affect the resistance of asphalt mixtures to permanent deformation. Fatigue life tests reveal that fatigue life increases with the addition of DCRCAs. The dynamic modulus of DCRCA-asphalt mixtures are either comparable (when DCT1 is used) or better (when DCT2 is used) than that of control mix. The fatigue and dynamic modulus properties of DCRCA-asphalt mixes imply that Australian HMA made with RCAs behaves better than RCA-asphalt mixes that have been made in other parts of the world, such as the USA. Based on these results, DCRCAs could be potential replacements for granite aggregates in Australian HMA. Thus, the current Australian policy of not using recycled aggregates in HMA may need to be re-examined in order to reduce the increasing amounts of RCAs and maintain sustainable practices.

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List of Publications

The following publications are based on the work undertaken for this thesis.

Refereed journal papers:

1. Abbaas Inaayah Kareem, Hamid Nikraz, and Hossein Asadi (2018). Evaluation of the double coated recycled concrete aggregates for hot mix asphalt. *Construction and Building Materials*, *172*, 544-552.
2. Abbaas Inaayah Kareem, and Hamid Nikraz (2018). Development of a double coating technique for recycled concrete aggregates used in hot-mix asphalt. *World Academy of Science, Engineering and Technology, International Journal of Urban and Civil Engineering*, *12*(6).
3. Abbaas Inaayah Kareem, Hamid Nikraz, and Hossein Asadi (2019). *Performance of hot-mix asphalt produced with double coated recycled concrete aggregates. Construction and Building Materials*, *205*, 425-433.
4. Abbaas Inaayah Kareem, Hamid Nikraz, and Hossein Asadi (2019). Application of double coated recycled concrete aggregates for hot-mix asphalt. *Journal of Materials in Civil Engineering*, *31*(5): p. 04019036.

5. Abbaas Inaayah Kareem, Hamid Nikraz, and Hossein Asadi (2020). Characterization of Asphalt Mixtures Containing Double-Coated Recycled Concrete Aggregates. Journal of Materials in Civil Engineering, 32(2), 04019359.

Refereed conference papers:

1. Abbaas Inaayah Kareem, and Hamid Nikraz (2018). *Development of a double coating technique for recycled concrete aggregates used in hot-mix asphalt*. Paper presented at the 20th International Conference on Pavement Engineering and Infrastructure. June, 25-26, 2018. Paris, France.
2. Abbaas Inaayah Kareem, and Hamid Nikraz (2018). *The effects of using double coated recycled concrete aggregates on volumetrics of hot-mix asphalt.* Paper presented at the One Curtin International Postgraduate Conference. November, 26-28, 2018. Miri, Malaysia.

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1 Introduction

1.1 General background

Recycled concrete aggregate (RCA) is mainly produced from construction and demolition (C&D) activities and by natural disasters. It is mainly composed of natural aggregate with old cement mortar attached to its surfaces (Martinho & Picado-Santos, 2018). It also contains impurities such as brick, tiles, recycled asphalt pavement (RAP), ceramics, metals, wood, plastics and gypsum plaster (A. R. Pasandín & Pérez, 2014a). In recent years, the quantities of such solid waste materials have increased, as have efforts to mitigate the economic and environmental problems such solid waste can cause.

In Australia and other parts of the world, many studies have been carried out to investigate the use of RCAs for base and subbase applications (Arul Arulrajah, Disfani, Horpibulsuk, Suksiripattanapong, & Prongmanee, 2014; Arul Arulrajah, Piratheepan, Disfani, & Bo, 2012; A. Arulrajah, Piratheepan, & Disfani, 2014; Leite, Motta, Vasconcelos, & Bernucci, 2011). Furthermore, Marinković, Radonjanin, Malešev, and Ignjatović (2010) stated that the RCAs derived from C&D waste are primarily applied in pavement construction, such as for subbase and base constructions and embankment works. Such uses, however, can increase the pH of groundwater and, therefore, affect vegetation near roads (Robinson et al., 2004). Fine recycled aggregates can also dissolve in water, react with CO_2 in the air, and precipitate out calcium carbonate. As a consequence, this can potentially clog up the drainable layers (Wong, Sun, & Lai, 2007).

In contrast, when used in asphalt, use of both fine and coarse RCA particles is feasible, as the pores will be sealed up and provide protection against water damage (Zulkati, Wong, & Sun, 2013). Furthermore, a layer of bitumen on the surfaces of RCAs is believed to eliminate the lixiviation of chemicals (Z. Zhang, Wang, Liu, & Deng, 2016), which can happen in unbound roads and cause significant concerns for environmentalists. Accordingly, RCA has been evaluated for use in hot-mix asphalt (HMA) for its potential economic and environmental benefits.

Even though some states in Australia have managed their concrete recycling very well, the overall recycling rate in Australia is still low (Tam, 2009). In Australia, the use of RCAs for higher grade pavement applications, such as HMA production, is not yet permitted and is not encouraged. According to Austroads technical report AGPT04E/09 entitled *Guide to Pavement Technology Part 4E: Recycled Materials*, 30% of natural aggregates can typically be replaced by coarse RCAs in concrete but not HMA (Andrews & Rebecchi, 2009). Based on the available documentation, Australia is behind the USA (Jin & Chen, 2015) and Japan (Wijayasundara, Mendis, & Zhang, 2013) in RCA recycling. In this regard, the Department of Transportation in Michigan, USA, permits the use of crushed concrete aggregate (CCA) in HMA (Gonzalez & Moo-Young, 2004). However, the current Australian technical specification for recycled aggregate is mainly concentrated on utilizing it in unbound base- and subbase-layer construction (Tam, 2009).

There is no data on the consumption of natural aggregates in Australia; however, the use of virgin materials like natural aggregates in asphalt and concrete projects has increased due to economic and population growth (Wardle, Cosson, & Trinh, 2016), and increasing highway and concrete works (Leek et al., 2012). Based on these trends, the quantities of natural aggregates available to future generations will constantly be decreased and the use of RCAs derived from C&D waste in pavement and concrete works has to be increased.

1.2 Research problem

Hot-mix asphalt (HMA) produced with RCAs is documented to suffer from several issues. These issues can be divided into two main groups: binder-related and behaviour-related. The binder-related issue is associated with the asphalt absorption rate of RCAs, which is normally higher than those of natural aggregates. As a result, HMA made with RCAs has a higher optimum bitumen content (OBC) than that of a mix prepared with natural aggregates only (A. R. Pasandín & Pérez, 2014b; J. Zhu, Wu, Zhong, & Wang, 2012). The second issue is basically a result of utilizing lower quality aggregates, i.e. RCAs, in HMA production. This can decrease the resistance of asphalt mixtures to moisture damage and lower their performance at the same time.

Taking into consideration these two issues, some researchers recommend not using RCAs in HMA (Bhusal & Wen, 2013), while others suggest using various treatments in order to improve the durability and strength of RCA-asphalt mixtures. Some previously used treatments were based on heating the RCA before mixing it with asphalt (Wong et al., 2007), or heating the RCA-asphalt mixture in an oven before mixture compaction (A. R. Pasandín & Pérez, 2013). Other treatments were based on coating the RCA with different materials to achieve certain objectives (Yueqin Hou, Ji, Su, Zhang, & Liu, 2014; Lee, Du, & Shen, 2012; Pan et al., 2015; A. R. Pasandín & Pérez, 2014b; J. Zhu et al., 2012).

To date, none of these treatments has yet been used in the pavement industry. This implies a need to develop new treatments to make the performance of RCA-asphalt mixtures better than that achieved previously. Therefore, two state-of-the-art upgrading techniques are developed in the present study in order to increase the performance of Australian HMA type AC14, which is made with RCAs derived from construction and demolition (C&D) waste. Each technique is based on combining two treatments, so the new technique will be called the *double coating technique* (DCT). The RCAs treated with the DCT will hereafter be named *double coated recycled concrete aggregates* (DCRCAs). Two different DCTs will be developed after taking into account the results of previously used treatments. The treatments used in the DCTs will be modified to suit the material properties and objectives of current research.

1.3 The significance of the research

As mentioned before, the use of RCAs is not permitted in the Australian asphalt industry. In addition, only limited studies have been undertaken to investigate Australian HMA made with RCAs. During the last decade, some researchers have suggested different treatments to improve the behaviour of RCAs in HMA. The treatments were based on coating or heating treatments. However, none of these suggested treatments has yet been used in the pavement industry, indicating that the development of new upgrading technique is required.

For the purpose of this research, two techniques are developed on the basis of combining the advantages of two treatments. The new techniques are, therefore, expected to bring RCA-asphalt mix properties to an acceptable level of performance over that achieved by previous treatments. The significance of the DCT is to identify a new way to systematically enhance the properties of RCA-asphalt mixtures, which may encourage the use of low-quality RCAs in asphalt mixtures in Australia and other parts of the world.

Another advantage for current research is that the RCA is cheaper and lighter than granite aggregates (Leek et al., 2012), which can result in lower construction and transportation costs. The use of RCAs in asphalt mixtures can also mitigate supply shortages of natural aggregate for road construction (X. Zhang, Zhang, Chen, & Kuang, 2018), compensate for the cost of processing and transporting RCAs (Yueqin

Hou et al., 2014) and promote sustainable practices (Chica-Osorio & Beltrán-Montoya, 2018; J. P. Giri, M. Panda, & U. C. Sahoo, 2018). Therefore, its usage would facilitate the flow of RCAs within the community, reduce the burden on natural aggregate resources, and compensate for the construction and transportation costs of building new road infrastructure.

Past research on Australian HMA made with RCAs has also not investigated fatigue life and dynamic modulus (Paranavithana & Mohajerani, 2006). Therefore, in the present study, these two performance properties will be evaluated for asphalt mixtures made with RCAs treated with DCTs. This is important for maintaining the applicability of fatigue and dynamic modulus comparisons between asphalt mixes made with two types of DCRCAs and natural granite aggregates, and also to assess the degree of improvement achieved after applying the DCTs.

1.4 Scope and research objectives

The main goal of this study is to develop and assess two state-of-the-art DCTs to upgrade the engineering properties of RCAs, which could enhance the properties of asphalt mixtures. To achieve this goal, the specific objectives of this study were formulated and are summarised below:

1. Critically review the literature on RCA treatments for use in HMA.
2. Determine the pros and cons of RCA and asphalt mixture properties after previous applied treatment.
3. Determine a method to combine two previously used treatments systematically by taking into account their advantages and disadvantages. In addition, modify the selected treatments to suit the material properties and the objectives of the DCTs.

4. Evaluate the improvement achieved upon the utilization of the two DCTs by assessing the essential properties of asphalt mixtures made with different percentages of DCRCAs (0%, 20%, 40% and 60%). For this purpose, a detailed experimental program was carried out in the Geomechanics and Pavement Laboratory at Curtin University. The following HMA tests are to be carried out for this purpose:
   a) Marshall design method: this method consists of three main tests:
      - Determination of the bulk density of compacted asphalt mixture (Standards Australia/Standards New Zealand, 2014).
      - Stability and flow tests (Australian/New Zealand Standards, 2015).
      - Maximum density of asphalt mixture determination (MRWA, 2011).
   b) Indirect tensile strength (ITS) test (ASTM, 2017).
   c) Tensile strength ratio (TSR) test (Austroads, 2007b).
   d) Indirect tensile stiffness modulus (ITSM) test (Australian/New Zealand Standard, 2013).
   e) Rutting resistance test (Austroads, 2006a).
   f) Flow number test (AASHTO, 2014b).
   g) Four-point bending beam fatigue life test (Austroads, 2006b).
   h) Dynamic modulus test (AASHTO, 2014b).
5. Generate dynamic modulus master curves for HMAs made with natural aggregates and RCAs treated by DCTs to investigate the level of improvement post-DCTs.
6. Draw conclusions and make recommendations for the future sustainable use of RCAs treated by the DCTs.

1.5 Thesis structure

This thesis consists of six chapters as shown in Figure 1.1. Chapter 1 gives a general background of the current work, its research problems, the significance of the research, the scope and objectives of this thesis, and its structure. Chapter 2 presents a review of the literature with particular reference to HMAs made with untreated RCAs and RCAs treated by heating or coating. Chapter 3 describes the concepts of the two state-of-the-art DCTs. It explains the hypotheses and criteria used to develop the DCTs. It also describes the coating material properties, the DCT development process, and the tests used to evaluate the double-coated RCAs and asphalt mixtures with DCRCAs. Chapter 4 describes the research methodology, which is composed of a characterisation of the materials used in the study, the mixture design procedure, and the testing protocols. Chapter 5 presents and discusses the results collected during the experimental program. The aim of Chapter 5 is to present the results obtained for asphalt mixtures prepared with natural granite aggregates (control mixes) and those made with DCRCAs. The results and discussion in Chapter 5 are presented in three different stages. Stage 1 focuses on the Marshall tests, stage 2 focuses on the ITS, TSR and ITSM tests and, finally, stage 3 focuses on the performance testing. Chapter 6 summarizes the main outcomes of the thesis and gives recommendations for future work in this area of research.

Chapter 1 (Introduction)

- Identify research background and problem statement
- Identify research significance
- Identify research scope and objectives
- Identify thesis structure

Chapter 2 (Literature review)

- Identify characteristics of HMA made with RCAs
- Identify treatments used to upgrade the behaviour of RCA-asphalt mixtures
- Identify research gaps and needs

Chapter 3 (Development of the DCT)

- Identify hypotheses and criteria for the developed double coating techniques (DCTs)
- Identify coating materials properties
- Identify the development process of each DCT
- Identify tests used for the development processes

Chapter 4 (Research methodology)

- Determine the materials' properties
- Determine test protocols
- Determine testing machines

Chapter 5 (Results and discussion)

- Present/analyse characteristics of asphalt mixtures made with DCRCAs
    1. Stage 1: Marshall parameters
    2. Stage 2: ITS, TSR and ITSM tests
    3. Stage 3: Performance testing:
        - Permanent deformation resistance (Wheel tracking and Flow number tests)
        - Fatigue life testing
        - Dynamic modulus testing
- Develop master curves for HMAs produced Granite and DCRCAs
- Identify the level of improvement achieved post-DCTs

Chapter 6 (Conclusions and recommendations)

- Conclusions
- Recommendations for future work

Figure 1.1: Thesis structure

2. Literature review

2.1 General introduction

The addition of RCAs to asphalt mixtures has a great effect on the mechanical properties and performance of the mixture, such as its strength, durability, resistance to cracking, and resistance to deformation (Bhusal & Wen, 2013). Use of RCA can increase the amount of bitumen required to achieve the optimum bitumen content (A. R. Pasandín & Pérez, 2013) and decrease the moisture resistance of HMA compared to those prepared with natural aggregates (Zulkati et al., 2013). Therefore, asphalt mixtures containing RCAs exhibit a higher bitumen content (more expensive) and lower performance than those made with fresh aggregates. Several studies have shown that the main reason for such behaviour was the cement mortar attaches to recycled aggregate particles, and other RCAs' impurities such as bricks and tiles. In light of these findings, some researchers recommend not use RCAs in HMA production (Bhusal & Wen, 2013), while others have used different treatments to improve the engineering properties of RCA-asphalt mixtures. These treatments were based on heating or coating techniques.

To date, none of these treatments has been applied to flexible pavement construction, which implies the need for a new technique to improve the performance of RCA-asphalt mixtures over that achieved previously.

To achieve a better understanding of the performance of HMA containing RCAs, it is essential to review studies in this area of research. This review can clarify why treatment is required to upgrade the engineering properties of RCAs. In this chapter, the use of RCAs as aggregates in unbound and bound flexible pavement layers is discussed. An overview of the major distresses occurring in flexible pavement is made (stripping, fatigue, rutting, and thermal cracking). In addition, a review of the available technical studies on HMA made with RCAs is presented. Treatments previously used to improve RCA-mixture properties are introduced and discussed. The outcomes of this literature review are 1) identification of the current research problem and 2) identification of the need to develop new coating techniques. Figure 2.1 describes the structure of the literature review conducted in this chapter.

2.2 A brief history of using recycled aggregates in construction

After World War II, brick aggregate recycled from damaged and/or destroyed buildings was used in concrete construction (Vieira & Pereira, 2015). However, this was limited to non-structural applications in areas where natural aggregates were unavailable or more costly than recycled aggregates (Weggel, Hilger, Ogunro, & Diemer, 2012).

Although demolition debris has been used in the construction industry since World War II, studies on its properties, performance and durability have only been conducted in recent years (Ebrahim Abu El-Maaty Behiry, 2013). After World War II, the vast quantities of solid waste material produced from destroyed buildings encouraged their use in the construction of new infrastructure. Recently, large quantities of recycled aggregates have been generated from the construction booms of developed countries (Gul & Guler, 2014; Shi & Xu, 2006) and by natural disasters (J. Li, Xiao, & Zhou, 2009). The existence of these wastes makes recycled aggregates the most abundant sloid waste material in the entire world and, therefore, its utilisation has become an interesting topic for researchers in many countries.

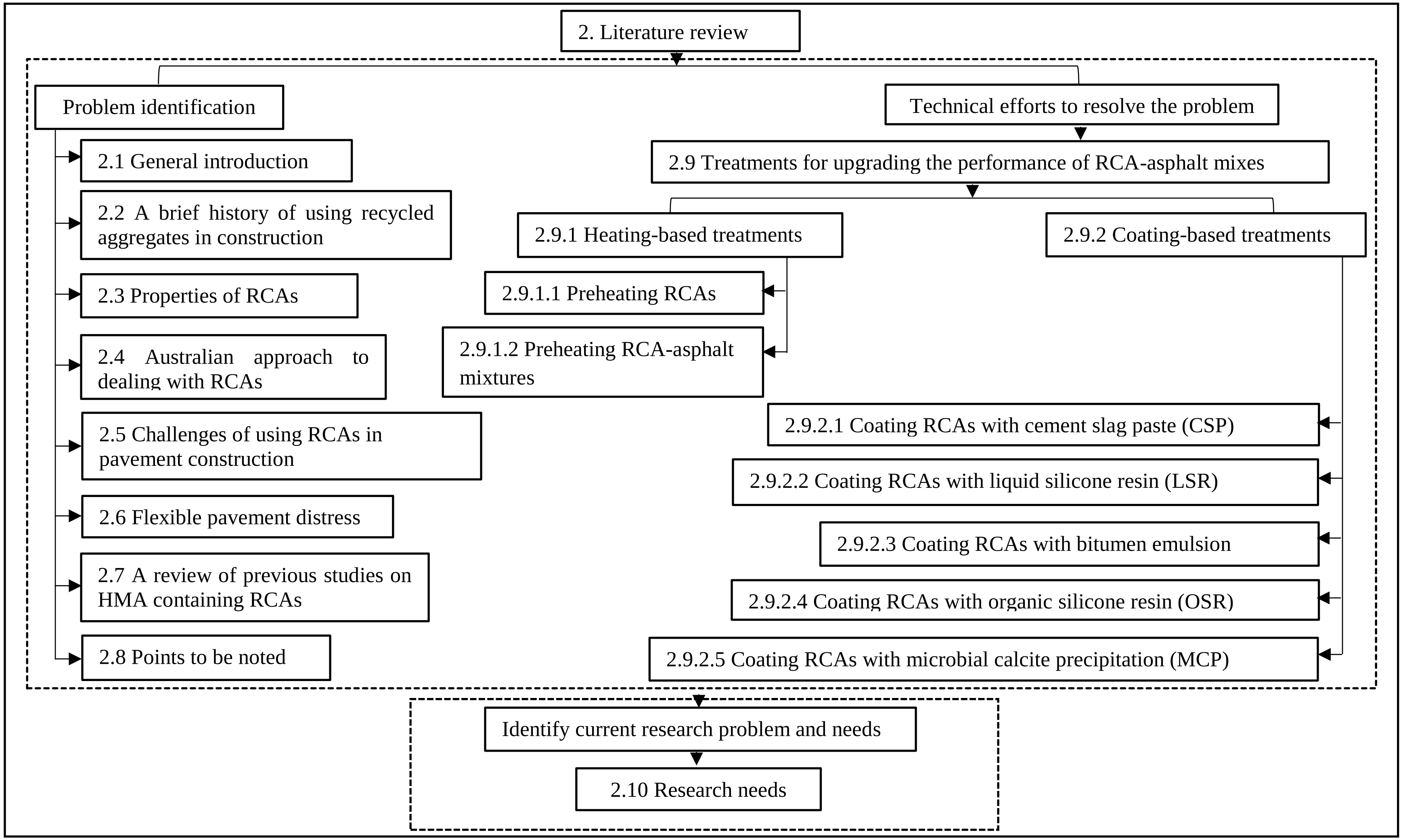


Figure 2.1: Structure of literature review

Recycled aggregate derived from C&D wastes can generate a number of environmental and landfill problems (Calkins, 2008; Faisal & Kumar, 2019; Muniz de Farias, Quiñonez-Sinisterra, & Rondón-Quintana, 2018). When considering their transportation costs, sending these wastes to landfill can impose extra costs on the construction sector (Gul & Guler, 2014; Robinson, Menzie, & Hyun, 2004). The use of recycled aggregates derived from C&D waste is, therefore, considered as a key way to reduce the use of finite natural resources and improve environmental practices (Calkins, 2008; Shi & Xu, 2006; H. Zhang, Ji, Liu, & Su, 2018). The work presented in this thesis aims to promote the use of RCAs derived from C&D wastes in HMA after being upgraded by two different DCTs. This can mitigate the ecological and financial disadvantages of RCAs, ease its flow in the developed communities and make its usage as HMA aggregate more valuable.

2.3 Properties of RCAs

According to Cement Concrete & Aggregates Australia (CCAA), recycled aggregates (RAs) are aggregates derived from the processing of materials previously used in a product and/or in construction (CCAA, 2008). Usually, the RA is contaminated with impurities such as bricks, tiles, recycled asphalt pavement (RAP), wood, metals, ceramics and gypsum plasters. However, when only cement concrete samples or structures (i.e., dams and bridges) are recycled in the production process, a better quality of RA is obtained. This type of RA is typically named *crushed concrete aggregate* (CCA) (Pasandin & Perez, 2015). The most abundant type of RA is the *recycled concrete aggregates* (RCAs) which concrete is the main part of it (Pourtahmasb & Karim, 2014).

Compared to natural aggregates, RCA has lower strength and higher water absorption. In this respect, many researchers have reported that RCAs exhibit lower unit weight

(M. J. Chen & Wong, 2013; de Juan & Gutiérrez, 2009; A. Pasandín & Pérez, 2015; A. R. Pasandín & Pérez, 2014b; A. R. Pasandín, Pérez, Caamaño, Pérez-Barge, & Gómez-Meijide, 2018; J. Zhu et al., 2012) and abrasion resistance (de Juan & Gutiérrez, 2009; Yueqin Hou et al., 2014; A. R. Pasandín & Pérez, 2014b; J. Zhu et al., 2012) compared with natural aggregates. RCAs also have higher rates of water and bitumen absorption than fresh aggregates (Acosta Alvarez, Alonso Aenlle, & Tenza-Abril, 2018; M. J. Chen & Wong, 2013; de Juan & Gutiérrez, 2009; J. Giri, M. Panda, & U. Sahoo, 2018; Yueqin Hou et al., 2014; J. Li et al., 2009; A. R. Pasandín & Pérez, 2014b; Singh, Bisht, Aswathy, Chaurasia, & Gupta, 2018; J. Zhu et al., 2012). These findings are likely related to the attached cement mortar and other impurities present in RCA products. Consequently, the final product (e.g. concrete or asphalt mixture) containing RCAs usually behaves differently from that made with 100% high-quality natural aggregate.

2.4 Australian approaches to dealing with recycled aggregates

In Australia and other parts of the world, the demand for recycled aggregates from non-traditional sources is increasing. This has occurred in conjunction with increasing quantities of C&D waste that can be reused as alternative aggregates in road and concrete construction. In Australia, it is estimated that about 32.4 million tonnes of solid waste were generated in 2002-2003 (Andrews & Rebecchi, 2009), as shown in Figure 2.2. As can be seen in Figure 2.2, a total of 13.6 Mt of C&D waste was produced in 2002-2003. However, this amount increased to 19 million tonnes in 2008-2009 (Hyder consulting, 2011). Of this C&D waste stream, 8.5 million tonnes (45%) was sent to landfill. In 2014-2015, a total of 20 million tonnes of C&D waste was generated in Australia. From this amount, 64% was recovered while 36% went to landfill (Wardle et al., 2016). Although the recovery rate improved in 2014-2015 (64%)

compared with that in 2008-2009 (45%), some 7.1 million tonnes (36%) was still taken directly to landfill.

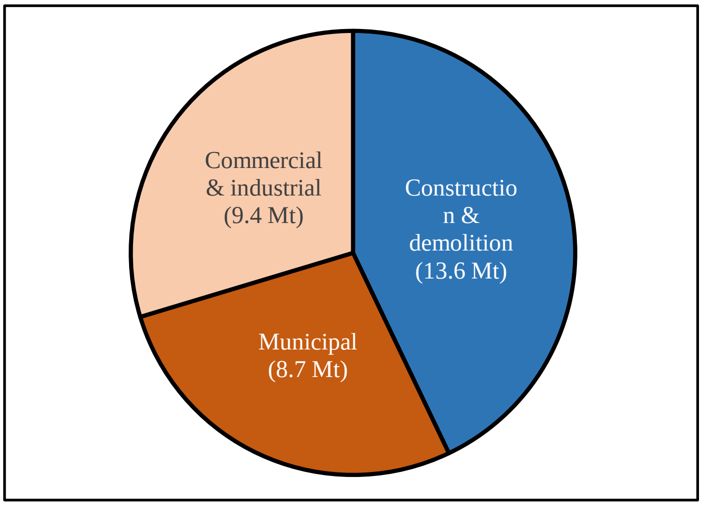


Figure 2.2: Quantities of solid waste produced in Australia (Andrews & Rebecchi, 2009)

In some Australian states such as Western Australia, the state government has committed to landfill levy increases for C&D waste in metropolitan areas from 2015 to 2019. The plan is to increase the landfill levy from about \$8/tonne in 2015 to \$70/tonne in 2019. This commitment requires significant industry investment in order to increase opportunities to reuse these wastes in practical ways. This commitment also encourages the enhancement of recycled aggregate products to improve overall waste recovery performance (Wardle et al., 2016).

Cement Concrete and Aggregates Australia (CCAA, 2008) reviewed the available types of manufactured, recycled and reused by-product aggregates in Australia (Figure 2.3). In the review, commercially available aggregates were described in terms of their sources, properties and manufacturing processes, along with their advantages and restrictions when used in road and/or concrete constructions. According to the CCAA review, RCA has been recognised as practical for use in low-grade concretes. In addition, it was indicated that the essential factors affecting the use of RCA in different applications are its availability and consistency in supply (CCAA, 2008). Sufficient performance data for air-cooled blast furnace slag is available, and continuous efforts are conducted to collect performance data for manufactured sand. However, in pavement applications, only limited data is available based on field trials conducted by road authorities (CCAA, 2008).

Both Australia and New Zealand have developed waste reduction, reuse and recycling strategies to manage materials used in road construction as presented in Figure 2.4 (Andrews & Rebecchi, 2009; Technical Committee 4.3 Road Pavements, 2008). According to these strategies, the least preferred option is to dispose of solid C&D waste in landfill. In order to achieve the objectives of reducing, reusing and recycling, it would be more wise and practical to use these wastes for construction purposes. One of the available options for highway engineers is to reuse the RCAs derived from C&D wastes in HMA production (Qasrawi & Asi, 2016).

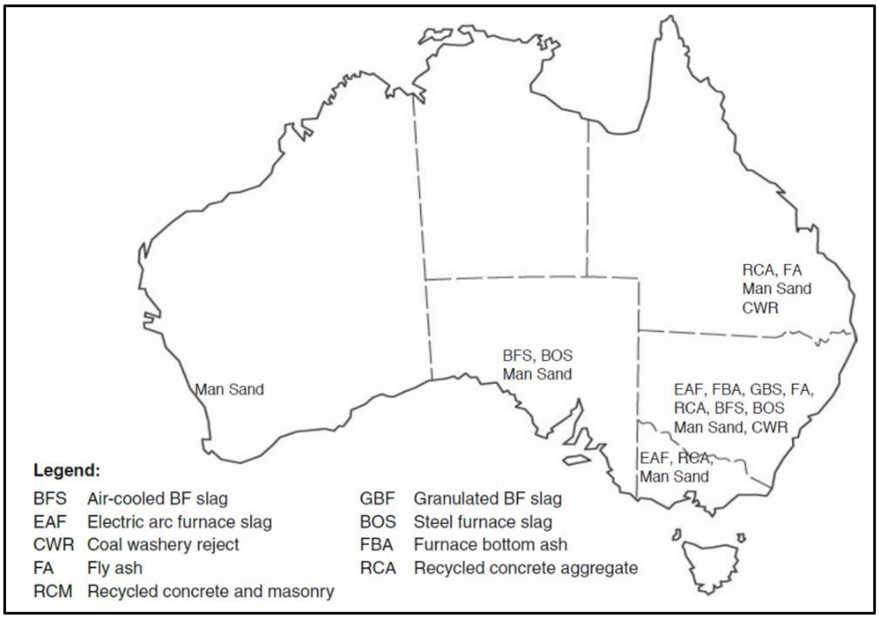


Figure 2.3: Sources of recycled aggregates in Australia (CCAA, 2008). Man Sand stands for Manufactured Sand.

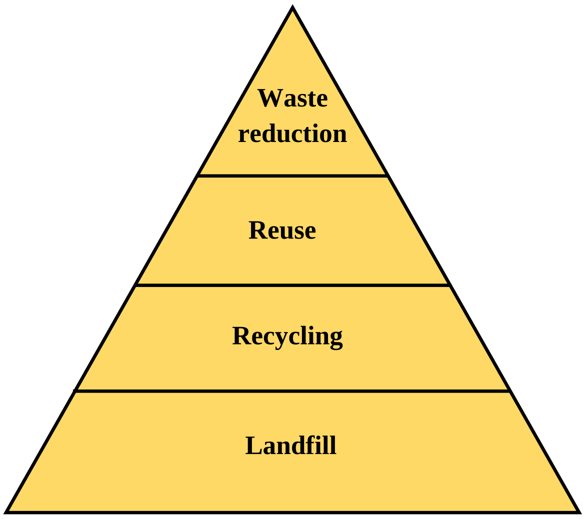


Figure 2.4: Australia and New Zealand waste hierarchy strategies for pavement materials

2.5 Challenges of using RCAs in pavement construction

The use of recycled aggregates derived from C&D waste can reduce the amount of solid waste sent to landfill (Bhusal, Li, & Wen, 2011), preserve natural aggregates for future generations (Shi & Xu, 2006), and promote sustainable development (J. Li et al., 2009). It also can reduce greenhouse emissions and construction costs when considering the costs of transportation and disposal (Jin & Chen, 2015). However, the fact that RCA is different from natural aggregates restricts its usage in the construction of new asphalt infrastructure. In the following sections, the challenges of using RCAs in pavement construction are presented.

2.5.1 Challenges of using RCAs in unbound pavement layers

One possible solution for RCAs derived from C&D waste is to use it as aggregate for unbound pavement layers. Many researchers have investigated the use of recycled aggregates as granular materials for base and subbase-layer construction (A. Arulrajah et al., 2014; Leite et al., 2011; Park, 2003). The results of these investigations indicate that RCAs could be used as aggregates for base and subbase-layer construction in low-volume roads.

Unfortunately, the use of RCAs in unbound layers can increase the pH of groundwater and affect vegetation growing near roads (Gul & Guler, 2014; Robinson et al., 2004). Moreover, fine RCAs can dissolve in water, react with CO_2 in the air, and precipitate out calcium carbonate which, in turn, potentially clogs up the drainable layers (Wong et al., 2007).

In Australia, RCAs are primarily utilised as aggregates for unbound pavement layers (Jitsangiam et al., 2014; Leek et al., 2012; Tam, 2009). Therefore, considering the vast quantities of RCAs generated in Australia and the sustainability strategies of reducing, reusing and recycling that mentioned in Section 2.4, there is a pressing need to investigate the feasibility of using RCAs derived from C&D waste for structural purposes such as by using them as aggregates in HMA.

2.5.2 Challenges in using RCAs in bound pavement layers

The use of RCAs in HMA production has been studied for its economic and environmental advantages (Bhusal et al., 2011; Gul & Guler, 2014; I Pérez, Pasandín, & Gallego, 2012; Pourtahmasb & Karim, 2014; Tahmoorian & Samali, 2018). Moreover, the use of RCAs in asphalt mixtures can mitigate supply shortages of natural aggregate for road construction and reduce the costs of processing and transporting RCAs (Yueqin Hou et al., 2014). Additionally, the environmental and practical issues associated with the use of RCAs in unpaved road constructions do not apply to their use in asphalt mixtures. In contrast to unbound pavement layers, in the presence of asphalt, the use of both fine and coarse RCA particles will be feasible (Zulkati et al., 2013). The pores in RCAs can be sealed with bitumen, which provides the required protection against water damage.

However, due to RCAs' high rate of water absorption, the optimum bitumen content (OBC) of HMA prepared with RCAs increases with the proportion of RCA in the mix (Bhusal & Wen, 2013; Zulkati et al., 2013). Additionally, Zulkati et al. (2013) stated that asphalt mixtures made with RCAs have more potential to be damaged in the presence of water due to their thinner binder film and uncoated broken RCAs. A. R. Pasandín, Pérez, Gómez-Meijide, and Pérez-Barge (2015) mentioned that RCA particles are more difficult to coat with asphalt because of their rough surface texture, which degrades the mixture's resistance to water damage. Thus, asphalt mixes produced with RCAs are expected to be more expensive than those made with fresh natural aggregates, while their resistance to moisture damage is lower.

In the US, Bhusal and Wen (2013) conducted an experimental study to examine the feasibility of using RCAs in HMA. The results indicate that the use of RCAs degrades the performance of asphalt mixtures. The authors concluded that RCAs should not be used in HMA, even when the volumetric properties of the asphalt mixtures satisfy standard requirements. It was also noted that RCAs tend to break under the mechanical loads induced by mixing and compaction (Cho, Yun, Kim, & Choi, 2011; Gómez-Meijide & Pérez, 2016; Leite et al., 2011; Paranavithana & Mohajerani, 2006; Zulkati et al., 2013). This can degrade the stripping resistance of asphalt mixtures. Cho et al. (2011) concluded that the dynamic loading used in the Marshall method could produce unexpected results due to the breakage of RCAs. In order to moderate the effects of RCA breakage on aggregate gradation, Gómez-Meijide and Pérez (2016) suggested selecting the aggregate gradation in a way that maintains it within the limits recommended by the standard after the mixing and compaction processes.

2.6 Flexible pavement distress

When an asphalt mixture is designed and appropriately compacted, it should be able to withstand traffic and environmental conditions for many years. However, asphalt pavements undergo different types of stresses resulting from traffic, moisture, day/night temperature fluctuations, and subgrade movement. These stresses can induce minor defects which can develop into various distresses with time if proper treatment is not applied (Lavin, 2003). The main four distresses in flexible pavement are moisture sensitivity, fatigue cracking, permanent deformation (rutting), and low-temperature cracking. For the purposes of understanding these distresses, a brief description of each type is presented in the next four sub-sections, with a focus on HMAs containing RCAs.

2.6.1 Moisture sensitivity (stripping) of HMA

This type of distress is one of the main problems in asphalt mixtures. It increases the stripping of bitumen film from aggregate particles in the presence of water (Kanitpong & Pummarin, 2010). Moisture damage (stripping) is a complex phenomenon that may happen during any stage of the life of an asphalt mixture. It has been documented that it is difficult to measure HMA stripping in a reproducible way (Kiggundu & Roberts, 1988). Therefore, to evaluate stripping, several simulation tests have been developed to measure HMA resistance to moisture damage. In this regard, the AASHTO T 283 (Modified Lottman) test method is widely used to measure stripping of HMA (Brown, Kandhal, & Zhang, 2001; N. Hunter, Self, & Read, 2015). The Australian version of this testing method is the AG:PT/T232 standard, which is used to evaluate stripping of HMA made in the present study (Austroads, 2007a).

The main factor that can affect bitumen-aggregate adhesion is the type of aggregate. Aggregates with a high content of silica oxide such as quartz and granite are, in general, more difficult to coat with bitumen than basalt and limestone aggregates (N. Hunter et al., 2015). In addition, the presence of dust on aggregate surfaces, the acidity of the water, and the surface texture of aggregates affect the initial and/or final bonding achieved (N. Hunter et al., 2015). Also, properties of the bitumen, asphalt mixture and external factors may affect the aggregate-bitumen bond. Table 2.1 presents the main factors that affect aggregate-bitumen adhesion.

In Western Australia, granite is the most available aggregate and is widely used in HMA production. As mentioned above, granite is a siliceous aggregate and is hard to coat with bitumen. Thus, it is recommended to add 1.5% hydrated lime to asphalt mixtures prepared in the Perth metropolitan area to improve the bonding between aggregate particles and bitumen and increase moisture resistance (MRWA, 2017b).

The addition of RCA to HMA can degrade its resistance to moisture damage (Y Hou, Ji, Li, & Li, 2018). On the one hand, the RCA particles are expected to absorb a high percentage of the added bitumen and, thus, a lesser amount of bitumen will be left to coat the RCA particles. On the other hand, the rough surfaces of RCA particles decrease the ability of bitumen to coat recycled particles compared with that of non-recycled aggregate particles. It is, therefore, essential to reduce the porosity of RCAs and improve their adhesion with bitumen to obtain satisfactory resistance to moisture damage and maintain bitumen content within acceptable limits. This indicates the importance of the surface texture modification of RCAs through a systematic coating technique. The technique should be able to control the absorptive nature of RCAs and improve the interfacial bonding (adhesion) between treated RCAs and bitumen.

Table 2.1: Properties of aggregate, bitumen, asphalt mixtures, and external factors that can affect bitumen-aggregate bonding (N. Hunter et al., 2015)

Aggregate properties	Bitumen properties	Mixture properties	External factors
Mineralogy	Rheology	Void content	Rainfall
Surface texture	Electrical polarity	Permeability	Humidity
Porosity	Constitution	Bitumen content	Water pH
Dust	Surface free energy	Bitumen film thickness	Presence of salts
Durability	-	Filler type	Temperature
Surface area	-	Aggregate grading	Temperature cycling
Surface free energy	-	Type of mixture	Traffic
Absorption	-	-	Design
Moisture content	-	-	Workmanship
Shape	-	-	Drainage
Weathering	-	-	-

2.6.2 Fatigue cracking

Fatigue cracking is one of the main distresses in flexible pavement structural deterioration (Q. Li, Lee, & Kim, 2012; Nunn, 1998; A. Pasandín & Pérez, 2017). This distress is mainly caused by cyclic loading induced by traffic, which can lead to noticeable reductions in flexible pavement serviceability (Nejad, Azarhoosh, & Hamedi, 2013; Shu, Huang, & Vukosavljevic, 2008). Fatigue cracking, sometimes called *alligator cracking*, is influenced by binder characteristics, compaction level, and aggregate characteristics and grading. It also affected by layer characteristics such as layer thickness, stress/strain levels, load repetition period, layer stiffness, and temperature (Alderson, 2008). The higher the fatigue cracking resistance of the mixture, the better its capability to withstand repeated traffic loading without significant loss of pavement serviceability.

When a vehicle passes over a pavement layer (Figure 2.5), it creates stress and induces strain in the pavement. The strain is either low or high. In the case of low strain and an elastic material, the strain can be considered as ‘recoverable’. In the case of high (permanent) strain and a viscous material, a molecular reorientation occurs. At low temperatures, the molecular mobility of viscoelastic materials such as bituminous mixtures is restricted. Under such conditions, wheel load-induced stresses will produce strains that produce molecular scission at the bonding level until damage (cracks) appear in the pavement. The area of damaged material cannot support vehicle stresses, which creates higher stresses in that zone. Thus, a crack will grow as the number of wheel passes (load cycles) increases (Moreno-Navarro & Rubio-Gámez, 2016).

In light of this, to control this mode of distress, the aim of the pavement designer is to maintain the tensile strain at the bottom of the asphalt layer so that it is recoverable. The main characteristic that can make an asphalt layer resist fatigue cracking is to design the mixture with the maximum percentage of bitumen permitted without affecting its resistance to rutting (Brown et al., 2001). Furthermore, the use of a finer grade of aggregate also improves the resistance to fatigue cracking (Newcomb, Buncher, & Huddleston, 2001). In addition, keeping the filler-to-bitumen percentage low, not overheating the bitumen during preparation, constructing an HMA with a low air void content, and selecting a suitable type of bitumen are all factors that can ensure better resistance to fatigue cracking distress (Brown et al., 2001).

Different laboratory protocols have been introduced to evaluate the resistance of hot-mix asphalt (HMA) to fatigue cracking; for example, centre point flexural tests, two-point bending tests, four-point loading tests (also known as third-point loading), trapezoidal beam tests, and direct axial loading tests (tension or compression). In Australia, the test method adopted is the four-point loading test (Austroads, 2006b).

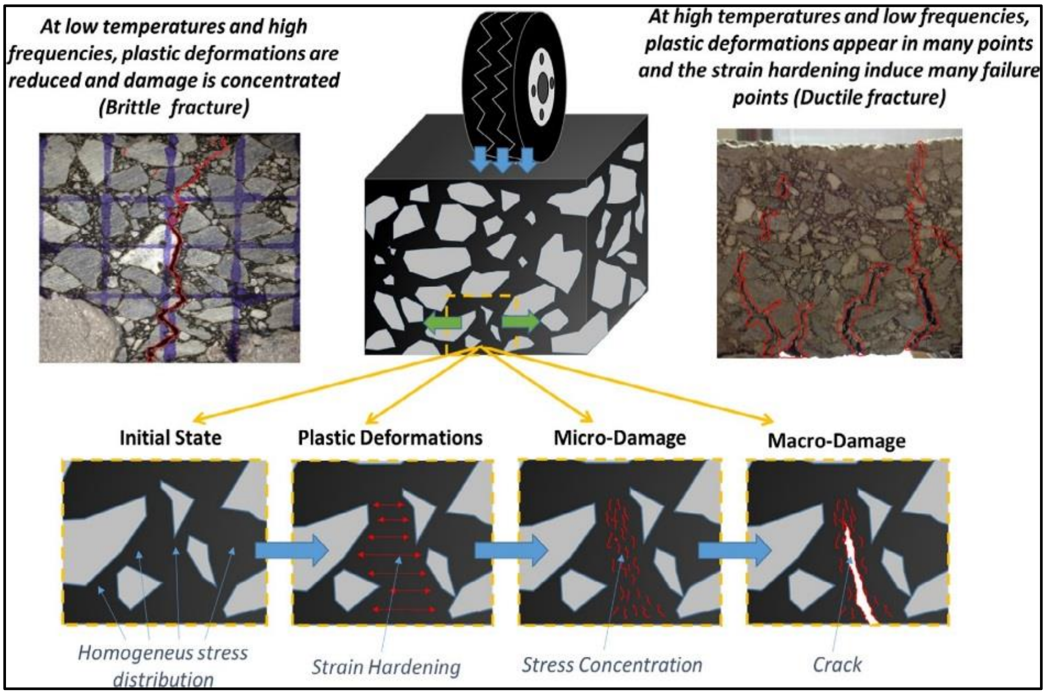


Figure 2.5: Stresses transmitted by wheel loads over asphalt structures (Moreno-Navarro & Rubio-Gámez, 2016)

The test can be run in two different modes: *controlled stress mode* (a fixed load amplitude is applied on the specimen and the resulting deflection is measured) and *controlled strain mode* (a fixed deflection amplitude is used on the beam and the load produced is monitored); (Austroads, 2006b). The controlled stress test is more applicable for thick pavement layers (> 75 mm) while the controlled strain test is more suitable for thin pavement layers (< 75 mm). In Australia, asphalt layer thicknesses are usually less than 75 mm. Thus, the controlled strain mode test should be used to evaluate the fatigue life of the majority of HMA made in Australia (Austroads, 2006b).

In the area of RCA-asphalt mixtures, past research has not evaluated the fatigue life of Australian HMA made with RCAs (Paranavithana & Mohajerani, 2006). Therefore, no such data is available. In the present study, the fatigue performance of asphalt mixtures produced with different percentages of DCRCAs will be evaluated. This is important for comparative purposes and to measure the degree of improvement achieved by using the DCTs.

2.6.3 Permanent deformation (rutting)

Rutting or permanent deformation is considered to be one of the basic failure modes of flexible pavement (Radziszewski, 2007). It is generated gradually as the unrecoverable strain induced by traffic loading accumulates with time. This form of distress is mainly associated with high ambient temperatures and slow-moving loads. Permanent deformation (rutting) can increase driving hazards and discomfort (Miljković & Radenberg, 2011). According to Brown et al. (2001) and Miljković and Radenberg (2011), rutting consists of two different mechanisms. The first mechanism is *densification* (due to asphalt layer compaction, i.e., decrease in volume and increase in density). This mechanism is followed by a shear deformation/lateral movement mechanism with no volume change. The second rutting mechanism can occur when unsatisfactory materials are used in the HMA layer or any other pavement layers. A characterisation of rutting is given in Figure 2.6.

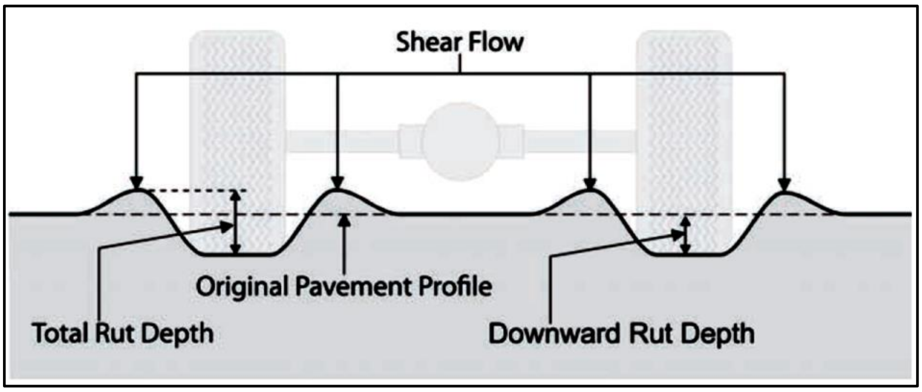


Figure 2.6: Characterisation of rutting in an asphalt structure (Kandhal & Cooley Jr, 2003)

Rutting is characterised in the laboratory using various tests, such as dynamic creep tests and wheel tracking tests. The latter is adopted in Australia for rutting evaluation. Aggregate properties such as surface texture and gradation, bitumen properties such as stiffness, and mixture properties such as bitumen, void contents and method of compaction, are all factors affecting the resistance of HMA to permanent deformation (Table 2.2).

Table 2.2: Factors influencing asphalt mixture rutting resistance (Miljković & Radenberg, 2011)

Mix component	Factor	Change in factor	Effect on rutting resistance
Aggregate	Surface texture	Smooth to rough	Increase
	Gradation	Gap to continuous	Increase
	Shape	Rounded to angular	Increase
	Size	Increase in maximum size	Increase
Bitumen	Stiffness	Increase	Increase
Mixture	Bitumen content	Increase	Decrease
	Air void content	Increase	Decrease
	VMA	Increase	Decrease
	Method of compaction	*	*
*The method of compaction may affect the structure of the mixture and therefore the tendency for permanent deformation.			

Asphalt mixtures produced with RCAs are expected to exhibit higher rutting resistance than those made with fresh aggregates. This is because the rough, porous surfaces of RCAs increase friction among mixture particles, thus generating greater resistance to aggregate dislocation forces when traffic loads are applied. However, this is only a general expectation. The tendency of RCAs to break under mechanical loading, however, could affect the rutting potential of asphalt mixtures containing RCAs.

In a previous study conducted in Australia, it was found that both control and RCA-asphalt mixtures exhibited comparable resistance to rutting. However, these results were based on dynamic creep tests, which are no longer recommended for rutting evaluation in Australia (Alderson, 2008). According to Alderson (2008), wheel tracking test was selected as the most suitable testing protocol for rutting evaluation in Australia. Consequently, it is crucial to carry out the wheel tracking test to evaluate the rutting behaviour of Australian HMA made with RCAs.

2.6.4 Thermal cracking

This type of flexible pavement distress is caused by the temperature dropping to a critical level, which produces extreme contraction and, subsequently, cracks (Hao Yin, Ghassan R. Chehab, Shelley M. Stoffels, Tanmay Kumar, & Premkumar, 2010). As the asphalt layer is continuous, low-temperature transverse cracking will be created because of the high thermal strains and stresses induced in the asphalt pavement layer. This distress is generated due to two mechanisms: *single-event low-temperature cracking* and *thermal fatigue cracking*. In extremely cold weather conditions, thermal cracking distress may generate through the whole depth of the asphalt layer after only one or a few cooling cycles (single-event mechanism). In the case of fatigue cracking, thermal cracking is created at a slower rate and the full-depth crack only occurs after many cooling cycles (Alavi, Hajj, & Sebaaly, 2017). Controlling this distress is important in mitigating the substantial damage that may be produced when water forms a pathway into the pavement structure (Dave & Hoplin, 2015; Marasteanu et al., 2007).

The mechanism of thermal cracking is illustrated in Figure 2.7. It can be seen that the thermal stresses change with depth as the temperature changes from top to bottom. Thus, these stresses are not uniform and can cause cracks to develop and propagate at the surface and then spread downwards (N. Hunter et al., 2015).

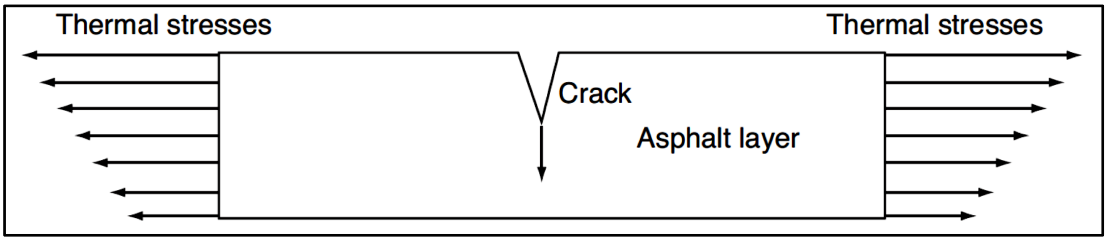


Figure 2.7: Thermal cracking mechanism (N. Hunter, Self, & Read, 2015)

The mechanical properties of asphalt mixtures and the magnitude and cycles of low-temperature weather are key factors in the formation of low-temperature cracking

(Dave & Hoplin, 2015). Brown et al. (2001) mentioned that the stiffness of the mix has a great effect on the occurrence and intensity of thermal cracking. Also, the layer thickness and environmental factors can influence the resistance of the asphalt layer to low-temperature cracking. The properties of asphalt mixtures (i.e., type and amount of bitumen, aggregate type and gradation, volumetric properties) greatly affect the thermal cracking of asphalt pavements (Alavi et al., 2017). To minimise thermal cracking, it is important not to overheat the bitumen during HMA production, and to compact the pavement layer to the lower allowable void content in order to minimise oxidation (Brown et al., 2001).

In Australia, thermal cracking is not incorporated into pavement design, based on the hypothesis that there are no specific regions of low temperature (Wibisono, 2014). Keeping with this, the Austroads technical report (AP-T100/08) entitled *Testing Asphalt in Accordance with the Austroads Mix Design Procedures* (Alderson, 2008) did not include any testing method to evaluate this type of asphalt pavement distress. Therefore, thermal cracking investigation is beyond the scope of the present thesis.

2.7 A review of previous studies on HMA containing RCAs

In this section, the performance of asphalt mixtures containing RCAs is reviewed. Different tests have been carried out to investigate the effects of RCAs on the properties of hot-mix asphalt. The findings of past technical efforts using Marshall tests, TSR tests, wheel tracking tests, dynamic creep tests, flow number tests, fatigue life tests, resilient modulus tests, and dynamic modulus tests are presented in the following sub-sections.

2.7.1 Marshall parameters

The Marshall procedure is one of the most popular design methods for road asphalt mixtures. The goal of the Marshall design method is to determine the optimum bitumen content (OBC) by considering the strength, flow, volumetric and density properties of asphalt mixtures. According to Australian practices, the Marshall method is applicable to HMA containing aggregates not exceeding a 20 mm nominal size (Australian/New Zealand Standards, 2015). It should be noted that the Marshall method is an empirical procedure, which cannot describe the stress-strain characteristics of hot-mix asphalt.

The majority of the researchers have used this procedure to design HMA containing RCAs (Cho et al., 2011; Lee et al., 2012; A. R. Pasandín & Pérez, 2013; I. Pérez, Pasandín, & Medina, 2012; Wong et al., 2007; J. Zhu et al., 2012) (M. J. Chen & Wong, 2013; A. R. Pasandín & Pérez, 2014b; Pourtahmasb & Karim, 2014; Rafi, Qadir, Ali, & Siddiqui, 2014; Z. Zhang et al., 2016). However, some researchers have used the Superpave design method to compute the volumetrics of HMA made with RCAs (Bhusal et al., 2011; Mills-Beale & You, 2010; Zulkati et al., 2013).

The design process is mainly aimed at determining the optimum bitumen content (OBC) of asphalt mixes and other parameters such as density and volumetric properties. It should be mentioned that both Marshall and Superpave parameters are not enough to judge the behaviour of an asphalt mixture, especially those RCA-containing RCAs. Therefore, performance tests are essential to ensure the durability and behaviour of RCA-asphalt mixes (Bhusal & Wen, 2013).

2.7.1.1 Optimum bitumen content (OBC)

It is essential to design an HMA at its OBC to achieve the best strength, durability and performance. In the case of HMA made with RCAs, the amount of absorbed bitumen can significantly affect the cost of the prepared mix. In this regard, many studies show that inclusion of RCAs in HMA results in an increased amount of bitumen being required to achieve the OBC level (Arabani, Moghadas Nejad, & Azarhoosh, 2013; I

Pérez & Pasandín, 2017). The OBC results for RCA mixes reported by several studies are graphically presented in Figure 2.8.

It can be seen that there is an increase in the percentage of bitumen required to achieve the OBC as the dosage of RCA increases. This is mainly due to the higher porosity and surface area of recycled aggregates compared to natural aggregates (Radević, Đureković, Zakić, & Mladenović, 2017). In addition, Pourtahmasb and Karim (2014) and Radević et al. (2017) studied the effect of including fine and coarse RCAs on HMA. They concluded that mixes made with fine RCAs require more bitumen to achieve the OBC than those produced with coarse RCAs, as shown in Figure 2.9.

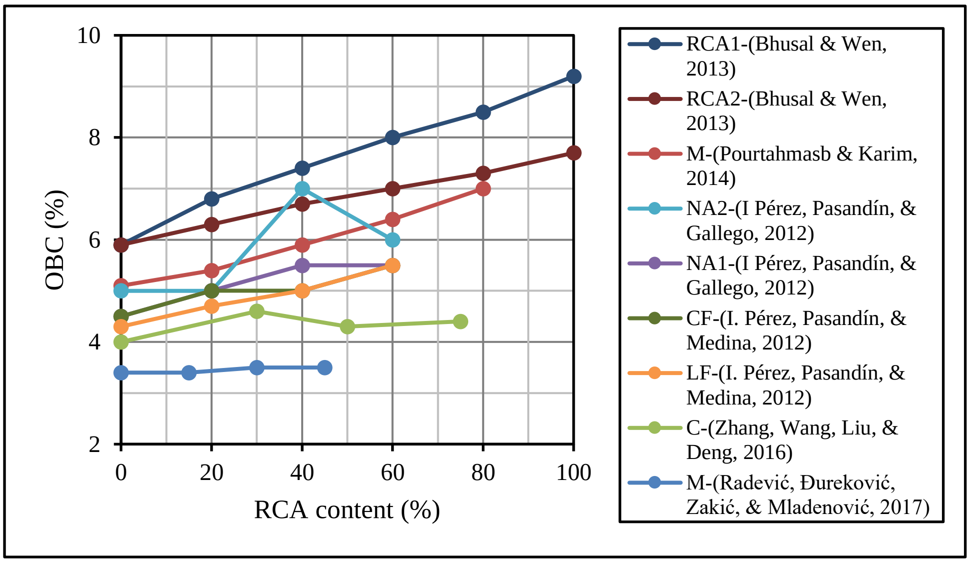


Figure 2.8: The effect of RCA percentage on the OBC (%).NA1 = natural aggregate type 1, NA2 = natural aggregate type 2, C = coarse RCA, M = fine and coarse (mixed) RCAs, CF = cement filler, and LF = lime filler.

These results can be explained by the lower density and higher water absorption of fine RCAs compared to those of coarse RCAs (Radević et al., 2017). As a result, Bhusal et al. (2011) recommended using only coarse RCAs in HMA in order to control the cost of the final mix. Keeping with this, I. Pérez et al. (2012) investigated the effect of coarse RCAs on the stripping of asphalt mixtures. They prepared asphalt mixtures with two types of fillers: cement and hydrated lime. The results from these investigations reveal that the type of filler can affect the OBC of HMA made with RCAs.

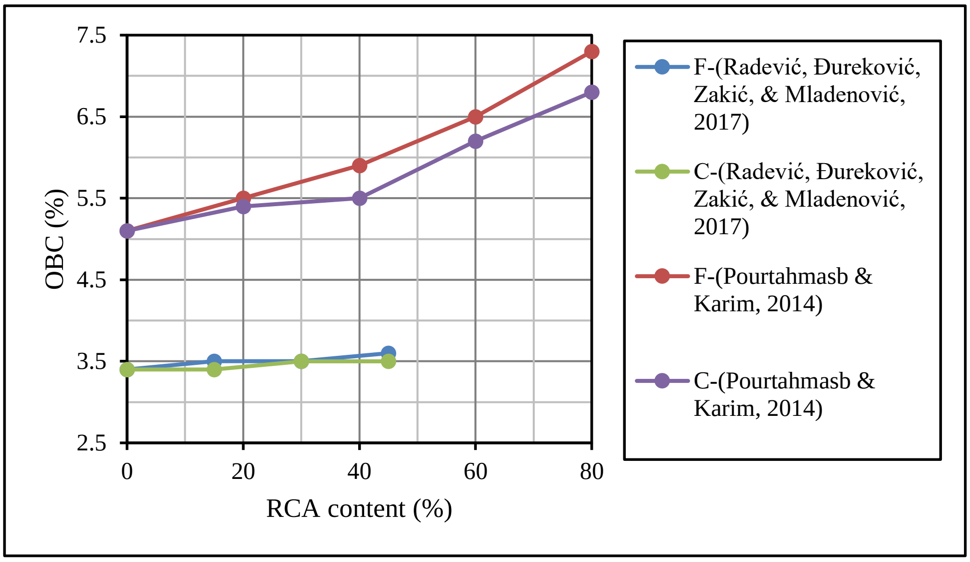


Figure 2.9: The effect of RCA gradation on the OBC. F = fine RCAs, C = coarse RCAs

To mitigate the absorptive nature of RCAs, coating is recommended to seal the pores and cracks in their surfaces (Mills-Beale & You, 2010). J. Zhu et al. (2012) coated RCAs with a liquid silicone resin (LSR) to reduce bitumen absorption. Furthermore, Yueqin Hou et al. (2014) used organic silicone resin (OSR) to activate the RCA to reduce its permeability (water absorption). The results of J. Zhu et al. (2012) showed that, after treatment, RCA particles still absorbed more bitumen than natural aggregate particles to achieve the OBC. Furthermore, the asphalt mixture made with 100% RCA coated with LSR required almost twice the amount of bitumen (OBC = 6.98%) than that of a control mix (OBC = 3.57%) to achieve the OBC. However, the OSR treatment used by Yueqin Hou et al. (2014) reduced the OBC of HMA made with treated RCAs compared to that made with untreated RCAs. Further discussion regarding RCA treatments and findings are introduced in section 2.9.

2.7.1.2 Stability and flow

According to several studies, application of RCA increases the stability of asphalt mixtures (I Pérez et al., 2012; Zulkati et al., 2013), as shown in Figure 2.10. However, this is true for a specific increment in the RCA based on other studies. According to Radević et al. (2017), the use of fine RCA, coarse RCA, and a mixture of fine and coarse RCA produced an increase in Marshall stability up to 30% replacement of natural aggregates. Beyond this percentage, the stability starts to decrease. Additionally, in some cases, the stability either follows a non-definite trend (Z. Zhang et al., 2016) or decreases as the RCA dosage increases (I. Pérez et al., 2012). The heterogeneity of the RCAs, type of natural aggregates, quality of RCAs, types and amounts of impurities, and type of mineral filler are factors that might contribute to the variety of reported results. It should be mentioned that all the Marshall stability results reviewed in this section were higher than the 8 kN required for asphalt mix type AC14 according to Australian standards (Standards Australia, 2005). This means that, in most cases, strength compliance is achieved by these asphalt mixtures and they can withstand traffic loadings in Australia.

Figure 2.11 illustrates the flow results reported in several studies for asphalt mixtures made with RCAs. The flow values should range from 2 to 4 mm based on Australia standards (Standards Australia, 2005). Generally, the flow value increases as the RCA dosage increases in the mix. This trend may be related to the higher OBC of RCA-asphalt mixes, which increases with the RCA content. There was only one exception, where the RCA mixes tended to exhibit lower flow values than control mixes in Zulkati et al. (2013) study. This is might be because Zulkati et al. (2013) designed both control and RCA mixes at the same OBC (5%). A. R. Pasandín and Pérez (2013) concluded that the higher flow values exhibited by RCA-asphalt mixes indicate that increases in RCA content will produce asphalt mixtures more susceptible to rutting.

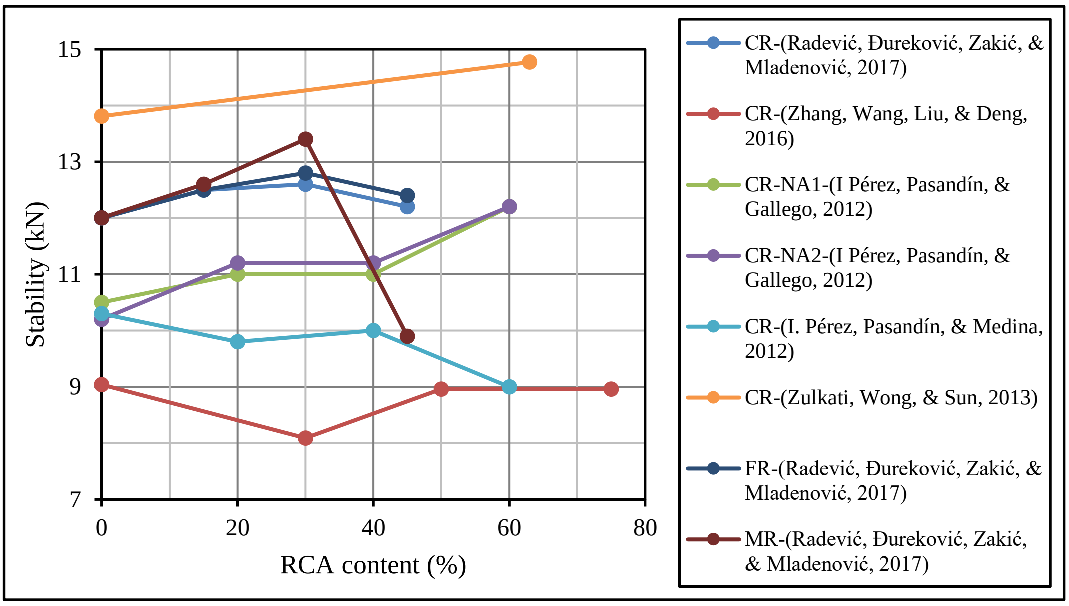


Figure 2.10: Effect of RCA content on Marshall stability. CR = coarse RCA, FR = fine RCA, MR =mixed (fine and coarse) RCA

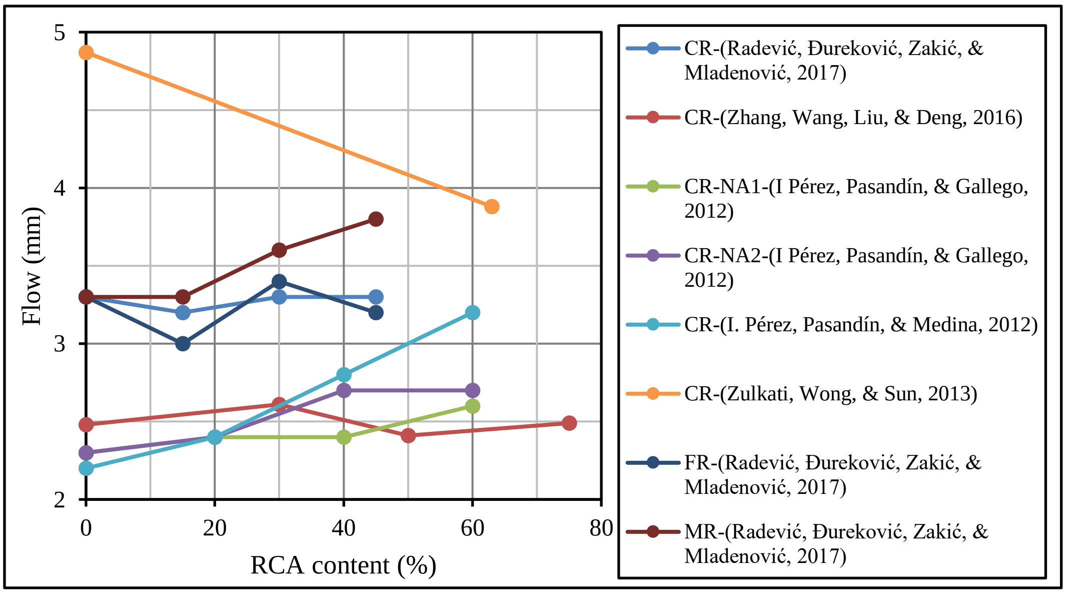


Figure 2.11: Effect of RCA content on the flow of HMA. CR = coarse RCA, FR = fine RCA, MR = Mixed (fine and coarse) RCA

2.7.1.3 Volumetric properties

Many roads authorities specify limits for air voids or the voids in the total mix (VTM), voids in mineral aggregates (VMA), and voids filled with bitumen (VFB) to control the quality of asphalt mixtures. According to technical studies conducted in the area of RCA-asphalt mixtures, the addition of RCA can change the volumetric properties of asphalt mixtures. A volumetric analysis was, therefore, carried out to evaluate the VTM, VMA, and VFB of RCA-asphalt mixtures. The VTM, VMA, and VFB results obtained by different studies for asphalt mixtures made with RCAs are graphically shown in Figure 2.12, Figure 2.13, and Figure 2.14 respectively.

As can be seen in Figure 2.12, in general, the addition of RCAs produces an increase in the air void contents. The higher porosity of RCAs compared to natural aggregates seems to directly affect the VTM values, even though the OBCs of these mixes were higher than those of control mixes (I Pérez et al., 2012).

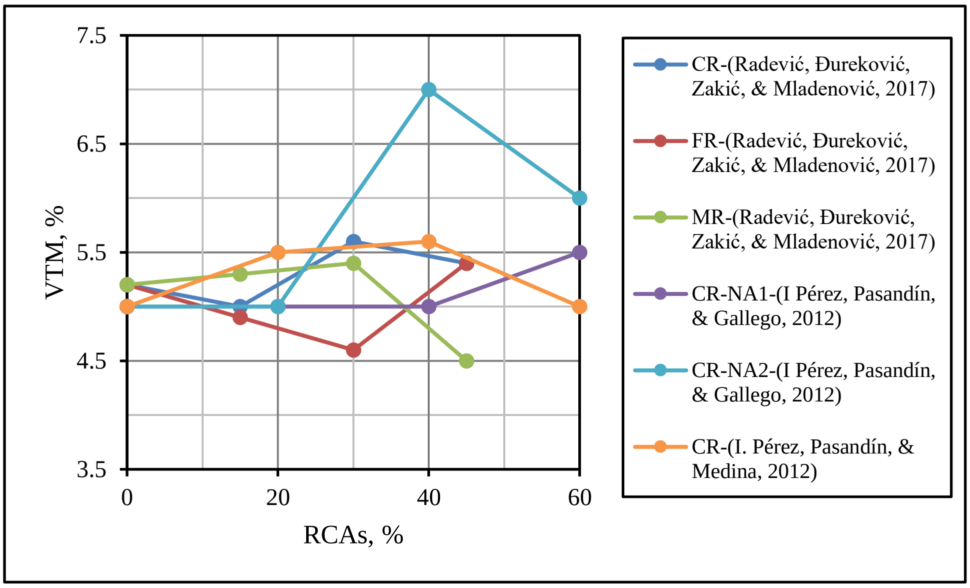


Figure 2.12: Reported relationships between VTM and RCA content. CR = coarse RCA, FR = fine RCA, MR = fine and coarse (mixed) RCAs

The inclusion of the RCAs in HMA production can also affect the VMA and VFB values. The VMA and VFB results collected from several studies are variable (see Figure 2.13, and Figure 2.14, respectively). In some cases, the increase in RCA content produces decreases in VFB and VMA (Pasandin & Perez, 2015). Zulkati et al. (2013) attributed this to the dense aggregate structure of HMAs containing RCAs. The dense aggregate structure in RCA mixes is due to the breakage of RCA after being subjected to mechanical loading during mixing and compaction. In contrast, in studies conducted by I Pérez et al. (2012) and Z. Zhang et al. (2016), the addition of RCAs into asphalt mixtures produced an increase in the VMA.

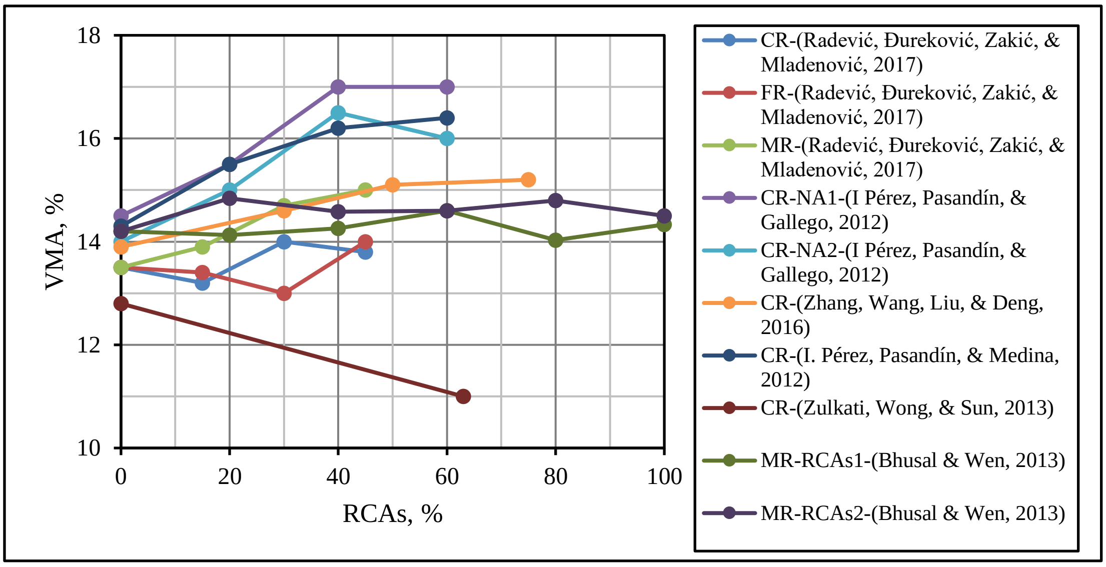


Figure 2.13: VMA versus RCA percentage in several studies. Where CR = coarse RCA, FR = fine RCAs, and MR = fine and coarse (mixed) RCAs

Furthermore, in some studies, the values of VFB and/or VMA fluctuated as the RCA dosage increased in the mix. Such findings are evident in the VMA results obtained by Bhusal and Wen (2013) and Radević et al. (2017). The results obtained in Radević et al. (2017) study also indicate that the RCA gradation can change the values of VFB and VMA. The variation in volumetric properties between different studies might be understood in the light of RCA's inhomogeneity and also the use of different types of fillers, natural aggregates, gradations and design procedures.

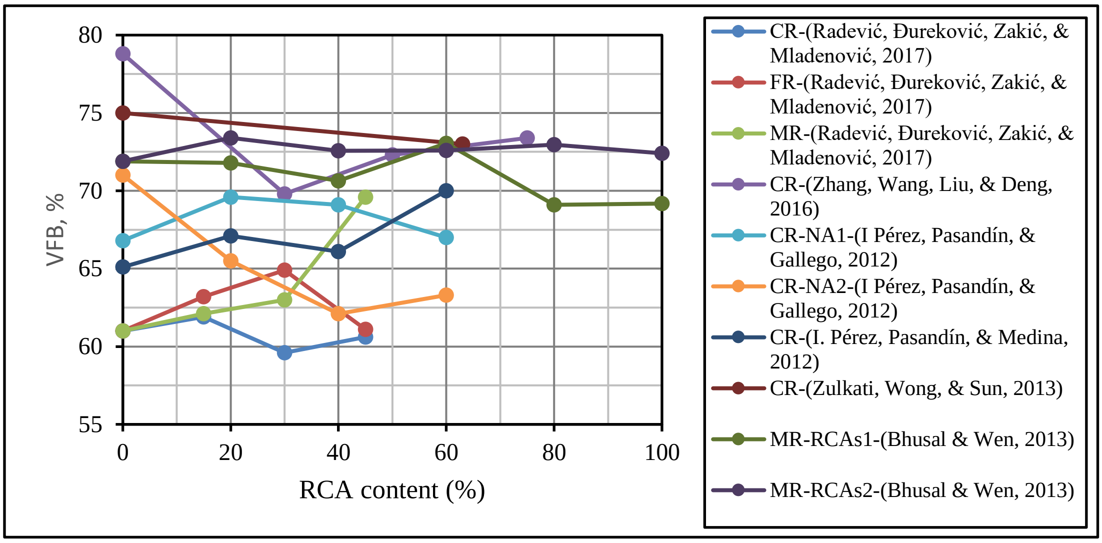


Figure 2.14: Relationships between VFB and RCA according to several studies. CR = coarse RCA, FR = fine RCA, MR = fine and coarse (mixed) RCAs

2.7.1.4 Bulk density

Figure 2.15 illustrates the relationship between the bulk density of hot-mix asphalt made with RCAs and the RCA content according to several studies. It is evident that the utilization of RCA in HMA gives it a lower density than HMA containing regular aggregate. I Pérez et al. (2012) stated that the addition of RCA is the main reason for this trend. In addition, according to Radević et al. (2017), the use of lower density aggregates (RCAs) and the higher OBC of RCA mixes lead to this result.

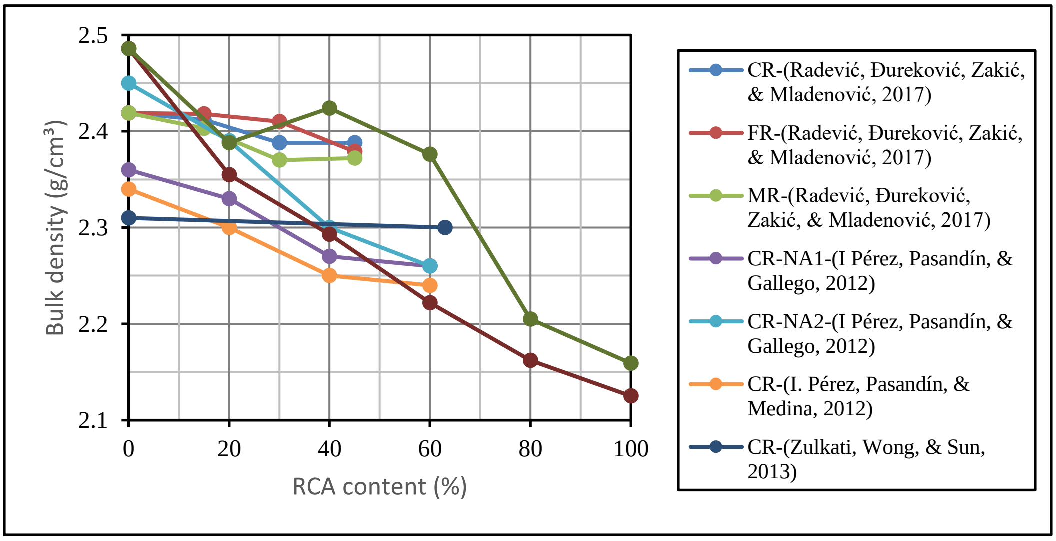


Figure 2.15: Relationship between bulk density and RCA content. CR = coarse RCA, FR = fine RCA, MR = fine and coarse (mixed) RCAs

2.7.2 Resistance to moisture damage

The resistance of asphalt mixtures to moisture damage can be a very complicated phenomenon, particularly in mixtures made with RCAs. The resistance to moisture damage can give an indication of the strength of the bond between the bitumen and the RCA. Strong bonding will result in better water resistance in the field. Absorption nature of RCAs in hot-mix asphalt expects to leave a lesser amount of bitumen to coat the RCA, which leads to a thinner bitumen film on RCAs' surfaces. Consequently, there will be a large chance of RCA-asphalt mixtures being damaged by water. For this reason, investigation of the moisture sensitivity of RCA-asphalt mixtures is crucial.

To evaluate the moisture sensitivity of HMA, different laboratory tests were used. Rolling bottle tests and boiling water tests are performed to examine bitumen-aggregate adhesion in loose asphalt mixtures. Moreover, tensile strength ratio tests, retained Marshall stability tests, and immersion-compression tests are used to evaluate the moisture sensitivity of compacted asphalt mixtures. The following sections present an overview of the reported moisture damage sensitivity of asphalt mixtures made with RCAs.

2.7.2.1 Moisture sensitivity of compacted asphalt mixtures

The presence of RCAs in HMA is one of the factors that can influence its resistance to moisture damage. Therefore, in many studies, this property was evaluated to ensure that reasonable resistance to moisture damage was achieved. Table 2.3 summarises the findings of several studies on the moisture damage resistance of RCA-asphalt mixes.

As seen in Table 2.3, the results of several studies indicate that RCA-asphalt mixtures have a lower resistance to moisture damage than mixes made with high-quality aggregates (Bhusal & Wen, 2013; Mills-Beale & You, 2010; I. Pérez et al., 2012; Shenghua Wu, Muhunthan, & Wen, 2017; Z. Zhang et al., 2016). Surprisingly, only one study found that increasing the content of recycled aggregate derived from C&D waste produced an increase in the TSR (Fatemi & Imaninasab, 2016).

Based on Mills-Beale and You (2010) and Bhusal and Wen (2013), increasing the RCA content decreases the moisture resistance of the mix produced. In addition, based on I. Pérez et al. (2012), the type of filler can affect the resistance of RCA-asphalt mixtures to moisture damage. According to Shaopeng Wu, Zhong, Zhu, and Wang (2013), the RCA gradation can affect the stripping resistance of HMA. They found that an asphalt mixture containing fine RCAs had higher resistance to moisture damage than the control mix. However, Cho et al. (2011) obtained different results. They reported that mixtures made with fine or coarse RCAs both had comparable resistance to stripping. In addition, Shenghua Wu et al. (2017) mentioned that the higher rate of water absorption of RCAs leads to a thinner bitumen film around the RCA particles and, thus, a lower resistance to stripping is expected. Higher water absorption and poor adhesion with bitumen are the main reasons identified by Z. Zhang et al. (2016) contributing to the lower water stability of RCA-asphalt mixtures.

To improve the resistance of RCA mixes to moisture damage, Bhusal and Wen (2013) used various amounts of an anti-stripping agent. I. Pérez et al. (2012) also suggested using an anti-stripping agent to enhance the moisture resistance of HMA made with RCAs.

Table 2.3: Studies conducted to investigate the moisture resistance of compacted HMA made with RCAs

Authors/country	Standard	RCA (%)	Test	Result	Limits	Comments
(Bhusal & Wen, 2013)/US	WSDOT T178	0% (RCA1)	TSR	88%	≥80%	Increasing the RCA dosage in the mix decreased its moisture resistance. Adding an anti-stripping agent to a mix prepared with RCA can mitigate this issue.
		20% (RCA1)		87%		
		40% (RCA1)		82%		
		60% (RCA1)		80%		
		80% (RCA1)		77%		
		100% (RCA1)		76%		
		0% (RCA2)		88%		
		20% (RCA2)		84%		
		40% (RCA2)		82%		
		60% (RCA2)		81%		
		80% (RCA2)		81%		
		100% (RCA2)		80%		
(Cho et al., 2011)/Korea	ASTM D4867 and KS F 2398	0%	TSR	>70%	≥70%	Even though all tested mixtures comply with the Korean standard for TSR, Mixes 2 and 3 showed comparable resistance to moisture damage.
		100% (RCA < 4.75 mm)		>70%		
		100% (RCA > 4.75 mm)		>70%		
		100% (RCA)		>70%		
(Fatemi & Imaninasab, 2016)/Islamic Republic of Iran	AASHTO T-283	0% (C&D waste)	TSR	>80%	≥80%	Surprisingly, as the C&D waste increased, the TSR increased despite the tendency of these wastes to absorb more water than natural aggregates. The authors stated that this was due to a rough surface texture, which allowed the C&D waste to maintain sufficient adhesion with the bitumen binder.
		10% (C&D waste)		>80%		
		20% (C&D waste)		>80%		
		30% (C&D waste)		>80%		
		40% (C&D waste)		>80%		
(Mills-Beale & You, 2010)/USA	ASTM D 4867	0%	TSR	>80%	≥80%	The authors concluded that increasing the RCA content increased the moisture damage produced.
		25%		>80%		
		35%		>80%		
		50%		>80%		
		75%		<80%		

Table 2.3 Continued

Authors/country	Standard	RCAs %	Test	Result	Limits	Comments
(I. Pérez et al., 2012)/Spain	NLT-162 standard	0% (cement filler)	Retained Strength Ratio	85%	≥75%	Asphalt mixtures prepared with RCA had insufficient resistance to moisture damage. Only asphalt mixtures with 20% RCA and lime filler achieved the Spanish specification threshold of 75%. The use of anti-stripping agent was recommended.
		20% (cement filler)		59%		
		40% (cement filler)		54%		
		60% (cement filler)		63%		
		0% (lime filler)		69%		
		20% (lime filler)		79%		
		40% (lime filler)		51%		
		60% (lime filler)		53%		
(Shaopeng Wu et al., 2013)/China	Chinese standard (RIOH)	0%	Residual Marshall Stability	>95%	>80%	The water stability of a control mix was better than mixtures made with fine and coarse RCAs. RCA absorption led to decreased bitumen film thickness around aggregate particles and lower moisture resistance. Also, the water resistance of mixes prepared with fine RCAs was better than that of mixtures made with coarse RCAs.
		100% (RCA > 4.75 mm)		>90%		
		100% (RCA < 4.75 mm)		>95%		
		0%	TSR	>85%	>80%	
		100% (RCA > 4.75 mm)		>80%		
		100% (RCA < 4.75 mm)		>84%		
(Z. Zhang et al., 2016)/China	JTG E20-2011	0%	Residual Marshall Stability	86%	≥75%	The water stability of RCA mixes was significantly lower than that of a control mix. Poor asphalt-RCA adhesion and higher water absorption were the main reasons for this result.
		30%		80.70%		
		50%		78.90%		
		75%		82.10%		
		0%	TSR	68%	≥75%	
		30%		62.90%		
		50%		61%		
		75%		64.30%		

2.7.2.2 Moisture sensitivity of loose asphalt mixtures

Tests conducted on loose asphalt mixtures to assess stripping are considered to be more economical and much easier to perform. However, these tests did not include any consideration of the loading stresses induced by traffic or pore water pressure when a significant change in ambient temperature occurs. Based on the technical review undertaken, the boiling water tests (as described in ASTM D3625 standard or the JTGE42-2005 Chinese standard) were used in some studies to evaluate the moisture sensitivity of loose RCA-asphalt mixtures. In addition, the rolling bottle tests (as described in Spanish standard European Norm EN 12697-11 *Test methods for hot bituminous mixtures - Part 11: Determination of the affinity between aggregates and bitumen*) were used in other studies for this purpose. A summary of the studies conducted to evaluate the strength of bond between RCAs and bitumen can be found in Table 2.4.

According to A. R. Pasandín and Pérez (2014a) and Z. Zhang et al. (2016), RCAs have poor adhesion with bitumen and, thus, lower resistance to moisture is expected in RCA-asphalt mixtures. In addition, A. R. Pasandín et al. (2015) stated that the use of different types of fillers can affect the stripping resistance of RCA mixes. A. R. Pasandín and Pérez (2014a) concluded that RCAs treated by heating or coating demonstrated better RCA-bitumen bonding than untreated RCAs.

According to the results presented in Table 2.4, the addition of RCAs into HMA can greatly affect its resistance to moisture damage. It is, therefore, vital to investigate the susceptibility of RCA-asphalt mixtures to stripping, especially when asphalt mixtures are to be made with low-quality RCAs derived from C&D waste.

Table 2.4: Studies conducted to investigate the adhesion between RCAs and bitumen

Author/country	Standard	Binder used	Test	Samples	Parameter	Result	Requirement	Comments
(A. R. Pasandín & Pérez, 2014a)/ Spain	ASTM D3625	50/70 Penetration-grade bitumen from Venezuela	Boiling water test	RCA	Coverage %	20	85-90	RCA showed poor water resistance. Using the cement filler and treatments (conditioning the mixtures in an oven for 4 hours/coating the RCA with 5% bitumen emulsion) improved the adhesion between RCA and bitumen.
				RCA + RCA filler		60		
				RCA + 1% hydrated lime		65		
				RCA + commercial limestone		50		
				RCA + grey cement		80		
				RCA + fly ash		50		
				RCA + 4 hours in oven		95		
				RCA + 5% bitumen emulsion		90		
	European Norm EN 12697-11		Rolling bottle test	RCA		60	-	
				RCA + RCA filler		65		
				RCA +1% hydrated lime		75		
				RCA + commercial limestone		75		
				RCA + grey cement		85		
				RCA + fly ash		75		
				RCA + 4 hours in oven		80		
				RCA + 5% bitumen emulsion		82		
(A. R. Pasandín et al., 2015)/Spain	European Norm EN 12697-11	B40/50	Rolling bottle test	RCA (Portland cement filler)	Coverage %	90	85-90	RCA with cement filler achieved better or similar bitumen coverage than RCA with hydrated lime.
		B40/50		RCA (Hydrated lime filler)		90		
		B40/50		RCA (Hydrated lime added to SSD RCA)		95		
		B40/50		RCA (Slurry of hydrated lime mixed with RCA)		80		
		B60/70		RCA (Portland cement filler)		85		
		B60/70		RCA (Hydrated lime filler)		80		
		B60/70		RCA (Hydrated lime added to SSD RCA)		85		
		B60/70		RCA (Slurry of hydrated lime mixed with RCA)		75		
		B150/200		RCA (Portland cement filler)		80		
		B150/200		RCA (Hydrated lime filler)		80		
		B150/200		RCA (Hydrated lime added to SSD RCA)		80		
		B150/200		RCA (Slurry of hydrated lime mixed with RCA)		60		

Table 2.4: Continued

Author/country	Standard	Binder used	Test	Samples	Parameter	Result	Requirement	Comments
(A. R. Pasandín et al., 2015)/Spain	ASTM D 3625	B40/50	Boiling water test	RCA (Portland cement filler)	Coverage %	80	-	RCA with cement filler achieved better or similar bitumen coverage than RCA with hydrated lime.
		B40/50		RCA (Hydrated lime filler)		55		
		B40/50		RCA (Hydrated lime added to SSD RCA)		55		
		B40/50		RCA (Slurry of hydrated lime mixed with RCA)		70		
		B60/70		RCA (Portland cement filler)		80		
		B60/70		RCA (Hydrated lime filler)		75		
		B60/70		RCA (Hydrated lime added to SSD RCA)		55		
		B60/70		RCA (Slurry of hydrated lime mixed with RCA)		75		
		B150/200		RCA (Portland cement filler)		75		
		B150/200		RCA (Hydrated lime filler)		65		
		B150/200		RCA (Hydrated lime added to SSD RCA)		55		
		B150/200		RCA (Slurry of hydrated lime mixed with RCA)		65		
(Z. Zhang et al., 2016)/China	JTGE42-2005	Heavy traffic road asphalt (AH-70), 60–80 penetration grade.	Boiling water test	RCA (4.75-9.50 mm)	Adhesion level	2	≥4	RCA had poorer adhesion with bitumen compared with natural aggregates.
				RCA (4.75-13.2 mm)		3		
				RCA (9.5-19.0 mm)		4		
				Natural aggregate (4.75-9.50 mm)		5		
				Natural aggregate (4.75-13.2 mm)		5		
				Natural aggregate (9.5-19.0 mm)		5		
				Natural aggregate (2.36-4.75 mm)		1		

2.7.3 Resistance to permanent deformation

Permanent deformation (rutting) is considered to be one of the most common distress failure modes of flexible asphalt pavement (Tayfur, Ozen, & Aksoy, 2007; T. Zhu, Ma, Huang, & Wang, 2016). In general, the rough surface texture of RCAs can mobilize higher friction forces, thereby generating high resistance to aggregate dislocation when traffic loads are applied. However, this is only a general expectation of what could happen to a mixture made with RCAs when rutting is evaluated. The tendency of RCAs to break under mechanical loading might affect the rutting performance of RAC-asphalt mixtures. Thus, a lower rutting performance may be produced when RCAs are added to HMA. Therefore, in many studies, this vital property has been evaluated to assess rutting performance of RCA-asphalt mixtures. In the following sub-sections, the results of wheel tracking tests, dynamic creep tests, and other types of tests conducted to investigate the rutting resistance of asphalt mixtures made with RCAs are summarised.

2.7.3.1 Wheel tracking test results

The wheel tracking test is a simulative test used to evaluate the resistance to permanent deformation of HMA. This test is widely used to investigate the rutting performance of RCA-asphalt mixes.

Table 2.5 gives an overview of studies undertaken to evaluate the ability of HMA made with RCAs to resist permanent deformation. The outcomes of these studies are inconsistent and show that the major influences are the source of the RCAs, their gradation and the type of mineral filler used.

Based on the results obtained by Mills-Beale and You (2010) and I. Pérez et al. (2012), the addition of RCAs to asphalt mixtures degrades their resistance to rutting. In addition, Rafi et al. (2014) found that a 10% RCA content enhanced the permanent deformation resistance of an asphalt mixture. Any further increase in RCA content decreases the rutting resistance. Z. Zhang et al. (2016) investigated the high-temperature stability of HMA made with RCAs. They indicated that the dynamic stability (the number of passes requires to create a rut of 1 mm depth) of mixes made with RCAs increased and the final rut depth also increased. The authors suggested increasing the degree of initial compaction of asphalt mixtures produced with RCAs to mitigate this issue. Pourtahmasb and Karim (2014) concluded that the utilization of fine and coarse RCAs in HMA degrades its resistance to permanent deformation. However, they stated that the use of mixed RCAs (fine and coarse fractions) leads to better resistance to rutting.

In contrast to these studies, the results obtained by Radević et al. (2017) showed that the inclusion of fine RCA and coarse RCA in HMA improves its rutting resistance, while the addition of mixed RCAs negatively affects this property. Additionally, it was documented that the filler type also affects rutting resistance. I. Pérez et al. (2012) found that the use of cement filler in HMA made with RCAs improves its rutting resistance compared to those prepared with lime fillers. The authors attributed this behaviour to the higher stiffness of mixes containing cement filler.

Table 2.5: Results of studies using wheel tracking tests on mixtures containing RCAs

Authors/ Country	Standard	Samples	Result	Requirements	Note/s	Comments
(Mills-Beale & You, 2010)/USA	AASHTO T 62-03	0% RCAs	<2 mm	≤8 mm	Rut depth measured after 8000 cycles. The test temperature was 52° C.	As the RCA percentage increases, the potential for rutting increases. However, the rutting resistance of mixtures made with RCAs is unlikely to be problematic at up to 75% RCAs.
		25% RCAs	<2 mm			
		50% RCAs	<2 mm			
		65% RCAs	<4 mm			
		75% RCAs	<6 mm			
(Cho et al., 2011)/ Korea	KS F 2374	0%	<3 mm	-	Rutting measured after 2500 load cycles at 60 °C.	Asphalt mix rutting resistance was ranked as (from highest to lowest) mixture with fine RCAs, mixture with coarse RCAs, mixture with natural aggregates and mixture with 100% RCAs.
		100% (RCA<4.75 mm)	<2 mm			
		100% (RCA>4.75 mm)	<3 mm			
		100% (RCA)	<4 mm			
(Pérez et al., 2012)/Spain	NLT-173	0% (cement filler)	12.97	Rate of rut depth (deformation velocity) must be lower than 20 µm/min	The test temperature was 60° C.	Mixtures made with natural aggregates performed better than those made with RCAs. The performance of mixtures with cement filler was better because it acts as a stiffener
		20% (cement filler)	14.08			
		40% (cement filler)	13.26			
		60% (cement filler)	-			
		0% (lime filler)	11.32			
		20% (lime filler)	13.15			
		40% (lime filler)	16.25			
		60% (lime filler)				
(Rafi et al., 2014)/ Pakistan	EN12697-22	0% RCAs	6.6 at 3000 cycles	≤20 mm after 5000 load cycles (10,000 passes)	The test temperature was 60° C.	The rutting performances of all mixtures were similar up to 2000 load cycles but changed beyond this level. Mixtures with 10% RCA showed higher rutting potential than mixtures made with virgin aggregates.
		10% RCAs	6.24 at 3000 cycles			
		20% RCAs	7.54 at 3000 cycles			
		30% RCAs	8.88 at 3000 cycles			
		0% RCAs	7.98 at 4000 cycles			
		10% RCAs	7.36 at 4000 cycles			

Table 2.5: Continued.

Authors/ Country	Standard	Samples	Result	Requirements	Note/s	Comments
(Rafi et al., 2014)/Pakistan	EN12697-22	20% RCAs	9.05 at 4000	≤20 mm after 5000 load cycles (10,000 passes)	The test temperature was 60° C.	The rutting performances of all mixtures were similar up to 2000 load cycles but changed beyond this level. Mixtures prepared with 10% RCA showed higher rutting potential than mixtures made with virgin aggregates.
		30% RCAs	11.5 at 4000			
		0% RCAs	8.9 at 5000 cycles			
		10% RCAs	8 at 5000 cycles			
		20% RCAs	10.2 at 5000			
		30% RCAs	14 at 5000 cycles			
(Pourtahmasb & Karim, 2014)/Malaysia	BS 598-110	0	2.9 mm	≤4 mm for moderate to heavily stressed sites	The test temperature was 45° C.	Rut depths increased with RCA content in HMA. HMA specimens made with 100% fine RCAs (fine and 100% coarse RCAs had lower performance than control mixes with up to a 40% replacement level. The performance of specimens with 100% mixed (fine and coarse) RCA was better than corresponding specimens made with fine and coarse RCAs.
		20% (RCA<4.75 mm)	2.96 mm			
		40% (RCA<4.75 mm)	3.31 mm			
		60% (RCA<4.75 mm)	4.47 mm			
		80% (RCA<4.75 mm)	7.08 mm			
		20% (RCA>4.75 mm)	2.91 mm			
		40% (RCA>4.75 mm)	3 mm			
		60% (RCA>4.75 mm)	4.09 mm			
		80% (RCA>4.75 mm)	6.33 mm			
		20% RCA	<3 mm			
		40% RCA	<3 mm			
		60% RCA	<4 mm			
		80% RCA	<4.5 mm			
(Fatemi & Imaninasab, 2016)/Iran	BS 598-110	0% (C&D) - 25° C	<3 mm			The final rut depth measured after 120 minutes followed the following trend (from highest to lowest): 30% C&D mix, 40% C&D mix, 20% C&D mix, 10% C&D mix, and 0% C&D mix. It was concluded that the addition of C&D waste improved the resistance of asphalt mixes to rutting.
		10% (C&D) - 25° C	<2 mm			
		20% (C&D) - 25° C	<2 mm			
		30% (C&D) - 25° C	<1.5 mm			
		40% (C&D) - 25° C	<1.5 mm			

Table 2.5: Continued.

Authors/ Country	Standard	Samples	Result	Requirements	Note/s	Comments
(Fatemi & Imaninasab, 2016)/Iran	BS 598-110	0% (C&D) - 60° C	<7 mm			The final rut depth measured after 120 minutes followed the following trend (from highest to lowest): 30% C&D mix, 40% C&D mix, 20% C&D mix, 10% C&D mix, and 0% C&D mix. It was concluded that the addition of C&D waste improves the resistance of asphalt mixes to rutting.
		10% (C&D) - 60° C	<6 mm			
		20% (C&D) - 60° C	<6 mm			
		30% (C&D) - 60° C	<4 mm			
		40% (C&D) - 60° C	<4 mm			
(Zhang et al., 2016)/China	(JTG E20-2011 JTG)	0%	5.54 mm	(-)	The test was performed at 60° C	According to the results, the dynamic stability of mixtures made with RCAs was better than that of a control mix. However, the final rut depth was higher. To reduce the accumulated final rut depth, the initial compaction degree should be improved in the real project.
		30%	6.2 mm			
		50%	6.51 mm			
		75%	7 mm			
(Radević et al., 2017)/Serbia	EN 12697-22	0% RCAs	5%	<7% proportional rut depth (determined by dividing the rut depth after *n* cycles (10,000) by the initial specimen height.	The test temperature was 60° C. The rut depth was measured after 10,000 cycles (20,000 passes)	The addition of RCAs greatly improves rutting resistance. The RCA structure and texture (rough surfaces and sharp edges), increased the specific surface area and friction between aggregate particles and helped to improve resistance to permanent deformation for both fine and coarse RCA mixtures. However, the combined effects of an increased bitumen content and RCA led to slightly higher permanent deformation for mixtures made mixed (fine and coarse) RCAs; although they were still below 7%, which is the limit according to technical requirements.
		15% (RCA<4.75 mm)	4.90%			
		30% (RCA<4.75 mm)	4.50%			
		45% (RCA<4.75 mm)	5.50%			
		15% (RCA>4.75 mm)	4.80%			
		30% (RCA>4.75 mm)	3.90%			
		45% (RCA>4.75 mm)	4.60%			
		15% RCAs	5.50%			
		30% RCAs	5.50%			
		45% RCAs	5.10%			

2.7.3.2 Dynamic creep test results

Table 2.6 summarises the findings of the dynamic creep tests conducted in several studies to investigate the rutting behaviour of asphalt mixtures containing RCAs. The results collected from the dynamic creep tests were comparable to a great extent. Paranavithana and Mohajerani (2006) stated that RCA and control mixes exhibited comparable resistance to creep. Similar results were obtained by Wong et al. (2007) for asphalt mixtures made with RCA filler. Wong et al. (2007) also mentioned that the addition of RCAs (particles < 3.15 mm) improved the creep performance of RCA-asphalt mixtures compared to control mixes.

Furthermore, Zulkati et al. (2013) indicated that during the early stage of loading, HMA made with RCAs showed comparable creep behaviour to those made with fresh aggregates. However, after a high number of loading cycles, these mixes showed more creep resistance than that offered by control mixes. The findings of Paranavithana and Mohajerani (2006) and Fatemi and Imaninasab (2016) suggest that increasing the amount of bitumen in asphalt mixtures containing RCAs can affect the creep characteristics of the mixes. According to Fatemi and Imaninasab (2016), the creep resistance of HMA increased up to 30% with RCA addition. However, at 40% RCA addition, creep resistance decreased. The authors attributed this decrease to the tendency of RCAs to break and to the high percentage of bitumen (OBC) in the mix.

Table 2.6: Results of studies using dynamic creep test on mixtures containing RCAs

Authors/country	Standard	Samples	Results	Parameter	Comments
(Paranavithana & Mohajerani, 2006)/Australia	AS 2891.12.1	0% RCAs (80 gyration-5% Bitumen)	2.682	Minimum creep slope (µƐ/pulse)	The mixes containing RCA behaved similarly to conventional mixes in regard to creep. For the control mix, creep testing was conducted only for the 5.0% bitumen and 80 gyrations sample. The determined value for minimum creep slope was 2.682 µƐ/pulse, which was higher than those found for mixes containing RCA.
		100% (coarse RCAs-80 gyration-5.1% Bitumen)	1.699		
		100% (coarse RCAs-80 gyration-5.5% Bitumen)	1.608		
		100% (coarse RCAs-80 gyration-6% Bitumen)	3.445		
		100% (coarse RCAs-80 gyration-6.5% Bitumen)	1.148		
		100% (coarse RCAs-120 gyration-5.1% Bitumen)	1.212		
		100% (coarse RCAs-120 gyration-5.5% Bitumen)	1.334		
		100% (coarse RCAs-120 gyration-6% Bitumen)	1.798		
		100% (coarse RCAs-120 gyration-6.5% Bitumen)	3.39		
(Wong et al., 2007)/Singapore	AS 2891.12.1	0% RCAs	<2000	Load cycles at 10,000 µƐ	Mixes with 6% concrete substitution were comparable to control mixes. Mixes with 45% concrete substitution achieved better rutting resistance than the conventional mix. The specimens with 45% heat-treated concrete substitution showed the highest rutting resistance.
		6% RCAs (<0.075 mm)	<2000		
		45% RCAs (<3.15 mm)	<7000		
		45% heat treated RCAs (<3.15 mm)	<10,000		
(Zulkati et al., 2013)/Singapore	AS 2891.12.1	0% RCAs	<3000	Load cycles at 10,000 µƐ	The RCA mixture showed comparable creep resistance as the control mix. This result indicates that both mixtures exhibited similar deformation during the early stage of loading. However, after many cycles of loading, the RCA mixture showed better deformation resistance than those made with control aggregates.
		63% RCAs (>6.3 mm)	<3000		
		0% RCAs	<6000	Load cycles at 30,000 µƐ	
		63% RCAs (>6.3 mm)	<10,000		

Table 2.6 Continued.

Authors/country	Standard	Samples	Results	Parameter	Comments
(Fatemi & Imaninasab, 2016)/Iran	AS 2891.12.1	0% (C&D)	1920	Flow number	The addition of up to 30% RCA enhanced the rutting resistance. At less than 30% RCA, the friction property causes brittleness while more C&D waste materials have the opposite effect. In addition, considering binder content effects on rutting resistance, greater binder content makes mixes less rut-resistant, which is another possible reason why 40% RCA decreases rutting resistance.
		10% (C&D)	2319		
		20% (C&D)	2530		
		30% (C&D)	2712		
		40% (C&D)	2394		

2.7.3.3 Results of other tests

Besides wheel tracking and dynamic creep tests, other tests have also been performed to evaluate the rutting potential of RCA-asphalt mixtures. For example, the Kim test and flow number (FN) test were used by Cho et al. (2011) and Bhusal and Wen (2013), respectively, to investigate the rutting performance of HMA made with RCAs. The Kim test is a simulative test developed by Doh, Yun, Amirkhanian, and Kim (2007) to evaluate the resistance of asphalt mixtures to permanent deformation at high temperatures. In this test the load is applied in the directions of compaction and traffic load application in order to simulate the loading conditions in the field.

Based on Bhusal and Wen (2013) study, as the percentage of RCA increases in the mix, the rutting resistance decreases. Bhusal and Wen (2013) attributed this finding to bitumen being absorbed into RCA pores, which expanded at the high testing temperature. On the contrary, Cho et al. (2011) indicated that asphalt mixtures manufactured with a fine recycled aggregate fraction achieved higher deformation strength than those prepared with natural and coarse recycled aggregates. The authors attributed this to the combined effect of friction and the higher bitumen absorption of fine recycled aggregates.

2.7.4 Fatigue life test results

As explained in Section 2.6.2, fatigue cracking is one of the main forms of flexible pavement distress in asphalt mixtures (Sabouri & Kim, 2014). The investigation of fatigue distress is, therefore, crucial to evaluating the possibility of RCAs for use in HMA. Several standard tests have been agreed on to assess the fatigue behaviour of asphalt mixtures, such as repeated indirect tensile fatigue tests, monotonic tensile strength tests, and cyclic fatigue life tests. In the following sub-sections, a summary of the results of studies undertaken to investigate the fatigue performance of RCA-asphalt mixtures is introduced.

2.7.4.1 Monotonic test results

Bhusal and Wen (2013) evaluated the fatigue life of HMA made with RCAs using indirect tensile testing (IDT). The researchers computed the fracture energy (FE) of the IDT as the area under the stress-strain curve up to the ultimate peak stress. The researchers utilized two types of RCAs to prepare the asphalt mixtures. Based on the results obtained, increases in RCA percentage decreased the tensile strength (i.e., fatigue life) of the mix. Bhusal and Wen (2013) found that a mix made with 80% of RCA type 1 had the highest FE while the mix made with 20% of RCA type 2 had the lowest FE. The authors did not give any explanation for this performance and mentioned that the reason behind it is unknown.

Shenghua Wu et al. (2017) carried out a laboratory investigation to check the effectiveness of the monotonic fracture approach to fatigue life prediction. They investigated the fatigue life of asphalt mixtures made with RCAs using indirect tensile tests. The fatigue life can be predicted based on the assumption that the energy required to fracture a specimen is equal to the energy dissipated during the cyclic fatigue test (Stowell, 1966). Based on the test results and analysis, Shenghua Wu et al. (2017) found that the addition of RCAs into HMA decreases its fatigue life. It was reported that increasing the RCA content lowered the number of load cycles to failure. It was also stated that the fatigue life was underestimated when monotonic indirect tensile tests are used. This is, however, not observed when indirect cyclic loading tests are used to evaluate the resistance of RCA-asphalt mixtures to fatigue. Shenghua Wu et al. (2017) attributed this reduction in the number of load cycles to failure to the loading mode and frequency used in both tests. During the cyclic test, there was a significant rise in the temperature, which might affect the sample's stiffness. Consequently, different fatigue performance is calculated when indirect cyclic loading is utilized.

2.7.4.2 Cyclic test results

As mentioned in the previous section, indirect tensile monotonic tests cannot account for hysteresis loss and healing effects, which can happen in the cyclic loading mode. It was also mentioned that a considerable rise in temperature can occur when cyclic testing is used to evaluate fatigue life. As a result, the temperature increase will affect the performance of the RCA-asphalt mixture (Shenghua Wu et al., 2017). Therefore, it is more appropriate to conduct fatigue testing in the cyclic loading mode.

In this regard, indirect tensile fatigue testing (ITFT) was carried out by (Bocci, Cerni, & Colagrande, 2016). The results of ITFT for specimens manufactured with 0%, 15%, 30% and 50% C&D waste are shown in Figure 2.16 and Figure 2.17. The results indicate that the mixes made with 15% and 30% C&D exhibited behaviour comparable to that of control mix. However, the behaviour was different when 50% C&D was added. It can be seen that the curve of the 50% C&D mix is located below the others. Bocci et al. (2016) stated that the addition of a high dosage of C&D (≥ 50%) encouraged fragile failures in the C&D-asphalt mix. The researchers recommended to paying special attention when designing an asphalt mixture with a high percentage of C&D (≥ 50%).

Furthermore, Shenghua Wu et al. (2017) performed indirect tensile cyclic testing to assess the ability of RCA-asphalt mixtures to resist fatigue cracking. Asphalt mixtures made with different percentages of RCAs (0%, 20%, 40%, 60%, 80% and 100%) were assessed. Tests were terminated when the indirect tensile modulus reached 50% of its initial value. According to the results of this study, the stiffness decreases as the number of load cycles increases. This implies that damage is initiated and develops during cyclic testing. It was concluded that the inclusion of RCA adversely affected the number of cycles required to halve the initial stiffness. Therefore, the authors recommended not adding higher percentages of RCA to asphalt mixtures, as this could lower their resistance to fatigue cracking.

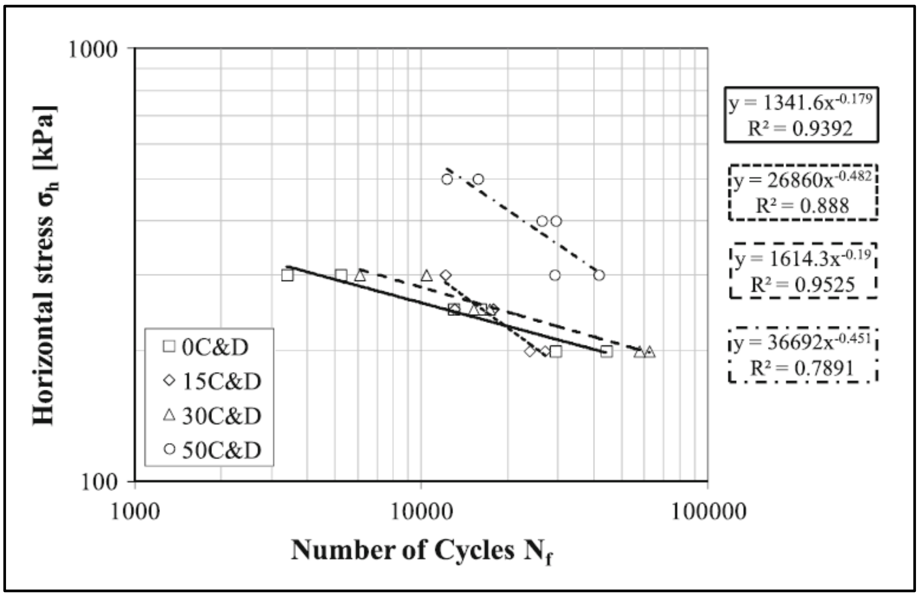


Figure 2.16: Fatigue life results in terms of horizontal stress as a function of number of cycles (Bocci et al., 2016)

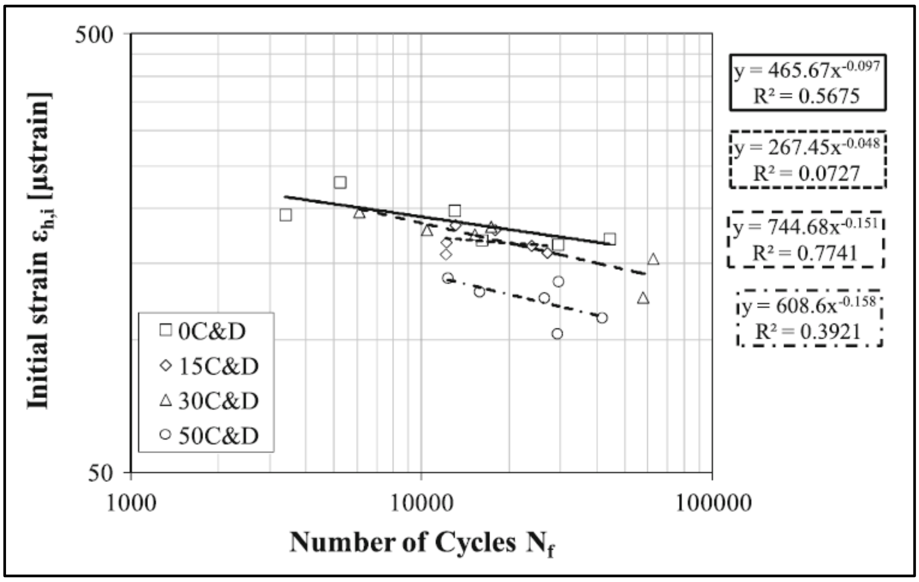


Figure 2.17: Fatigue life results in terms of initial horizontal strain as a function of number of cycles (Bocci, Cerni, & Colagrande, 2016)

2.7.5 Resilient modulus test results

The load spreading capacity of an asphalt layer is a function of two factors: layer thickness and material stiffness. Therefore, when a material with less stiffness is used in construction, greater thickness is needed to achieve the same magnitude of pressure on the layer underneath. This fact is explained in Figure 2.18 (N. Hunter et al., 2015).

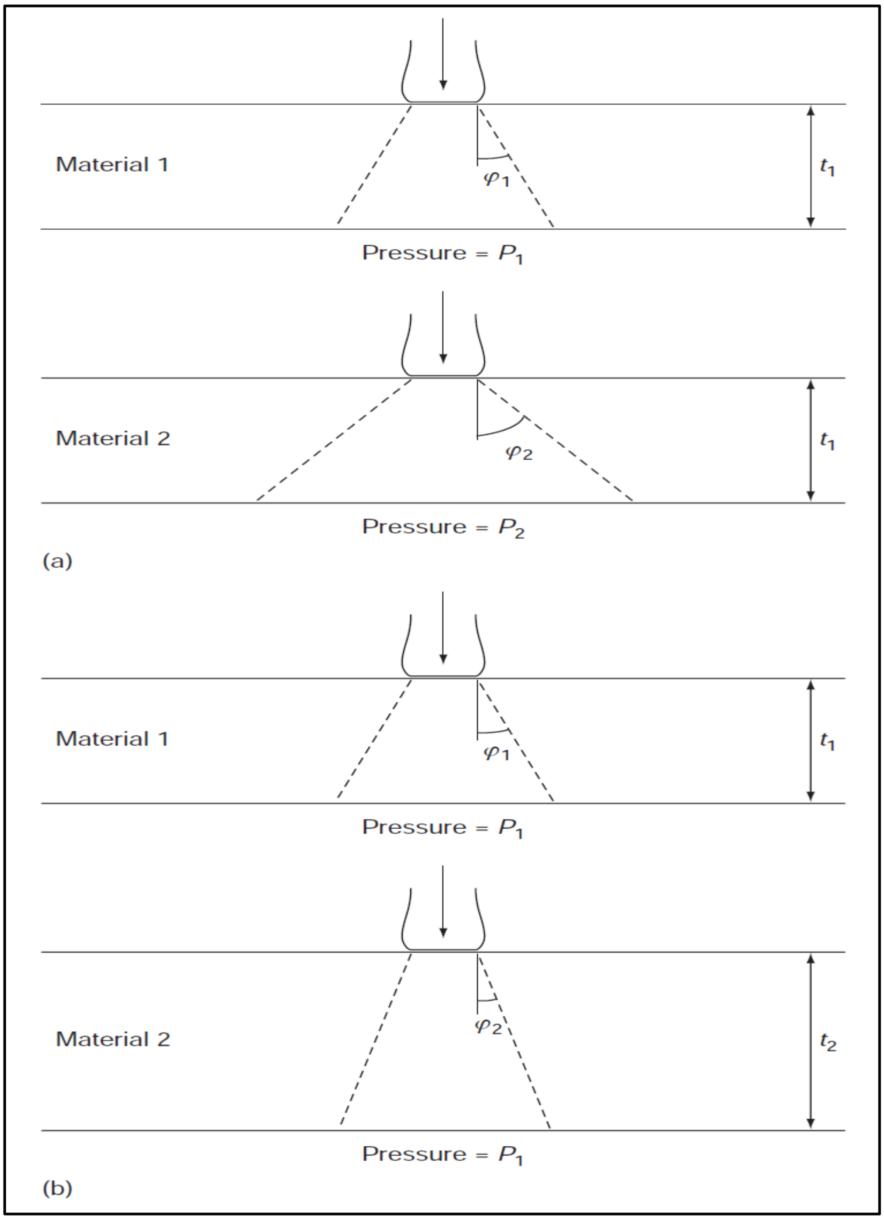


Figure 2.18: Stiffness of pavement layers: (a) equal thicknesses, different material stiffnesses, different resultant pressures; (b) different thicknesses, different material stiffnesses, equal resultant pressures (Courtesy of Dr Robert N Hunter).

The stiffness modulus or resilient modulus is a property essential to the characterisation of an asphalt mixture's performance. It is a crucial input to the mechanistic pavement design approach, as it explains the ability of an asphalt mixture to spread loads and withstand traffic loadings. As mentioned at the beginning of this chapter, the inclusion of RCA can affect the mechanical properties of asphalt mixtures. Therefore, it is important to ensure that RCA-asphalt mixes satisfy stiffness requirements. In Australia, the Australian Pavement Research Group has proposed typical values for different pavement layers based on the application and type of bitumen used (Alderson, 2008). These values are presented in Table 2.7.

Table 2.7: Typical laboratory resilient modulus (MPa) for HMA made in Australia (Alderson, 2008)

Application	Bitumen type				
	Class 170	Class 320	Multigrade or Class 600	A35P	AE10
Wearing course	2200	3000	4000	3000	2000
Intermediate layer/base course	3000	4000	5000	4000	2000
Fatigue course	2200	3000	4000	3000	2000

The resilient modulus of RCA-asphalt mixtures has been investigated in several studies to measure the effect of RCA addition on this property. In the laboratory, the indirect tensile stiffness modulus (ITSM) test is widely used to measure the resilient modulus of asphalt mixtures. Zulkati et al. (2013) and Bocci et al. (2016) made their asphalt mixtures: the control mix and RCA mixes, at the same bitumen content (5%). The findings of these studies indicate that the inclusion of more than 30% RCAs in asphalt mixtures improves resilient modulus. Bocci et al. (2016) stated that as the percentage of RCAs increases, the thickness of the bitumen film around the aggregate particles decreases. As a consequence, friction between particles is increased. Therefore, an increase in the resilient modulus of RCA mixes is expected. Zulkati et al. (2013) added another reason for such finding. The dense aggregate structure of RCA-asphalt mixtures helps achieving higher stiffness compared to control mix made with 100% natural aggregates. The RCA particles break due to mechanical loading during mixing and compaction, which changes the aggregate gradation and results in a denser structure in the RCA mix.

Furthermore, in many studies, the RCA-asphalt mixtures were made at their OBC. The outcomes of these studies are opposite to those observed when RCA and control mixes were made with the same percentage of bitumen. For example, Paranavithana and Mohajerani (2006) and Pourtahmasb and Karim (2014) showed that the incorporation of RCAs in HMA decreases its resilient modulus. Paranavithana and Mohajerani (2006) attributed this reduction to the lower strength of the old mortar and the lower quality of the RCAs.

In opposition to these results, Fatemi and Imaninasab (2016) designed four RCA-asphalt mixtures at their OBC. Increases in the resilient modulus were observed with up to 30% RCA content. At 40% RCA content, the stiffness decreased but was still higher than that computed for the control mix. Fatemi and Imaninasab (2016) connected this behaviour to the roughness and brittleness of the RCAs used. The former property (roughness) contributes to increased stiffness, while the latter one (brittleness) encourages decreased stiffness. Table 2.8 summarises the findings of studies that have evaluated the resilient modulus of asphalt mixtures made with RCAs.

Table 2.8: Results of studies using resilient modulus tests on mixtures containing RCAs

Results of several studies of resilient modulus test on mixtures containing RCAs.				
Authors/country	Standard	Samples	Resilient modulus (MPa)	Comments
(Paranavithana & Mohajerani, 2006)/Australia	AS 2891.13.1	0% RCAs (80 gyration-5% Bitumen)	7394	The resilient modulus of asphalt mixtures containing RCAs was decreased as the percentage of bitumen increases in the mix. The authors in this study attributed these findings to the lower strength of cement mortar attached to the RCA surfaces. It was indicated that the resilient modulus increased as the level of compaction increases from 80 gyrations to 120 gyrations.
		0% RCAs (80 gyration-5.5% Bitumen)	4400	
		0% RCAs (80 gyration-6% Bitumen)	5807	
		0% RCAs (120 gyration-5% Bitumen)	7561	
		0% RCAs (120 gyration-5.5% Bitumen)	6652	
		0% RCAs (120 gyration-6% Bitumen)	5807	
		100% (coarse RCAs-80 gyration-5.1% Bitumen)	4424	
		100% (coarse RCAs-80 gyration-5.5% Bitumen)	3393	
		100% (coarse RCAs-80 gyration-6% Bitumen)	2760	
		100% (coarse RCAs-80 gyration-6.5% Bitumen)	3395	
		100% (coarse RCAs-120 gyration-5.1% Bitumen)	4445	
		100% (coarse RCAs-120 gyration-5.5% Bitumen)	3846	
		100% (coarse RCAs-120 gyration-6% Bitumen)	3526	
		100% (coarse RCAs-120 gyration-6.5% Bitumen)	3147	
(Mills-Beale & You, 2010)/USA	ASTM D 4123-82	0% RCAs (tested at 5 °C)	> 1500	The results approved that the addition of RCAs into asphalt mixtures decreases the resilient modulus at all three testing temperatures.
		25% RCAs (tested at 5 °C)	> 1100	
		35% RCAs (tested at 5 °C)	> 900	
		50% RCAs (tested at 5 °C)	> 800	
		75% RCAs (tested at 5 °C)	> 700	

Table 2.8 Continued				
Authors/country	Standard	Samples	Mr (MPa)	Comments
(Mills-Beale & You, 2010)/USA	ASTM D 4123-82	0% RCAs (tested at 25 °C)	> 900	The addition of RCAs into asphalt mixtures decreased their resilient modulus at all three tested temperatures.
		25% RCAs (tested at 25 °C)	> 800	
		35% RCAs (tested at 25 °C)	> 400	
		50% RCAs (tested at 25 °C)	> 250	
		75% RCAs (tested at 25 °C)	> 200	
		0% RCAs (tested at 40 °C)	> 800	
		25% RCAs (tested at 40 °C)	> 600	
		35% RCAs (tested at 40 °C)	> 100	
		50% RCAs (tested at 40 °C)	≈ 100	
		75% RCAs (tested at 40 °C)	< 100	
(Zulkati et al., 2013)/Singapore	AS 2891.13.1	0% RCAs	> 3000, and < 3500	The RCA mix demonstrated better stiffness than the control mix. This is can be attributed to the thinner asphalt film thickness in the hybrid mix due to the higher absorption of the RCA. The thin film of asphalt creates higher resistance to dislocation shear forces among aggregate particles upon testing.
		63% RCAs (>6.3 mm)	> 4000, and < 50000	
(Pourtahmasb & Karim, 2014)/Malaysia	ASTM D4123	0	> 3100	The inclusion of various percentages of different grades of RCAs significantly affected the resilient modulus of the asphalt mixtures. The greater the RCA content, the greater the reduction in the resilient modulus.
		20% (RCA<4.75 mm)	> 3000	
		40% (RCA<4.75 mm)	> 2700	
		60% (RCA<4.75 mm)	> 2500	
		80% (RCA<4.75 mm)	> 2000	
		20% (RCA>4.75 mm)	> 3100	
		40% (RCA>4.75 mm)	> 2800	

Table 2.8 Continued				
Authors/country	Standard	Samples	Mr (MPa)	Comments
(Pourtahmasb & Karim, 2014)/Malaysia	ASTM D4123	60% (RCA>4.75 mm)	> 2800	The inclusion of different percentages of different grades of RCAs significantly affected the resilient modulus of the asphalt mixtures. The greater the content of RCAs, the greater the reduction in the resilient modulus.
		80% (RCA>4.75 mm)	> 2300	
		20% RCA (coarse and fine)	> 3000	
		40% RCA (coarse and fine)	> 2600	
		60% RCA (coarse and fine)	> 2600	
		80% RCA (coarse and fine)	> 2100	
(Fatemi & Imaninasab, 2016)/Iran	ASTM-D4124	0% (C&D) tested at 21° C	1782	Mixtures made with recycled aggregates exhibited higher resilient modulus than the control mix. The resilient modulus peaks at 30% C&D and, after this limit, it starts to decrease. Such a trend was attributed to the roughness of the recycled aggregate's surfaces, which increases internal friction.
		10% (C&D) tested at 21° C	2185	
		20% (C&D) tested at 21° C	2385	
		30% (C&D) tested at 21° C	2640	
		40% (C&D) tested at 21° C	2410	
(Bocci et al., 2016)/Italy	EN 12697-26	0% (C&D) tested at 20° C	2961	The results were comparable with up to 30% C&D addition. However, the inclusion of 50% C&D into the mix significantly increased the resilient modulus.
		15% (C&D) tested at 20° C	2957	
		30% (C&D) tested at 20° C	3041	
		50% (C&D) tested at 20° C	7303	

2.7.6 Dynamic modulus testing

The dynamic modulus is a fundamental property of asphalt mixtures, which controls the mechanical characteristics of flexible pavements (Ceylan, Schwartz, Kim, & Gopalakrishnan, 2009; Karki, Kim, & Little, 2015; Nega, Ghadimi, & Nikraz, 2015). Asphalt mixtures show viscoelastic behaviour under different loadings and temperatures, and their response to traffic loading depends on the peak of the loading frequency and temperature. The dynamic modulus test can define the stress-strain behaviour of asphalt mixtures (Al-Khateeb, Shenoy, Gibson, & Harman, 2006; M. W. Witczak, 2005). The dynamic modulus is a complex number which links the stress and strain of a viscoelastic material subjected to a continuous sinusoidal loading (Rahman, Mannan, & Tarefder, 2016). In addition, the calculated phase angle (ϕ) during the test indicates the elasticity or viscosity of the material. In a viscoelastic material such as an asphalt mixture, there is a time lag where the corresponding strain occurs sometime after the load application, as shown in Figure 2.19.

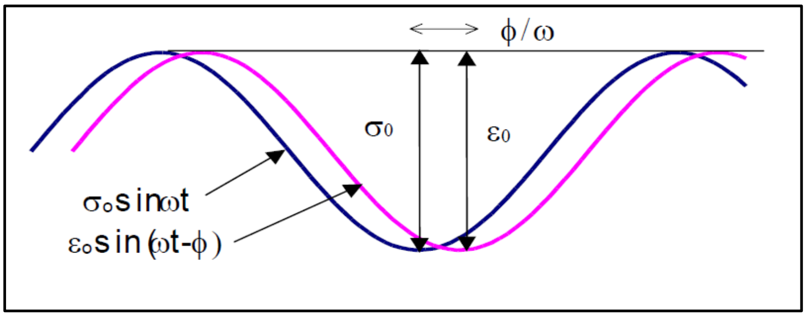


Figure 2.19: Dynamic (complex) modulus (M. Witczak & Bari, 2004)

By conducting dynamic modulus tests at different temperatures and frequencies, the stiffness of an asphalt mix can be determined over a wide range of temperatures and loading frequencies using the time-temperature superposition principle.

After reviewing the literature, it was noted that the dynamic modulus of RCA-asphalt mixtures was the least investigated property by researchers in this field. In this regard, Mills-Beale and You (2010) and Bhusal and Wen (2013) evaluated the dynamic modulus of an asphalt mixture made with RCA. A summary of the testing temperatures and frequencies used in these two studies is shown in Table 2.9. Both studies obtained similar results. It was evident that the dynamic modulus of RCA mixes decreased with increases in RCA content. Based on Bhusal and Wen (2013) results, the type of RCA used can change the dynamic modulus of the final product.

Table 2.9: Overview of reported dynamic modulus of RCA-asphalt mixes

Authors/country	Samples	Temperature (°C)	Frequency (Hz)	Remarks
(Mills-Beale & You, 2010)/USA	Mixtures with 0%, 25%, 35%, 50%, and 75% RCA were evaluated	13, 21.3, and 39.2	25, 10, 5, 1, 0.1	The greater the RCA content, the lower the dynamic modulus
(Bhusal & Wen, 2013)/USA	Mixtures with 0%, 20%, 40%, 60%, 80% and 100% coarse RCA1 or RCA2 were investigated	7, 15, 21, and 37	25, 10, 5, 1, 0.5, 0.01	The addition of RCAs decreased the dynamic modulus of the mix. As more RCA is added, the dynamic modulus decreases.

2.8 Points to be noted

RCA is different from natural aggregates mainly because of the old cement mortar attached to its surfaces, which can directly affect the properties of the asphalt mixture produced. The performance of an asphalt mixture containing RCAs is, therefore, different from that exhibited by one made with high-quality aggregates. Based on the technical review, the following points summarise the outcomes of several studies conducted to evaluate the behaviour of asphalt mixtures containing RCAs:

a) In recent years, the scarcity of natural aggregates in some countries, adoption of new environmental regulations in many countries, and the development of special engineering standards for low-quality aggregates are factors that have prompted researchers worldwide to use RCAs in construction.

b) In keeping with the previous point, only limited studies have been undertaken to evaluate the effectiveness of RCA incorporation in HMA, and their results are mostly variable. The attached cement mortar and other impurities present in RCAs, such as bricks, tiles, ceramics, timber, metals, etc., have helped obtain such conclusions.

c) Compared with natural aggregates, RCAs absorb more bitumen to achieve the optimum bitumen content (OBC). Also, fine recycled aggregate absorbs more bitumen than coarse RCA. This has prompted many researchers to only use coarse recycled aggregates in their mixtures.

d) As the dosage of RCA increases, the bulk density of the mix decreases and, in general, the strength increases. This is likely because of the higher porosity and rougher surface texture of RCAs compared to natural aggregates. The higher porosity decreases the density, while the roughness contributes to strength improvement of the final product. Additionally, the breakage of RCAs during mechanical mixing and compaction causes RCA-asphalt mixtures to exhibit denser structures than control mixes, which might give them higher strength.

e) According to the accepted Australian standards, RCAs are mainly utilized as granular materials for base and subbase layers in pavement construction. As a result, very limited research has been performed to explore the feasibility of using RCAs for HMA production in Australia.

f) RCA-asphalt mixtures have lower moisture resistance than those made with natural aggregates. The thin bitumen film that coats RCA particles, the high water absorption of recycled aggregates, and their poor adhesion with bitumen are factors influencing the moisture sensitivity of RCA-asphalt mixtures.

g) The resistance of RCA-asphalt mixtures against rutting is variable. Some authors attribute the decreased rutting resistance of RCA-mixtures to breakage of the RCA and their high percentage of absorbed bitumen. Others attribute the increased resistance to permanent deformation of HMA made with RCAs to the high friction mobilized by the recycled particles and bitumen absorption. Therefore, it is of high importance to assess the potential of RCA mixtures to rutting.

h) The resilient modulus of an asphalt mixture can indicate its ability to spread traffic loads over a wide area at the layer underneath. The results collected by several studies are inconsistent and dependent on the amount of bitumen added to the mix. Some researchers attributed increases in the stiffness of RCA-asphalt mixtures to the roughness of recycled aggregates. Others attributed the decrease in stiffness to the weak cement mortar attached to the RCA surfaces. Therefore, it was recommended to limit the percentage of RCA in the mix to keep its stiffness at an acceptable limit.

i) Cyclic fatigue tests are more suitable for measuring the resistance of asphalt mixtures to fatigue than monotonic tests. The former can take into account increases in specimen temperature throughout testing, which can significantly affect the performance of asphalt mixtures. Results collected from both tests indicate that the inclusion of RCAs into asphalt mixtures decreases their fatigue-cracking resistance. In light of discouraging results, it is vital to evaluate the fatigue life of HMA made with RCAs.

j) The dynamic modulus is a key property of asphalt mixtures, which controls strain and displacement in pavement structures. However, the dynamic modulus of RCA-asphalt mixtures is the least investigated property in this field. The results of limited studies indicate that the addition of RCAs to HMA decreases its dynamic modulus compared to that of a control mix. For this reason, it is crucial to investigate this fundamental property, especially for Australian HMA made with low-quality RCAs, because it has not been evaluated in previous studies.

2.9 Treatments for upgrading the performance of HMA made with RCAs

It is evident that the addition of RCAs to HMA can lead to a decline in a range of asphalt mixture performance properties. For instance, the inclusion of RCAs in HMA decreases its moisture resistance, resilient modulus, and dynamic modulus and fatigue life. In addition, the breakage of RCA particles can degrade the resistance of the mix against rutting and moisture damage. As a result, some researchers recommend not using RCA in HMA production because of the discouraging performance of asphalt mixtures containing RCAs (Bhusal & Wen, 2013). However, some researchers suggest using different treatments to upgrade the quality of RCAs used for HMA production. The objectives of these treatments are either to upgrade the engineering properties of the RCAs before being used in HMA production or to improve the performance of RCA-asphalt mixtures after they have been mixed and compacted. The suggested treatments are based on either heating or coating procedures. In the following sections, a brief description of these treatments in terms of their aims, methodologies and outcomes is presented.

2.9.1 Heating-based treatments

Two heating-based treatments have been suggested by Wong et al. (2007) and A. R. Pasandín and Pérez (2013) to enhance the quality of RCAs or RCA mixes. In these treatments, the RCA or RCA-asphalt mix is heated in an oven for a specific period of time.

2.9.1.1 Preheating RCAs

Wong et al. (2007) preheated fine recycled aggregates (<3.15 mm) in a Nabertherm furnace over different temperatures and time intervals, as shown in Figure 2.20. This process is called *calcination* and is conducted before mixing with bitumen. The aim of heating is to produce lime from calcium carbonate, which can improve the mixture's performance and enhance its moisture resistance. To evaluate the treatment, asphalt mixtures made with 0% RCA, 6% unheated RCA (<3.15 mm), 45% unheated RCA (<3.15 mm) and 45% heat-treated RCA (<3.15 mm) were designed and tested.

$$\boxed{27\ ^\circ\mathrm{C} \xrightarrow{1\ \mathrm{h}} 450\ ^\circ\mathrm{C} \xrightarrow{1\ \mathrm{h}} 450\ ^\circ\mathrm{C} \xrightarrow{1\ \mathrm{h}} 950\ ^\circ\mathrm{C} \xrightarrow{2\ \mathrm{h}} 950\ ^\circ\mathrm{C} \xrightarrow{1\ \mathrm{h}} 27\ ^\circ\mathrm{C}}$$

Figure 2.20: Heating regime used for RCA calcination (Wong, Sun, & Lai, 2007)

The results obtained by Wong et al. (2007) showed that an asphalt mix made with unheated fine RCA requires less bitumen than one made with heated RCAs. Also, the volumetric properties, stability and flow values were compliant with the Singapore Land Transport Authority's requirements. The mixes made with unheated and heated fine recycled aggregates exhibited comparable resilient modulus which was higher than that measured for a control mix. The mix with heat-treated RCA achieved the highest rutting resistance; however, mixes prepared with untreated RCA achieved good resistance to permanent deformation as well. Asphalt mixtures made with heated and unheated RCAs achieved higher resistance against rutting compared to those made with natural aggregates.

The results indicate that the unheated RCA mix achieved better stiffness modulus and higher resistance against permanent deformation compared to that of control mix. Furthermore, the mix made with unheated RCAs exhibited comparable stiffness and rutting resistance as compared with the mix containing heat-treated RCAs. Most importantly, the unheated RCA mix requires less bitumen than that required for heat-treated RCA mix to achieve OBC. This implies that the cost of asphalt mixtures produced with unheated RCAs would be much lower than those made with heat-treated RCAs. The *cost* here refers to the costs of the bitumen and energy of heating process.

2.9.1.2 Preheated RCA-asphalt mix

A. R. Pasandín and Pérez (2013) suggested curing RCA-asphalt mixtures in an oven at 170 °C for 4 hours to improve stripping resistance. During the heating process, the RCA is expected to absorb more bitumen, which reduces water absorption and leads to better moisture resistance. Asphalt mixtures made with 0%, 5%, 10%, 20% and 30% coarse RCAs were designed and evaluated. The results show that the RCA-asphalt mixes achieved better stiffness modulus, performed well against rutting, achieved similar fatigue life with up to 20% natural aggregate substitution, and behaved better against stripping compared to control mix. The RCA-asphalt mixes, however, absorb more bitumen to achieve the OBC. The authors mentioned that this treatment requires development of a new storage system in the asphalt plant to keep the temperature at 170 °C after mixing. Tables 2.10 and 2.11 give an overview of the properties of RCA and RCA-asphalt mixtures after heating-based treatment.

Table 2.10: Aggregate properties after heating-based treatment

Authors/country	Aggregate properties after heating based treatments.	
	Specific gravity	Aggregate crushing strength (%)
(Wong et al. 2007)/Singapore	Specific gravity of RCA was lower than that of natural aggregate	The aggregate impact value (AIV) of RCA was higher than that of natural aggregate, i.e. the RCA was weaker than natural aggregate
(Pasandín & Pérez 2013)/Spain	-	-

Table 2.11: RCA-asphalt mixture properties after heating-based treatment

Authors/country	RCA-asphalt mixtures properties after treatment.									
	Samples	OBC (%)	Volumetric properties (%)	Stability and flow	Stiffness (MPa)	ITS	Permanent deformation	Fatigue	Moisture sensitivity	Low-temperature cracking
(Wong et al. 2007)/Singapore	Concrete substitutions at 6% (untreated-<0.075 mm), 45% substitution (untreated-<3.14 mm) and 45% substitution (heat-treated-<3.14 mm)	The OAC increased with the % of substitution. Treatment increased the OBC from 5.3% for control mix to 7% for a mixture with 45% heat-treated aggregate.	VTM values are within the limit of specification but the VFB of mixtures made with 6% and 45% untreated aggregate did not satisfy the standard	Stability and flow results were within the limits of specification. Treatment increased stability and decreased flow.	MR values for mixtures with 45% treated and untreated aggregate were higher than that for a control mixture.	Not assessed	Samples made with heat-treated aggregate performed better against rutting than untreated aggregate samples. However, both mixes performed better than a control mix.	Not assessed	Not assessed	Not assessed
(Pasandín & Pérez 2013)/Spain	0, 5, 10, 20 and 30% of course natural aggregate were substituted with RCA.	Mixtures with 20% and 30% RCA had higher OBC than a control mix.	VMA and air void AV were within the limit of specification. AV increased as the % of RCA increased. This is explained by the roughness of attached mortar, which prevents suitable compaction.	Mixtures with 5% and 30% RCA exhibited higher stability than a control mix. Flow values increased with the % of RCA.	Mixtures prepared with RCA cured in an oven at 170 ºC were stiffer than a control mix at 0 ºC and 20 ºC.	In general, ITS values decrease as the RCA percentage increases in both dry and wet conditions.	As the dosage of RCA increases, rutting increases.	RCA-mixes showed similar fatigue resistance as the control mixture. However, the mix with 30% RCA had poorer fatigue life.	All mixtures complied with Spanish requirements for moisture damage. The mixture with natural aggregate had the lowest moisture resistance.	The produced mixture could be used in temperate regions but is not recommended for cold regions.

2.9.2 Coating-based treatments

In addition to heating-based treatments, coating-based treatments have been used to enhance the engineering properties of weak RCA particles. Five different methods have been developed using a variety of products and methodologies. In some cases, heating was also used to achieve the final objectives of the coating. In the following sub-sections, an overview of coating-based treatments is introduced, giving special attention to their aims, methodologies and findings.

2.9.2.1 RCA coated with cement slag paste (CSP)

To improve RCA resistance to crushing and upgrade the performance of RCA-asphalt mixtures, Lee et al. (2012) coated the coarse RCAs with cement slag paste (CSP). The RCA particles were coated at various thicknesses: 0.25 mm, 0.45 mm and 0.65 mm. The optimal thickness was determined to be 0.25 mm after analysing the results of experiments and visual inspections. Asphalt mixes with 0%, 25%, 50%, 75%, and 100% pre-coated RCA were evaluated.

The results revealed that the properties of asphalt mixes made with pre-coated RCA satisfied the specifications for Taiwan. In addition, it was stated that coating the RCAs with CSP decreased the bulk specific gravity and increased the rate of water and asphalt absorption. As a result, resistance against stripping and stability decreased as the RCA percentage increased in the mix. Furthermore, HMA made with RCA coated with CSP showed higher tensile strength and rutting resistance. The authors attributed these findings to the multiple crushed faces and rough surfaces of RCA coated with CSP, which enhances its resistance to shear dislocation forces.

In terms of aggregate properties, the coated RCAs showed lower specific gravity, higher water absorption, and higher resistance to crushing and abrasion compared with uncoated RCAs. This implies that such treatments can improve the strength of RCAs but also degrade their permeability simultaneously.

2.9.2.2 RCA pretreated with liquid silicone resin (LSR)

J. Zhu et al. (2012) suggested a different strengthening technique to improve the engineering indexes of RCAs. They treated coarse RCA particles with liquid silicone resin (LSR) to improve strength and reduce permeability. The RCAs coated with LSR showed higher resistance to crushing and a lower rate of water and asphalt absorption compared to that of untreated coarse RCAs. In order to assess the feasibility of RCAs with LSR for HMA production, asphalt mixtures with 100% fine and coarse RCA, 100% coarse RCA, 80% coarse untreated RCA, 80% coarse treated RCA, and 0% RCA were investigated.

The results show that the LSR treatment improved the resistance of the asphalt mixtures to moisture and low-temperature flexibility compared to untreated RCA mixes. In addition, the LSR treatment increased the rutting potential of the asphalt mixtures but these mixes still performed better against rutting than untreated RCA mixes. It was concluded that RCAs coated with LSR can be utilized in the production of HMA. Despite these results, this treatment requires curing the treated RCAs in an oven maintained at 60 °C for 24 hours to solidify the LSR. This requires considerable amounts of energy in addition to the cost of LSR treatment.

2.9.2.3 RCA coated with bitumen emulsion

A. R. Pasandín and Pérez (2014b) coated RCAs with bitumen emulsion. The aim of the treatment was to improve the mixture’s resistance to moisture damage. Coarse RCAs were coated with 5% bitumen emulsion (type ECL-2d containing 61.2%

bitumen). Mixtures with 5%, 10%, 20% and 30% coated RCA were prepared and evaluated. Each mixture was made at three bitumen contents: 3.5%, 4% and 4.5%.

This treatment homogenised the performance of RCA-asphalt mixes in terms of resilient modulus, rutting, fatigue life and moisture sensitivity. The good performance of RCA mixes was attributed to the homogenisation achieved after bitumen emulsion treatment.

2.9.2.4 RCA activated with organic silicone resin (OSR)

Another treatment was suggested by Yueqin Hou et al. (2014) for improving RCA strength and reducing absorption. Three types of activators were used for this purpose: organic silicone resin (OSR), metatitanic resin acceptor and silane resin acceptor. To achieve activation, a specific amount of activator was sprinkled onto the RCA and the mixture stirred continuously for 5 minutes. Then, the activated RCA was allowed to set outside for 3-4 hours. The OSR performed best so it was selected to activate the RCAs. The treatment increases the bulk specific gravity, reduces water absorption, and improves the crushing resistance of RCAs. For evaluation, mixtures made with 0%, 30%, 60% and 100% activated RCA were assessed.

The results of this investigation indicate that the Marshall indices of the mixtures met the requirements of bituminous mixtures for highway engineering (JTJ 052-2000). Furthermore, the mixes made with RCAs coated with OSR also showed good rutting and stripping resistance. The mixes made with up to 60% RCA satisfied all the requirements for bituminous mixtures in China. It should be mentioned that asphalt mixes containing OSR-activated RCA required more bitumen to achieve the OBC (Yueqin Hou et al., 2014). In this regard, an asphalt mix made with 100% coated RCAs required twice the amount of bitumen compared to the control mix to reach the OBC.

The RCAs coated with OSR absorbed 51.6% of the OBC (6.98%), which is higher than the OBC of the control mix (3.57%).

2.9.2.5 RCA treated with microbial calcite precipitation (MCP)

Pan et al. (2015) treated RCAs with microbial calcite precipitation (MCP) in order to enhance their poor adhesion with bitumen. The enhancement effects of MCP are achieved through two phases: *bio-deposition* and *bio-cementation*. The bio-deposition phase refers to bacteria-induced $CaCO_3$ precipitation that seals the pores of RCA. Bio-cementation involves bacteria-induced $CaCO_3$ precipitation that acts as a bonding material to reinforce weak RCA (De Muynck, De Belie, & Verstraete, 2010)

This treatment reduces water absorption and increases the bulk specific gravity of RCA. The treatment also improves the chemical affinity between RCA and bitumen. Disappointedly, no testing was undertaken to investigate the feasibility of this technique for use with HMA. Thus, the performance of asphalt mixtures made with RCAs treated with MCP has not yet been evaluated.

Tables 2.12 and 2.13 give a detailed overview of the properties of aggregate and RCA-asphalt mixtures after receiving coating-based treatments.

Table 2.12: RCA properties after coating-based treatment

Author/country	RCAs properties after coating based treatments.						
	Treatment and aims	Specific gravity	Water absorption (%)	Soundness	LA abrasion %	Flat and elongated particle content	Aggregates crushing strength (%)
(Lee et al., 2012)/Taiwan	RCAs were coated with CSP to improve their ability to resist crushing and enhance the engineering properties of RCA mixes.	Treatment decreased the specific gravity	As the coating thickness increased, water absorption increased.	The soundness of RCA particles improved after coating with CSP	The CSP treatment improved abrasion resistance	The treatment decreased the content of flat & elongated particles	The CSP coating enhanced the crushing strength
(J. Zhu et al., 2012)/China	RCA was coated with LSR particles to improve the strength and adhesion with asphalt and reduce the permeability.	Treatment produced RCA particles with lower specific gravity	Water absorption decreased after LSR treatment as a result of blocking and isolating the pores	Not assessed	The LA % decreased due to solidification of the LSR after treatment.	Not assessed	Decreased as a result of solidifying the LSR after treatment.
(A. R. Pasandín & Pérez, 2014b)/Spain	Coarse RCA was coated with 5% ECL-2d asphalt emulsion to improve the RCA-mixture durability	Not assessed	Not assessed	Not assessed	Not assessed	Not assessed	Not assessed
(Yueqin Hou et al., 2014)/China	OSR was used to coat coarse RCA to improve the engineering properties of RCA mixes and reduce bitumen absorption.	Bulk specific gravity increased.	Water absorption decreased.	Not assessed	Not assessed	Not assessed	The crushed value was reduced by treatment.
(Pan et al. 2015)/China	RCA was covered by a layer of alkaline calcite precipitate to enhance bonding between its particles and bitumen.	Specific weight of aggregates increased as a result of treatment.	Porosity of RCA samples was reduced, i.e. water absorption decreased.	Not assessed	Not assessed	Not assessed	Not evaluated

Table 2.13: RCA-mix properties after coating-based treatment

Author/Country/ Treatment	Mixtures	OAC	Volumetric properties	Stability and flow	Stiffness (MR)	ITS (MPa)	Permanent deformation	Fatigue	Moisture sensitivity	Low-temperature cracking
(Lee et al. 2012)/Taiwan/CSP	Mixtures with 0%, 25%, 50%, 75%, and 100% coarse RCA were evaluated.	OAC increased with the percent of coated RCAs	All values of VMA and VFB were within specified requirements.	All values were within specified requirements.	Not assessed	Indirect tensile strength increased with RCA content	Resistance to rutting increased as the substitution ratio of coated RCAs increased	Not assessed	Resistance to moisture damage decreased as RCA content increased.	Not assessed
(Zhu et al. 2012)/China/LSR	CR + FR, CR + FL, CR (80%) + CL (20%) + FL, TCR (80%) + CL (20%) + FL and CL + FL mixtures were made	OAC decreased by 88.8% when control aggregate was replaced by coarse RCA treated with LSR	All values were within the specifications.	All values satisfied the specifications	Mixtures prepared with coarse treated RCAs fulfilled the Chinese specification for moisture damage.	LSR increased the tensile strength of asphalt mixes made with treated RCAs	Mixtures without LSR exhibited higher rutting resistance than those prepared with natural or treated aggregate	Not assessed	Mixtures with LSR demonstrated higher resistance to moisture than mixtures prepared with untreated RCA.	Mixes made with LSR exhibited better resistance to low-temperature cracking than those made with untreated RCA, and less resistance than control mixes.

Table 2.13 Continued

Author/Country/ Treatment	Mixtures	OAC	Volumetric properties	Stability and flow	Stiffness (Mr)	ITS (MPa)	Permanent deformation	Fatigue	Moisture sensitivity	Low temperature cracking
(Pasandín & Pérez 2014)/Spain/Bitumen emulsion	Mixtures with 0%, 5%, 10%, 20%, and 30% RCA were investigated.	All mixes were made with 3.5%, 4%, and 4.5% bitumen	VMA and VFA complied with specifications.	Not assessed	Treatment slightly stiffened mixtures prepared with RCA.	Not assessed	All RCA mixes showed less resistance to rutting than control mixes.	Mixtures with RCA showed similar fatigue life to control mixes.	TSR values for all RCA specimens satisfied the requirements of Spanish standards.	Not assessed
(Hou et al. 2014)/China/OSR	Mixes with 0%, 30%, 60% and 100% fine and course aggregate were replaced by RCA treated with LSR	OAC increased with RCA content.	All volumetric properties were within the specifications.	Stability decreased and flow increased as the RCA content increased.	Not assessed	Not assessed	Mixes with 30% and 60% RCA showed higher dynamic stability than the control mix.	Not assessed	All mixtures satisfied the specification for moisture damage in China.	Resistance to low-temperature cracking was acceptable with up to 60% RCA content.
(Pan et al. 2015)/China/MCP	No testing was undertaken to assess the feasibility of the MCP treatment for RCA-asphalt mixtures.									

2.10 Research needs

Hitherto, none of the previously suggested treatments has yet been adopted for flexible pavement construction. Thus, development of new techniques to improve the performance of RCA-asphalt mixtures is still required. As explained in the previous section, treatment sometimes improves the strength of RCAs but degrades other properties such as permeability. This conclusion is also true when considering RCA-asphalt mixture properties. In this regard, keeping an RCA mix in an oven at 170 °C for 4 hours can improve its resistance to moisture damage while limiting its use to hot areas only (A. R. Pasandín & Pérez, 2013).

In light of this, it would be wise and logical to use a second treatment to remedy the remained and/or produced defects in RCAs after the first treatment. Considering this conclusion, two states of art coating techniques are to be developed in this thesis. These techniques are based on combining two treatments, and are thus called *double coating techniques* (DCTs). The philosophy behind the DCT is maximizing the total net benefits that are expected when applying specific treatment by using a second treatment.

The significance of the present study is in identifying new techniques for RCA by combining two treatments to enhance the performance of asphalt mixtures containing recycled aggregates. This represents a new approach to RCA improvement which may help in producing HMA with better engineering properties and, therefore, promote its use in asphalt mixtures in Australia and other parts of the world. Furthermore, this could reduce the demand for natural aggregate resources and promote sustainable practices.

3 Development of the Double Coating Technique

3.1 Introduction

As discussed in the literature review, the disposal of RCA wastes can produce ecological and financial impacts. Thus, a number of studies have examined the use of RCAs in flexible pavement applications, thereby diverting them from disposal in landfill. These studies indicate that the high porosity of RCAs, which gives them higher rates of water and asphalt absorption than fresh aggregates, is the main feature preventing their wider usage in asphalt and concrete applications. Some researchers, therefore, have suggested different coating and heating treatments to improve the behaviour of RCAs, either by reducing their porosity or increasing their strength. However, none of the previous treatments has yet been adopted in HMA production. With such a situation, developing a new treatment that allows RCAs to be used effectively in HMA production becomes a strategic priority in this regard. The new treatment may mitigate the impacts of RCA wastes and ease their usage in asphalt mixtures. In order to fulfil this strategic research priority, the following should be considered:

- More support for research in the area of RCA-asphalt mixtures is needed in order to make the use of RCAs more valuable, ease the flow of RCAs within communities and mitigate RCA disposal problems.
- Development of a new technique for RCAs to be used effectively in HMA is possible in view of below steps:

1) Critically review the outcomes of previously used RCA treatments.
2) Evaluate the pros and cons of each treatment by looking at the properties of RCA products and RCA-asphalt mixes post-treatment.

3) Explore a systematic way to remedy the defects that still exist after applying a specific treatment. That is, by using a suitable second treatment based on the review carried out in step (1) above.
4) Modify previously used treatments in order to suit the properties of the materials used in this research, and to fulfil the objectives of the new technique in improving the performance of RCA-asphalt mixes over that of past treatments.

Following this, two state-of-the-art coating techniques are to be developed in this research. These techniques aim to improve the performance of asphalt mixtures made with low-quality RCAs derived from C&D wastes. The techniques are based on combining two previously used treatments, and are thus called *double coating techniques* (DCTs) as mentioned in section 2.10.

The philosophy behind the DCT is to maximize the total net benefits that can be achieved when applying specific treatment by using a second treatment. The second treatment aims to remedy RCA defects remaining after the first treatment. The steps that have been followed by previous researchers in applying their treatments will be adjusted to fit the material properties and the objectives of the DCT.

This chapter demonstrates the processes that led to the development of the DCT. It also describes the hypotheses behind the DCT, the criterion for coating and heating RCAs or RCA-asphalt mixtures, the process of developing the DCT, and the tests used to assess the quality of RCAs after treatment. Furthermore, this chapter summarizes the post-treatment properties of RCAs and explains how to maintain the applicability of the DCT into the HMA industry.

3.2 Hypotheses of the DCT

Two research hypotheses have been set as a basis for developing the new state-of-the-art DCTs:

Hypothesis 1: *A treatment may improve some engineering properties of RCAs and asphalt mixtures while simultaneously degrading others. It is, therefore, necessary to remedy RCA defects remaining after a first treatment by using a second treatment.*

Hypothesis 2: *Pores and cracks in RCA that were not sealed efficiently by the first treatment may degrade the performance of the final product. These pores/cracks may increase water and bitumen absorption which, in turn, affects the durability and cost of the mix produced. Thus, it would be wise to seal these pores/cracks using a second treatment.*

According to Hypothesis 1, the issue that needs to deal with is related to the used treatment. However, in accordance with Hypothesis 2, the use of RCA, which is considered a lower quality material than natural aggregate, is the main reason for the degradation after using a specific treatment. In both cases, there is a need to systematically select a suitable second treatment to remedy the defects that remain after a first treatment.

3.3 RCA activation treatments

Based on the review carried out in Chapter 2 (Sections 2.9), several treatments were used to upgrade the performance of RCA-asphalt mixtures. These treatments were either based on heating the RCAs or RCA-asphalt mixes or coating the RCAs with different materials. The aims of these treatments were to 1) improve the strength of RCA (Yueqin Hou et al., 2014; Lee et al., 2012; J. Zhu et al., 2012), 2) decrease water and bitumen absorption (Yueqin Hou et al., 2014; A. R. Pasandín & Pérez, 2013, 2014b; J. Zhu et al., 2012), and 3) improve RCA-bitumen adhesion (Pan et al., 2015; J. Zhu et al., 2012).

In this regard, sometimes the treatment was found to upgrade some of the properties of the mix and degrade others simultaneously. For instance, although treatments suggested by Lee et al. (2012) and Yueqin Hou et al. (2014) were reported to enhance the resistance of coated RCAs against abrasion, they increased the rates of water and bitumen absorption at the same time. Therefore, there is a need to use a second treatment to remedy the degradation that may remain after the first treatment. Additionally, curing RCA- asphalt mixes in an oven at 170 °C for 4 hours increased the stiffness of RCA mixes (A. R. Pasandín & Pérez, 2013). However, the treatment limited the usage of RCA-mixes in hot regions. In addition, the latter treatment was documented to have a detrimental impact on the engineering properties of bitumen due to thermal and oxidative effects (Ana R. Pasandín, Pérez, Oliveira, Silva, & Pereira, 2015). It is therefore important to cure asphalt mixes made with RCAs for less than 4 hours. This is important to limit heating disadvantages, promote energy and time saving, and not dramatically affect the engineering properties of bitumen and asphalt mixes after heat-treatment.

3.4 Concepts of the DCT

Based on the two hypotheses set out for this research and described in Section 3.2, the use of two systematically selected treatments to upgrade the performance of RCA-asphalt mixtures, is logical. Therefore, for the purposes of this research, two DCT are developed, as described in the following sections.

3.4.1 DCT 1: Coating RCAs with CSP and Sika Tite-BE

According to Hypothesis 1, coating RCAs with two different materials is suggested (i.e. cement slag paste [CSP] and Sika Tite-BE in this study). The properties of these coating materials will be described in the following sections. The two coats (CSP coat, and Sika Tite-BE coat) are expected to produce RCA-asphalt mixes with better performance. In this regard, RCA coated with CSP is reported to have better abrasion resistance compared with uncoated RCA (Lee et al., 2012). However, HMA made with CSP-coated RCA is affected by the RCA content. This situation, in fact, is comparable to that described in Hypothesis 1. The issue that remains after CSP treatment is related to the treatment itself. It was reported that coating with CSP increases water absorption and degrades stripping resistance of asphalt mixtures (Lee et al., 2012). Accordingly, there is a need to remedy such degradation to avoid further damage in the future.

It can be seen from the results of tensile strength ratio (TSR), dynamic stability (DS), and indirect tensile strength (Figures 3.1, 3.2, and 3.3, respectively) that asphalt mixes made with RCAs coated with CSP were affected by the percentage of coated RCAs in the mix (Lee et al., 2012). In contrast, the bitumen emulsion treatment homogenises the performance of asphalt mixtures regardless of the percentage of RCA or bitumen content in the mix and improves stripping resistance (see Figure 3.1, Figure 3.4 and Figure 3.5; (A. R. Pasandín & Pérez, 2014b).

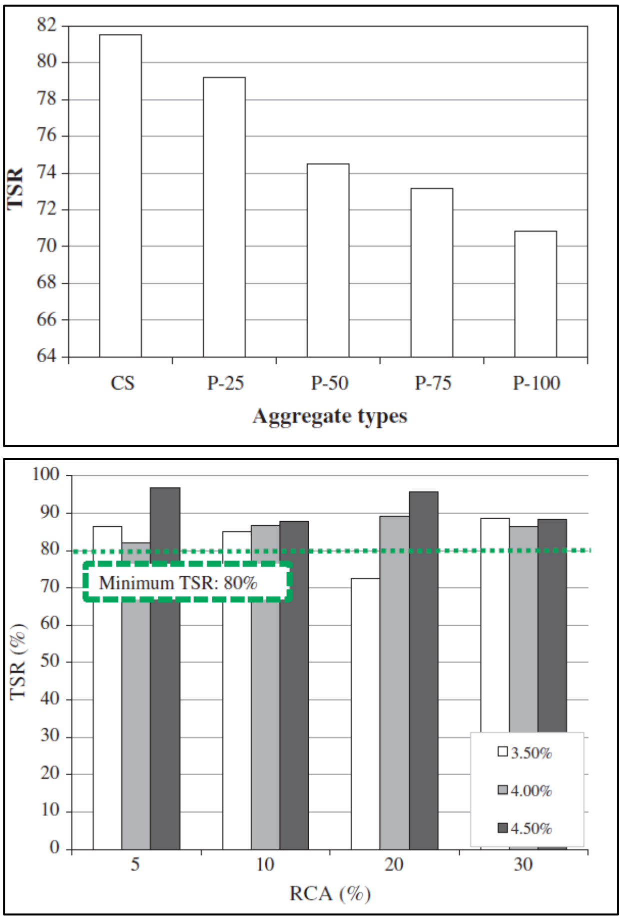


Figure 3.1: TSR results: above (after Lee et al., 2012), and below (after Pasandin and Perez 2014)

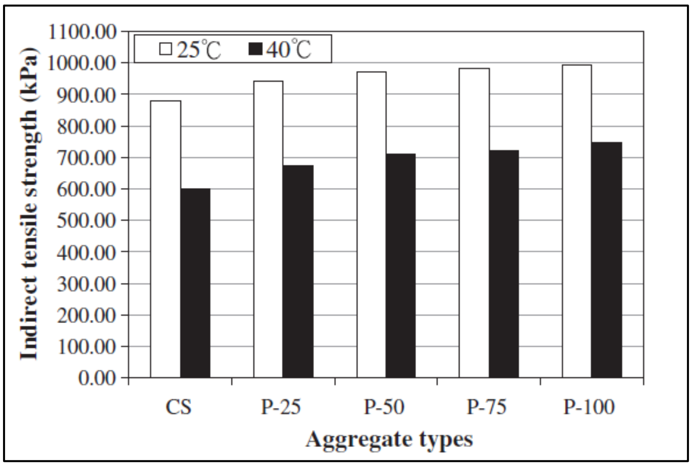


Figure 3.2: Indirect tensile strength of several aggregate types at two temperatures (Lee et al., 2012)

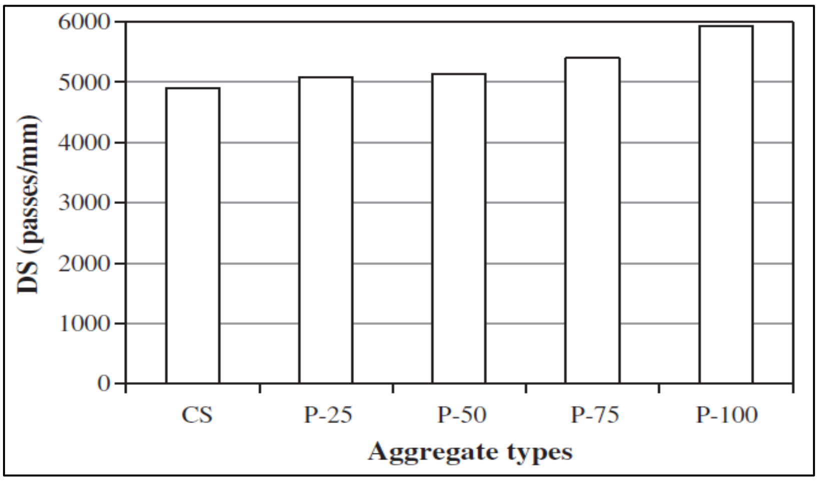


Figure 3.3: Dynamic stability of asphalt mixtures (Lee et al., 2012)

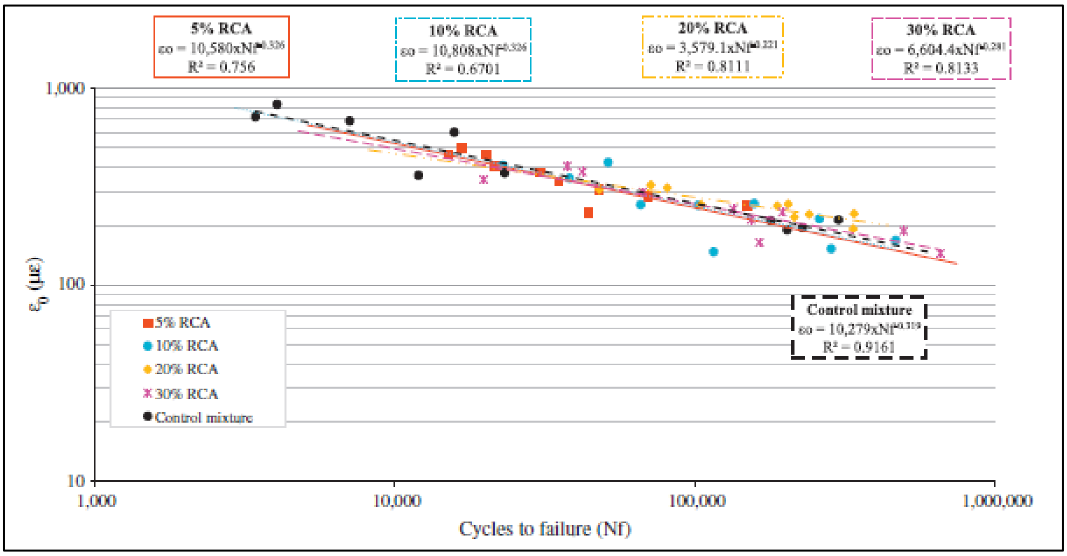


Figure 3.4: Fatigue life of several RCA mixes (Pasandin and Perez, 2014)

Therefore, it is expected that coating the RCA particles with CSP and then with Sika Tite-BE before mixing with bitumen will produce asphalt mixtures with better performance. The first coat (CSP coat) is expected to reinforce the weak RCA particles, while the second coat (Sika Tite-BE coat) is expected to mitigate absorption of RCA coated with CSP and, thus, upgrade the mix’s durability and adhesion between bitumen and DCRCAs.

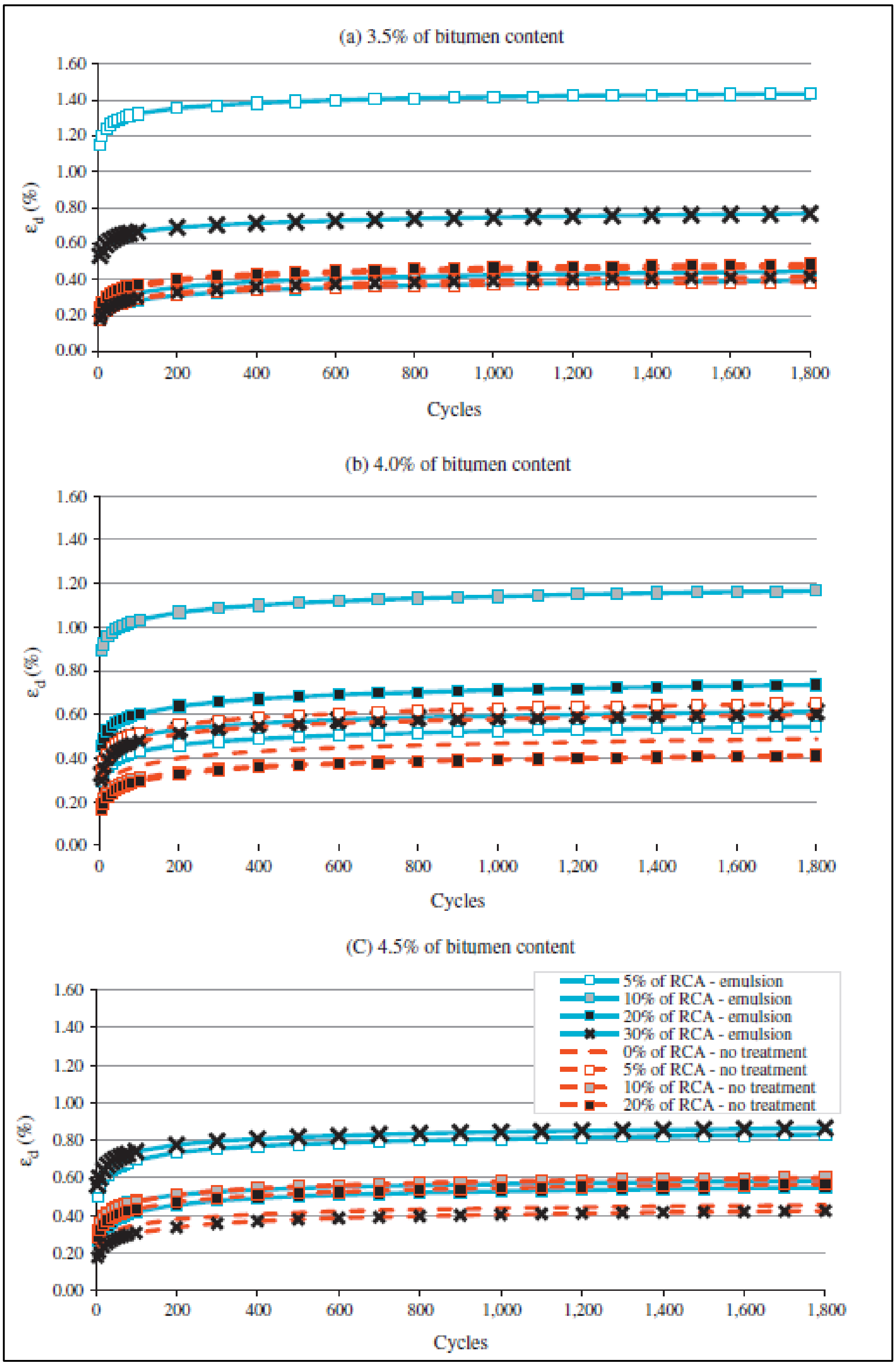


Figure 3.5: Permanent deformation test results (Pasandin and Perez, 2014)

Figure 3.6 describes the concept of the first DCT developed for the purposes of this research.

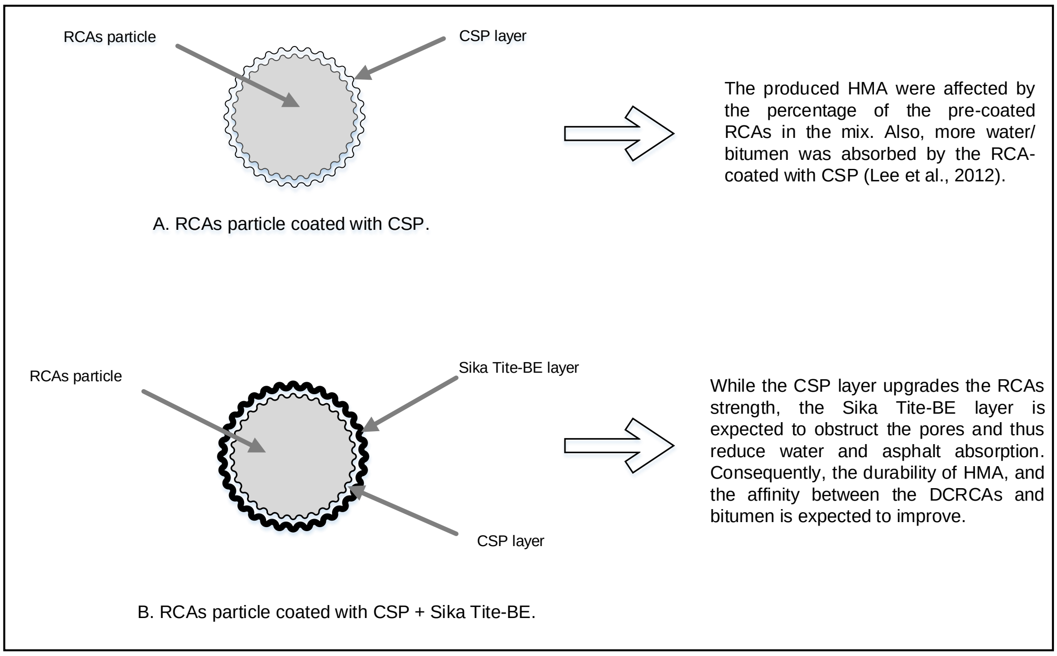


Figure 3.6: The concept of DCT 1 (coating RCA with CSP + Sika Tite-BE)

3.4.2 DCT 2: Coating RCAs with Sika Tite-BE followed by heat-treatment

As stated in Hypothesis 2, the issue that prevents the modified mix from achieving better performance may be related to defects in the RCAs. In such a situation, there is a need to use a second treatment to remedy RCA defects remaining after the first treatment. Therefore, coating RCA particles with asphalt emulsion and then preheating the mixture in an oven for 1½ hours only may be worth studying. The aim of heating the mixture in an oven for 4 hours was to allow the RCA particles to absorb high amount of asphalt, thus obstructing the pores and improving the resistance of RCA mixtures to moisture damage (Pasandín & Pérez, 2013). This aim, however, can be achieved by coating RCA particles with an asphalt emulsion (Pasandín & Pérez, 2014). Coating the RCAs with asphalt emulsion is expected to reduce the rate of water and bitumen absorption and improve adhesion with bitumen.

Moreover, keeping the mixture in an oven at 170 °C for 4 hours is expected to damage the engineering properties of asphalt cement (Ana R. Pasandín et al., 2015). Therefore, reducing the heating time to less than 2 hours may limit these damages and provide some time to obstruct the remaining pores remaining on the RCA surfaces after the bitumen emulsion treatment. In this research the heating time is limited to 1½ hours to mitigate heating disadvantages and promote energy and time saving as described in Section 3.3. The combined coating and heating treatments are expected to effectively obstruct the RCA's pores and direct the effects of DCT2 towards the RCA itself without damaging the engineering properties of the bitumen and RCA-asphalt mix. Figure 3.7 describes the concept of DCT2 developed in this investigation. As shown in the figure, the technique is based on combining the coating and heating treatments and reducing the heating period to 1½ hours. The importance of the latter step is to limit the impact of heat on the bitumen and asphalt mixes as explained earlier, such that only the RCAs are affected by this technique.

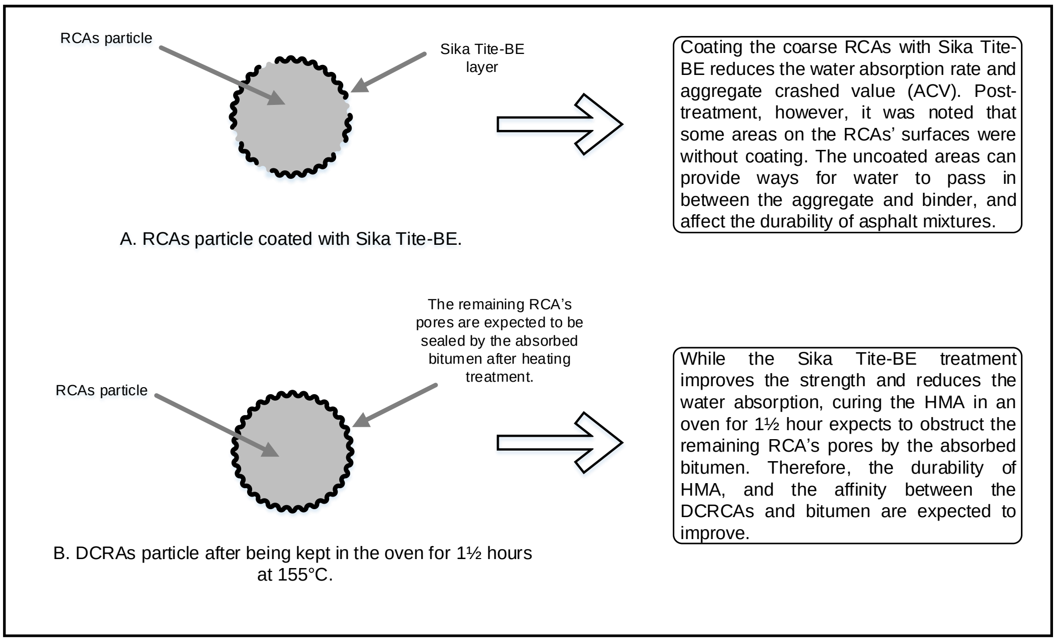


Figure 3.7: The concept of DCT2 (coating with Sika Tite-BE plus heat-treatment)

3.5 Criteria of the DCT

Based on the two suggested hypotheses, and to achieve the objectives of the two DCTs, two basic criteria need to be applied. The first criterion is that the coating of RCAs has to be verified in a way that maintains the applicability of the technique to site practices. The second criterion is that the heating of RCA-asphalt mixes has to be verified in a way that minimises heating effects on bitumen and mix engineering indexes.

3.5.1 Criterion for coating RCAs with CSP

Coarse RCA is to be coated by CSP at four theoretical thicknesses: 0.05 mm, 0.1 mm, 0.2 mm and 0.4 mm. The optimal thickness will be selected in accordance with testing results [Los angles (LA) abrasion %, aggregates crushed value (ACV) %, water absorption, apparent particle density, particle density on a dry basis, and particle density on saturated surface dry (SSD) basis) and visual inspection]. The visual inspection of the coated RCA is of great importance because it can assess the applicability of the coating to field conditions. The optimal CSP thickness is that which is sufficient to shield the coarse RCA particles without causing them to cement to each other and form lumps.

3.5.2 Criterion for coating RCAs with Sika Tite-BE

In the present study, Sika Tite-BE, an acrylic-based bitumen waterproofing membrane manufactured by Sika Australia Pty Ltd, was used to coat the RCAs. The information about Sika Tite-BE can be found in section 3.6.4. According to its technical data sheet, Sika Tite-BE has to be applied in two successive coats. The percentage of added Sika Tite-BE by RCA weight should only be sufficient to shield the coarse recycled particles, no more, no less. The Sika Tite-BE requires dilution with water before coating. The mixture of Sika Tite-BE/water prepared for the first coat has to have a lower Sika Tite-BE content than that made for the second coat. The first coat is expected to penetrate deep into small pores and cracks while the second coat is expected to seal the remaining pores and cracks.

3.5.3 Criterion of heating the RCA-asphalt mixtures

Keeping the RCA-asphalt mixtures in an oven after mixing has been suggested previously to improve moisture resistance (A. R. Pasandín & Pérez, 2013). This technique can affect HMA indexes and bitumen properties, as explained in Section 3.3. To fit the objectives of the DCT, the heating time has to be limited to less than 2 hours. The conditioning time used in this study, 1½ hours (90 minutes), will allow the pores and cracks remaining after the Sika Tite-BE treatment to be sealed with bitumen after heat-treatment, and concentrate the effect of heating on RCA particles without damaging the mixture and/or bitumen properties.

3.6 Materials used in the DCT

In this section, a detailed description of the materials used in the DCTs is provided. The material types, sources and properties are presented. These materials consisted of type GP grey cement, ground granulated blast furnace slag (GGBFS), Sikament® NN superplasticiser, and Sika Tite BE. These materials are used to upgrade the RCA strength and improve its water resistance before being used as aggregate in HMA production. The RCAs used in this investigation are shown in Figure 3.8. This type of RCA was derived from C&D waste, so it considered a low-quality RCA compared to that derived from recycling of pure concrete structures.

Figure 3.8: Sample of the RCA used in this study

3.6.1 GP grey cement

Type GP grey cement was used as the primary material to coat the RCAs. The physical and chemical properties of this type of cement are provided by the supplier, BGC CEMENT, and shown in Table 3.1. The particle size distribution (PSD) of this cement has been checked and the results tabulated in Table 3.2. The apparent particle density was also checked in accordance with the AS 1141.7 standard, and the result was 3.145 t/m³ (Standards Australia, 2014).

Table 3.1: Physical and chemical properties of PG grey cement used (BGC CEMENT)

Property	Comment/value
Appearance	Grey powder
Odour	Odourless
pH	Alkaline
Boiling point	Not available
Specific gravity	3.0-3.4

Table 3.2: PSD of PG grey cement used.

Sieve size	% passing
0.6 mm	100
0.3 mm	99.4
0.15 mm	98.3
0.075 mm	96.7

3.6.2 Ground granulated blast furnace slag (GGBFS)

In order to pursue sustainability in the cement coating technique, Ground granulated blast furnace slag (GGBFS) was also used in coating of RCAs. Although the target of sustainable development in cement coating is not easy to achieve, a total replacement of 15% (by weight) of type GP grey cement with GGBFS may support this goal.

The chemical composition information for this type of GGBFS, along with some of its physical and chemical properties, are given in Tables 3.3 and 3.4, respectively, as provided by the manufacturer.

Table 3.3: Chemical composition of GGBFS (BGC CEMENT).

Ingredient	Identification	Content
Quartz	CAS: 14808-60-7, EC: 238-878-4	<1%
Chromium trioxide	CAS: 1333-82-0, EC: 215-607-8	<0.1%
GGBFS	CAS: 65996-69-2	>90%
Calcium sulphate dehydrate	CAS: 10101-41-4, EC: 600-148-1	2 to 5%

Table 3.4: Physical and chemical properties of GGBFS (BGC CEMENT).

Property	Comment/value
Appearance	Fine off-white to grey powder
Odour	Odourless
pH	>10
Solubility (water)	Reacts

3.6.3 Sikament® NN

In the present study, the Sikament® NN high range water reducer (HWR) admixture was added at a fixed percentage of 0.8% to all CSP mixes. According to AS 1478.1, the admixture shall be added to the mix in the way recommended by the manufacturer. According to the material data sheet of the product, it is usually added at a percentage between 500 to 1500 millilitres/100 kg cementitious. Thus, 0.8% of Sikament® NN is compiled with the added percentage specified by the supplier. It was also confirmed that this superplasticiser satisfies all AS 1478.1 requirements for HWR admixtures (Standards Australia, 2000c). The physical and chemical properties of the Sikament® NN superplasticiser are given in Table 3.5, as provided by the manufacturer.

Table 3.5: Physical and chemical properties of Sikament® NN (Sika Australia Pty. Ltd.)

Appearance	Dark brown liquid
Odour	Characteristic
Boiling point	>100 °C
Evaporation rate	As for water
Vapour pressure	18 mm Hg at 20 °C
Specific gravity	1.19–1.23 g/cm^3
pH	8-9 (approximately)

3.6.4 Sika Tite-BE

In the present study, Sika Tite-BE, an acrylic-based bitumen waterproofing membrane manufactured by Sika Australia Pty Ltd, was used to coat the RCAs. A ten-litre sealed plastic container of Sika Tite-BE (Figure 3.9) was purchased from Bunnings warehouse in Cannington, Western Australia. This product was used in this study to coat the coarse RCAs (particles > 4.75 mm).

In DCT1 (coating with CSP and Sika Tite-BE), the product was used to form the second coating to reduce the permeability of RCAs coated with CSP and, hence, improve aggregate-bitumen bonding and durability.

In DCT2 (coating with Sika Tite-BE and heating), the product was used to form the first coating in order to seal the majority of pores and cracks present on the RCA surfaces.

According to the technical data sheet of the product, Sika Tite-BE is suitable for application to regular building materials such as concrete, rendering, masonry, fibre cement, sheet, timber, clay brick, concrete blocks, aerated concrete and asphalt. Furthermore, when Sika Tite-BE is used for waterproofing purposes, it needs to be applied with a minimum of two successive coats (Sika Australia Pty Limited, 2010). The physical and chemical properties of Sika Tite-BE are shown in Table 3.6.

Figure 3.9: Sika Tite-BE 10 litre sealed plastic container

Table 3.6: Physical and chemical properties of Sika Tite-BE (Sika Australia Pty Limited, 2010)

Physical state	Liquid
Colour	Black
Odour	Characteristic
pH	Not available
Density	~1.2 g/cm³ (20 °C)
Boiling point	100 °C
Flash-point	>93.3 °C

3.7 DCT development process

As mentioned previously in this chapter, two DCTs (i.e., DCT1 and DCT2) were developed for the purposes of this investigation. In the following sub-sections, the development process of DCT1 and DCT2 is introduced.

3.7.1 Development process for DCT1

3.7.1.1 Coating RCA with CSP

Lee et al. (2012) studied the performance of HMA made with coarse RCAs coated with cement slag paste (CSP). The authors evaluated three CSP coating thicknesses: 0.25, 0.45, and 0.65 mm, with the results shown in Table 3.7. It can be seen that as the CSP thickness increases, the water absorption increases while LA abrasion decreases. In keeping with this, and according to the criterion of coating RCAs with CSP (Section 3.5.1), the optimal CSP thickness is that which is sufficient to shield the coarse RCA particles without causing clumping. The CSP coating thicknesses evaluated in the present study were 0.05 mm, 0.1 mm, 0.2 mm, and 0.4 mm, respectively.

Table 3.7: Properties of pre-coated RCAs (PCRCA) after Lee et al. (2012)

The properties of PCRCA aggregate.

Properties	Spec.	PCRCA coating			RCA	CS
		0.25 mm	0.45 mm	0.65 mm		
Bulk specific gravity, coarse	–	2.30	2.29	2.27	2.32	2.66
Absorption (%)	–	10.54	11.52	12.30	9.68	1.58
Sodium soundness (five cycles) (%)	12 max	8.05	9.91	11.88	13.02	2.97
L.A. abrasion (%)	40 max	29.92	30.52	31.31	34.68	25.35
Flat and elongated particles (%) 1:3	10 max	5.86	5.48	5.54	5.98	4.66
Particle crushing strength (MPa)	4 min	12.86	11.22	0.27	8.24	15.6

The optimal CSP thickness has to be verified through various measurements. The measurements are evaluated in two steps. In the first step, basic aggregate properties such as the Los Angles abrasion value (LA, %), aggregate crushing value (ACV, %), water absorption rate (WA, %), apparent particle density (Qa, t/m^3), particle density by dry basis (Qd, t/m^3), and particle density by saturated surface dry basis (Qs, t/m^3) were investigated. In the second step, visual inspection was carried out to assess the applicability of CSP coating to site practices.

3.7.1.2 Coating RCAs coated with CSP using Sika Tite-BE

Based on previous research carried out by Lee et al. (2012), it was confirmed that the water absorption rate of RCAs coated with CSP increases as the coating thickness increases. The resistance of asphalt mixtures to moisture-induced damage was also found to decrease as the percentage of RCAs coated with CSP increases. Lee et al. (2012) attributed this degradation in moisture resistance to the high asphalt absorption rate of RCAs after being coated with CSP. It was documented that one of the main mechanisms that accelerate moisture damage in HMA is the loss of adhesion between the aggregate and bitumen (Bhasin, Masad, Little, & Lytton, 2006). This degradation in moisture resistance can, however, be avoided by using a second treatment. The use of the second treatment (i.e., coating with Sika Tite-BE) is expected to reduce absorption of RCAs and improve adhesion (bonding interface) between recycled aggregates and bitumen simultaneously. The concept of using a second treatment to remedy the first treatment's defects is in accordance with Hypothesis 1 introduced in Section 3.2.

According to supplier recommendations, Sika Tite-BE has to be applied in a minimum of two coats (Sika Australia Pty Limited, 2010). On concrete and masonry materials, the prime coat has to be formed by mixing 1 part Sika Tite-BE with 3 parts water. However, on asphalt surfaces, the prime coat should be formed by mixing 1 part Sika Tite-BE and 1 part water. As the RCAs used in this study contained about 7% recycled asphalt pavement (RAP), the primary coat was formed by diluting 1 part Sika Tite-BE in 2 parts water. Only 5% (by weight of dry RCA) of this mixture (1:2 Sika Tite-BE: water) was added to coarse RCA particles and mixed thoroughly for about 2 minutes. After 3 to 4 hours, a second coat was applied. The second coat was formed by mixing 2 parts Sika Tite-BE with 1 part water. Only 3.5% (by weight of dry RCA) of the latter mix was added to the RCAs and mixed thoroughly for about 2 minutes. The RCA coated with CSP and Sika Tite-BE was then kept in a well-ventilated area at Curtin University's Geomechanics Laboratory to allow water to evaporate. The air-dried RCAs samples were then kept in plastic containers for future testing.

3.7.2 Development process for DCT2

3.7.2.1 Coating RCAs with Sika Tite-BE

The process of coating coarse RCAs (particles > 4.75 mm) with Sika Tite-BE is similar to that explained in Section 3.7.1.2. The first coat was formed by mixing 1 part Sika

Tite-BE and two parts water. This mixing ratio was adopted because the RCA used contains about 7% RAP. Then, 5% (by weight of dry RCA) of the Sika Tite-BE/water mixture was added to coarse RCAs and mixed thoroughly for about 2 minutes. Then, after 3-4 hours, another coat was applied. The second coat was formed by mixing 2 parts Sika Tite-BE and 1 part water. The concentration of Sika Tite-BE in the mixture prepared for the second coat was increased to effectively seal the pores and cracks present on the RCA surfaces. The final product was then kept in a well-ventilated area at Curtin University's Geomechanics Laboratory to allow water to evaporate. The coated RCAs samples were then kept in plastic containers for use in future experiments.

3.7.2.2 Heating of RCA-asphalt mix

In order to assess the quality of the Sika Tite-BE coating, some coated RCA particles were investigated under an optical microscope. Although a high coating ratio was achieved, there were some areas appeared without coating, and the coating thickness was variable, as shown in Figure 3.16 in Section 3.10.1 of this chapter. The uncoated areas can provide ways for water to pass between the aggregate particle and binder, which will affect the durability of the asphalt mixture.

In order to avoid such a scenario, it was decided to condition the mix prepared with RCA coated with Sika Tite-BE in an oven maintained at 155 °C for 1½ hours. Previously, (A. R. Pasandín & Pérez, 2013) cured mixes made with RCA coated with bitumen emulsion in an oven for 4 hours at 170 °C. This was to allow the RCAs to absorb bitumen, which could increase the water resistance. However, A. R. Pasandín and Pérez (2013) stated that keeping the asphalt mix in an oven for 4 hours at 170 °C increases the stiffness of the produced mix, which could limit its usage to hot areas only. Nevertheless, Ana R. Pasandín et al. (2015) stated that the 4 hours conditioning time may affect the engineering properties of the asphalt binder.

In order to save time and energy, and to limit these two drawbacks, the conditioning time was reduced to 1½ hours in the present investigation. This is expected to concentrate the generated effect by heating on the RCA particles without affecting the HMA/bitumen properties. During the conditioning time, an absorption mechanism occurred. The bitumen was expected to be less viscous and flow through the remaining uncoated areas on the RCA surfaces. This absorption mechanism was expected to improve the moisture resistance of the mix and increase the bonding strength between binder and double coated recycled concrete aggregates (DCRCAs).

3.8 Tests used for DCT evaluation

Different tests were carried out in order to evaluate the final product after each DCT treatment.

3.8.1 Surface texture evaluation

To assess the quality of the coating treatment, the surface texture of coated and uncoated RCAs was investigated. Uncoated and coated RCAs were subjected to microscopic examination in order to inspect their surface textures. An optical microscope with a digital camera at the Centre for Material Research (CMR) of the Department of Physics, Curtin University of Technology was employed for this purpose. This microscope has the important benefit of digital image capture via image processing software installed on a connected computer. Figure 3.10 shows the optical microscope used for surface texture evaluation.

Figure 3.10: Optical microscope used in this study

3.8.2 Evaluation of uncoated RCA and coated RCA

Uncoated and coated RCAs were evaluated in two steps: 1) basic aggregate properties were investigated, and 2) a visual inspection was conducted to check the applicability of the coating to field practices.

3.8.2.1 Properties of uncoated RCA and coated RCA

In the first step of aggregate evaluation, some basic properties of the coated RCAs were carried out to examine the efficiency of the DCT used. These properties were measured using several standard tests. A detailed list of these properties is given in Table 3.8 with an indication of the Australian standard used to measure each property. Measurement of these properties allows assessment of the effectiveness of RCA coatings by comparing their properties before and after treatment.

Table 3.8: List of tests carried out to evaluate RCAs after DCT

No.	Property name	Standard	Limits
1	Apparent particle density (Qa), t/m³	AS 1141.6.1	-
2	Particle density on a dry basis (Qd), t/m³	AS 1141.6.1	-
3	Particle density on a saturated surface dry basis (Qs), t/m³	AS 1141.6.1	-
4	Water absorption, %	AS 1141.6.1	≤2
5	LA value, %	AS 1141.23	<35
6	Aggregate crushing value (ACV), %	AS 1141.21	-

3.8.2.2 Visual inspection

In the second step of RCA evaluation, a visual inspection was performed to investigate the applicability of the coating to field practices.

3.8.3 Asphalt mixture evaluation

The feasibility of the current DCTs in HMA production cannot be verified only by assessment of some RCA properties and visual inspection post-DCT. It is crucial to evaluate the performance characteristics of asphalt mixtures made with RCAs upgraded through DCT1 and DCT2. Therefore, an extended experimental program was carried out for this purpose. This program aimed to evaluate asphalt mixes made with different percentages of RCAs coated in accordance with the DCT hypotheses, criteria and processes introduced in this chapter. The program consisted of several testing protocols used to characterise the asphalt mixes. Table 3.9 gives a detailed list of these testing protocols along with their Australian and American standard codes. It should be noted that this chapter does not contain the results of these tests, which are presented and discussed in Chapter 5.

Table 3.9: Testing protocols used for HMA evaluation

No.	Test	Standard
1	Bulk density of compacted asphalt mixture	AS 2891.9.2
2	Stability and flow	AS 2891.5
3	Maximum density determination of asphalt	WA 732.2
4	Indirect tensile strength	ASTM D6931
5	Stripping potential of asphalt	AG:PT/T232
6	Indirect tensile stiffness modulus (ITSM)	AS 2891.13.1
7	Deformation resistance of asphalt mixtures	AG:PT/T231
8	Flow number	AASHTO TP 79-13
9	Fatigue life of compacted bituminous mixes	AG:PT/T233
10	Dynamic modulus	AASHTO TP 79-13

3.9 Fabrication of double-coated RCAs

Two DCTs were used in this research. DCT1 is based on using two coating treatments: a CSP coating treatment and a Sika Tite-BE coating treatment. In addition, DCT2 is based on a combination of two treatments: coating with Sika Tite-BE and heating treatment. The double-coated RCAs based on DCT1 will be named DC1, where DC refers to the double coating technique used, and 1 refers to the first type of DCT used. Similarly, the double-coated RCAs based on DCT2 will be named DC2. In the following sub-sections, the fabrication processes of DC1 and DC2 are described.

3.9.1 DC1 fabrication

The first coating layer (CSP layer): It is obvious that assuming values for CSP coating thickness is critical because it relates to the amount of cementitious material needed to achieve the pre-specified thickness (Bayomy, 1983). However, the correct amount of cementitious materials can still be controlled through the practicality of the coating process. In order to simplify the process of calculating the amount of cementitious material required for each size of RCA, the following assumptions are to be considered for ideal circumstances (Figure 3.11):

1. RCA coarse particles of the same size are spheres of diameter *d*.
2. CSP coating thickness is a constant (t).

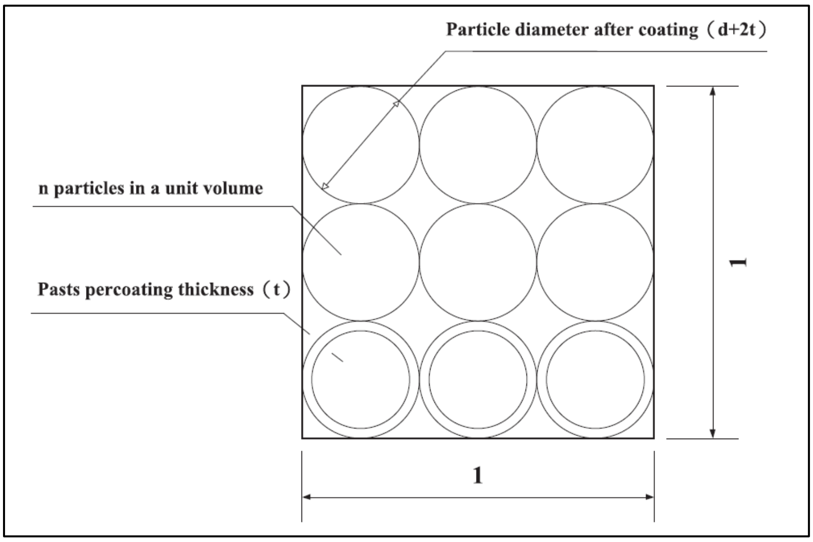


Figure 3.11: Ideal spherical RCAs coated with CSP (Lee et al. 2012)

Taking into account these two assumptions, the amount of coating paste for each RCA size can be computed through the following steps (Lee et al., 2012):

- ✓ Select CSP theoretical thickness, t.
- ✓ Compute the volume of RCA (V_{RCAs}) for each sieve size using Equation 3.1:

$$V_{RCAs} = W_{RCAs}/\gamma_{water} \tag{3.1}$$

- ✓ Compute the surface area of RCA (SA_{RCAs}) for each size of RCA as per Equation 3.2:

$$SA_{RCAs} = V_{RCAs} \times SA_{Representing\ value} \tag{3.2}$$

- ✓ Compute the coating volume of CSP for each size of RCA (Equation 3.3):

$$V_{CSP} = SA_{RCAs} \times t^{th} \tag{3.3}$$

✓ Compute the specific gravity of CSP using Equation 3.4 below:

$$G_{CSP} = \frac{W_{Cement} + W_{GGBFS} + W_{Water}}{\frac{W_{Cement}}{\gamma_{Cement}} + \frac{W_{GGBFS}}{\gamma_{GGBFS}} + \frac{W_{Water}}{\gamma_{Water}}} \tag{3.4}$$

✓ Compute the weight of CSP based on the theoretical thickness (t) using Equation 3.5:

$$W_{CSP} = V_{CSP} \times G_{CSP} \tag{3.5}$$

Where W_{RCAs}= the weight of RCA, and $SA_{Representing\ value}$= the representing surface area of each RCA size introduced in Table 3.10. W_{Cement}, W_{GGBFS}, and W_{Water} are the weights of GP grey cement, GGBFS, and water, respectively. In addition, γ_{Cement}, γ_{GGBFS}, and γ_{Water} are the densities of GP grey cement, GGBFS, and water, respectively.

Table 3.10: SA representing values of each RCA size

RCA particle size (mm)	Average size (mm)	SA representing value*
19 passing-13.2 retained	16.1	372.7
13.2 passing -9.5 retained	11.53	520.4
9.5 passing -6.7 retained	12.85	466.9
6.7 passing -4.75 retained	5.73	1047.1
*SA representing value = 6000/average size (Lee et al., 2012).		

To select the optimal coating CSP that suits the material properties and criteria of DCT, four CSP coating thicknesses were evaluated: 0.05, 0.1, 0.2, and 0.4 mm. The process of coating RCAs with CSP is illustrated in Figure 3.12. Firstly, the water required to bring the coarse RCAs into a saturated surface dry condition (W1) was added and the mixture was mixed for 60 seconds. Then, the mixture of GP grey cement and GGBFS was added to the saturated surface dry RCAs and mixed for 60 seconds. Finally, the second part of water (W2 = 0.45 × cement) was added to obtain fresh RCAs coated with CSP.

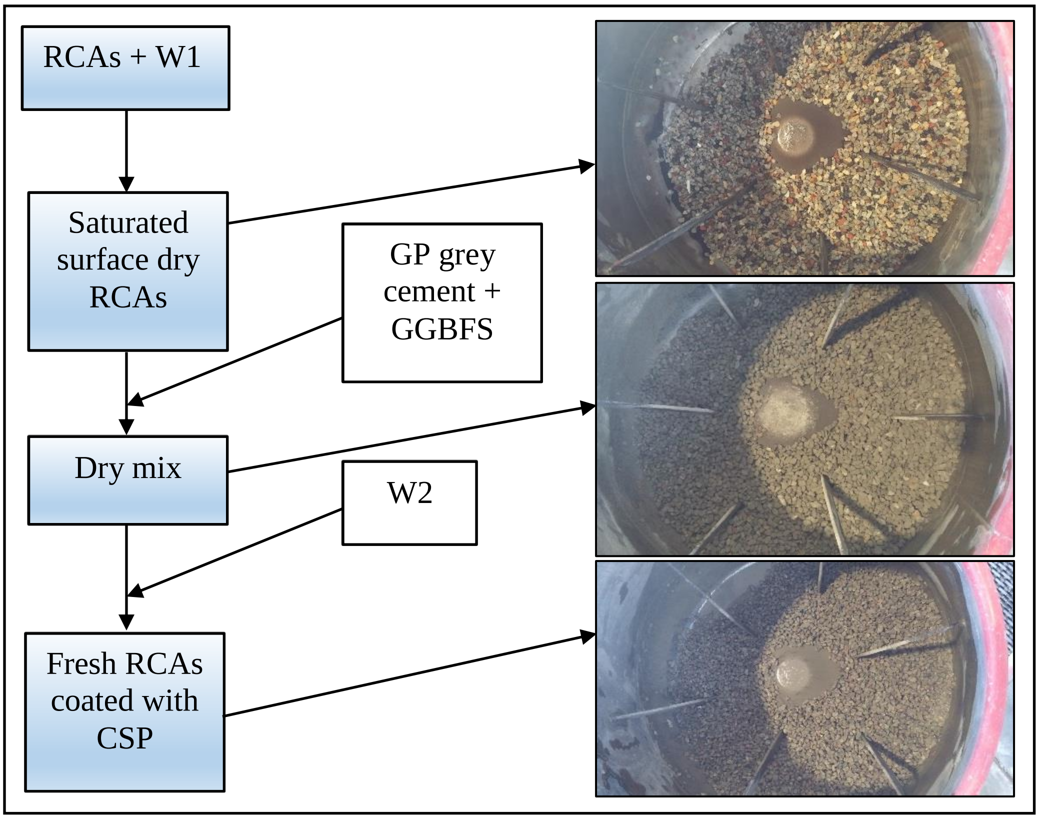


Figure 3.12: Process of coating RCAs with CSP

The final product was then left to set for 24 hours. After 24 hours, these particles were allowed to hydrate in clean water for 7 days. After being hydrated, the coated RCAs were kept in plastic containers for evaluation.

The second coating layer (Sika Tite-BE layer): As per the supplier's recommendation, Sika Tite-BE was applied in two successive coats (Sika Australia Pty Limited, 2010). As explained in Section 3.7.1.2., on concrete/masonry materials, the first coat has to be formed by mixing 1 part Sika Tite-BE with 3 parts water. However, on asphalt/bitumen, the prime coating layer needs to be formed by mixing 1 part Sika Tite-BE with 1 part water. Since the RCAs used contained about 7% RAP, the first coat was formed by mixing 1 part of Sika Tite-BE with 2 parts of water. Some 5% (by weight of dry RCA-coated with CSP) of this mixture (1 Sika Tite-BE/2 water) was added to the RCAs coated with CSP and mixed thoroughly for about 2 minutes.

After 3 to 4 hours, a second coat was applied as recommended by the supplier. The coat was formed using 2 parts Sika Tite-BE and 1 part water. Some 3.5% (by weight of dry RCA-coated with CSP) of this mix was applied to the coated recycled aggregates and mixed thoroughly for about 2 minutes. The concentration of Sika Tite-BE in the second coat was increased to effectively seal the pores present on the RCAs coated with CSP and, thus, the absorptive nature of these particles could be mitigated. The Sika Tite-BE coating layer was expected to reduce water and bitumen absorption by RCAs coated with CSP and upgrade its affinity with bitumen. Figure 3.13 shows the systematic process of coating CSP-coated RCA with Sika Tite-BE.

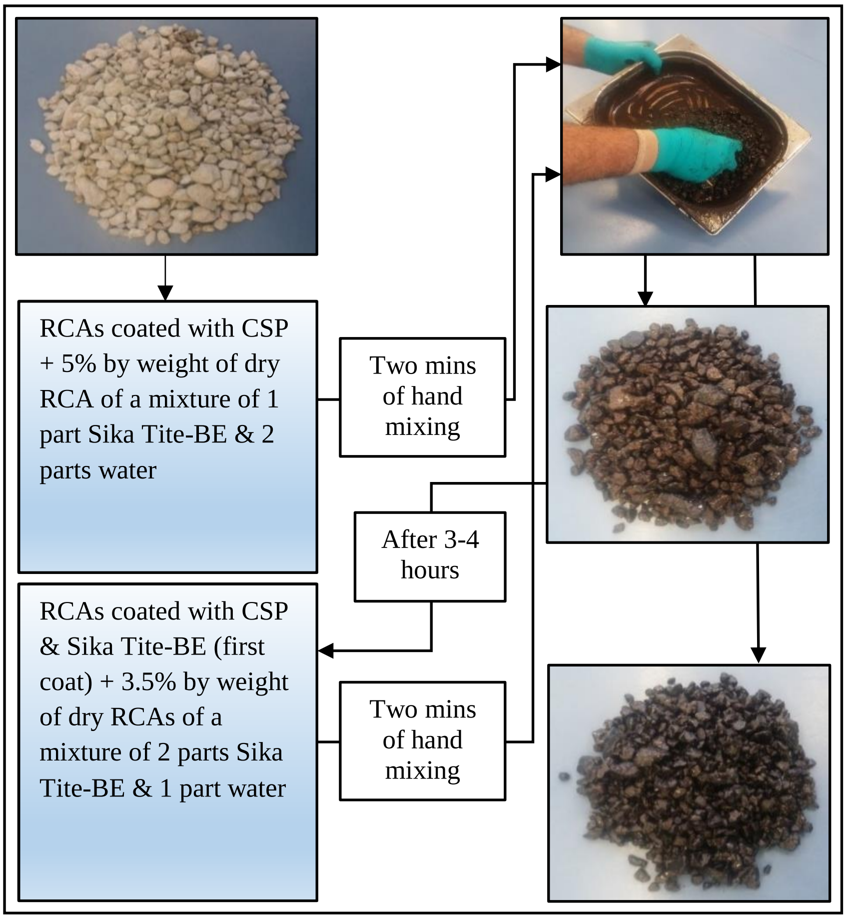


Figure 3.13: Systematic process of coating the CSP-coated RCA with Sika Tite-BE

3.9.2 DC2 fabrication

The first coating layer (Sika Tite-BE layer): the process of coating RCAs with Sika Tite-BE was explained previously in Section 3.7.2.1. The coating process is detailed in Figure 3.14. The first coat was formed by mixing 1 part Sika Tite-BE and 2 parts water, while the second coat was formed by mixing 2 parts Sika Tite-BE and 1 part water. Only 5% by weight of dry RCAs of the former mix was added to recycled aggregate and mixed thoroughly for about 2 minutes. The product was then left for 3-4 hours to set. After that, 3.5% of the mix was added to the RCA and mixed carefully for about 2 minutes.

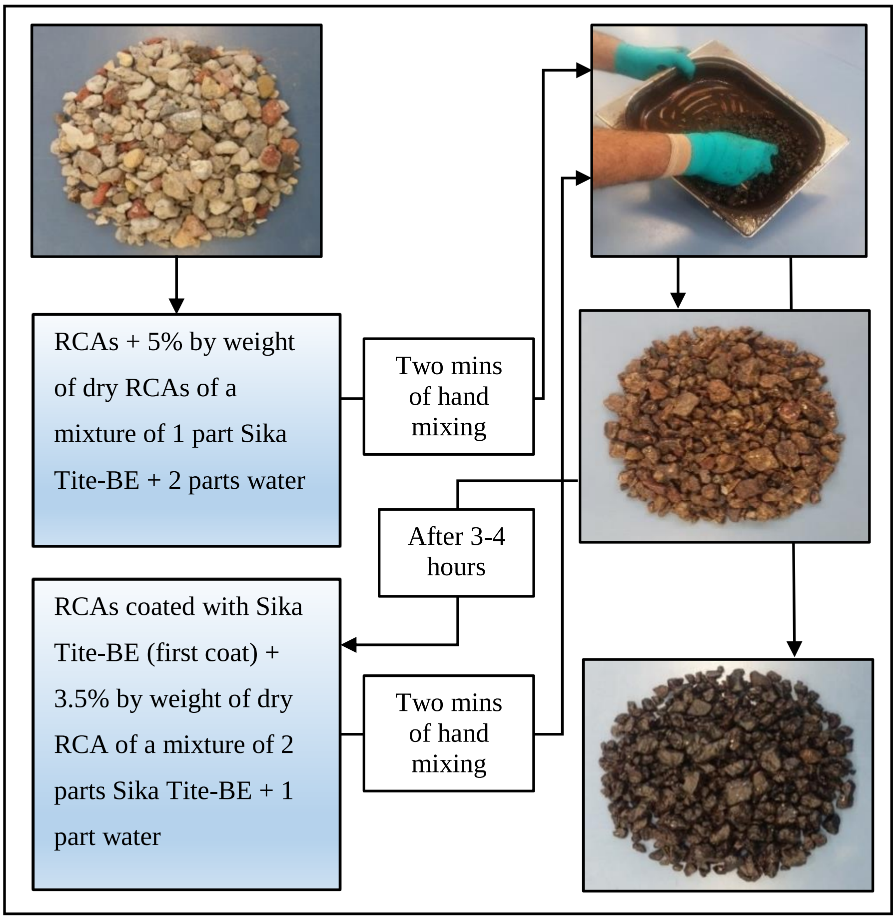


Figure 3.14: Systematic process of coating RCA with Sika Tite-BE

It should be noted that the second coat was applied during heating of the asphalt mixtures in an oven at 155 °C for 1½ hours. The coating occurs via an absorption mechanism. The bitumen is expected to be less viscous at 155 °C and flow through the areas on the RCA surfaces remaining uncoated after the first treatment, as explained in Section 3.7.2.2.

3.10 Results of uncoated and coated RCAs

3.10.1 Surface texture results

To examine the quality of coating with CSP and Sika Tite-BE, and the quality of coating with Sika Tite-BE, a modern optical microscope with a digital camera was used (Figure 3.10). The microscope was used to investigate the RCA surface texture after coating treatment. Figures 3.15, 3.16, 3.17, and 3.18 are magnified images of uncoated RCAs and RCAs coated with CSP, RCAs coated with CSP and Sika Tite-BE, and RCAs coated with Sika Tite-BE, respectively.

As shown in Figure 3.15, the surface texture of uncoated RCA particles was rough, with large pores and cracks. This situation can lead to high water absorption (high permeability), high abrasion (low strength) and poor bonding with bitumen (poor adhesion). After coarse RCAs were coated with CSP (Figure 3.16), the particle surfaces became smoother. This is because the coating paste entered the pores and cracks and sealed them up. However, small pores can be clearly seen on the particle surfaces after CSP coating, as shown in Figure 3.16.

Figure 3.17 shows magnified images of RCAs coated with CSP and Sika Tite-BE. It can be seen that the small pores are sealed with Sika Tite-BE, which can reduce asphalt and water absorption, and improve adhesion between RCA particles and bitumen.

In Figure 3.18, magnified images of RCAs coated with Sika Tite-BE are shown. In these images, uncoated areas can be clearly seen after Sika Tite-BE treatment. These uncoated areas can provide ways for water to enter between the RCA and bitumen, which will degrade the moisture resistance of asphalt mix produced. It was, therefore, decided to allow asphalt mixes made with such particles to cure in an oven at 155 °C for 1½ hours. During heating, the bitumen is expected to be less viscous and, thus, flow through uncoated areas on the RCA surfaces, as explained in Section 3.7.2.2. This would improve the resistance of the mix to moisture and enhance the affinity between the DCRCA and bitumen.

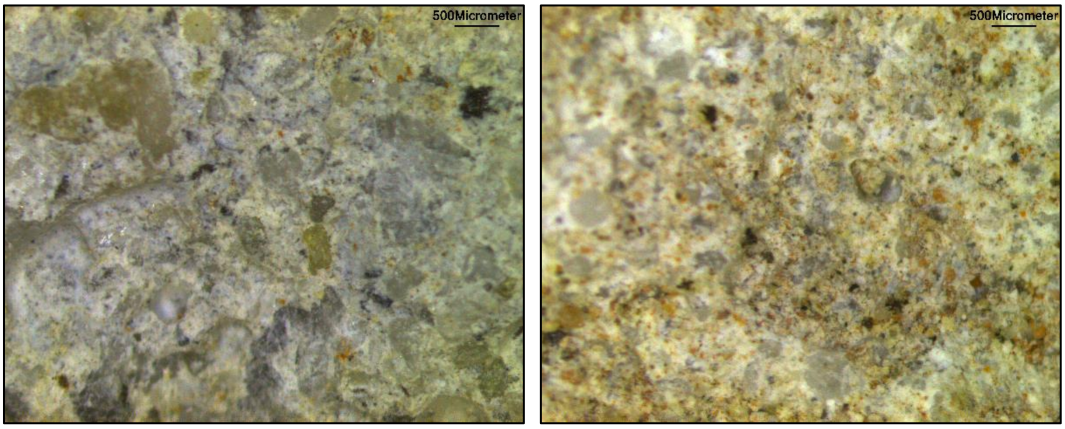


Figure 3.15: Magnified images of uncoated RCA

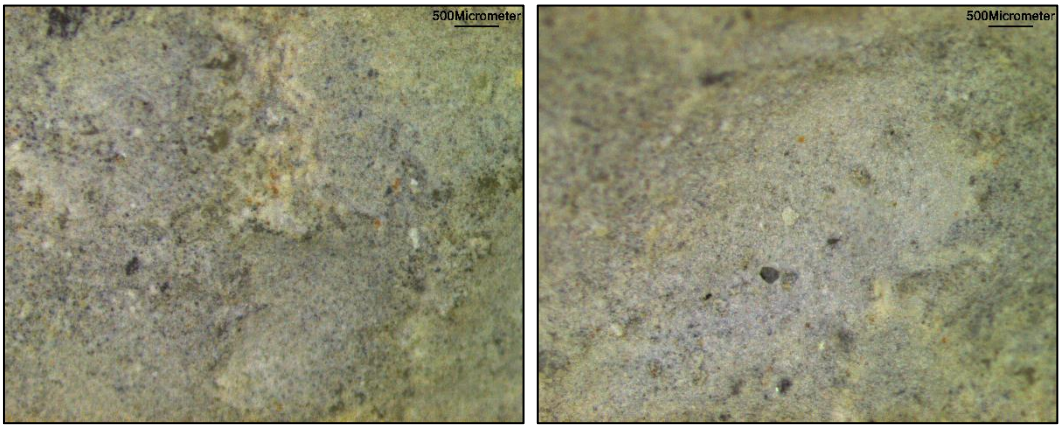


Figure 3.16: Magnified images of RCA coated with CSP

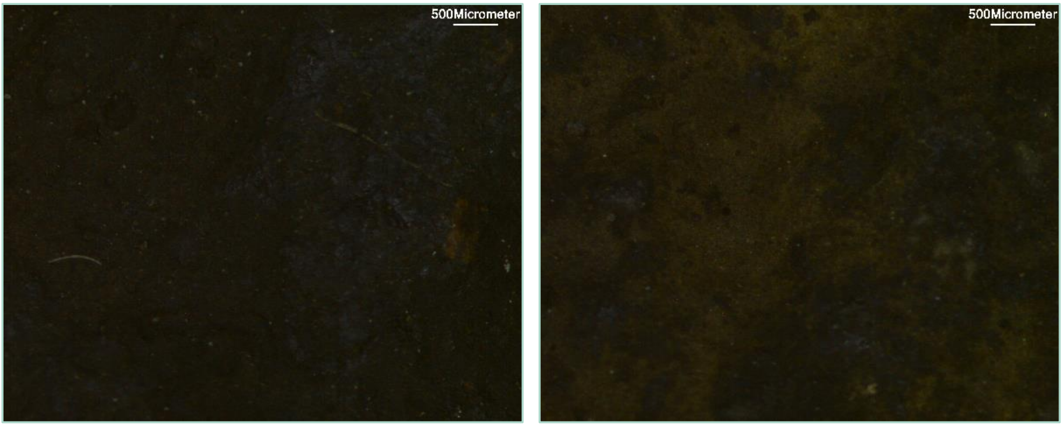


Figure 3.17: Magnified images of DC1 RCA coated with CSP + Sika Tite-BE

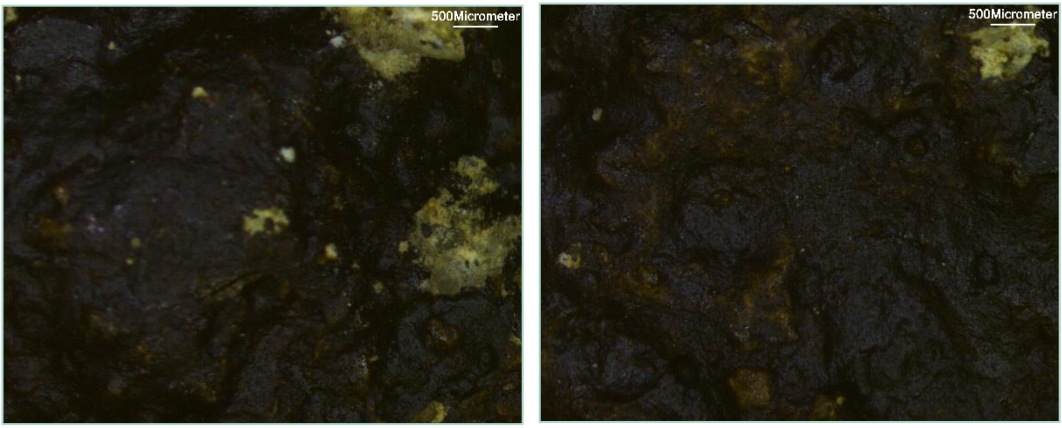


Figure 3.18: Magnified images of RCA coated with Sika Tite-BE

3.10.2 Properties of RCAs after treatment

As mentioned in Section 3.8.3, this chapter will not introduce any results for the RCA-asphalt mixes made in this study. However, this chapter presents results for uncoated and coated RCAs to allow comparison of their properties before and after treatment.

3.10.2.1 Properties of uncoated RCAs

The RCA used in this study was sourced from Capital Recycling, one of the main suppliers of recycled aggregates in the Perth region. This type of RCA is derived from C&D waste, so it is considered as a low-quality recycled aggregate. This RCA consisted of 83.4% crushed recycled concrete, 7.8% high-density brick and tile, and about 7% recycled asphalt pavement (RAP). The remainder was metals, glass, ceramics, plastics, plaster and organic matter. The natural aggregates use in this research was granite aggregates. The granite was sourced from a local quarry in Perth, Western Australia, in fractions of 0/5, 5, 7/10 and 14 mm. More details about these two types of aggregates (granite and RCA) will be provided in Chapter 4. The properties of the RCA and granite aggregates were evaluated in accordance with Australian standards and the results are presented in Table 3.11.

Table 3.11: Basic properties of granite aggregate and recycled concrete aggregate

Course aggregates				
Specification	Property	Granite	RCA	Limit
AS 1141.6.1	Apparent particle density, g/cm³	2.692	2.549	-
	Particle density on a dry basis, g/cm³	2.663	2.230	-
	Particle density on a saturated surface dry basis, g/cm³	2.674	2.355	-
	Water absorption, %	0.4	5.6	≤2
AS 1141.23	LA value, %	24.2	40.7	<35
AS1141.21	Aggregates crushing value (ACV), %	23.9	32.4	
Fine aggregates				
AS 1141.5	Apparent particle density, g/cm³	2.697	2.679	-
	Particle density on a dry basis, g/cm³	2.633	2.256	-
	Particle density on a saturated surface dry basis, g/cm³	2.657	2.414	-
	Water absorption, %	0.6	7.0	≤2

The results indicate that the coarse and fine RCAs had lower densities and higher water absorption than the fine and coarse granite aggregates. The water absorption rates of the coarse and fine RCAs were 14.0 and 11.7 times higher than those of coarse and fine granite aggregates, respectively. The RCAs did not comply with the Los Angeles abrasion coefficient requirement and water absorption limitation (Standards Australia, 2009b). This is due to old cement mortar being attached to the RCA surfaces, along with other impurities such as bricks and tiles. It was, therefore, decided to use only the coarse RCA (particles >4.75 mm) in this research. The following points are the primary reasons for this decision:

- Some lightweight material such as wood and plastics can be removed during a washing process, while others (i.e. metal and gypsum plaster) can be eliminated by visual inspection and, hence, the subsequent removal of impurities is more applicable in coarse fractions (Figure 3.19).
- It would be difficult to remove the impurities present in the fine fractions of recycled aggregates (A. R. Pasandín & Pérez, 2013), as shown in Figure 3.20.
- The water absorption of fine recycled aggregates is higher than that of coarse recycled aggregates, which could affect the durability of the mix produced and increase the amount of bitumen required to achieve the optimum bitumen content (OBC).

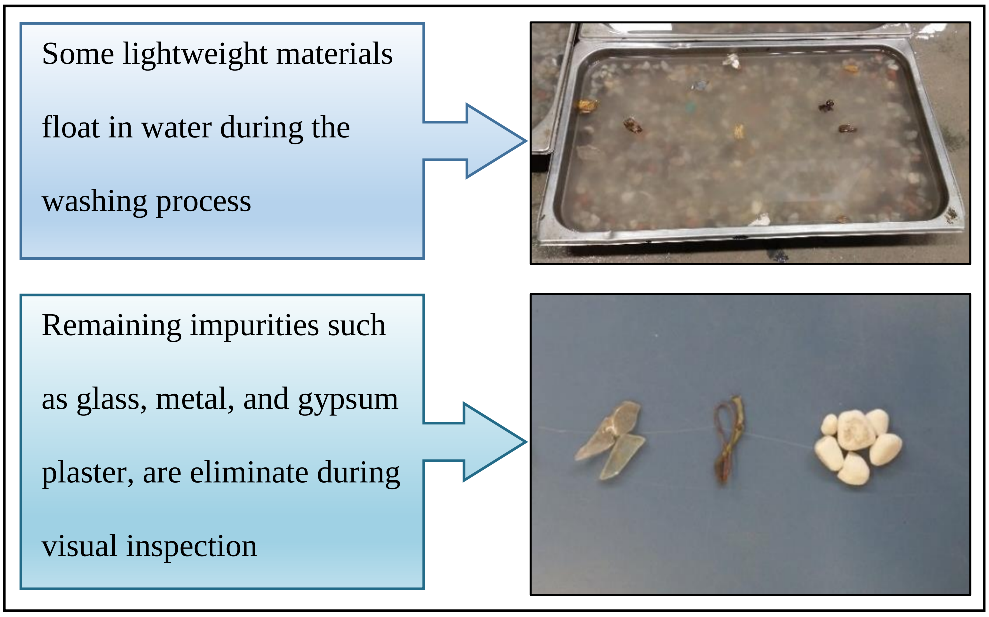


Figure 3.19: Removal of RCA impurities

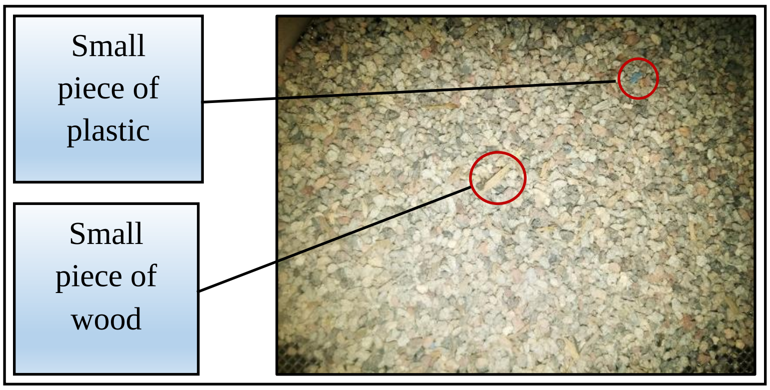


Figure 3.20: Fine impurities retained on a 2.36 mm sieve

3.10.2.2 Properties of coated RCAs

Two techniques were used to upgrade the quality of coarse RCAs in this thesis. The DCT1 method is based on coating coarse RCA with CSP and Sika Tite-BE, while DCT2 is based on coating the coarse RCAs with Sika Tite-BE then heating them. Bothe types of double-coated RCAs (i.e., DC1 and DC2) were evaluated in accordance with Australian standards.

DC1-RCAs coated with CSP & Sika Tite-BE: Table 3.12 shows the properties of RCAs coated with CSP. It was found that as the CSP coating thickness increased, the water absorption rate increased. Therefore, the CSP treatment produces a degrading effect when considering the water and bitumen absorption. The results of LA testing showed an improvement in the abrasion resistance of the RCAs coated with CSP. It can be seen that when the CSP coating thickness increased, the resistance to abrasion increased until 0.2 mm CSP. After that, the LA abrasion resistance decreased. In this regard, at a thinner coating thickness, the pores were filled with CSP, thereby reinforcing weak RCA particles. However, the increase in CSP coating thickness produces a decrease in LA resistance and density and an increase in water absorption, and causes RCA particles to cement to each other.

After Sika Tite-BE treatment, the water absorption rate of the DC1 decreased by 12.3% and 26.1% compared with uncoated RCAs (see Table 3.11) and RCAs coated with 0.1 mm CSP (see Table 3.12). The results also reveal that the Sika Tite-BE treatment slightly improved the aggregate crushing value (ACV) of the DC1, as shown in Table 3.12. This is possibly due to filling the pores in the CSP layer.

Table 3.12: Properties of RCA coated with CSP and the DC1

Property	RCAs coated with CSP				RCAs coated with 0.1 mm CSP & Sika Tite-BE (DC1)	Standard
	CSP coating thickness, mm					
	0.05	0.1	0.2	0.4		
LA abrasion, %	39.3	38.8	37.8	38.5	-	AS1141.23
ACV, %	-	30.7	-	-	30.3	AS1141.21
Qa[a], g/cm³	2.581	2.585	2.581	2.578	2.437	AS1141.6.1
Qd[*], g/cm³	2.203	2.199	2.193	2.152	2.172	
Qs[×], g/cm³	2.349	2.348	2.343	2.317	2.280	
WA[××], %	6.62	6.77	6.84	7.67	5	
[a] = Apparent particle density, [*] = particle density on dry basis, [×] = particle density on saturated surface dry basis, and [××] = water absorption.						

RCAs coated with Sika Tite-BE: In Table 3.13, some of the basic properties of the RCAs coated with Sika Tite-BE are shown. According to the results, the coarse RCA coated with Sika Tite-BE exhibited lower ACV and water absorption than uncoated RCA. It can be seen that the ACV and water absorption rates of RCAs coated with Sika Tite-BE were 10.2% and 23.2% lower than those of untreated RCAs. This strength and durability improvement can be explained in light of sealing the majority of the pores after Sika Tite-BE treatment.

Table 3.13: Properties of RCAs coated with Sika Tite-BE

Property	Value	Standard
Aggregate crushing value (ACV)	29.1	AS1141.21-1997
Apparent particle density (Qa), t/m³	2.435	AS 1141.6.1-2000
Density on dry basis (Qd), t/m³	2.203	
Density on saturated surface dry basis (Qssd), t/m³	2.298	
Water absorption, %	4.3	

3.10.2.3 Visual inspection

The visual inspection showed that the RCA particles passing a 13.2 mm sieve tended to cement to each other after being coated with CSP at a thickness ≥ 0.2 mm, Figure 3.21. Thus, the optimal CSP coating thickness should be < 0.2 mm in order to ensure the applicability of the coating for site practices. The RCAs coated with 0.05 mm CSP had a lower rate of water absorption than those coated with 0.1 mm CSP; however, the RCAs coated with 0.1 mm CSP had a higher resistance to crushing than that coated with 0.05 mm CSP. Based on the results presented in Table 3.12 in conjunction with visual inspection, the CSP coating thickness selected was 0.1 mm.

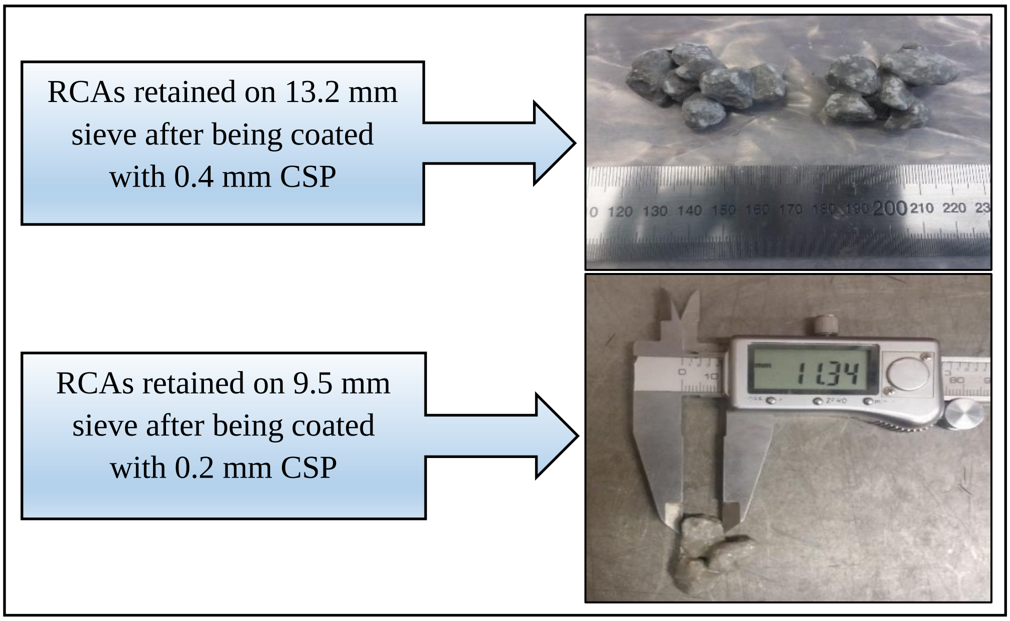


Figure 3.21: Visual inspection of RCAs after coating with 0.2 mm and 0.4 mm CSP

3.11 Improvement of RCA properties after treatment

Before defining the level of improvement achieved after the RCA being coated with CSP, CSP + Sika Tite-BE, and Sika Tite-BE, it is necessary to examine how the addition of RCA affects the properties of granite-RCA mixtures. In order to verify this, the water absorption rate and LA value of different granite-RCA mixtures were evaluated, and the results are presented in Figure 3.22 and Figure 3.23, respectively. It can be seen that both the water absorption rate and LA value increased as more RCAs were added to the aggregate mixtures. Linear regression models were generated using the results collected from the tests. These regression models are shown in conjunction with their coefficients of determination (R^2). Based on the values of R^2 shown in Figure 3.22 (R^2 water absorption = 0.9991) and Figure 3.23 (R^2 LA = 0.9959), it can be concluded that the addition of RCAs into granite-RCAs mixtures linearly affected the permeability and strength of the granite aggregates.

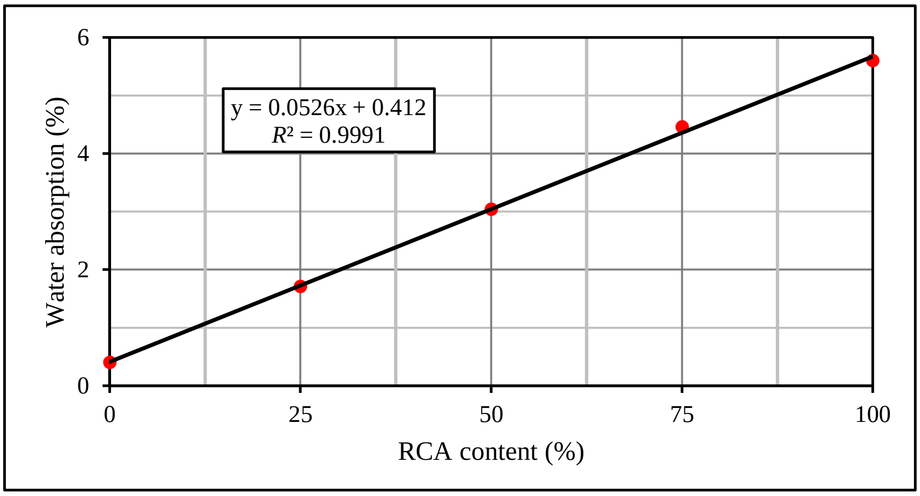


Figure 3.22: Effect of RCA content on water absorption by granite-RCA mixtures

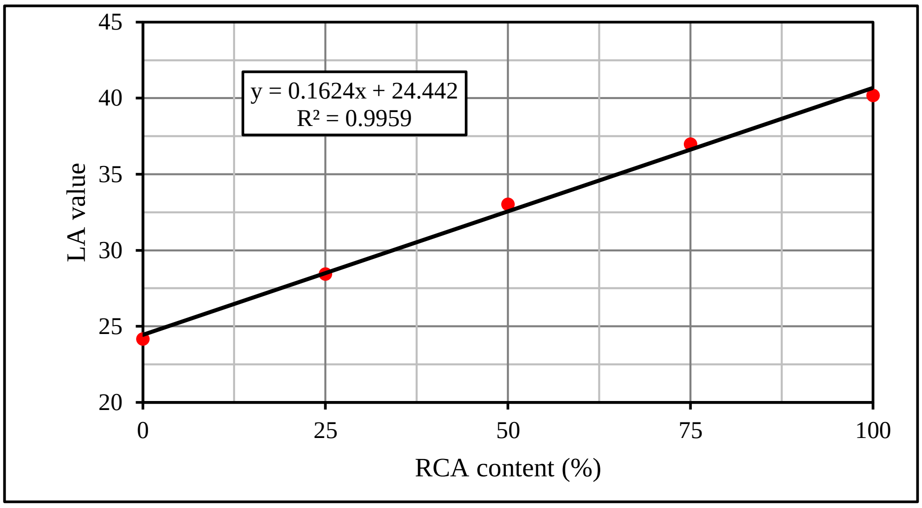


Figure 3.23: Effects of RCA content on the LA value of granite-RCA mixtures

Therefore, the level of improvement in permeability and strength of RCA after treatment can be determined based on the results obtained in Tables 3.11, 3.12, and 3.13. Figure 2.24, Figure 3.25, Figure 3.26 and Figure 3.27 describe this improvement. These figures are drawn assuming that the water absorption rate and ACV of granite aggregates change linearly after the addition of RCAs to the aggregate mix, as verified in Figure 3.22 and Figure 3.23. The zone of improvement of DCT1 is the zone enclosed between the red and green lines shown in Figure 3.24 and Figure 3.25 respectively. In addition, after the RCAs being coated with Sika Tite-BE, the first treatment in the DCT2, the zone of improvement is that enclosed by the black and green lines in Figure 3.26 and Figure 3.27 respectively. Based on these figures, the more RCA is added to the aggregate mixtures, the more degradation in permeability and strength is expected. However, the level of improvement achieved after treatment increased with the content of treated RCA.

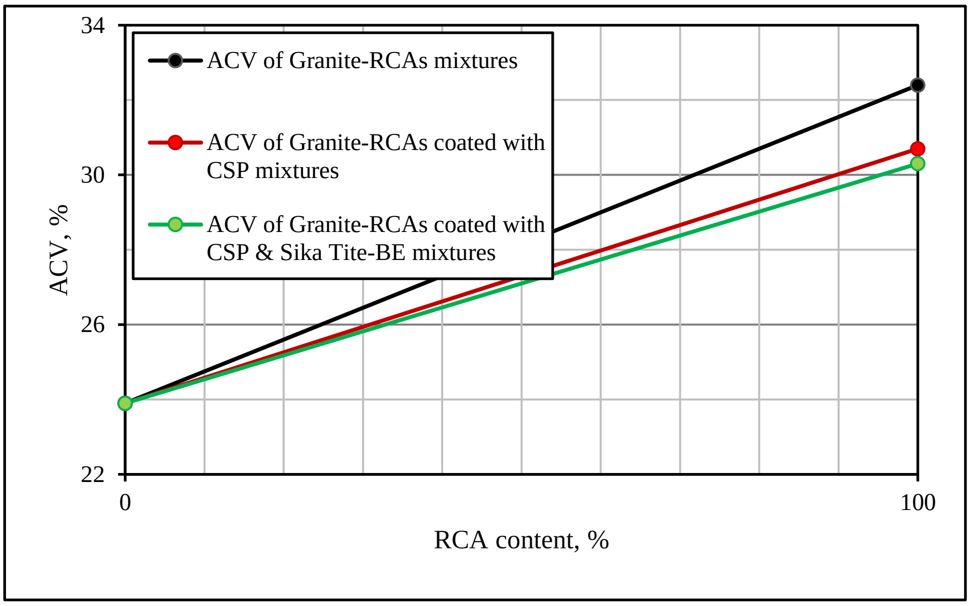


Figure 3.24: Effects of coating RCAs with CSP or CSP + Sika Tite-BE on ACV of granite-RCA mixtures

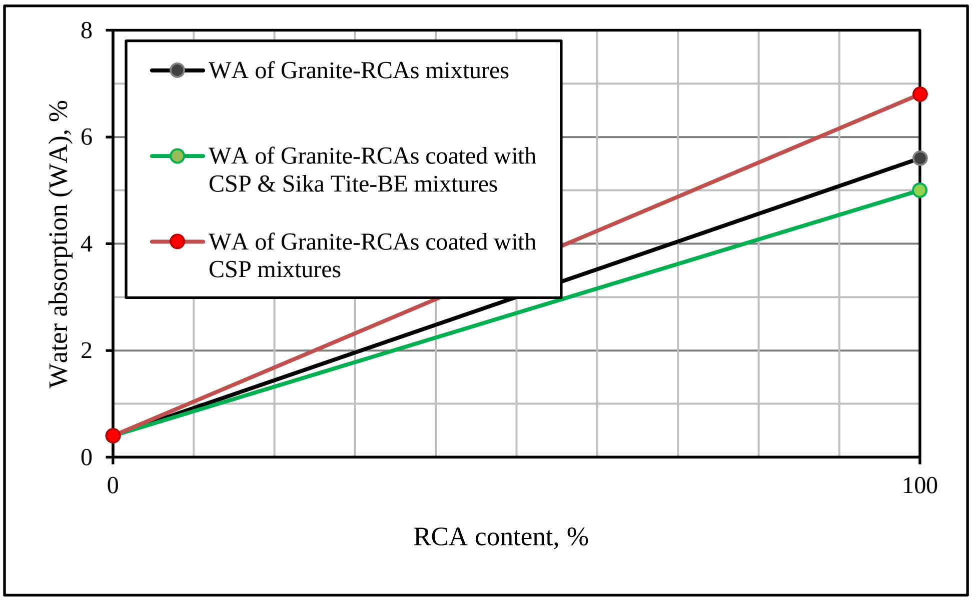


Figure 3.25: Effects of coating RCAs with CSP or CSP + Sika Tite-BE on WA rate of granite-RCA mixtures

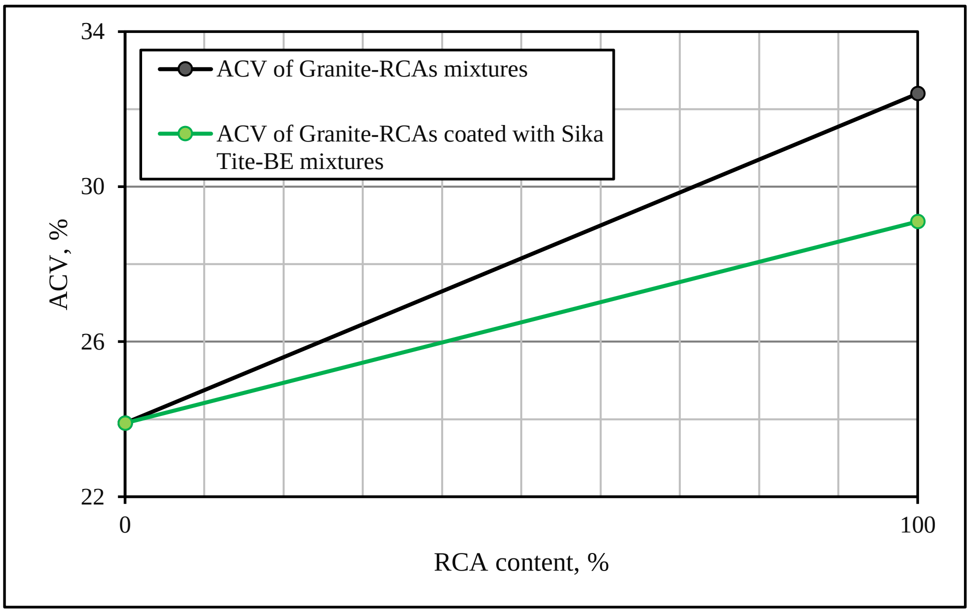


Figure 3.26: Effects of coating RCA with Sika Tite-BE on the ACV of granite-RCA mixtures

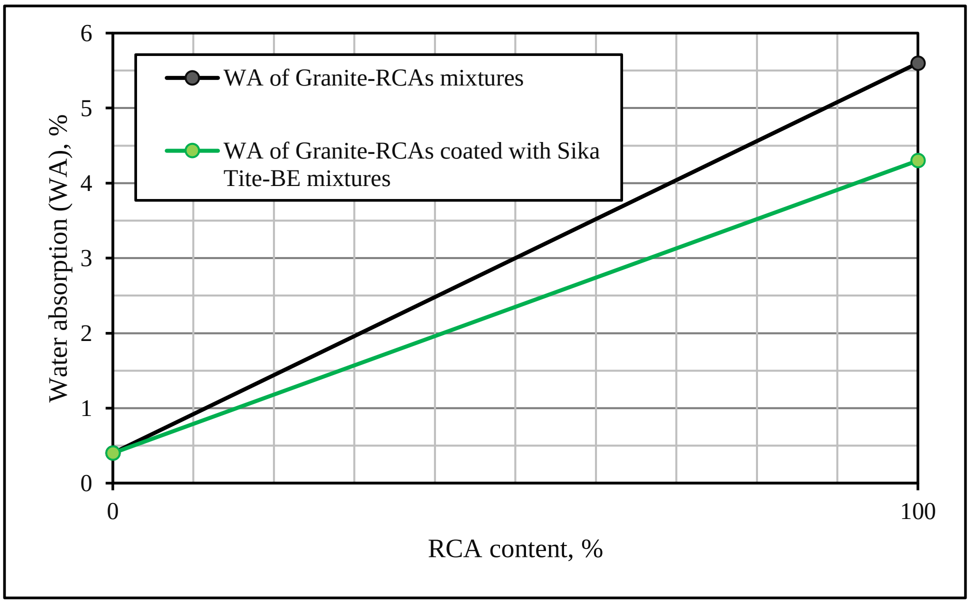


Figure 3.27: Effects of coating RCA with Sika Tite-BE on WA of granite-RCA mixtures

Although the results presented in the above figures are encouraging, a final judgment of DCT1 and DCT2 should only be made after assessing the performance characteristics of asphalt mixtures made with DC1 and DC2. Thus, a comprehensive laboratory program was carried out to assess and characterise the asphalt mixtures made with DC1 and DC2 as will introduced in Chapter 4.

4 Research methodology

4.1 Overview

Currently, the addition of recycled concrete aggregates (RCAs) derived from (construction and demolition) C&D waste to hot-mix asphalt (HMA) is not permitted in Australia. Moreover, only a very limited effort has been done to assess Australian HMA containing RCAs. The objective of the experimental program in this study, therefore, is to evaluate Australian RCA-asphalt mixtures made from local materials and assess them in accordance with Australian standards. A comprehensive laboratory testing program was conducted at Curtin University's Geomechanics Laboratory to evaluate asphalt mixtures made with RCAs derived from C&D waste. Two DCTs were developed for the purposes of this research to upgrade the quality of RCA-asphalt mixtures. In this chapter, a full description of the research methodology used is presented. The first part of this chapter gives an overview of the chapter, the second part describes the materials, while the third part explains the mix design process. Furthermore, the fourth part describes the sample preparation processes by introducing the mixing, quartering, conditioning and compacting methods. The last part presents the testing protocols used to evaluate asphalt mixtures made with granite aggregates, RCAs and two types of double-coated RCAs (i.e., DC1 and DC2). Figure 4.1 illustrates the research methodology workflow used in this thesis.

4.2 Materials for testing

The HMA used in this study was made to contain four basic materials: aggregate (granite, RCAs, DC1, and DC2), C320 bitumen, natural granite dust, and 1.5% hydrated lime. In this part of the chapter, the materials used, their properties and the standards used to characterise them are introduced.

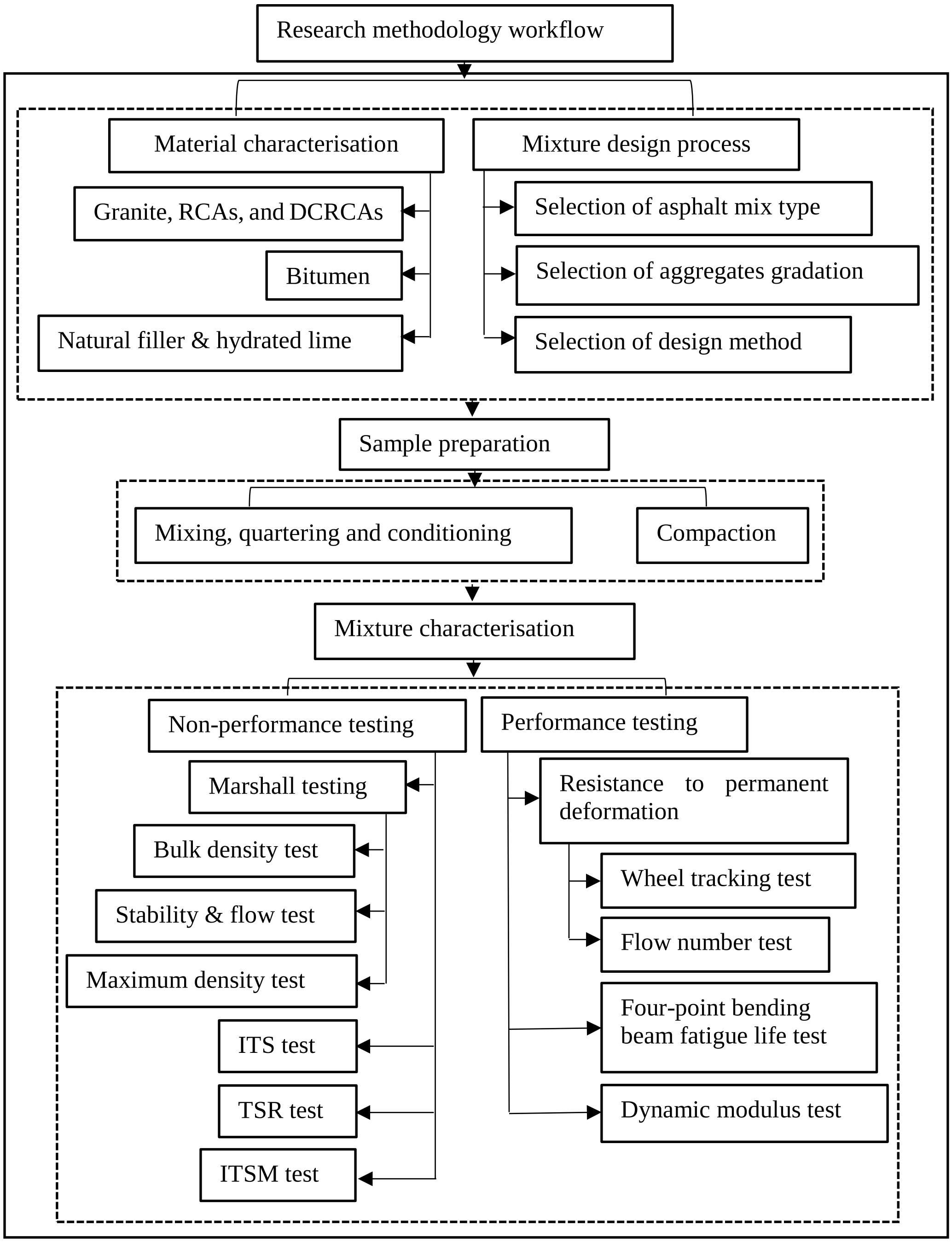


Figure 4.1: Research methodology workflow

4.2.1 Aggregate properties

4.2.1.1 Granite aggregates

Crushed granite sourced from a local quarry in Western Australia in fractions of 0-5, 5, 7, 10-7 and 14 mm was used as control aggregate. The granite aggregates were first dried at 105 °C until a constant mass was achieved. Then, aggregates from each fraction were sieved according to AS 1152 (Standards Australia, 1993). Later, the material retained on each sieve was washed and allowed to dry overnight at 105 °C in an oven. When a constant mass was reached, the aggregates were allowed to cool at room temperature and each fraction was stored separately in plastic buckets.

The particle density and water absorption of coarse and fine aggregates fraction were checked according to Australian Standards AS 1141.6 (Standards Australia, 2000b) and AS 1141.5 (Standards Australia, 2000a), respectively. Also, the Los Angeles (LA) abrasion values and aggregate crushing values (ACV) for coarse fractions were determined in accordance with AS 1141.23 (Standards Australia, 2009a) and AS 1141.21 (Standards Australia, 1997) respectively. The ACV can indicate the resistance of RCAs to crushing under a gradually applied compressive load. In order to be used in asphalt mixtures, the coarse and fine aggregates must comply with the limitations of AS 1141.6.1 and AS 1141.5; as the water absorption rate must be $\leq$ 2%. In addition, the LA abrasion value of the aggregates must be $\leq$ 35% to satisfy the requirement of AS 2758.5 (Standards Australia, 2009b) for HMA production. The granite properties were presented in Chapter 3, Table 3.11. It was confirmed that the granite aggregates complied with all Australian standards for use in HMA production.

4.2.1.2 RCAs derived from C&D wastes

The RCA used in this study was sourced from the Capital Company, one of the main producers of recycled aggregates in the metropolitan area of Perth, Western Australia. One of the key objectives of the Capital Company is to collect C&D wastes and separate them into three main categories: road pavement aggregates, drainage aggregates and clean fill sand. The RCA used satisfied the specifications of the Institute of Public Works Engineering Australia (IPWEA) for Class 1 RCAs. This type of RCA consists of a uniformly mixed blend of coarse and fine recycled aggregates resulting from the crushing of recycled concrete and other hard materials derived from C&D wastes (IPWEA, 2016). It also contains other materials such as clay brick, tiles, sand, glass, metals, gypsum plaster, wood and plastics according to the limits specified in Table 4.1. The particle size distribution (PSD) of the RCAs used in this study is shown in Figure 4.2.

Table 4.1: Limits of components of Class 1 RCAs, and the RCAs used in the study.

Material	Class 1	RCAs used
	Max. % by weight	
Crushed recycled concrete	95	83.4
Recycled asphalt pavement	10	7
High-density clay brick and tile	10	7.8
LDMs[a] (plastic, plaster, etc.)	1	0.3
Organic matter (wood, etc.)	0.5	0.1
Unacceptable HDMs[b] (metals, glass, ceramics > 4 mm)	2	1.1
Asbestos and other hazardous materials	0	0
[a] = low-density materials, and [b] = high-density materials		

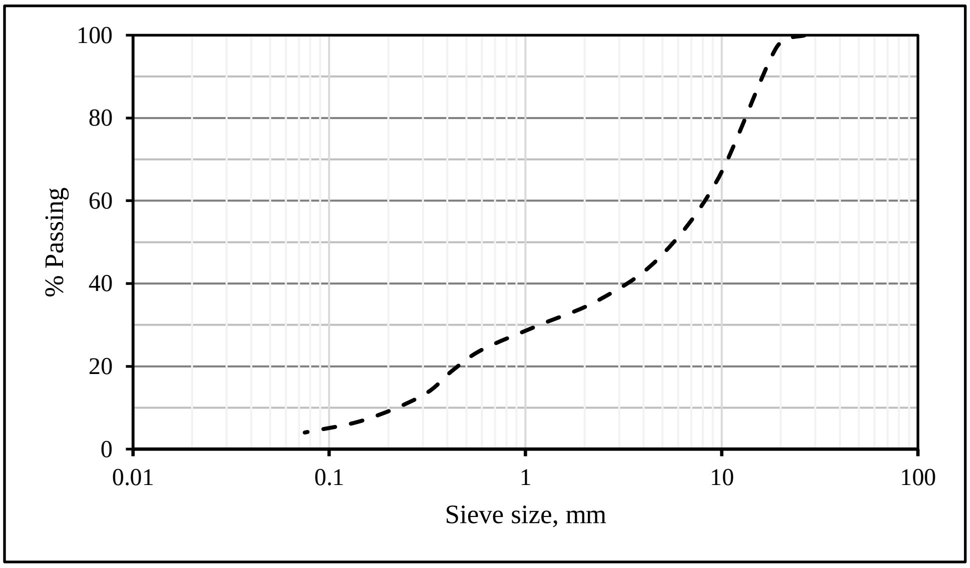


Figure 4.2: PSD of RCAs

4.2.1.2.1 RCAs as HMA aggregates

As explained in Section 3.10.2.1 in Chapter 3, based on the results obtained for RCAs, it was decided to use only coarse fractions (particles retained on a 4.75 mm sieve) in

HMA production. The reason for this decision was to control the amount and type of impurities present in the RCAs, which can affect the durability and cost of RCA-asphalt mixes. In order to obtain better RCA products for HMA, the following steps were performed to purify the RCAs, as shown in Figure 4.3:

1. Fine and coarse RCAs were dried at 105 °C in an oven overnight until a constant mass was reached.
2. The RCAs were then allowed to cool to room temperature after removal from the oven.
3. Then, the dried materials were sieved using a nest of sieves and a shaker until all material passing a 4.75 mm sieve was removed. Also, all impurities (e.g. wood, plastics, gypsum plaster and glass) that remained on larger sieves were removed.
4. Later, each coarse RCA fraction was soaked in water for about 2 hours to remove any fine clay particles attached to the recycled aggregate surfaces.
5. During soaking, all low-density materials and organic matter materials, which float on water, were removed.
6. The remaining materials were washed thoroughly in clean water until the discarded water appeared clean.
7. Then, the washed RCAs were dried at 105 °C overnight.
8. The dried RCAs were allowed to cool to room temperature after removal from the oven.
9. The dried coarse RCAs were stored separately in plastic buckets for later use in testing.

The particle density and water absorption rates of the coarse and fine RCAs were determined according to AS 1141.6.1 (Standards Australia, 2000b) and AS 1141.5

(Standards Australia, 2000a), respectively. The LA abrasion value and ACV were also investigated in accordance with AS 1141.23 (Standards Australia, 2009a) and AS 1141.21 (Standards Australia, 1997), respectively. The results collected from these tests are presented in Chapter 3, Table 3.11. It was concluded that the coarse RCAs had a higher water absorption rate and lower LA abrasion resistance.

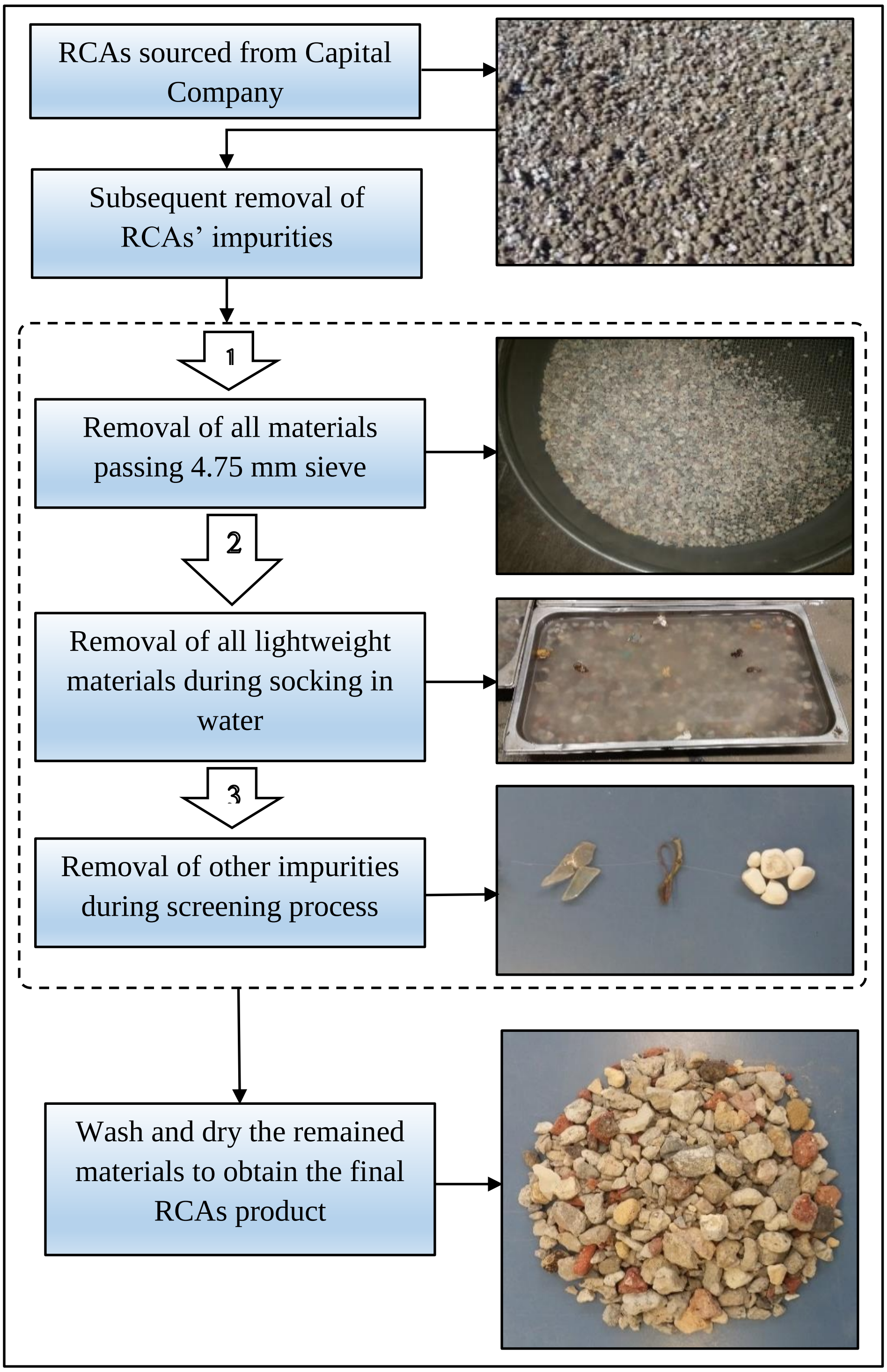


Figure 4.3: Subsequent removal of RCA impurities.

4.2.1.2.2 Maximum percentage of RCA to be used in HMA

As discussed in the previous section, it was verified by the results of the LA abrasion coefficient that the RCAs used had lower abrasion resistance than those of the control aggregates. As a result, the RCAs did not meet requirements of AS 2758.5 (Standards Australia, 2009b) for use in HMA production. It was, therefore, decided that abrasion resistance should be investigated by considering a mixture of granite and recycled aggregate. This was to measure the maximum percentage of RCAs that could be added into the asphalt mixtures in this study. As shown in Figure 4.4, it was found that the substitution of granite aggregate with up to 65% RCA was possible to keep the LA value within the AS 2758.5 limit (≤35%). This means that granite aggregate can be replaced with 0–65% RCA.

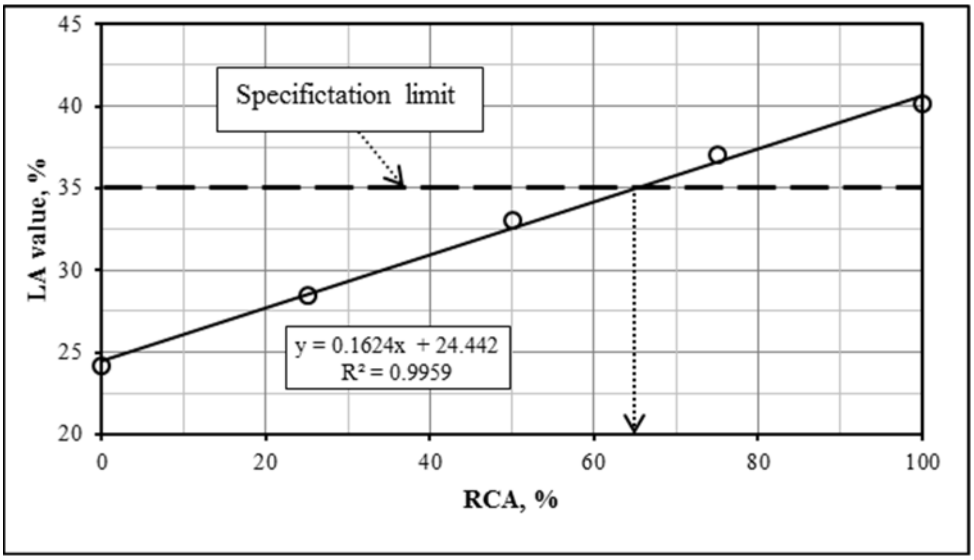


Figure 4.4: Determination of the maximum RCA percentage to be used in HMA

4.2.2 C320 bitumen

Class 320 bitumen binder was used to manufacture all asphalt mixes in this investigation. According to AS 2150, this bitumen is suitable for use in the construction of wearing and base pavement courses subjected to heavy loading conditions (Standards Australia, 2005). The physical and chemical properties of C320 bitumen are given in Table 4.2, as provided by the supplier. The C320 bitumen complied with all Australian standards requirements.

Table 4.2: Physical and chemical properties of C320 bitumen

Tests	Standard	Units	Result
Density, 15 °C	AS 2341.7	kg/L	1.03
Mean shear rate	AS 2341.2	1/s	1.64
Dynamic viscosity, 60 °C	AS 2341.2	Pa.s	320
Viscosity, 135 °C	AS 2341.3	Pa.s	0.547
Penetration, 25 °C	AS 2341.12	0.1 mm	50
Flash point	AS 2341.14	°C	>300
Toluene insoluble	AS 2341.8	%mass	<0.1
Mean shear rate (After RTFOT)	AS 2341.2	1/s	0.75
Dynamic viscosity, 60 °C (After RTFOT)	AS 2341.2	Pa.s	665
Viscosity of residue, 60 °C as % of original	AS 2341.10	%	208

4.2.3 Filler and hydrated lime

The filler in asphalt mixtures should consist of mineral matter such as natural rock dust, granulated blast furnace slag, hydrated lime, cement or other fine materials suitable for this purpose (ASTM, 1995). The aim of adding filler to the HMA is to satisfy gradation requirements and to obtain the desired performance. In this study, natural granite dust, which is normally produced during the crushing of granite aggregates, was used as a filler.

Additionally, it is well documented that the use of hydrated lime in HMA can upgrade its resistance against moisture damage (Airey, Collop, Zoorob, & Elliott, 2008; Behiry, 2013; Gorkem & Sengoz, 2009; Moghadas Nejad, Hamedi, & Azarhoosh, 2012). Keeping with this, according to the Main Roads Western Australia (MRWA) standard, it is recommended to add 1.5% (by mass of dry aggregate) hydrated lime into asphalt mixes (MRWA, 2017b).

Therefore, 1.5% hydrated lime (by weight of dry aggregate) was added to the HMA to improve its resistance to moisture-induced damage. Natural granite dust and 1.5% hydrated lime were mixed together and added to the asphalt mixtures. The particle size distribution (PSD) of the filler is shown in Table 4.3. In addition, the apparent particle density of the filler was determined in accordance with AS 1141.7 (Standards Australia, 2014), and the test result was 2.450 t/m³.

Table 4.3: PSD of natural filler used in this investigation.

Australian standard	Sieve size, mm	% Passing	Limits
AS 2150:2005	0.6 mm	100	100
	0.3 mm	100	95-100
	0.15 mm	100	-
	0.075 mm	98.4	75-100

4.3 Mix design

This section presents the process of designing the hot-mix asphalt mixtures used in this study. The following sub-sections explain the steps of the design procedure: the selection of asphalt mix, the selection of aggregate gradation, the method used for replacement of granite aggregates by RCAs and DCRCAs, and the selection of the design method.

4.3.1 Selection of asphalt mix

A dense HMA type, AC14, was chosen for investigation in the present study. According to AS2150, the AC14 mix is recommended for use in wearing courses for most heavy duty and intermediate applications (Standards Australia, 2005). As mentioned before, Class 320 binder was selected as it is suitable for use in base and wearing courses subjected to heavy loading according to Australian practices (Standards Australia, 2005). As explained in Section 4.2.1.2.2, the maximum percentage of RCAs allowed to be used in asphalt mixtures is 65%. This percentage was verified by the results collected from LA abrasion testing, as introduced in Figure 4.4. Accordingly, in this investigation, asphalt mixtures with 0%, 20%, 40% and 60% of RCA, DC1 and DC2 are to be evaluated.

As mentioned in Chapter 3, the RCA coated based on DCT1 (coating with CSP and Sika Tite-BE) was named DC1, and that coated based on DCT2 (coating with Sika Tite-BE and heating) was named DC2.

Table 4.4 presents the asphalt mix types based on their contents of RCA, DC1 or DC2, and their abbreviations. For instance, "0R mix" refers to an asphalt mix prepared with 100% granite aggregate (control mix), while "40DC1" refers to an asphalt mix made with 40% DC1, and "20DC2" refers to a mix made with 20% DC2.

Table 4.4: Characteristics of AC14 asphalt mixes investigated in this study

RCAs in the mix (%)	Mix type	Abbreviation
0	Control mix	0RCAs
20	20% RCAs	20RCAs
40	40% RCAs	40RCAs
60	60% RCAs	60RCAs
20	20% DC1	20DC1
40	40% DC1	40DC1
60	60% DC1	60DC1
20	20% DC2	20DC2
40	40% DC2	40DC2
60	60% DC2	60DC2

4.3.2 Selection of aggregate gradations

The first step in the preparation of HMA is the selection of the aggregate gradations. Aggregate gradations for bituminous mixtures should be selected to control bitumen content and gradation during HMA production (Birgisson & Ruth, 2001). According to the Australian standard AS 2150, asphalt mixture type AC14 can serve as wearing/intermediate course for heavy traffic conditions (i.e., 75 Marshall blow)

(Standards Australia, 2005). The selected gradation was chosen to comply with the upper and lower limit requirements. As shown in Figure 4.5, gradation curve was located near the centre of the upper and lower standard limitations of AC14 (Standards Australia, 2005).

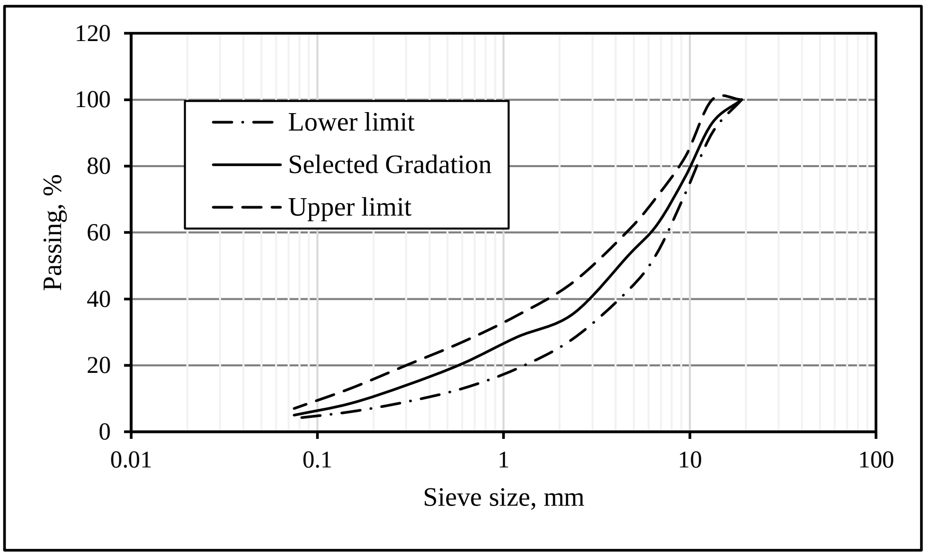


Figure 4.5: Selected aggregate gradation in relation to AC14 limits

4.3.3 Replacement of granite by uncoated RCAs and double-coated RCAs

The RCAs used in this study were derived from C&D waste. Therefore, they had a lower unit weight and higher water absorption than high-quality granite aggregates. Thus, when the replacement of granite aggregate by RCA was done on a weight basis, a greater volume (surface area) of recycled aggregate will be provided. However, when replacement is made on a volume basis, the same surface area of aggregate will be obtained (Lee et al., 2012). The replacement of granite aggregates by RCAs was, therefore, made on a volume basis in this research.

All coarse fractions retained on 13.2 mm, 9.5 mm, 6.7 mm and 4.75 mm sieves were replaced with their equivalent uncoated/double-coated RCA sizes. First, the weight of specified granite aggregates was translated to a volume, then the volume was translated to a weight of recycled aggregate. Table 4.5 gives an example of the aggregate batches required to prepare three Marshall specimens of asphalt mixture made with 0%, 20%, 40% and 60% RCA, respectively. For example, 60% of granite aggregate retained on a 13.2 mm sieve is equal to 144.9 g, see Table 4.5. This weight was converted to a volume using the specific gravity of granite (2.67), and the result was 53.3 cm³. Then, the calculated volume was translated to a weight by multiplying the determined volume by the specific gravity of the RCA (2.23), and the result was 121 g. It can be seen that as the dosage of coarse RCA increases in the mix, the final calculated weight of the aggregate batch decreases. Granite aggregate was replaced by DC1 and DC2 using the same procedure.

Table 4.5: Weight of granite and RCA in a batch of three Marshall specimens

Sieve size, mm	Mix type									
	0RCAs	20RCAs			40RCAs			60RCAs		
	G*	G	RCAs	G+RCA	G	RCAs	G+RCA	G	RCAs	G+RCA
19	0	0	0	0	0	0	0	0	0	0
13.2	241.5	193.2	40.3	233.5	144.9	80.7	225.6	96.6	121.0	217.6
9.5	552.0	441.6	92.2	533.8	331.2	184.4	515.6	220.8	276.6	497.4
6.7	500.3	400.2	83.6	483.8	300.2	167.1	467.3	200.1	250.7	450.8
4.75	310.5	248.4	51.9	300.3	186.3	103.7	290.0	124.2	155.6	279.8
2.36	621.0	0	0	621.0	0	0	621.0	0	0	621.0
1.18	241.5	0	0	241.5	0	0	241.5	0	0	241.5
0.6	276.0	0	0	276.0	0	0	276.0	0	0	276.0
0.3	224.3	0	0	224.3	0	0	224.3	0	0	224.3
0.15	189.8	0	0	189.8	0	0	189.8	0	0	189.8
0.075	120.8	0	0	120.8	0	0	120.8	0	0	120.8
Pan	172.5ˣ	0	0	172.5ˣ	0	0	172.5ˣ	0	0	172.5ˣ
Total	3450	1283.4	268.0	3397.1	962.6	536.0	3344.3	641.7	803.9	3291.4

* = Granite, ˣ = 51.75 g (weight of hydrated lime) + 120.75 g (weight of natural dust) = 172.5 g

In this study, different asphalt performance tests were performed to evaluate the behaviour of HMA made with RCAs. Each test required a different asphalt sample. Therefore, different aggregate batches were prepared for each test. For instance, three batches of approximately 7 kg each were required to make one fatigue slab. Additionally, two batches of about 6 kg each were required to manufacture one slab for the wheel tracking test, while an approximately 6.5 kg batch was required to make one sample for dynamic modulus and flow number tests.

4.3.4 Selection of design method

The main objective of the mix design method is to achieve the required balance between the materials used in the mix; i.e., aggregate, bitumen and filler. In this study, the Marshall method of mix design was used to determine the optimum bitumen content (OBC) of the asphalt mixtures.

The target of the Marshall procedure is to obtain the OBC for a combination of aggregates, mineral filler and asphalt binder. To determine the OBC, the Marshall procedure considers the strength of the mix (stability), its deformation (flow), and the density and volumetric properties of asphalt specimens compacted in the laboratory or cored from field-compacted pavement. According to AS 2891.5, the Marshall procedure is only applicable to HMA containing aggregates with a nominal size of 20 mm or less (Australian/New Zealand Standards, 2015).

4.4 Sample preparation

The asphalt mixtures made in this study were designed using available local materials in accordance with acceptable Australian practices. Asphalt mix type AC14 with a nominal maximum size of aggregates of 14 mm prepared with C320 was designed and evaluated throughout the present study. Asphalt samples were made with 0%, 20%, 40%, and 60% proportions of RCA, DC1 or DC2 for the purpose of HMA characterisation. The materials used in the preparation of the asphalt samples were preheated, mixed, conditioned and compacted as per Australian standards. The following sub-sections describe the related experimental procedures carried out according to relevant Australian standards for asphalt mixtures preparation in this research.

4.4.1 Mixing, quartering and conditioning of asphalt mixtures

In order to prepare the mineral components and bitumen of an asphalt mix for mixing, first, the required amount of fine aggregate and filler were mixed together, while the coarse RCA/DC1/DC2 were mixed with coarse granite aggregate (Figure 4.6). Then, the coarse and fine fractions were mixed together and placed in an oven maintained at 175 ± 5 °C overnight. The bitumen was heated separately in an oven maintained at 155 °C until it reached the required mixing temperature (150 ± 5 °C). The bitumen should be heated to the mixing temperature only once and should be used within 4 hours (Australian/New Zealand Standards, 2014b). The binder remaining after the completion of the preparation of asphalt mixes should be discarded. This is crucial, especially when a bituminous mixture is made for performance testing such as rutting, fatigue and dynamic modulus.

The mixing bowl and beater, trays and spatulas were also heated at 150 ± 5 °C to control the loss of temperature during mixing and quartering. To proceed with mixing, the preheated aggregates were put in a mixing bowl and a crater was formed. Immediately, the required amount of C320 binder was added to the components. Before using the mechanical mixer, the mixture was given about 30 seconds of hand mixing. Then, the components were mixed thoroughly in a mechanical mixer for about one minute until a uniform bitumen film formed around the aggregate particles. The whole process of mixing (and quartering if required), as shown in Figure 4.7, should not exceed 3 minutes (Australian/New Zealand Standards, 2014b).

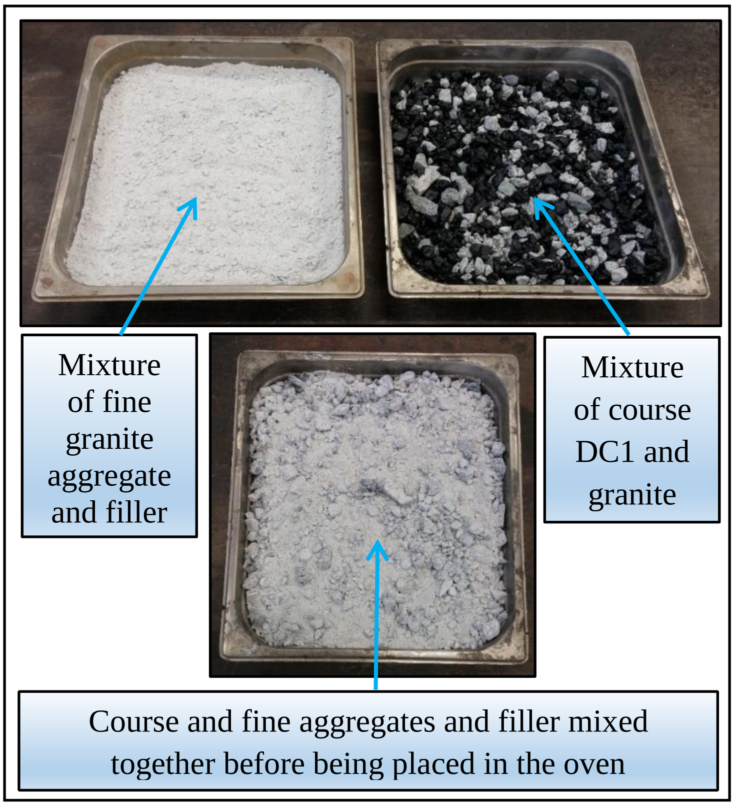


Figure 4.6: Mixing of mineral components before preheating in the oven

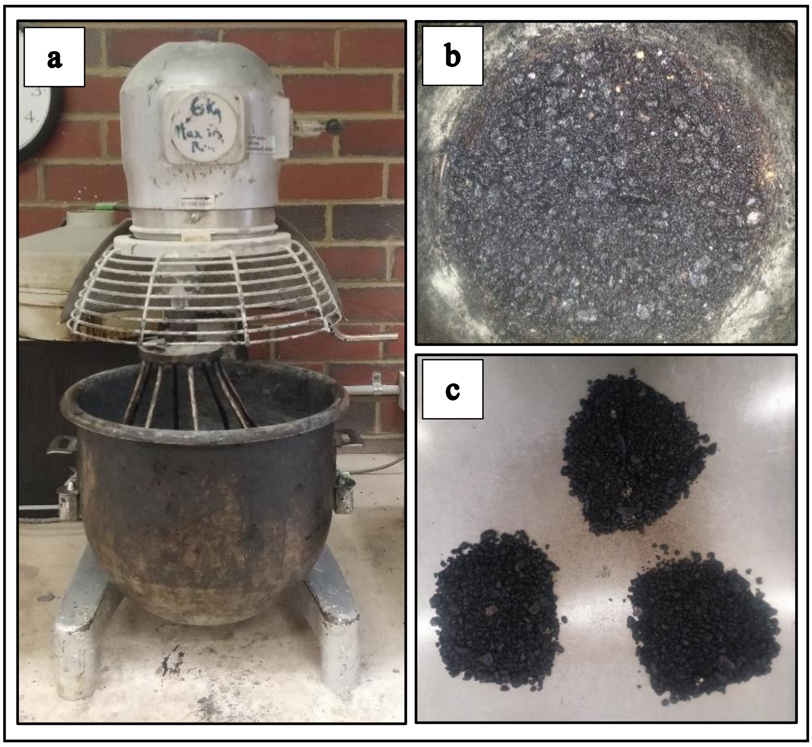


Figure 4.7: Mixing and quartering of asphalt mixture: (a) mechanical mixer, (b) asphalt mixture, and (c) quartering of asphalt

Once the mixing was complete, the asphalt mixture was placed on a quartering tray to obtain a representative portion of asphalt. Quartering may be neglected when large asphalt samples are to be made, such as those prepared for dynamic modulus samples. The temperature of the mix needs to be checked and should be within 10 °C of the conditioning temperature (150 ± 5°C) mentioned in the Australian Standards (Australian/New Zealand Standards, 2014b). Then, each mix portion was placed in a preheated tray and placed in a 150 ± 5 °C oven for 60 ± 10 min. It should be mentioned that the conditioning time of asphalt mixtures made with DC2 was 90 ± 10 min, where a further 30 minutes was added to allow time for uncoated areas to be sealed after bitumen absorption as explained in Section 3.4.2 in Chapter 3.

4.4.2 Compaction of samples

Based on the Australian Standards requirements, the compaction of asphalt samples prepared with class 320 bitumen should be carried out at 150 ± 5°C (Australian/New Zealand Standards, 2014b). Three methods of compaction were used to make five types of asphalt samples, as shown in Figure 4.8.

The Marshall compaction method was used to compact asphalt samples for the Marshall method of mix design. In addition, the gyratory compaction method was used for compacting asphalt samples for indirect tensile strength testing, tensile strength ratio (TSR) testing, indirect tensile stiffness modulus (ITSM) testing, dynamic modulus testing and flow number testing. Furthermore, the slab compaction method was used to produce slabs for rutting and fatigue life tests.

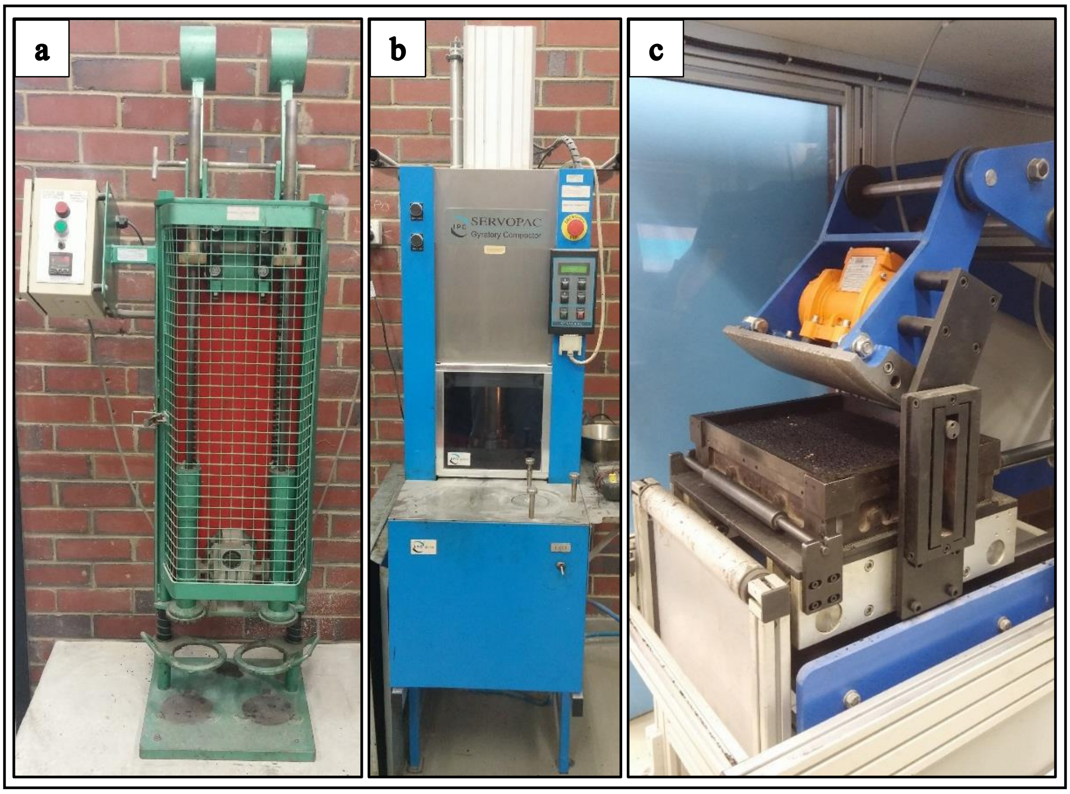


Figure 4.8: Methods of compaction: (a) Marshall method, (b) gyratory method and (c) slab roller compactor

4.4.2.1 Marshall mechanical compaction

The Marshall-compacted samples in this study were used in the determination of bulk density, stability and flow, and maximum density and void analyses. To compact a Marshall specimen, a paper disc is placed at the bottom of a Marshall mould. Then, a portion of asphalt conditioned at 150 ± 5 °C is poured into the mould. Immediately after, the mixture is compacted using a steel rod 15 times around the perimeter and 10 times in the centre. Another paper disc is placed on the top of the mould and the specimen is positioned on the compaction pedestals and hammered using an automated Marshall compacter (Figure 4.8 a) by applying 75 blow/face. The dimensions of the compacted specimen should be 100 mm in diameter and about 65 mm in height. The height of the specimen must be checked and any specimen having a height greater than 70 mm or less than 57 mm should be discarded. The specimen was then allowed to cool down so no deformation could occur upon extraction from the mould. Figure 4.9 shows asphalt specimens compacted using a Marshall mechanical compacter after being extracted from the mould.

Figure 4.9: Compacted Marshall specimens

4.4.2.2 Gyratory compaction

Gyratory-compacted samples were used in the determination of indirect tensile stiffness modulus, indirect tensile strength, tensile strength ratio, dynamic modulus and phase angle, and flow number. These specimens were compacted by means of a Servopac gyratory compactor, as shown in Figure 4.8 (b). The Servopac machine is designed to compact asphalt specimens of 100 mm and 150 mm diameter to a specific air void contents. During the gyratory compaction process, the asphalt sample was subjected to shearing forces like those experienced in the field under a roller compactor.

The gyratory compaction machine should be capable of applying vertical loads of 240 kPa at a rate of 60 ± 5 rev/min (Australian/New Zealand Standards, 2014a). Compaction can be conducted under a fixed gyratory angle between 0° and 3° measured at the centre of the specimen. The specimen is either compacted to a predetermined height or predetermined number of gyrations. The gyratory compactor is equipped with a revolution counter, a force measuring device and sensors so that it can stop applying pressure at a predetermined number of revolutions or when a predetermined height is achieved (Australian/New Zealand Standards, 2014a). Figure 4.10 shows two gyratory specimens made in this study.

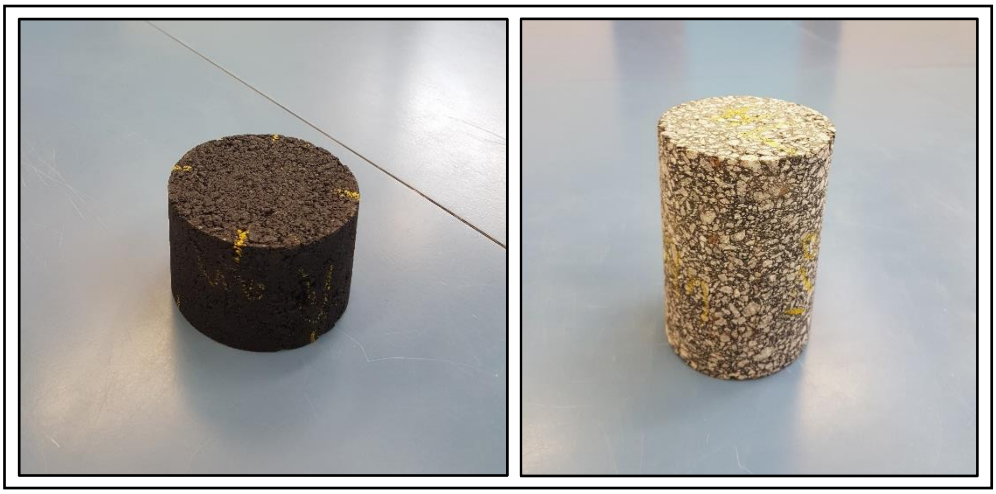


Figure 4.10: Gyratory-compacted specimens

4.4.2.3 Slab compaction

In order to prepare slabs for the wheel tracker test (rutting test) or for the flexural bending fatigue life test, a Cooper roller slab compactor was used (Figure 4.8 c). The slabs were made in accordance with Austroads Standard AG:PT/T220 (Austroads, 2005).

To manufacture a slab for rutting and fatigue testing, a sufficient weight of HMA mixture was put into the specified mould and uniformly levelled to avoid segregation (for rutting, a 300 × 300 × 50 mm slab mould was used; while for fatigue life testing, a 400 × 305 × 75 mm slab mould was used). The aim of compaction was to achieve an air void content of 5 ± 1% (Austroads, 2006a) or 5 ± 0.5% (Austroads, 2006b) for rutting and fatigue testing, respectively. At the beginning, the weight of HMA poured into the slab mould was determined depending on the maximum density of asphalt mixture and desired air void contents. This weight, however, had to be adjusted by trial and error until the required content of air voids was achieved.

After HMA was poured in the mould and levelled, the mould setup was placed in the Cooper compactor which was adjusted to manage that specific type of slab mould. Then, different loadings were applied so that the target height of the slab was achieved. On completion of compaction, the prepared slab was allowed to cool to the ambient temperature before being removed from the slab mould.

The slab for rutting test was marked with the direction of compaction so that the tests proceeded in the same direction. The slab prepared for fatigue life testing was required to be cut down into three beams (Figure 4.11). The bulk density and air void contents of then beams had to be checked. If the air void content did not fit with that specified for a rutting/fatigue life specimen, the specimen was rejected and the weight of the HMA poured into the mould was altered accordingly. The specimens (slabs/beams) satisfy with the air content requirements were stored on a flat surface and allowed to dry for four days in a well-ventilated area until a constant mass was achieved. The specimens should be stored at an ambient temperature of less than 30 °C until testing. Furthermore, the tests should be completed within 30 days of compaction. (Austroads, 2006a).

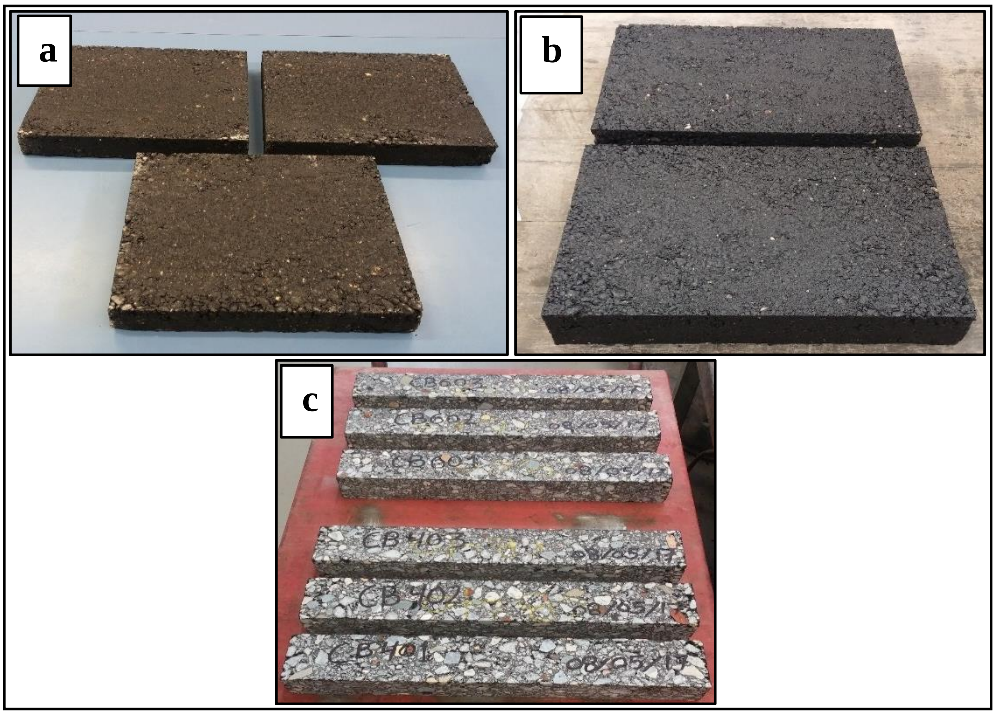


Figure 4.11: Prepared samples of (a) slabs for rutting testing, (b) slabs for fatigue testing and (c) beams for fatigue testing

4.5 HMA characterisation

For characterisation of the AC14 mixtures made in this study, different non-performance and performance tests were performed at Curtin University's Geomechanics Laboratory.

Figure 4.12 shows a flow chart of the asphalt mixture characterisation process. The figure mentions the types of mixtures investigated in conjunction with the outcomes of the investigation after each step of the experimental program. As shown in the figure, the characterisation of asphalt mixtures was conducted in three stages. The first stage was to determine the OBC. In the second stage, different tests were carried out to evaluate the indirect tensile strength, TSR and resilient modulus properties. Finally, the permanent deformation, fatigue and dynamic modulus/complex modulus characteristics were examined through wheel tracking, flow number, four-point fatigue life and dynamic modulus tests. In the following sub-sections, a brief description of each test is made.

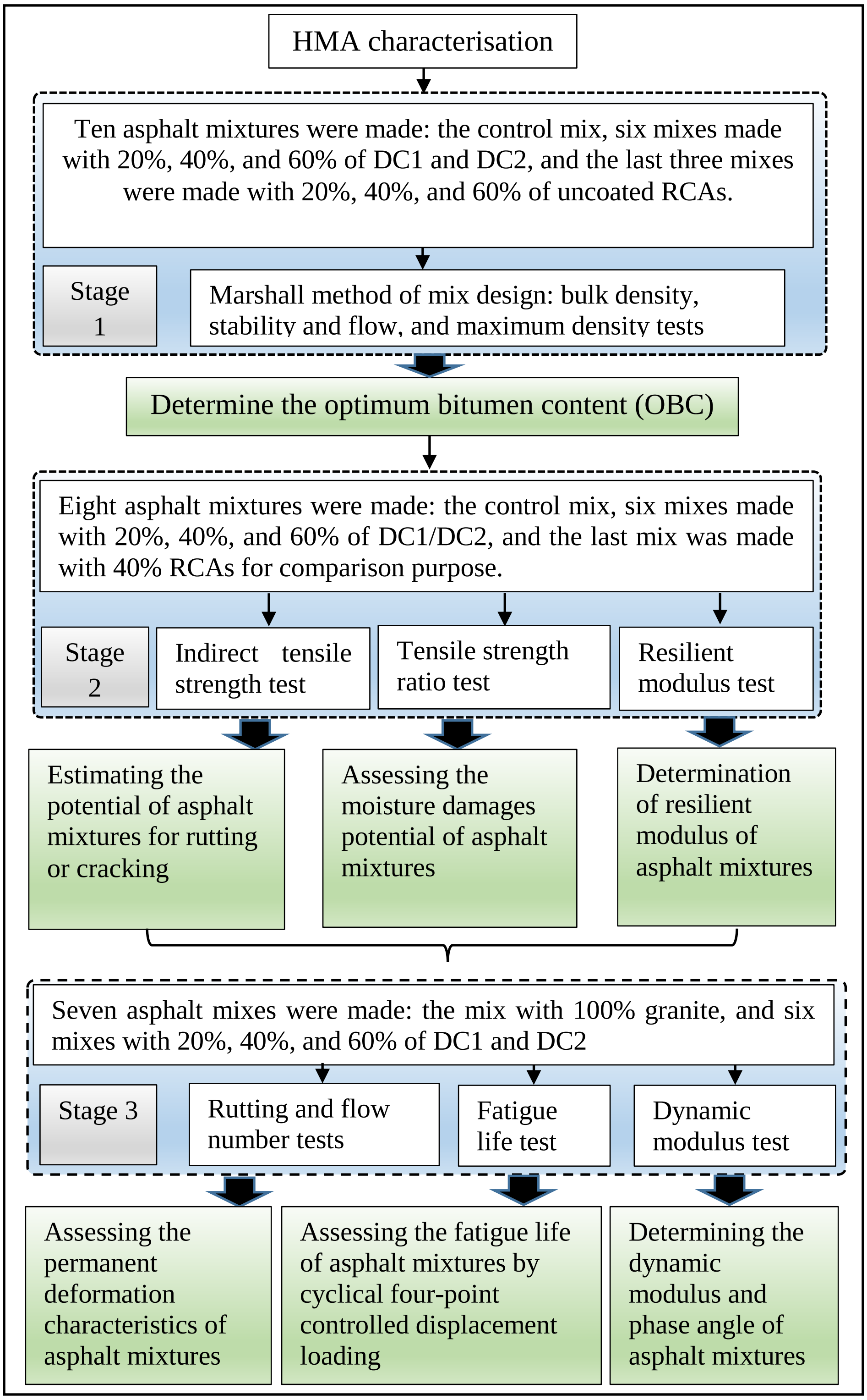


Figure 4.12: Flow chart of asphalt mixtures characterization.

4.5.1 Non-performance tests

4.5.1.1 Marshall method of mix design

As mentioned in Section 4.3.4, the Marshall method of mixture design was used in this investigation to determine the OBC. The Marshall method consists of three main tests: the bulk density test, stability and flow test and maximum density test. The following sub-sections introduce these tests as per the requirements of Australian standards.

4.5.1.1.1 Bulk density of compacted asphalt mixture

Test method AS/NZS 2891.9.2 (Standards Australia/Standards New Zealand, 2014) was used to determine the bulk density of compacted asphalt mixtures made with different dosages of RCA, DC1 and DC2. This test method is only applicable to dense graded asphalt mixtures which have a nominal aggregate size ≥ 14 mm (Standards Australia/Standards New Zealand, 2014). The test method consists of determining three masses of compacted Marshall specimen:

- The air-dried specimen was weighed to determine the dry mass (*m1*).
- The air-dried specimen was immersed in water bath for 5 min to determine its mass under water (*m2*).
- The specimen was removed from the water, surface dried with a damp towel and immediately weighed to check the saturated surface dry mass (*m3*).

After these three masses were recorded, the bulk density, in t/m³, of the compacted asphalt mixture was calculated using Equation 4.1:

$$\rho bulk = \frac{m1 * \rho w}{m3 - m2} \qquad 4.1$$

Where *m1* = the mass of air-dried specimen (g), ρw = the density of water at the test temperature (in this study = 25 °C) in t/m³, *m3* = the mass of the saturated surface-dry specimen (g) and *m2* is the mass of the saturated specimen in water (g).

4.5.1.1.2 Stability and flow testing

The Marshall procedure as outlined in AS/NZS 2891.5 (Australian/New Zealand Standards, 2015) was used to measure the stability and flow of the compacted asphalt specimens.

After the bulk density testing of the compacted asphalt specimen was completed, stability and flow testing was performed using the Marshall testing machine shown in Figure 4.13.

Figure 4.13: Marshall testing machine

Any specimen having an average height and volume less than 57 mm and 459 cm³ or higher than 70 mm and 572 cm³ should be discarded. To proceed with the test, the Marshall specimen was placed in a water bath maintained at 60 ± 1 °C for 30–40 minutes. After that, each specimen was removed from the bath and placed centrally in the breaking head assembly. Tests should be completed within 30 seconds once the specimen is removed from the bath. The testing machine was driven at a constant rate of travel of the plates of 51 ± 3 mm/min to apply the load. The load was applied until it began to decrease, and the peak force (kN) and displacement at failure (mm) were recorded using a data acquisition system. The calculated peak force was corrected using a factor related to the specimen's height/volume according to Equation 4.2:

$$S = L * F \tag{4.2}$$

Where S = stability of each asphalt specimen (in kN to the nearest 0.1 kN), L = the load at failure (kN) and F = a correction factor.

The values of the stability correction factor (F) can be obtained from Table 4.6.

Table 4.6: Values of stability correction factor (F)

Height of specimen (mm)	Volume of specimen (cm³)	Correction factor (F)
57	456-466	1.19
58	467-474	1.16
59	475-482	1.13
60	483-490	1.10
61	491-499	1.07
62	500-507	1.04
63	508-515	1.01
64	516-523	0.99
65	524-531	0.96
66	532-539	0.94
67	540-547	0.92
68	548-555	0.90
69	556-563	0.88
70	564-572	0.86

The average flow value was recorded to the nearest 0.1 mm. The Marshall quotient, to the nearest 0.1 kN/mm, was also calculated by dividing the average stability (kN) by the average flow (mm) of each asphalt mix (Australian/New Zealand Standards, 2015).

4.5.1.1.3 Maximum density of asphalt mixture – Rice method

The maximum density (void-less density) of loose asphalt mixes was determined in accordance with the WA 732.2 test method (MRWA, 2011). At least 1550 g of loose asphalt is required to conduct the test by using a Buchner filter flask with rubber and an appropriate vacuum pump. The Buchner flask should be capable of withstanding a vacuum of at least 1 atm.

Three masses were measured during the tests:

- The mass of the Buchner flask immersed in a water bath maintained at 25 ± 0.1°C, in grams (*m1*).
- The mass of the loose asphalt specimen in air, in grams (*m2*).
- The mass of the Buchner flask and loose asphalt in water after removing entrapped air from the sample, in grams (*m3*).

After calculating these three masses, the maximum density of the asphalt mixture was calculated using Equation 4.3:

$$\rho max = \frac{m2 * \rho w}{m2 - (m3 - m1)} \qquad 4.3$$

Where: ρmax = maximum density of loose asphalt (t/m³), ρw = density of water at 25 °C (t/m³), *m1* = mass of the Buchner flask in water (g), *m2* = mass of the loose test specimen (g) and *m3* = mass of the flask and loose specimen in water (g).

4.5.1.1.4 Voids and volumetric properties

After determining the bulk density and maximum density of the compacted asphalt mixtures, other Marshall volumetric properties can be calculated. These properties were the percentage of air voids (AV), the percentage of voids in the mineral aggregate (VMA), the percentage of voids filled with bitumen (VFB), the percentage of binder absorbed by aggregates (b), and the percentage of effective binder (Be). These percentages were calculated as per the following equations:

$$AV = \frac{\rho\text{max} - \rho\text{bulk}}{\rho\text{max}} * 100 \qquad 4.4$$

$$VMA = 100 - \frac{\rho\text{bulk}}{\rho\text{a}} (100 - B) \qquad 4.5$$

The ρa is calculated using Equation 4.6:

$$\rho a = \frac{100}{\frac{Pc}{\rho c} + \frac{Pf}{\rho f} + \frac{Pfiller}{\rho filler}} \qquad 4.6$$

Where ρa = the particle density of the combined mineral aggregates (t/m³), Pc = proportion of combined coarse aggregates (percentage), Pf = proportion of combined fine aggregates (percentage), P_{filler} = proportion of filler (percentage), ρc = particle density of the combined coarse aggregates (t/m³), ρf = particle density of the combined fine aggregates (t/m³), and ρfiller = apparent particle density of filler (t/m³).

$$VFB = \frac{VMA - AV}{VMA} * 100 \qquad 4.7$$

$$b = B - \rho b \left(\frac{100}{\rho max} - \frac{(100 - B)}{\rho a} \right) \qquad 4.8$$

Where ρb = density of C320 binder at 25 °C (t/m³) and B = proportion by mass of bitumen in asphalt mix (t/m³).

$$Be = B - b \tag{4.9}$$

4.5.1.2 Indirect tensile strength (ITS) tests

The ITS tests can be used to examine some performance properties of an asphalt mix, such as rutting and surface cracking (ASTM, 2017; Lee et al., 2012). In this investigation, the ASTM D6931 specification was used to measure the ITS of asphalt mixtures.

For each AC14 mix prepared, nine asphalt specimens were made. These specimens were divided into three groups of three specimens each. Two groups were made, with 5 ± 0.5% and 8 ± 0.5% air void contents, and tested at 25 °C in order to examine the effect of air void content on the ITS of the asphalt mixes. The last group was made with air void contents of 5 ± 0.5% and tested at 40 °C to assess the effect of temperature on ITS. The ITS of a specimen was tested at a loading rate of 50 mm/min and calculated using Equation 4.10.

$$TS = \left(\frac{2P}{\pi * H * D}\right) * 10^6 \tag{4.10}$$

Where TS = tensile strength (kPa), P = maximum applied force measured by the testing machine (kN), and H and D = the height and diameter of a specimen, respectively (mm). Figure 4.14 shows the setup used for ITS testing.

Figure 4.14: Setup used for the ITS testing

4.5.1.3 Moisture sensitivity testing

Moisture-induced damage to bituminous mixtures is one of the most important sources of distress to bituminous pavements (Mehrara & Khodaii, 2013). Moisture is a fundamental cause of deterioration in bituminous asphalt mixtures (X. Chen & Huang, 2007; Cheng, Little, Lytton, & Holste, 2003). Such distress is difficult to detect because pavement surface distresses can take several forms, such as rutting, shoving, and cracking (X. Chen & Huang, 2007). This distress is also known as *stripping* because it is a result of detachment of asphalt film from aggregate surfaces. In the current study, the Australian standard AG:PT/T232 was used to investigate moisture-induced damage in the AC14 mixes (Austroads, 2007a). This standard has been adapted from ASTM D 4867-92 and AASHTO T 283-85 standrads.

As mentioned previously in Section 4.4, eight AC14 mixes were made to evaluate their resistance to moisture-induced damage: a control mix with 100% granite aggregate, three mixes with 20%, 40% or 60% DC1, three mixes with 20%, 40% or 60% DC2, and one mix with 40% RCA. For each asphalt mix, six cylindrical Marshall specimens 100 mm in diameter by 65 mm in height were prepared using the gyratory compactor to achieve 8 ± 1% air voids (Austroads, 2007b). Any specimen outside this range of air voids was discarded and a new specimen manufactured by altering the amount of materials poured into the mould accordingly.

The samples were then divided into dry and wet groups. According to the AG:PT/T232 standard, the difference between the average void contents of dry and wet groups should be ≤ 0.5% (Austroads, 2007a). Each sample in the wet group was put in 50 ± 5 °C water in a vacuum desiccator. The water covered the specimen by at least 25 mm. Then, a vacuum pressure of 600 ± 25 mmHg was applied for 10 minutes (Figure 4.15). The specimen was then removed from the vacuum desiccator and wiped with a damp towel and weighed. The degree of saturation was calculated using Equations 4.11, 4.12 and 4.13:

$$Degree\ of\ saturation\ (\%) = \left(\frac{\mathrm{mps-md}}{\mathrm{Va}}\right) * 100 \qquad 4.11$$

The value of Va can be determined using Equation 4.12:

$$Va\ (cm^3) = \frac{\mathrm{AV*Vd}}{100} \qquad 4.12$$

The value of Vd can be calculated based on Equation 4.13:

$$Vd\ (cm^3) = \frac{\mathrm{m3-m2}}{\mathrm{F}} \qquad 4.13$$

where *mps* = mass of partially saturated sample (g), md = mass of dry sample in air, *Va* = volume of air in the specimen under test (cm³), *Av* = air void content (%), *Vd* = volume of dry specimen (cm³), *m3* = mass of the saturated surface dry specimen in air (g), *m2* = mass of the specimen in water (g) and *F* = density of the water used in the test (0.997 t/m^3).

Figure 4.15: Set-up of vacuum pump and desiccator for TSR testing

According to the Australian Standard AG:PT/T232, the degree of saturation has to be between 55-80%. When a degree of saturation less than 55% is obtained, the procedure was repeated under a different vacuum pressure or water temperature. In cases where the degree of saturation was > 80%, the specimen was assumed to be damaged and was discarded. To avoid the possibility of losing asphalt samples during partial saturation, two further specimens were made, as recommended by the Australian Standard used (Austroads, 2007b). On completion of the saturation of the three wet specimens, each specimen was wrapped in 2-3 layers of plastic cling wrap and placed in a plastic bag. A total of 10 mL of water was added to the specimen in the bag. The specimen was then immediately placed in a freezer at −18 ± 3 °C for 18 ±1 hours, as shown in Figure 4.16.

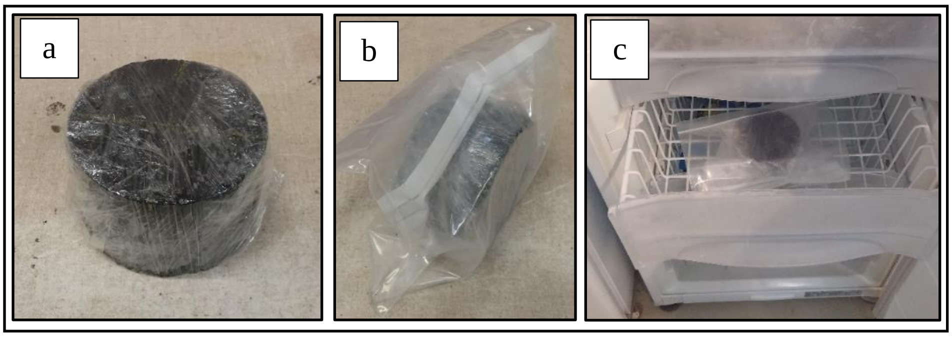


Figure 4.16: (a) Specimen wrapped in plastic film, (b) specimen put in a plastic bag, and (c) specimen put in a freezer at -18 ± 3 °C

After completion of the freeze-conditioning of the wet group specimens, they were immediately unwrapped and transferred into a temperature-controlled water bath at 60 ± 1 °C and kept there for 24 ± l hours. Then, specimens were placed into a 25 ± 1 °C water bath for 2 hours ± 5 minutes. The dry group specimens were placed in a temperature-controlled cabinet at 25 ± 1 °C for 2 hours ± 5 minutes. It should be noted that the dimensions of the three wet specimens were measured before and after conditioning to check for swelling. Swelling was determined using Equation 4.14:

$$Swell\ (\%) = \left(\frac{Vmc - Vd}{Vd}\right) * 100 \tag{4.14}$$

Where: *Vmc* = volume of the specimen after conditioning (cm^3) and *Vd* = volume of the specimen before conditioning (cm^3).

Once all the specimens in the dry and wet sets were conditioned, their ITSs were tested at 25 °C with a loading rate of 51 ± 3 mm/min, as shown in Figure 4.13. The wet and dry tensile strengths were calculated using Equation 4.10. The ratio of wet to dry tensile strength was used to define the moisture-induced damage to the asphalt mixture. This ratio is called the tensile strength ratio (TSR), and can be computed using Equation 4.15:

$$TSR\ (\%) = \frac{\text{TSW}}{\text{TSD}} \times 100 \qquad 4.15$$

Where TSR = tensile strength ratio (%), TSW = average tensile strength in wet conditions (kPa) and TSD = average tensile strength in dry conditions (kPa).

Figure 4.17 illustrates a typical loading curve for ITS testing of an asphalt specimen. It can be seen that the indirect tensile strength of an asphalt specimen reaches its maximum value after less than 5 seconds, then starts to decrease gradually (Alderson, 2008). In general, a TSR of 80% is the minimum acceptable value for an asphalt concrete mix to be considered not susceptible to stripping (Alderson, 2008). In addition, according to MRWA specification 510, the minimum accepted tensile strength in dry conditions (TSD) and tensile strength in wet conditions (TSW) should be 850 kPa and the 750 kPa, respectively (MRWA, 2017a).

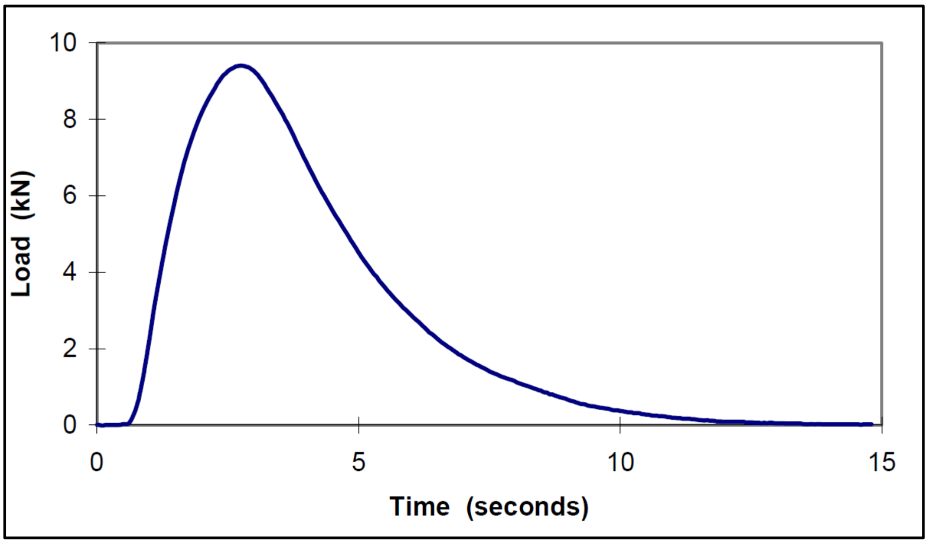


Figure 4.17: Typical loading curve for a tensile strength test (Alderson, 2008)

4.5.1.4 Indirect tensile stiffness modulus (ITSM) test

For determination of the resilient modulus of AC14 mixtures made with granite aggregates, uncoated RCAs, and two types of double coated RCAs (DC1 and DC2), the indirect tensile stiffness modulus (ITSM) test was performed according to

Australian Standard AS/NZS 2891.13.1 (Australian/New Zealand Standard, 2013). In this test, a load is applied vertically along the vertical diameter and the induced displacement is measured along the horizontal diameter plane of the sample (Figure 4.18**).** According to AS 2891.13.1, a recovered horizontal of 50 ± 20 $\mu\epsilon$ should be achieved in the specimen under testing. This is to ensure that a sufficient amount of deformation occurred and was measured precisely by the linear variable displacement transducers (LVDTs) whilst the response of the sample remained elastic (Alderson, 2008). The test was conducted at two temperatures (25 °C and 40 °C) to investigate the effect of temperature on the stiffness of asphalt mixtures made with 40% RCA and 20%, 40% and 60% DC1 and DC2. The results were compared with those obtained for mixtures made with control granite aggregates.

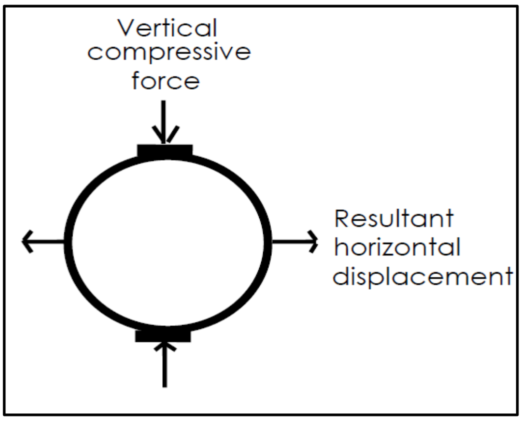


Figure 4.18: Schematic of the indirect tensile stiffness modulus (ITSM) test (Austroads 2008)

Testing machine: A universal testing machine (UTM 25) was used to perform the ITSM tests in the present study. This machine was designed by IPC Global and was capable of applying triangular/haversine load pulses with a rise time of 0.025–0.1 ± 0.005 s. The UTM 25 was capable of applying a load pulse with a peak load of 0.4–3.9 ± 0.05 kN. The repetition interval for each load pulse was adjustable over a range of 0.5–10 ± 0.005 s (Australian/New Zealand Standard, 2013). Figure 4.19 shows the force and horizontal deformation pulse shapes applied by the UTM 25 testing machine (Australian/New Zealand Standard, 2013).

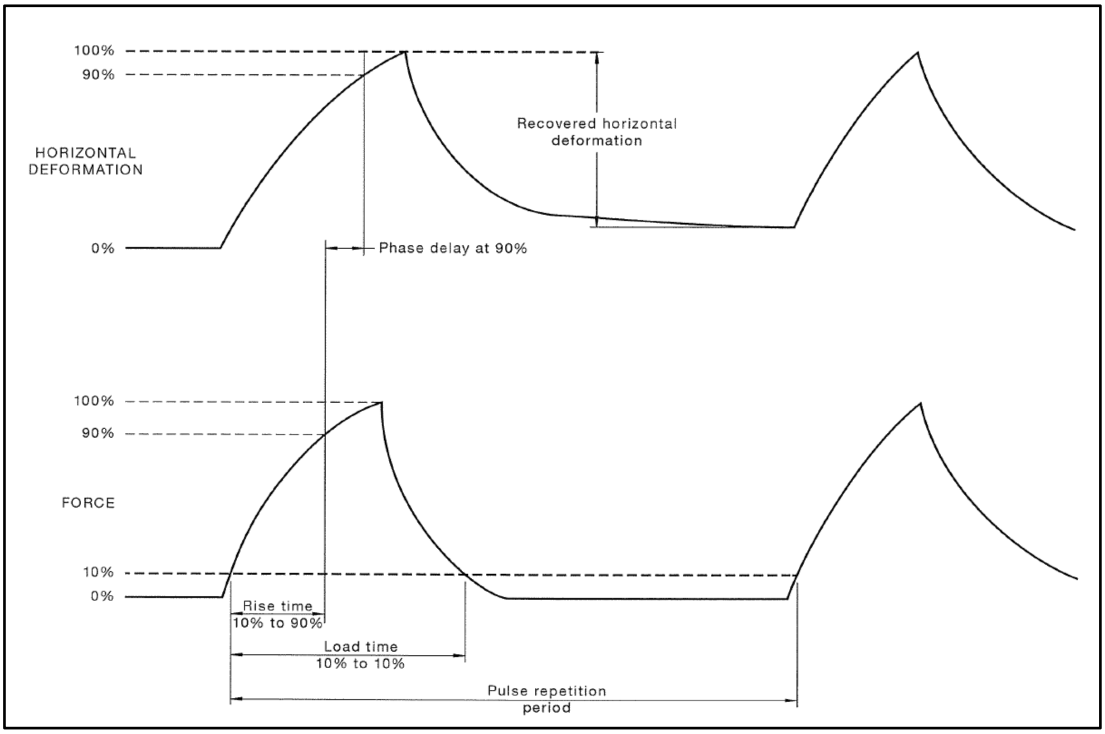


Figure 4.19: Force and horizontal deformation pulse shapes during the ITSM test

The temperature-controlled cabinet of the UTM 25 machine is capable of maintaining the temperature of the loading frame, a dummy specimen and at least three test specimens at the desired test temperature. To ensure that the asphalt specimen in the cabinet had achieved the target test temperature, a temperature measuring tool was installed at the centre and at the surface of the dummy specimen. Asphalt specimens were placed in the temperature-controlled cabinet for a minimum of two hours' conditioning time to achieve the target test temperature.

Specimen fabrication and testing: Depending on the maximum particle size of the asphalt used in the present study, a cylindrical asphalt specimen of 100 ± 2 mm diameter and 35–75 mm height was used for the ITSM tests (Australian/New Zealand Standard, 2013). In the present study, the target height was 65 ± 1 mm and the desired air void content was achieved by controlling the amount of loose asphalt poured into the mould before compaction. The process was done by trial and error until the target air void content (5 ± 0.5%) was achieved. The specimens were compacted using the gyratory compactor as explained in Section 4.4.2.2. The bulk density of each compacted specimen was determined as described in Section 4.5.1.1.1, and the air void contents were computed according to Equation 4.4. When the target air content (5 ± 0.5%) was not achieved, a new specimen was made by varying the weight of the loose asphalt poured into the gyratory mould.

Before testing, the specimen was placed in the temperature-controlled cabinet maintained at 25 ± 0.5 °C or 40 ± 0.5 °C for a minimum of two hours. The specimen was placed centrally on the lower loading platen in the loading device. Displacement measuring apparatus was placed on the asphalt specimen along the horizontal diameter and fixed onto the surface of the specimen. Then, the top loading platen was placed onto the specimen centrally such that the vertical plane halved the width of the upper loading platen.

Once the specimen had reached the test temperature, the test was carried out. Figure 4.20 shows the setup for the resilient modulus test before the test proceeded. After that, the installed software was opened and the displacement measuring device was readjusted to sit in the centre of its travel range. Immediately, the five conditioning pulses were applied. During the five conditioning pulses, the testing machine increased the load applied from 10% to 90% of the peak load. Additionally, the specimen under testing had to achieve a recovered horizontal of 50 ± 20 $\mu\epsilon$. When these standard requirements were achieved, the five testing pulses were applied and the resilient modulus was automatically calculated by the software based on Equation 4.16:

$$E = P \times \frac{v + 0.27}{H \times hc} \tag{4.16}$$

Where E = resilient modulus (MPa), P = peak load (N), v = Poisson ratio (0.4 was assumed for all asphalt mixes), H = recovered horizontal deformation of specimen after application of load (mm) and hc = specimen height (mm).

As per AS 2891.13.1, a minimum of three specimens were tested and a mean value of the resilient modulus was obtained. Any test providing a result that differed by more than ±15% from the mean resilient modulus was repeated (Australian/New Zealand Standard, 2013). Figure 4.21 shows some typical results of the ITSM test. In addition, typical values for laboratory-made HMA containing C320 bitumen are presented in Table 4.7. These values are reported in the Austroads technical report AP-T100/08 (Alderson, 2008).

Figure 4.20: Set-up for the ITSM tests

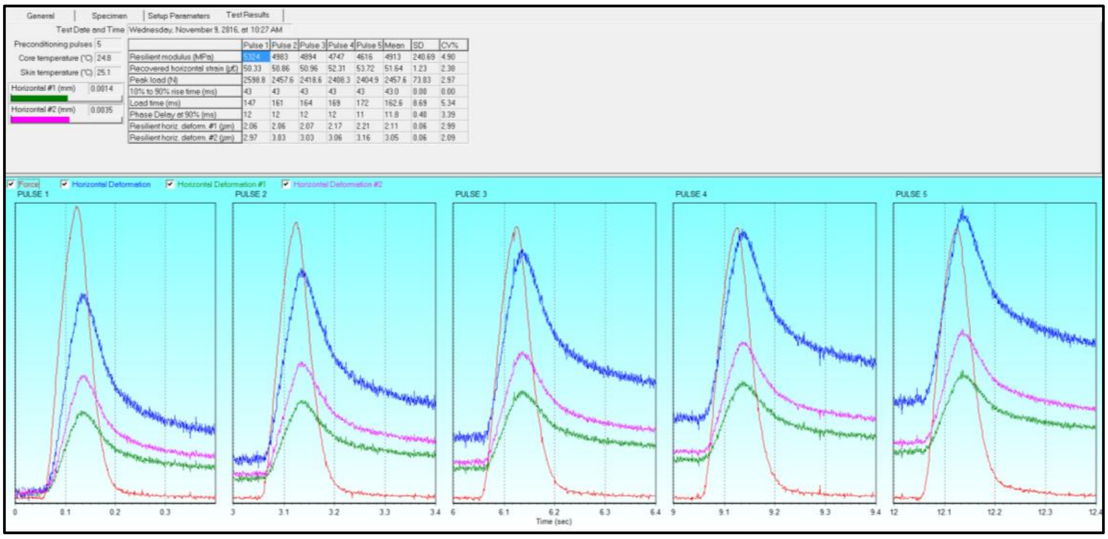


Figure 4.21: Typical ITSM test results

Table 4.7: Typical resilient modulus values (MPa) for different laboratory-made HMAs made with Class 320 bitumen

Application	Resilient modulus (MPa)
Wearing course	3000
Intermediate layer or base course	4000
Fatigue course	3000

4.5.2 Performance testing

In addition to the tests described in previous sections, the wheel tracking, flow number, dynamic modulus, and fatigue tests were also carried out in this research. In the following sub-sections, a brief description of performance tests is provided.

4.5.2.1 Resistance to permanent deformation

In this study, the wheel tracking test was used to rank the rutting resistance of asphalt mixtures made with conventional granite aggregates and two types of double-coated RCAs (DC1 and DC2). In addition to wheel tracking tests, flow number (FN) tests were also used to predict the rutting behaviour of HMA (J. Zhang, Alvarez, Lee, Torres, & Walubita, 2013). The latter test was also conducted in this study to evaluate the permanent deformation potential of HMA made with granite aggregate and different percentages of DC1 and DC2.

4.5.2.1.1 Wheel tracking testing

Wheel tracking testing was performed in accordance with Australian standard AG:PT/T231 (Austroads, 2006a). This test is basically used to simulate the effect of traffic loading at elevated temperatures. The test was performed using the Cooper wheel tracking machine. This device is a simple testing machine consisting of a 200-205 mm loaded steel wheel with a non-treated rubber which bears on an asphalt slab specimen. The slab was attached to a moving table and a 700 ± 20 N load was applied. The table was moved at a rate of 42 ± 0.5 passes/minute. Figure 4.22 shows the set-up for the wheel tracking test.

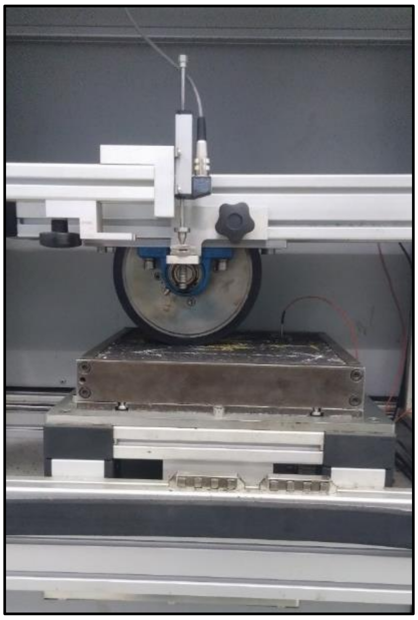


Figure 4.22: Set-up for wheel tracking testing

Rut depth calculation: The wheel tracking device comprised a displacement measuring device capable of measuring the vertical depth of rutting. The test was terminated when one of the termination conditions were reached: either 10,000 passes or a 15 mm rut depth. According to Australian standard AG:PT/T231, the rut depth was calculated in seven locations: the centre, and at ± 7.5 mm, ± 22.5 mm and ± 37.5 mm from the centre, with a ± 2.5 mm tolerance for these locations. It should be noted that the Wheeltracker EN software measures the rut depth in accordance with the European Standard (EN 12697-22) not Australian Standard (AG:PT/T233). Based on European Standard-EN 12697-22, the rut depth was measured at 27 locations, not at 7 locations as per Australian Standard AG:PT/T233.

According to the European Standard, the following locations were used to calculate the rut depth: the centre, and at ± 4 mm, ± 8 mm, ± 12 mm, ± 16 mm, ± 20 mm, ± 24 mm, ± 28 mm, ± 32 mm, ± 36 mm, ± 40 mm, ± 44 mm, ± 48 mm and ± 52 mm from the centre. Therefore, the final measurement might obtain some differences in rut depth to those measured based on Australian standard AG:PT/T233. Also, the software neglected the first ten passes while based on Australian standard the rut depth should be calculated from the first pass (Austroads, 2006a).

Therefore, two steps were considered in order to achieve accurate measurements of rut depth in accordance with Australian standard AG:PT/T233:

a) The rut depth was only measured at the centre, and at ± 8mm, ± 24 mm and 36 mm from the centre, because these locations satisfy the AG:PT/T233 limitations when taking a tolerance of ± 2.5 mm into consideration.
b) As the available device was set to work based on the European Standard, it neglected the first ten passes. So, in this study, the rut depth was considered to start from zero and was calculated from the first pass, and the test was terminated at 10,010 passes not at 10,000 passes.

Table 4.8 summarises the test conditions of the Cooper wheel tracking test used in the present investigation.

Table 4.8: Conditions of wheel tracking tests conducted in this study

Parameter	Units	Standard limits
Test temperature	°C	60 ± 1
Slab thickness	mm	50 ± 5 (for mixes with nominal sizes of 10 mm & 14 mm)
Air void content	%	5 ± 1
Vertical load	N	700 ± 20

Slab preparation: The rutting slabs prepared in this study were compacted according to Australian standard AG:PT/T220 (Austroads, 2005). The target air void content was achieved by controlling the weight of the loose asphalt poured into a 300 × 300 × 50 mm slab mould. When the loose asphalt was poured and levelled in the slab mould, it was transferred to the Cooper compacter which was adjusted to manage that size of slab mould. The rest of the compaction process was completed as described in Section 4.4.2.3. After the bulk density of the produced slab was determined, the air void content was calculated as described in Section 4.5.1.1.4. When the target air void was not achieved (5 ± 1%), the slab was discarded and a new slab was prepared by altering the amount of loose asphalt poured into the slab mould accordingly.

The slab was left to dry in a well-ventilated area at Curtin University's Geomechanics Laboratory. Once the slab had reached a constant mass, a small hole was made at the slab's corner and filled with silicon grease. Then, the slab was attached to the moving table of the wheel tracking device and left in a temperature-controlled cabinet for at least two hours to achieve the desired temperature (60 ± 1 °C). Once the slab had reached the target temperature, the wheel was released and attached to the slab surface. Also, a 700 N load was hung on the loading arm and the displacement measuring device was attached to the wheel.

According to Australian standard AG:PT/T231, two parameters can be used to assess the performance of HMA against rutting: the rut depth (mm) and the steady-state tracking rate (SSTR; mm/kPasses). The rut depth was obtained after 10,000 wheel passes, while the SSTR was determined using Equations 4.17 and 4.18:

$$Tangential\ Slope = \frac{rut\ depth\ at\ 10000\ passes - rut\ depth\ at\ 4000\ passes}{600} \qquad 4.17$$

$$SSTR\ \left(\frac{mm}{kPasses}\right) = Tangential\ slope \times 1000 \qquad 4.18$$

From the readings taken for each pass, an average rut depth was calculated and plotted against the number of passes to produce a graph similar to that shown in Figure 4.23. Table 4.9 presents some typical rut depth data and how they are classified depending on their values (Alderson, 2008).

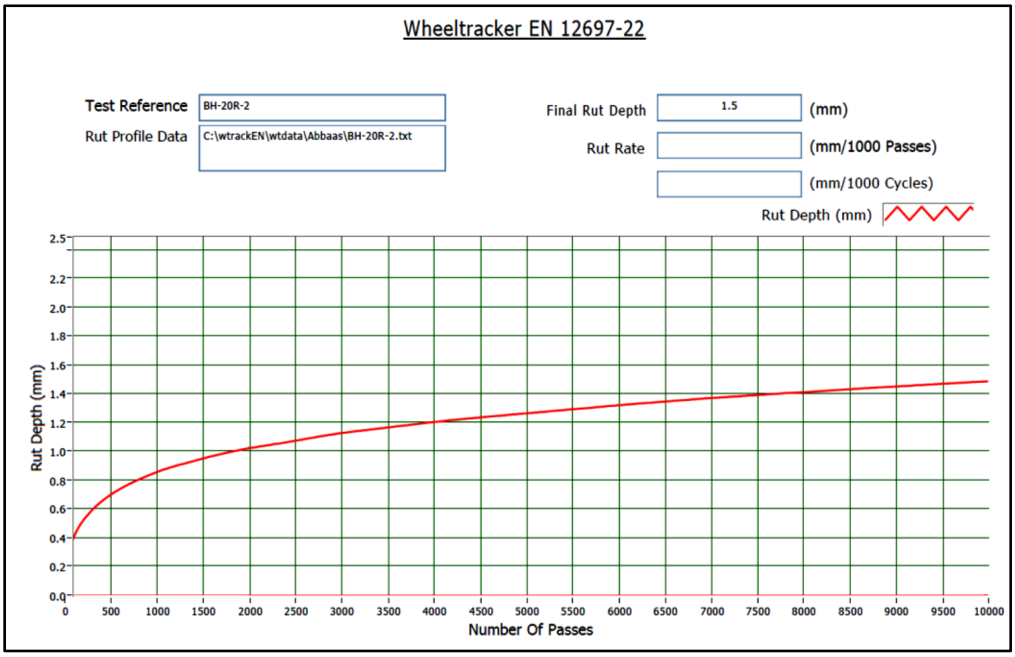


Figure 4.23: Typical output graph of Wheeltracker EN software

Table 4.9: Typical rut depth values (mm) of HMA and their classification

Superior performance	Good performance	Medium performance	Low performance
<3.5	3.5-8	8-13	>13

4.5.2.1.2 Flow number test

Flow number (FN) testing is documented as an appropriate method for investigating the rutting performance of asphalt mixtures (Rodezno, West, & Taylor, 2015). In this study, this test was performed to predict the rutting behaviour of HMA made with granite and two types of double-coated RCA (DC1 and DC2). The test was carried out in accordance with AASHTO TP 79 using the Asphalt Mixture Performance Tester (AMPT) (AASHTO, 2014a). The asphalt specimens used in dynamic modulus testing were used again in the FN tests because the former test is non-destructive. The height of the samples was around 150 mm and the diameter was 100 mm.

The FN test is a dynamic creep and recovery test in which the sample is subjected to a haversine loading pulse of 0.1 s followed by a rest period of 0.9 s under unconfined or confined conditions (Roy, Veeraragavan, & Murali Krishnan, 2015). In this research, the FN tests were performed at 54 °C (Bhusal & Wen, 2013) using zero confining pressure, 600 kPa deviatoric stress and 30 kPa contact deviatoric stress. The FN value was determined when the permanent axial strain rate changed from negative to positive. A mixture with high FN is expected to show high rutting resistance. Three samples of each mix were made and tested, and the mean value obtained to give the asphalt mix flow number. Figure 4.24 shows a flow number sample under testing using the AMPT machine.

The flow number is identified the initiation of the tertiary flow of an asphalt mix or the minimum point of the strain rate graph (Kök & Çolak, 2011). Figure 4.25 shows the typical outcomes of FN testing of asphalt mixtures. It can be seen that the strain rate decreases as the number of load cycles increases (primary stage). Then, the strain rate remains somewhat constant (secondary stage). In the tertiary stage, the minimum strain rate starts to increase significantly, which indicates that the specimen starts to deform (flow).

Figure 4.24: Set-up for flow number testing

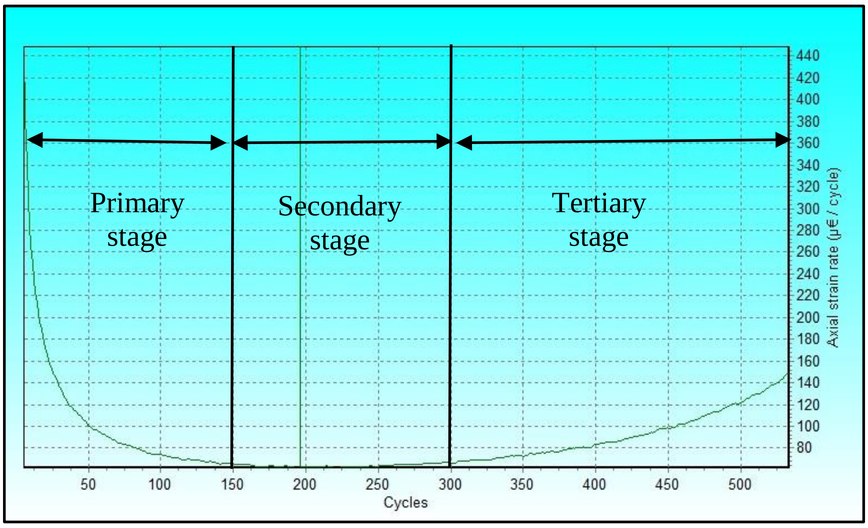


Figure 4.25: Strain rate versus cycle number: outcomes of the flow number test

4.5.2.2 Four-point bending beam fatigue life test

The fatigue life of asphalt mixtures can describe the ability of flexible pavement to resist cracking under successive traffic loading. In the field, repetitive traffic loading induced strain and stress occur in constructed pavements layers. As the load repetitions increase, the pavement stiffness decreases and the deflection increases and, consequently, fatigue cracking occurs. In the present research, the four-point beam fatigue life test was performed to evaluate this property according to Austroads testing standard AG:PT/T233 (Austroads, 2006b). This testing method was based on the Strategic Highway Research Program (SHRP) standard *Standard Method of Test for Determining the Fatigue Life of Compacted Bituminous Mixtures Subjected to Repeated Flexural Bending*. However, the test method was modified to suit Australian conditions (Austroads, 2006b).

There are two main versions of fatigue life testing for asphalt mixtures: the controlled stress test method and the controlled strain test method (Alderson, 2008). In controlled stress tests, a constant load is applied and the induced strain is measured. When the strain is doubled, the stiffness reaches half of its initial value. In controlled strain tests, a constant strain is applied throughout the test and the stress required to maintain the initial strain is recorded until the stiffness of the mix becomes half its initial value. The fatigue life is the number of load repetitions needed to reduce the stiffness of the mix by 50% (Khiavi & Ameri, 2013; Poulikakos, Pittet, Dumont, & Partl, 2015).

Controlled stress testing is most accurate when used to model stress in thick pavement layers (e.g. > 75mm) but controlled strain testing is more applicable for thin layers (e.g. < 75mm). In Australia, pavement layers are normally constructed with a thickness of less than 75 mm. Therefore, the controlled strain mode was used in this study to evaluate the fatigue life of asphalt mixtures made with natural granite aggregates and two types of double coated recycled concrete aggregates (DCRCAs), i.e., DC1 and DC2.

Testing machine: The fatigue tests were performed using the machine shown in Figure 4.26. This machine is a closed-loop, servo-controlled machine capable of applying either a constant load to the beam under test or moving the beam through a constant displacement at frequencies of up to 10 Hz. The machine consisted of an air supply system, beam cradle, data acquisition system, software and a temperature-controlled cabinet. The machine should be capable of applying the target peak tensile strain within the specified tolerance throughout the test. In each load cycle, the machine recorded the cycle number, the temperature of the beam, and time traces of the applied load and beam displacement. Furthermore, the temperature-controlled cabinet was capable of maintaining the beam temperature within ± 0.5 °C of the specified test temperature (Austroads, 2006b).

Figure 4.26: Fatigue testing machine

Sample preparation and testing: The specimens prepared for fatigue life characterisation were made with natural aggregates and two types of double-coated RCA (DC1 and DC2). These specimens were cut from slabs 400 mm in length, 280 mm in width and 75 mm in thickness. The loose asphalt mixtures were prepared based on the AS/NZS 2891.2.1 standard (Australian/New Zealand Standards, 2014b), as described in Section 4.4.1. Once the loose mix was conditioned, the loose asphalt was poured into a fatigue slab mould and compacted according to AG:PT/T220 (Austroads, 2005), as described in Section 4.4.2.3. The slab was then left to cool.

Each slab was cut into three beams transverse to the direction of compaction using an auto saw machine, as presented in Figure 4.27. According to Austroads standard AG:PT/T233 (Austroads, 2006b), the slab should be cut to produce three beams of 390 ± 5 mm in length, 50 ± 5 mm in depth and 63.5 ± 5 mm in width. The bulk density of each beam was calculated and the air void content was checked. When the air void content did not satisfy the target (5 ± 0.5 °C), the beams were rejected and a new slab was compacted by adjusting the weight of loose asphalt poured into the slab fatigue mould. All beams were stored on a flat, stiff surface and tested within 30 days of compaction, as recommended by the Austroads standard. Before testing, the dimensions of each beam specimen were checked in five locations and the average values of length, width and depth were recorded.

Figure 4.27: Slab cutting process using auto saw machine

Before testing, each beam was placed inside the temperature-controlled cabinet for a minimum of two hours to achieve the target temperature (20 ± 0.5 °C). The beam was then put in the beam cradle so that the four supports were just attached to the beam sample without applying load. The four clamps were then locked to hold the beam in place, while the supports were released by releasing the restraints. The fatigue testing set-up is shown in Figure 4.28. The beam was then left for a minimum of 30 minutes to allow for the clamping stresses to be released. The linear variable displacement transducer (LVDT) was adjusted and placed in the middle of its moving range. The test was then performed under the conditions summarised in Table 4.10.

Figure 4.28: Set-up for fatigue life testing

Table 4.10: Fatigue life testing conditions used in this study

Test parameter	Unit	Test condition
Temperature	°C	20 ± 0.5
Load frequency	Hz	10 ± 0.1
Loading type	-	Continuous haversine
Cycles to calculate the initial stiffness	Cycles	50
Peak tensile strain	$\mu\varepsilon$	400
Air void content	%	5 ± 0.5
Poisson ratio	-	0.4
Termination stiffness	-	50% of initial stiffness
Maximum number of cycles	Cycles	1,000,000

During testing, the flexural stiffness, the modulus of elasticity, phase angle and dissipated energy were automatically calculated. The test continued until the beam reached half its initial stiffness, which was calculated at the 50th load cycle or when 1,000,000 loading cycles were completed (Austroads, 2006b). An example of typical outcomes of the four-point bending beam fatigue test is shown in Figure 4.29. The flexural stiffness of each beam specimen was calculated using Equations 4.19, 4.20 and 4.21, below:

$$S_{Mix} = \frac{1000*\sigma_t}{\varepsilon_t} \quad 4.19$$

$$\sigma_t = \frac{\mathrm{LP}}{wh^2} * 10^6 \quad 4.20$$

$$\varepsilon_t = \frac{108\delta\mathrm{h}}{23L^2} * 10^6 \quad 4.21$$

Where s_{mix} = flexural stiffness (MPa), σ_t = peak tensile stress (kPa), ε_t = peak tensile strain (microstrain), L i = beam span (typically 356 mm), P i = peak force (kN), w i = beam width (mm), h i = beam height (mm) and δ = peak displacement (mm).

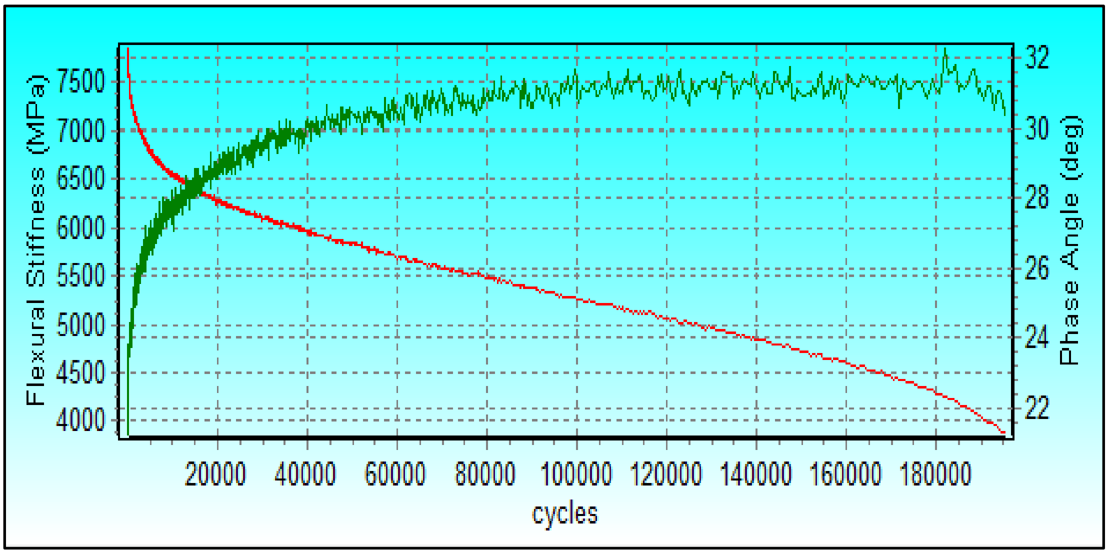


Figure 4.29: Typical results of a fatigue life test

4.5.2.3 Dynamic modulus testing

An asphalt mixture performance tester (AMPT) was used in the present study to measure the dynamic modulus and phase angle of mixtures made with granite aggregates and different dosages of DC1 and DC2 under a range of temperatures and loading frequencies. The tests were carried out based on the standard of the American Association of State Highway and Transportation Officials (AASHTO) Designation: TP 79-13 (AASHTO, 2014b). This test method is applicable for dense and gap-graded asphalt mixtures with nominal maximum aggregates sizes of up to 37.5 mm.

Testing machine: The machine used to perform the dynamic modulus and phase angle tests is shown in Figure 4.30. This machine was manufactured by IPC Global and consisted of the following items: a servo-hydraulic actuator of 25 kN capacity, loading cell, displacement measuring device, environmental chamber and data acquisition system. The environmental chamber was used to simulate the ambient temperature and was capable of maintaining an AMPT sample within a test temperature range of -15–60 °C.

Figure 4.30: Dynamic modulus testing machine

Sample preparation and testing: The specimens made for dynamic modulus testing were compacted in the laboratory as described in Section 4.4.2.2. The weight of loose asphalt required for each sample was calculated based on the maximum density of asphalt mixture, the desired air void content (5 ± 0.5%), and the dimensions of the asphalt sample. The process of mixing and conditioning of loose asphalt was carried out as described in Section 4.4.1. The samples were compacted to a target height of 170 mm using the gyratory compactor at a gyratory angle of 3° and a vertical stress of 240 kPa (Australian/New Zealand Standards, 2014a). After the compaction, and once the gyratory specimen had cooled down, a 100 mm core was taken from the 150 mm gyratory compacted material. Then, the top and bottom of the cored specimen were cut using the IPC Global auto saw to produce a specimen approximately 150 mm in height.

After that, the bulk density of the compacted asphalt mixture was computed and the air void content was determined. If the air void content was not within the target limit (5 ± 0.5%), a new specimen was made by adjusting the weight of the loose asphalt poured into the gyratory mould. In the present study, three samples were made of each AC14 mixture to evaluate their dynamic modulus and phase angle properties. The dimensions of each AMPT sample had to be measured and the average height and diameter were recorded. Before the testing was started, six knobs were attached to each AMPT sample using a two-part epoxy cement. The six knobs were attached so that three LVDTs with 120° separation from each other could be installed on the specimen to measure the deformations during the test. Figure 4.31 shows the different processes of preparing asphalt samples for dynamic modulus testing.

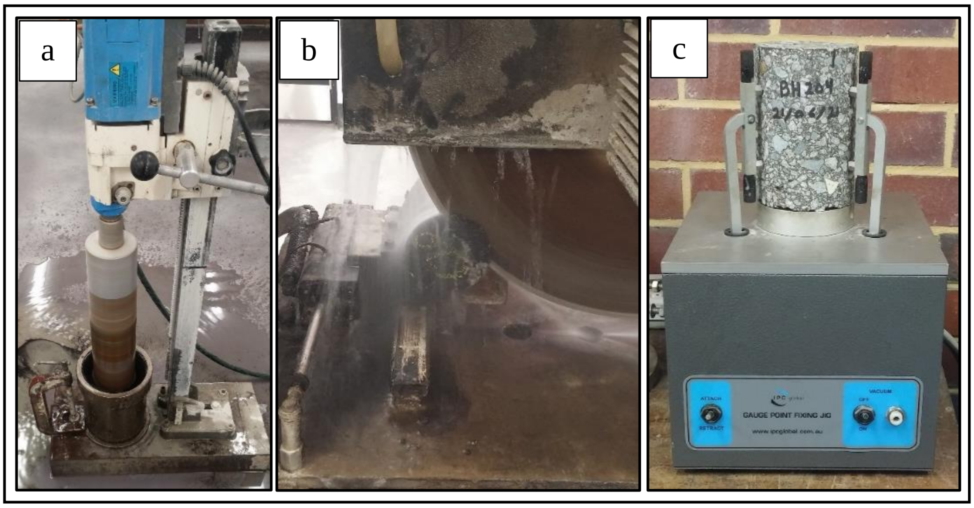


Figure 4.31: (a) Coring process, (b) cutting process, and (c) set-up for knob fastening

In this study, the test was carried out at three temperatures (4 °C, 20 °C and 40 °C) and four frequencies (0.01 Hz, 0.1 Hz, 1 Hz and 10 Hz). Different conditioning times were used as shown in Table 4.11, to allow the AMPT specimen to achieve different target temperature.

Table 4.11: Conditioning time for dynamic modulus testing

Test temperature, °C	Conditioning time, hours
4	Overnight
20	3
40	2

The test was performed under unconfined uniaxial compression test conditions. The test conditions used in this study are reported in Table 4.12. The sample under testing was subjected to sinusoidal (haversine) axial compressive stress over a wide range of temperatures and loading frequencies. This was to develop an asphalt mixture master curve for use in performance analysis. The AMPT test started at 4 °C and continued to 40 °C, and at 10 Hz proceeding to 0.1 Hz. The dynamic modulus and phase angle were calculated automatically by UTS 006 software. Some typical AMPT test results are shown in Figure 4.32.

Table 4.12: AMPT test conditions used in this study

Parameter	Unit	Value
Test temperature	°C	4, 20 and 40
Loading frequency	Hz	10, 1 and 0.1 at (4 °C and 20 °C) and 10, 1, 0.1 and 0.01 at 40 °C
Loading mode	-	Haversine
Air void content	%	5 ± 0.5

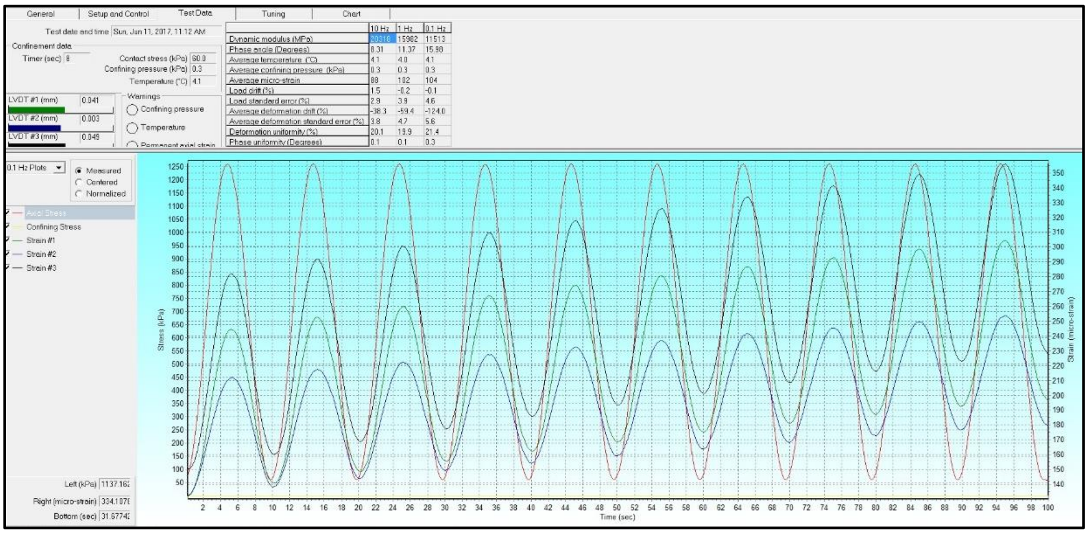


Figure 4.32: Typical results of an AMPT test

4.5.2.3.1 Dynamic modulus master curve construction

Dynamic modulus master curves for asphalt mixtures made with natural aggregates, DC1 and DC2 were constructed to determine their dynamic modulus and phase angle. The master curve enables researchers to predict asphalt mix properties under conditions that were not tested in the laboratory. In the new mechanistic-empirical pavement design guide (MEPDG), asphalt mix stiffness is determined from the master curve constructed at a reference test temperature (M. Witczak & Bari, 2004). The data collected at different temperatures were shifted to form a single smooth function using the time-temperature superposition principle. An example of a typical master curve for an AC10 asphalt mix containing 30% recycled asphalt pavement (RAP) is shown in Figure 4.33 (Denneman et al., 2015). It can be seen that only the shifted data (above and below the reference temperature) were affected by the shifting process.

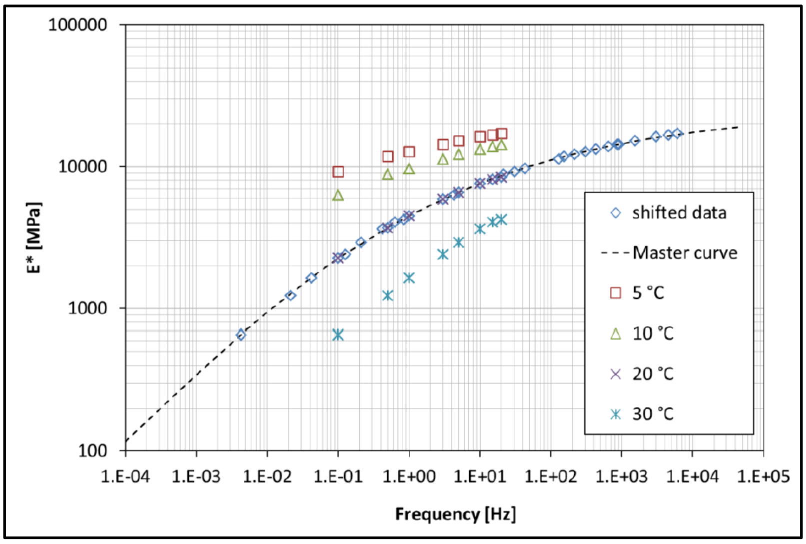


Figure 4.33: Master curve of AC10 asphalt mix at a 20 °C reference temperature (Denneman, Lee, Dias, & Petho, 2015).

Generally, the master curve can be constructed using a sigmoid equation, as per Equation 4.22 (M. Witczak & Bari, 2004):

$$\log|E^*| = \log(Min) + \frac{(\log(Max) - \log(Min))}{1 + e^{\beta + \gamma \log \omega_r}} \qquad 4.22$$

Where $|E^*|$ = dynamic modulus, ω_r = reduced frequency (Hz), *Max* = limiting maximum modulus (Ksi), *Min* = limiting minimum modulus (ksi), and β, and γ = parameters describing the shape of the sigmoid function.

In this research, ω_r was calculated using the Arrhenius equation, Equation 4.23:

$$\log \omega_r = \log \omega + \frac{\Delta E_a}{19.14714}\left(\frac{1}{T} - \frac{1}{T_r}\right) \qquad 4.23$$

Where ω_r = reduced frequency at the reference temperature, ω = loading frequency at the test temperature, *Tr* = reference temperature (20 °C in this study), *T* = test temperature (°C), and ΔEa = activation energy.

By substituting Equation 4.23 into Equation 4.22, the master curve equation can be written as shown in Equation 4.24:

$$\log|E^*| = \log(\min) + \frac{(\log(Max) - \log(Min))}{1 + e^{\beta + \gamma\left\{\log\omega + \frac{\Delta E_a}{19.14714}\left[\left(\frac{1}{T}\right) - \left(\frac{1}{T_r}\right)\right]\right\}}} \qquad 4.24$$

The shift factor for each test temperature is given by Equation 4.25:

$$\log[a(T)] = \frac{\Delta E_a}{19.14714}\left(\frac{1}{T} - \frac{1}{T_r}\right) \qquad 4.25$$

Where a (*T*) = shift factor at temperature T, *Tr* = reference temperature (20 °C in this study), *T* = test temperature (°C), and ΔEa = activation energy.

The limiting maximum modulus was estimated from the volumetric properties of the mix using the Hirsch model and a limiting bitumen modulus of 145,000 psi (1 GPa), as described in Equations 4.26 and 4.27, respectively (Bonaquist, 2008):

$$|E^*|_{max} = P_c\left[4{,}200{,}000\left(1 - \frac{VMA}{100}\right) + 435{,}000\left(\frac{VFA\ x VMA}{10{,}000}\right)\right] + \frac{1 - P_c}{\left[\frac{\left(1 - \frac{VMA}{100}\right)}{4{,}200{,}000} + \frac{VMA}{435{,}000(VFA)}\right]} \qquad 4.26$$

Where |E*| max = limiting maximum mixture dynamic modulus, VMA = voids in mineral aggregates (%), VFA = voids filled with asphalt (%), and P_c was computed using Equation 4.27:

$$P_c = \frac{\left(20 + \frac{435{,}000\,(VFA)}{VMA}\right)^{0.58}}{650 + \left(\frac{435{,}000\,(VFA)}{VMA}\right)^{0.58}} \qquad 4.27$$

The dynamic modulus is considered as a key parameter input for current MEPDG (Apeagyei, Diefenderfer, & Diefenderfer, 2012). The construction of the master curve for Australian HMA made with granite and two types of double-coated RCA (DC1 and DC2) is, therefore, of great importance in characterising the viscoelastic behaviour of these mixes over a wide range of temperatures and loading frequencies.

5 Results and discussion

5.1 Overview

This chapter presents and discusses the results of the tests introduced in Chapter 4. Two DCTs were developed to improve the performance of Australian HMA made with RCAs. The significance of the developed DCTs was assessed through Marshall tests, ITS tests, TSR tests, ITSM tests, wheel tracking tests, flow number tests, four-point bending fatigue life tests, and dynamic modulus tests. The results and discussion are presented in three stages. Stage 1 gives the results and discussion of Marshall tests, stage 2 presents the outcomes of ITS, TSR and ITSM tests, while stage 3 illustrates the findings of performance testing (i.e., rutting, fatigue and dynamic modulus tests).

5.2 Stage 1: Marshall mix design results

Marshall method of mix design was used to determine the OBC of Australian hot-mix asphalt (HMA) made with fresh granite aggregates, RCAs, and two types of DCRCAs (DC1 and DC2). According to Australian standard AS 2150, type AC14 asphalt mixtures should have the typical composition and volumetric properties shown in Table 5.1 (Standards Australia, 2005). In the following sub-sections, Marshall stability and flow, bulk density and maximum density of asphalt mixtures, optimum bitumen content (OBC), the percentage of absorbed and effective bitumen, and volumetric properties are presented and discussed.

Table 5.1: Typical composition and volumetric properties of AC14 mixes (Standards Australia, 2005)

Typical composition and volumetric properties of AC14 mix (Standards Australia, 2005)	
Property	Standard requirements
Stability, kN	≥ 8
Flow, mm	2-4
Air voids, %	3-7
VMA*, %	≥ 15
VFB*, %	60-80
* It is unusual to specify limits for both VMA and VFB (Standards Australia, 2005)	

5.2.1 Stability and flow results

Marshall stability and flow tests were conducted according to test method AS 2891.5 (Australian/New Zealand Standards, 2015). The results of asphalt mixtures made with natural aggregates, RCAs, DC1 and DC2 are shown in Table 5.2.

Table 5.2: Stability (kN) and flow (mm) of HMA made in this study

Stability (kN) and flow (mm) of HMA made in this study.			
Aggregate type	Mix type	Stability, kN	Flow, mm
Granite	0RCAs	17.6	2.3
Uncoated RCAs	20RCAs	17.5	2.6
	40RCAs	18.0	3.0
	60RCAs	18.6	3.2
DC1	20DC1	17.6	2.8
	40DC1	17.5	2.9
	60DC1	18.9	3.1
DC2	20DC2	19.9	2.7
	40DC2	19.5	2.8
	60DC2	18.1	3.0

The results generally indicate that asphalt mixes made with DC1 and DC2 exhibited higher stability than control (i.e., 0RCAs) and RCA mixes. In addition, DC1 and DC2 mixes demonstrated lower flow values than RCA mixes. The stability of an asphalt mix is a function of friction between aggregate particles and adhesion between aggregate and bitumen (Cubuk, Gürü, & Çubuk, 2009). Thus, the high stability of mixes made with DC1 and DC2 can be attributed to better aggregate-bitumen bonding. The mixes containing DC2 demonstrated the highest stability among the prepared asphalt mixes. This may prove that these mixes mobilized higher aggregate-bitumen bonding or higher friction between aggregates. In this regard, after CSP and Sika Tite-BE treatments, DC1 exhibited smoother surfaces, as introduced in Figure 3.16 and Figure 3.17, thus, lower friction was mobilised between aggregate particles.

The Marshall quotient (MQ) values were also calculated and shown in Figure 5.1. It was documented that the MQ can be used as a measure of asphalt mixture resistance to rutting (I. Pérez et al., 2012) and shear stress (Arabani & Azarhoosh, 2012). Thus, a mixture with a high MQ is expected to exhibit better resistance against rutting and cracking. The MQ results in this study can be ranked, from highest to lowest, as control mix (0RCA), DC2 mixes, DC1 mixes and RCA mixes. The inclusion of RCAs and double coated recycled concrete aggregates (DCRCAs), i.e., DC1 and DC2, decreases the MQ. However, the addition of DC1 and DC2 improves MQ values compared to mixes made with uncoated RCAs. This can indicate the possibility of adhesion enhancement between aggregates and bitumen when DCTs are used.

Based on the results of stability and flow and MQ, the DC2 mixes exhibited, in general, higher stability, lower flow, and higher MQ test results compared to asphalt mixtures made with different dosages of DC1 and RCAs. It can, therefore, be concluded that the DC2-asphalt mixes performed better than DC1-asphalt mixes regarding stability, flow and MQ.

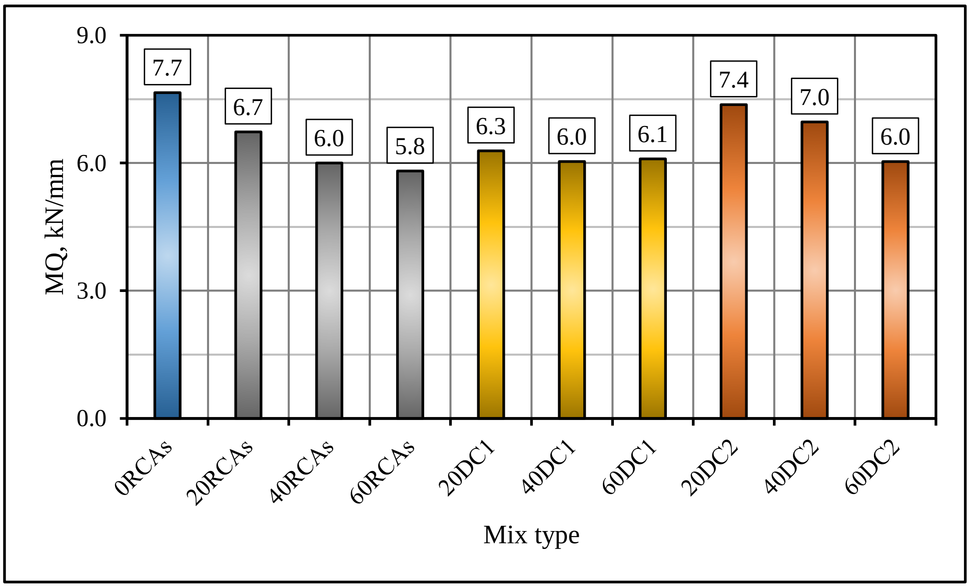


Figure 5.1: MQ values of HMA made in this study

5.2.2 Bulk and maximum densities of asphalt mixtures

According to the experimental results presented in Chapter 3, Section 3.10.2, the densities of uncoated RCA, DC1 and DC2 were lower than that of natural granite aggregate, while their water absorption rates were significantly higher. These results indicate the high porosity of the RCAs used in this study. Even though DCT1 and DCT2 helped reduce the water absorption rate of RCAs, both types of DCRCAs (DC1 and DC2) still absorbed more water than natural aggregate. The addition of RCAs, DC1 and DC2 into HMA is, therefore, expected to decrease the bulk density of compacted asphalt mixtures and the maximum density of loose asphalt.

Figure 5.2 and Figure 5.3 show the relationship between the percentage of RCA, DC1, and DC2 with the bulk density of compacted asphalt mixtures, and the maximum density of loose asphalt, respectively. It can be seen that the bulk and maximum densities of the Australian HMA decrease after the addition of RCA and the two types DCRCAs into the mix.

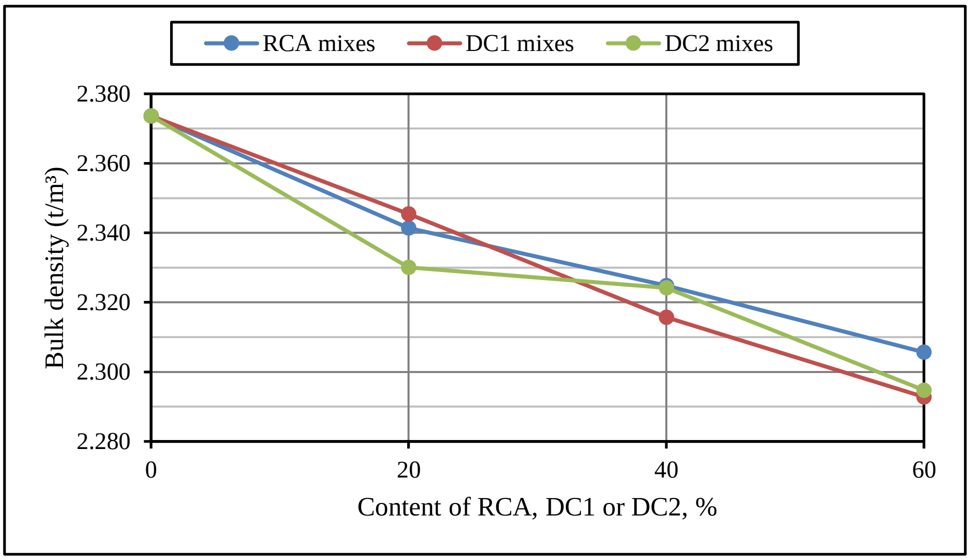


Figure 5.2: RCAs, DC1, and DC2 contents vs bulk density of compacted asphalt mixtures

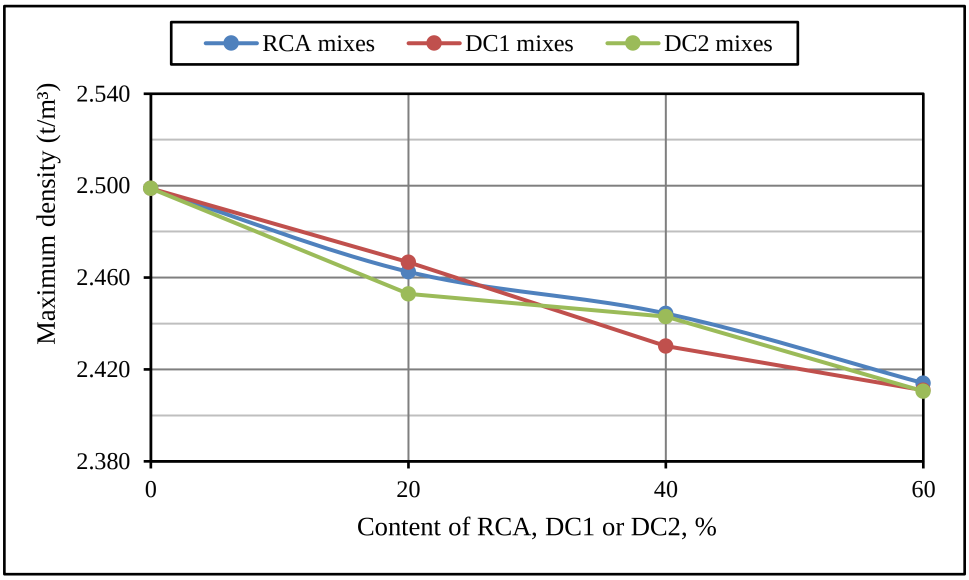


Figure 5.3: RCAs, DC1, DC2 contents vs maximum density of loose asphalt mixtures

It can be seen that the addition of DC1 or DC2 leads, in general, to decrease in the bulk density and maximum density of asphalt. The latter finding might be related to the lower densities of coating materials such as Sika Tite-BE and CSP. The lower density of the DC1 and DC2 mixes means that less materials need to construction of asphaltic roads compared to that required if granite or uncoated RCA mixes are used (Gómez-Meijide & Pérez, 2016).

5.2.3 Optimum bitumen content, absorbed bitumen, and effective bitumen

The Marshall method of mix design was mainly used to determine the OBC of Australian HMA made with granite aggregates, RCAs, and two types of DCRCAs (DC1 and DC2). The optimum bitumen content (OBC) of all asphalt mixtures made in this investigation was determined at 5% air void contents, as per accepted Australian practices. The OBC results of all the asphalt mixes are shown in Figure 5.4.

It can be seen that the OBC increases as the percentage of RCAs, DC1, and DC2 increases in the mix. According to the results obtained, asphalt mixtures made with DC1 or DC2 had lower OBCs than corresponding mixes made with RCA. One exception was noted: the 20DC1 mix had an OBC (4.47%) slightly higher than that of the 20RCAs mix (4.44%). However, both types of DCRCAs (DC1 and DC2) still had higher OBCs than that of fresh granite aggregate. This result can be explained by the water absorption rates of granite, RCA, DC1 and DC2 reported in Chapter 3, Section 3.10.2. The lowest OBC was obtained for the control mix, while the highest OBC was calculated for RCA mixes.

The rough and porous surface texture of RCAs leads to a high amount of bitumen being absorbed into pores presented onto RCA surfaces. However, the two DCTs (DCT1 and DCT2) obstruct the majority of pores and cracks present on the RCA surfaces, as shown in Chapter 3 (Figures 3.17 and 3.18), and thereby reduce the absorption of RCAs.

Based on the results shown in Figure 5.4, the DC2 mixes demonstrated lower OBCs than the corresponding DC1 and RCA mixes. This result can be explained by the effects of the water absorption rates of RCA, DC1 and DC2 introduced in Chapter 3, Tables 3.11, 3.12, and 3.13, respectively. The trend in water absorption rate for the different types of aggregate, from highest to lowest, was: RCA, DC1 and DC2. This was similar to the OBC trend of the asphalt mixes made in this investigation. The fact that asphalt mixes made with DC1 and DC2 tended to have lower OBCs than those of RCA mixes should encourage the use of DCTs in HMA production. The lower amount of bitumen needed for DC1 and DC2 mixes than for RCA mixes may compensate for the cost of the materials used in the DCTs.

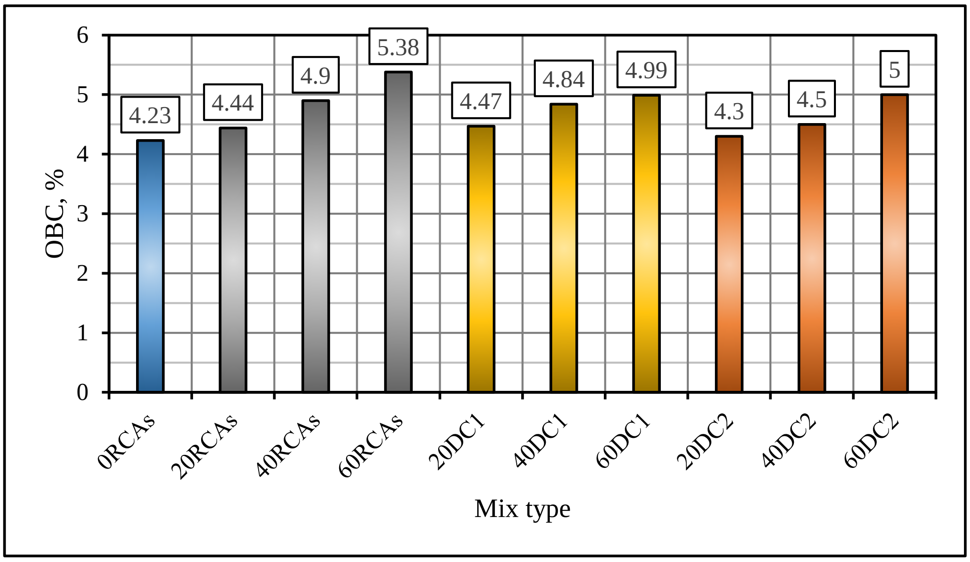


Figure 5.4: OBCs of the asphalt mixtures made in this study

The percentages of absorbed bitumen and effective bitumen of the asphalt mixtures made in this study were determined according to Equations 4.8 and 4.9 (Chapter 4) and are presented in Figure 5.5 and Figure 5.6. It can be seen that the amount of absorbed bitumen increased as the percentage of RCA, DC1 or DC2 increased in the mix. This explains the higher OBCs in the RCA, DC1 and DC2 mixes compared with that of the control mix.

Although DC1 mixes had lower OBCs than those obtained for corresponding RCA mixes, they were found to have slightly higher absorbed bitumen than RCA mixes. This might be related to the higher absorption rate of RCAs coated with CSP, as explained in Chapter 3 (Table 3.12). However, the percentages of effective bitumen (Be, %) of all asphalt mixtures were generally comparable, as graphically presented in Figure 5.6. This may prove that the differences in the OBCs of asphalt mixes mainly resulted from differences in the water absorption rates of the aggregates used.

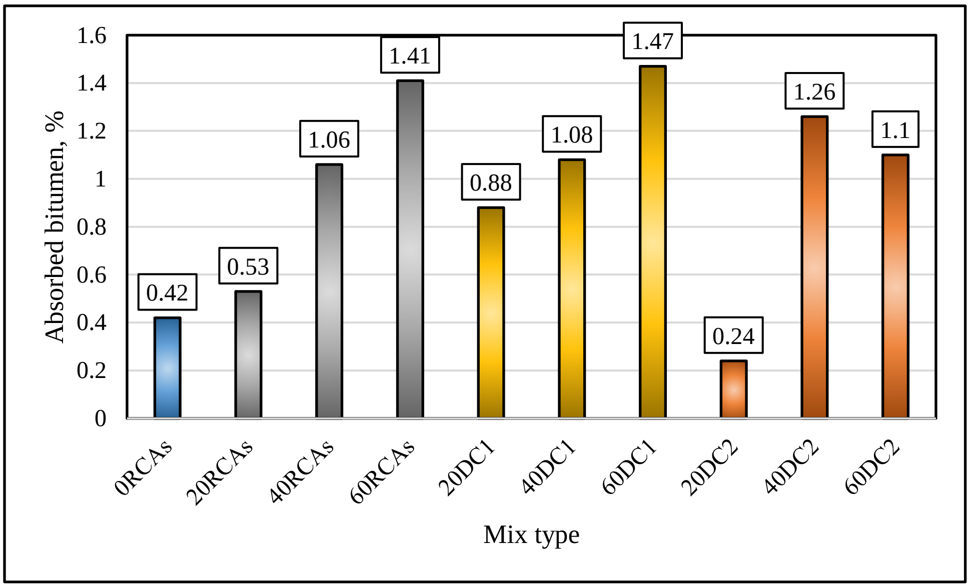


Figure 5.5: Percentage of absorbed bitumen (b) of asphalt mixes made in this study

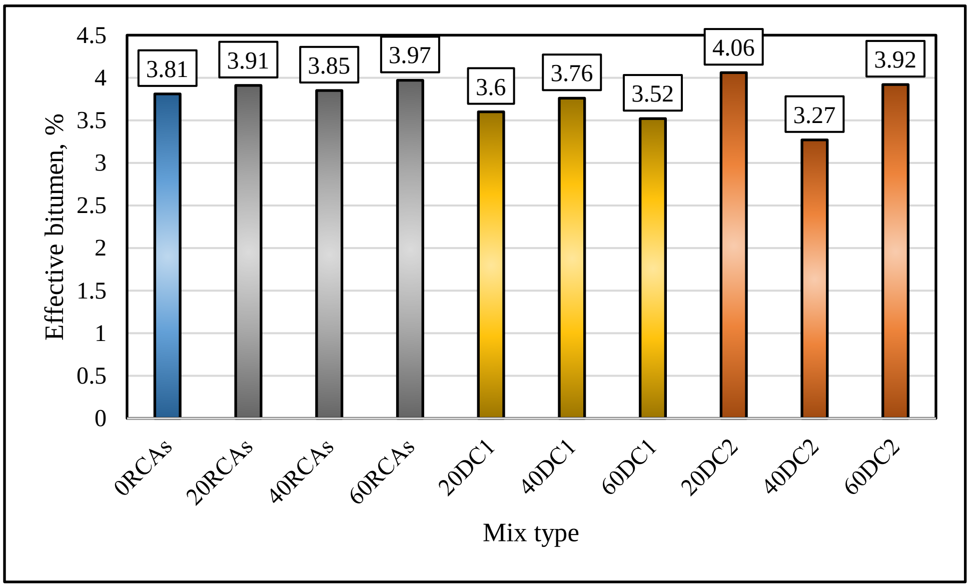


Figure 5.6: Percentage of effective bitumen of asphalt mixes produced in this study

5.2.4 VMA and VFB

Figure 5.7 and Figure 5.8 show the VMA and VFB values of asphalt mixtures made with granite aggregates (0% RCAs) and 20%, 40% or 60% of RCAs, DC1 and DC2. Although no clear trend is observed, especially when 20% or 60% of DC1 or DC2 was added to HMA, the addition of RCAs (uncoated or double coated) was, in general, found to decrease the VMA and VFB of the mixture. These results are in accordance with those obtained by (Fatemi & Imaninasab, 2016; Paranavithana & Mohajerani, 2006; Zulkati et al., 2013).

As illustrated in the VMA and VFB graphs, the following trend was observed, from highest VMA and VFB to lowest: DC2 mixes, RCA mixes, DC1 mixes. This may be linked to the amount of bitumen absorbed by these mixes; where the DC1 mixes absorbed higher bitumen than the RCA and DC2 mixes. A greater amount of absorbed bitumen will translate into less effective bitumen being available to fill the gaps between aggregate particles and vice versa. Consequently, lower VMA and VFB values were obtained in case of asphalt mixtures produced with DC1.

Furthermore, all VMA values were below 15%, which represents the minimum level of VMA for AC14 according to AS 2150. In addition, all VFB values were above 60%, which indicates that all asphalt mixes satisfied the VFB requirements (Standards Australia, 2005). It should be mentioned that based on the Australian standard, it is unusual to specify limits for both VMA and VFB for HMA with the nominal aggregate sizes of 10, 14, 20, 28 and 40 (Standards Australia, 2005).

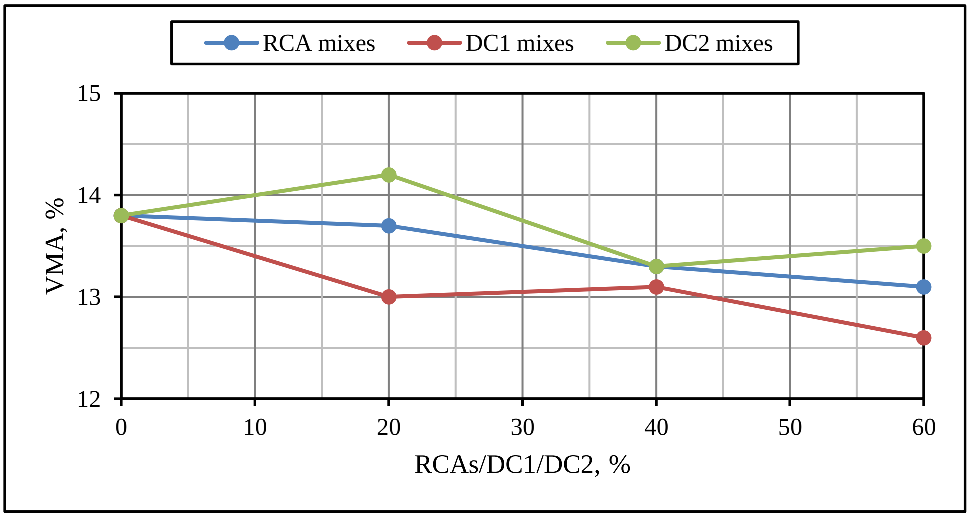


Figure 5.7: VMA of asphalt mixtures versus content of RCA, DC1 and DC2

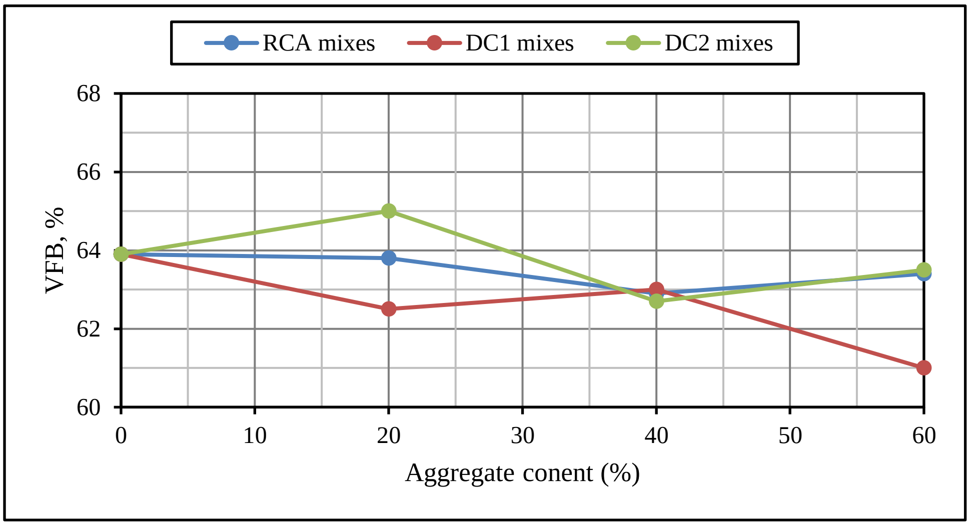


Figure 5.8: VFB of asphalt mixtures versus contents of RCA, DC1 and DC2

5.2.5 Comparison of DC1 and DC2 mixtures based on Marshall properties

According to the results of the Marshall tests results for the ten asphalt mixtures produced at this stage of the investigation, the following comparison between DC1 and DC2 mixes can be made:

- The stabilities of the DC1 and DC2 mixes were generally higher than those of the control and RCA mixes. This indicates the possibility of improving the adhesion between the two types of double coated recycled concrete aggregates (DCRCAs) and bitumen and, thus, the stability of DC1 and DC2 mixes was higher than that obtained for the control and RCA mixes. The DC2 samples had higher stability than the corresponding DC1 samples. This finding can be understood in light of the effects of the DCT1. Both CSP and Sike Tite-BE treatments decrease the ability of coated RCAs to mobilise high friction among aggregates, which made the DC1 samples exhibit generally lower stability than the DC2 samples. The DCT2, however, seems to improve the bonding between aggregate and bitumen while maintaining high friction between DC2 aggregates.

- The bulk and maximum densities of DC1 and DC2 mixes were generally lower than those obtained for RCA mixes. This is considered to be related to the lower densities of coating materials (CSP and Sika Tite-BE) compared to RCA. The DC2 mixes tended to demonstrate slightly higher bulk and maximum densities than DC1 mixes, particularly with 40% or 60% replacement of natural aggregate.
- According to the OBC results, DC1 and DC2 mixes exhibited lower OBCs than corresponding RCA mixes. The following trend of OBC was noted, from lowest to highest: 0R, DC2 mixes, DC1 mixes, and RCA mixes. The OBC results reflect the absorption rate of the aggregates used in HMA production. Thus, when the water absorption rate increases, the OBC increases accordingly. It can be said that the DCT2 (coating with Sika Tite-BE and heating) is more effective than DCT1 (coating with CSP and Sika Tite-BE) in mitigating the absorption nature of the RCAs used in this study.
- DC1 mixes demonstrated lower VMA and VFB than DC2 mixes. Furthermore, the greater the content of DC1 or DC2, the lower the VMA and VFB percentages. This finding can be related to the amount of bitumen absorbed by DC1 and DC2. Due to the CSP treatment, DC1 absorbs more bitumen and water than DC2 resulting in less bitumen being available to fill the gaps between the aggregates particles.

5.3 Stage 2: ITS, TSR, and ITSM results

After determining the Marshall properties of the asphalt mixtures, the indirect tensile strength (ITS), tensile strength ratio (TSR) and indirect tensile stiffness modulus (ITSM) tests were performed. Through these tests, the indirect tensile strength, moisture-induced damage and resilient modulus of type AC14 asphalt mixes were assessed. In this stage of the experiments, only eight asphalt mixtures were evaluated, as demonstrated in Chapter 4 (Figure 4.12). These mixes were the 0RCAs mix (control mix), mixes made with 20%, 40% or 60% of DC1 and DC2, and the mix with 40% RCA. The last mix was made and evaluated for comparison purposes. The following sub-sections present and discuss the results of these tests.

5.3.1 ITS test results

The ITS of asphalt mixtures can define their cracking and rutting characteristics. In this study, the indirect tensile strength of asphalt specimens of different mixes was measured using the ASTM D6931 specification, as mentioned in Section 4.5.1.2. In order to assess the effects of temperature and void contents on ITS, the tests were performed at 25 ± 1 °C and 40 ± 1 °C, and air void contents of 5 ± 1% and 8 ± 1%.

5.3.1.1 Effect of test temperature on ITS

The effect of testing temperature on the ITSs of AC14 asphalt mixtures is shown in Figure 5.9. In this graph, it is obvious that the test temperature greatly affected the ITSs of the mixes. It can be seen that the addition of DC1 into HMA decreased the ITS at 25 ± 1 °C and 40 ± 1 °C compared to that of the control mix and 40RCA mixes. However, asphalt mixes made with DC2 exhibited comparable or even higher ITSs than those of control mix and 40RCA mixes. In the case of DC1 mixes, the decrease in ITS could be related to the effect of CSP and Sike Tite-BE coatings. The double coating action of DCT1 helped to fill pores and cracks present on the RCA surfaces and, thus, a relatively smooth surface was produced. This situation led to lesser friction forces being mobilised by the DC1-asphalt specimens upon testing. However, the ITS results for the DC2 mixes demonstrated a different trend. The combined treatment of Sika Tite-BE coating and heating seemed to upgrade the adhesion between DC2 particles providing high ITS in case of DC1 samples.

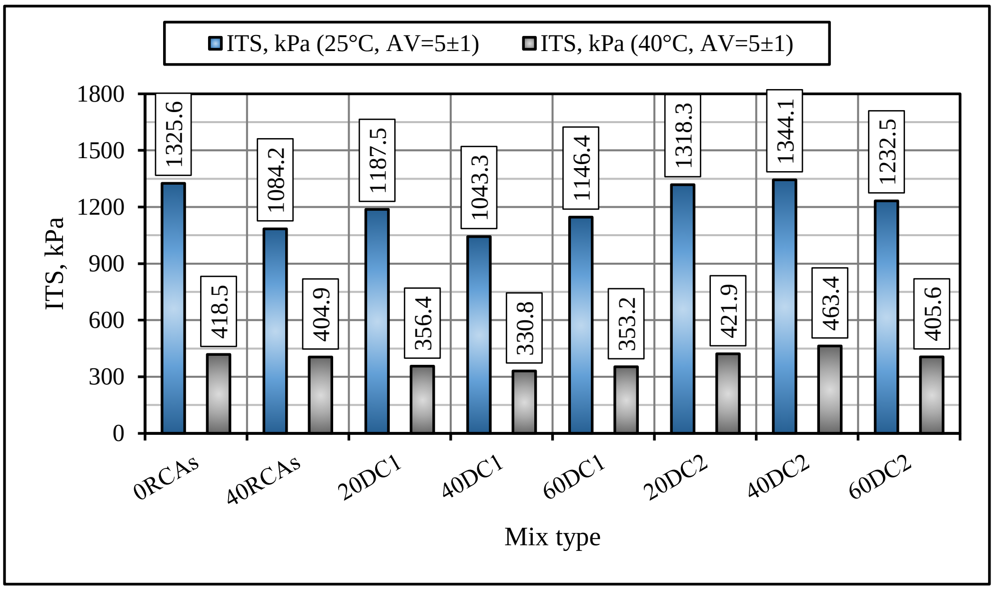


Figure 5.9: ITS of asphalt mixtures at 25 °C and 40 °C

Based on the ITS results at different temperatures, the addition of double-coated RCAs (DC1, and DC2) can in general decrease the ITS than that of control mix. For example, the ITSs of the 20DC1 and 20DC2 mixes tested at 25 °C were 1187.5 and 1318.3 kPa, respectively. However, the ITS of the 60DC1 and 60DC2 mixes tested at 40 °C were 353.2 and 405.6 kPa, respectively. The ITSs of DC2 mixes reveal that the inclusion of DC2 can improve the ITS of asphalt mixes containing up to 40% DC2 compared to that of 0RCAs mix. The DC2 mixes also exhibited better ITSs than the 40RCAs mix and corresponding DC1 mixes. This improvement in ITS in the DC2 mixes could be explained by better aggregates-bitumen bonding being achieved after Sika Tite-BE and heating treatments. The results also indicated that the use of DCTs to upgrade the behaviour of HMA made with RCA derived from C&D waste has helped to achieve comparable ITS values regardless the contents of DC1 and DC2 in the mix.

5.3.1.2 Effect of air voids on ITS

Asphalt samples were made to have 5 ± 1 (%) and 8 ± 1 (%) air void contents to examine the effects of void contents on the ITS properties of asphalt mixes. Figure

5.10 shows the ITS results for asphalt specimens at two air void contents (5 ± 1% and 8 ± 1%) and tested at 25 °C. It seems that the air void contents of asphalt mixes also have a definite effect on the ITS properties. For instance, the 0RCAs mix made with 5 ± 1% and 8 ± 1% air void contents had ITSs of 1325.6 kPa and 871.5 kPa, respectively. This means that the control mix lost 454.1 kPa of its ITS when the air void content increased by about 3%.

Also, the DCTs developed for the purposes of this study had a great effect on the ITS of asphalt mixtures. The addition of DC1 aggregate decreased the ITS of DC1 mixes with different air void contents, while the addition of DC2 increased the ITS with up to 40% granite substitution, as compared with the 0RCAs and 40RCAs mixes. This happened with 5 ± 1% and 8 ± 1% air void contents, as shown in Figure 5.10. It is clear that the DC2 mixes had higher ITS values than those obtained for DC1 mixes. It was stated by Lee et al. (2012) that CSP coating increased the ITS of asphalt mixes. However, the second coating used in DCT1 (Sika Tite-BE coating) generated a smooth surface texture after CSP coating. This, accordingly, decreased the ITS of the DC1 mixes. On the other hand, coating with Sika Tite-BE and heating at 155 °C for 1½ hours seemingly increased the ITS of HMA.

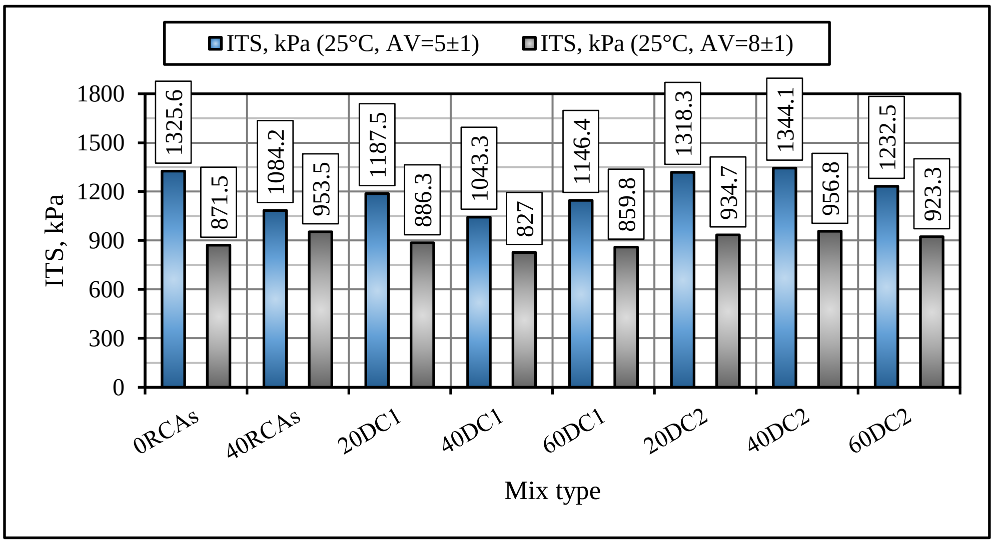


Figure 5.10: Effect of air void content on the ITS of the asphalt mixtures made in this study

5.3.1.3 Analysis of Variance (ANOVA) of ITS results

Statistical analysis is typically carried out to minimise the subjective criteria of researchers when analysing test results. Analysis of variance (ANOVA) is a statistical tool used to examine the reliability of the findings. In the following sections, ANOVA was carried out to assess the effects of DC1 and DC2 contents, and other test factors such as air void content and testing temperature on ITS. The effect of the individual independent variable is named as *main effect*, while the effect of both independent variables is named as *interaction effect*. ANOVA aims to determine the level of significance of the results based on a selected level of probability (p-value), which was 0.05 in this study. If the p-value is lower than 0.05, the null hypothesis is rejected, and the results are statistically significant.

5.3.1.3.1 Effects of DC1 content, air void content and test temperature on ITS

The results obtained for asphalt mixes made with different percentages of DC1 (0%, 20%, 40% or 60%) were statistically examined. A two-way ANOVA was performed to investigate the effects of DC1 dosage and air void content (5% or 8%) on ITS. Another two-way ANOVA was performed to test the effects of DC1 content and testing temperature (25 °C or 40 °C) on ITS. The first ANOVA indicated that both factors (%DC1 and air void content) were significant at the 95% confidence level ($p < 0.05$). In addition, the second ANOVA showed the significant effects of both investigated factors (%DC1 and test temperature) on ITS ($p = 0.000$). Based on ANOVA, any change in the DC1 percentage, test temperature or air void content will produce a change in the ITS of DC1 mixes. Based on the ANOVA results, the addition of DC1 can greatly affect the ITS properties of asphalt mixes. The combined effect of %DC1 and test temperature is higher than those obtained for %DC1 and air voids. This was verified by the p-value of interaction shown in Table 5.3 and Table 5.4, which equals 0.016 (for %DC1 and air void content) and 0.006 (for %DC1 and test temperature) respectively.

Table 5.3: Two-way ANOVA results for the effect of %DC1 and air void content on ITS

Two way ANOVA results of %DC1 and air voids						
Source of variation	SS	df	MS	*F*	*p-value*	*F* crit
%DC1	88309.60537	3	29436.54	8.243438	0.001527	3.238872
%Air voids	560359.3124	1	560359.3	156.9236	1.1E-09	4.493998
Interaction	49553.57173	3	16517.86	4.625678	0.016328	3.238872
Within	57134.48341	16	3570.905	-		
Total	755356.9729	23	-			

Table 5.4: Two way ANOVA results of %DC1 and test temperature

Two way ANOVA results of %DC1 and test temperature.						
Source of Variation	SS	df	MS	F	P-value	F crit
%DC1	112084.8	3	37361.6	20.50429	9.94E-06	3.238872
Test temperature	3859638.9	1	3859638.9	2118.194	1.97E-18	4.493998
Interaction	32806.3	3	10935.4	6.001432	0.006113	3.238872
Within	29154.2	16	1822.1	-		
Total	4033684.24	23	-			

5.3.1.3.2 Effect of DC2 content, air void content and test temperature on ITS

A two-way ANOVA analysis was performed to investigate the effects of %DC2 and air void content on ITS. Another two-way ANOVA was carried out to examine the effect of %DC2 and test temperature on ITS. The first ANOVA indicates that the effect of air void content was significant ($p = 0.000$) but the %DC2 was not ($p = 0.121$). The ANOVA results indicate that the DCT2 helped to produce an ITS comparable ITS regardless DC2 content in the mix,. However, the second ANOVA confirms that both individual factors, %DC2 and test temperature, significantly affected ITS ($p = 0.004$ and 0.000, respectively).

According on the ANOVA analysis, the test temperature and air void content had significant effects on ITS. In addition, the test temperature had a greater effect on the ITS of DC2 mixes (p = 2.96E-33) than air void content (p = 2.82E-29), as shown in

Table 5.5 and Table 5.6, respectively. Furthermore, the interaction effect of %DC2 and test temperature was significant (p = 0.049). However, the interaction between %DC2 and air void content was not (p = 0.116).

Table 5.5: Two-way ANOVA results for the effects of %DC2 and air void content on ITS

Two-way ANOVA results of %DC2 and air voids.						
Source of variation	SS	df	MS	*F*	*p*-value	*F* crit
%DC2	12103.82	3	4034.605072	2.14505	0.120941	3.008787
%Air voids	10763489	2	5381744.472	2861.274	2.82E-29	3.402826
Interaction	21827.89	6	3637.982474	1.93418	0.116115	2.508189
Within	45141.38	24	1880.890896	-		
Total	10842562	35	-			

Table 5.6: Two-way ANOVA results for the effects of %DC2 and test temperature on ITS

Two-way ANOVA results of %DC2 and test temperature.						
Source of variation	SS	df	MS	*F*	*p*-value	*F* crit
%DC2	14735.65	3	4911.881821	5.703227	0.004285	3.008787
Test temperature	10598165	2	5299082.337	6152.809	2.96E-33	3.402826
Interaction	13065.71	6	2177.618187	2.528451	0.048555	2.508189
Within	20669.91	24	861.24606	-		
Total	10646636	35	-			

5.3.1.4 DC1 mixtures versus DC2 mixtures based on ITS results

Based on the ITS values of asphalt mixes tested at different air void contents and temperatures, the following comparison between DC1 and DC2 mixes can be made:

- At 25 °C and 40 °C test temperatures, the DC2 mixes made with 5% air voids always demonstrated higher ITS values than corresponding DC1 mixes. The reason is related to the effects of the treatments used in DCT1 and DCT2. The second treatment of DCT1, i.e., Sika Tite-BE treatment, is expected to decrease the resultant friction forces between DC1 particles. However, the Sika Tite-BE and heating treatments seemingly increased the adhesion between DC2 and bitumen while maintaining high friction between DC2 particles. In this regard, the 40DC1 mix was exhibited lower ITS than the 40RCAs mix at 25 °C and 40 °C, which is also considered to be linked to the effects of CSP and Sika Tite-BE treatments.

- Additionally, the DC2 specimens made with 5% and 8% air voids and tested at 25 °C always showed higher ITS than corresponding DC1 specimens. This is also can be connected to the effects of the treatments used in the DCT1 and DCT2.
- According to the ANOVA results, the percentage of DC1 had a significant effect on ITS at different air void contents and testing temperatures. However, the percentage of DC2 had a significant impact on ITS only when the test carried out at different temperatures. The ANOVA indicates that the percentage of DC2 (20%, 40% or 60%) did not have a significant impact on the ITS of asphalt mixes when the DC2 specimens are made at different air void contents.

5.3.2 Tensile strength ratio (TSR) test results

The moisture-induced damage tests of asphalt mixes made in this study was evaluated following Australian standard AG:PT/T232. Eight asphalt mixtures were designed and evaluated. These are the mix made with 0% RCAs (the control mix), the mixes made with 20%, 40%, and 60% of DC1 and DC2 respectively, and 40RCAs mix. The last mix was made and evaluated for comparison purposes. The TSR is expressed as a percentage between the ITS of wet (conditioned) and dry (unconditioned) asphalt specimens and computed using Equation 4.15. The following sub-sections introduce and discuss the TSR test results.

5.3.2.1 Indirect tensile strength of dry and wet asphalt samples

Figure 5.11 shows the tensile strength of the dry (TSD) and wet (TSW) asphalt mixtures tested in the present study. According to the graph of dry-wet tensile strength, the 0RCAs and 60DC1 mixes, exhibited TSD higher than TSW. The addition of 1.5% hydrated lime (by the weight of dry aggregate) might explain such results (Nega, Nikraz, & Leek, 2013; Nega, Nikraz, Leek, & Ghadimi, 2013). Furthermore, the 40DC2 mix achieved the highest TSD and TSW compared with the 0RCAs, 40RCAs, and 40DC1 mixes.

It can be seen that both DCTs helped to decrease the gap between the TSD and TSW values of asphalt mixes made with 20%, 40% or 60% of DC1 and DC2 when compared to that exhibited by the 40RCAs mix. In addition, the DC1 mixes always demonstrated lower TSD and TSW values than those of corresponding DC2 mixes. Only one exception was observed: the 60DC2 mix demonstrated lower TSW than its equivalent 60DC1 mix. The results revealed the possibility of adhesion enhancement between bitumen and both types of DCRCAs (DC1 and DC2) especially when looking at TSW values.

Generally, the proposed DCT1 method decreases the average tensile strength of dry samples. These results contradict the results obtained by Lee et al. (2012), where the TSD was found to increase with increasing percentages of RCA coated with CSP (see Figure 3.2). Therefore, it could be said that after Sika Tite-BE coating, the friction forces mobilized by the DC1-asphalt mixes decreased.

Furthermore, both DCTs seem to be suitable for enhancement of water resistance in asphalt mixtures made with RCAs derived from C&D waste. These coating techniques help to seal the majority of pores present on RCAs, leaving no space for water to enter between bitumen and DCRCA surfaces. Therefore, in the case of DCT2, the TSWs of mixes made with 20%, 40% and 60% DC2 were close to the TSDs, as shown in Figure 5.11. The gap between dry and wet tensile strength was further reduced in the case of DC1 mixes. This is considered a result of Sika Tite-BE treatment, which decreases the TSD, as displayed in Figure 5.11.

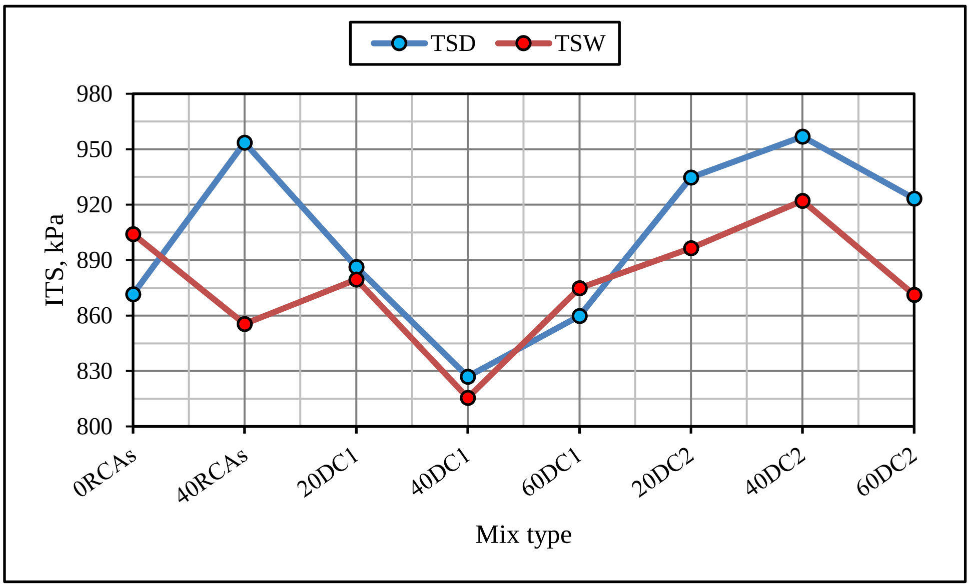


Figure 5.11: Tensile strength of dry and wet asphalt samples made in the present study.

5.3.2.2 TSR of asphalt mixes

The TSR results for the asphalt mixtures made in the present study are presented in Figure 5.12. The highest TSR (103.7%) was achieved by the control mix (0RCAs), followed by the 60DC1 mix (101.7%). Although the TSRs of the control mix and 60DC1 mix seem hard to understand, this result is reasonable based on acceptable Australian practices. According to the technical review carried out in this field, researchers in different studies obtained TSRs higher than 100% for Australian asphalt mixes (Nega, Nikraz, & Leek, 2013; Nega, Nikraz, Leek, et al., 2013). The researchers in these studies obtained TSRs of 103% and 112.9% for two types of Australian hot-mix asphalt made with natural aggregate. In this regard, the TSR of the 60DC1 mix might be understood in the light of DCT1 effects and the addition of hydrated lime.

In can be seen that the addition of 40% uncoated RCAs significantly affects moisture-induced damage in Australian HMA. The inclusion of 40% RCA has caused the TSR to decrease from 103.7% (control mix) to 89.7% (40RCAs mix). In keeping with this, the use of DCT1 and DCT2 seemingly improved the moisture resistance of asphalt mixes made with 40% DC1 or DC2.

However, the DC1 mixes always displayed higher TSRs compared with corresponding DC2 mixes. The use of DCT1 decreased the gap between TSD and TSW, as shown in Figure 5.11. As a result, TSRs of 99.2%, 98.6% and 101.7% were achieved by asphalt mixes made with 20, 40, and 60% DC1, respectively. However, the gap between the TSD and TSW of DC2 mixes was higher than that of DC1 mixes. This led to achieving TSRs of 95.9%, 96.4% and 94.4% by the 20DC2, 40DC2 and 60DC2 mixes, which are lower than those obtained for DC1 mixes.

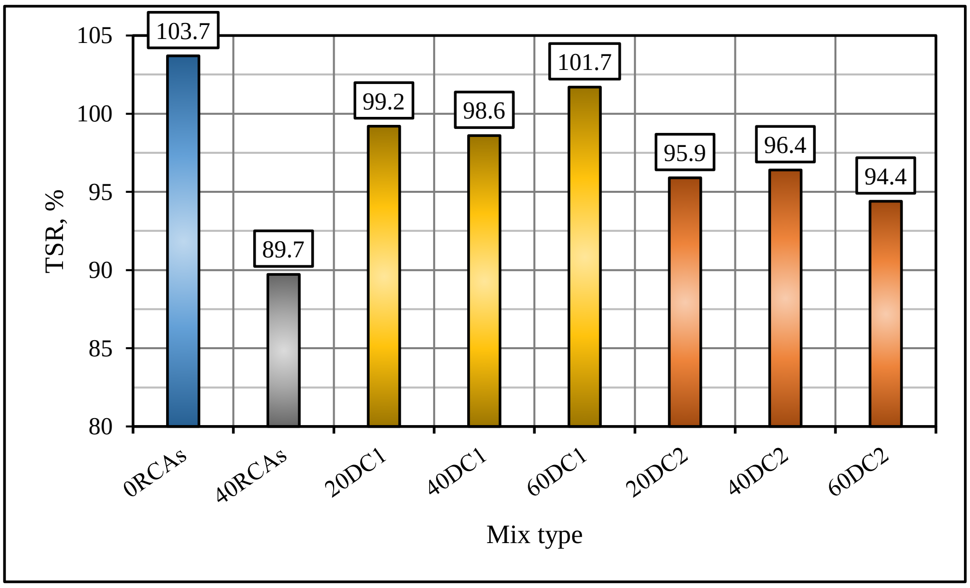


Figure 5.12: TSR of asphalt mixtures made in the present study.

5.3.2.3 Analysis of variance (ANOVA) of TSR test results

5.3.2.3.1 Effect of DC1 content on TSR

A one-way analysis of variance (ANOVA) was conducted to investigate the effect of DC1 percentage on the TSR of the asphalt mixes. The results of the first ANOVA are presented in Table 5.7. The ANOVA indicates that the percentage of DC1 did not significantly affect TSR (p = 0.958). This implies that the change in the DC1 percentage will not produce a significant change in the water resistance of the produced mix.

Table 5.7: One-way ANOVA results: effect of %DC1 on TSR

One-way ANOVA results: effect of %DC1 on TSR.						
Source of variation	SS	df	MS	*F*	*p*-value	*F* crit
Between groups	46.00253	3	15.33418	0.100269	0.957606	4.066181
Within groups	1223.445	8	152.9306	-		
Total	1269.447	11	-			

5.3.2.3.2 Effects of DC1 content and state condition on ITS

Two-way ANOVA was carried out to study the effects of DC1 percentage and state condition (dry or wet) on ITS (Table 5.8). The DC1% was not significant (p = 0.402). The two-way ANOVA also confirms that the dry and wet state were also not significant (p = 0.949). The results reveal that DCT1 provides protection to RCAs and decreases the difference between the tensile strengths in the dry and wet states, as explained in Section 5.3.2.1 and 5.3.2.2, regardless the percentage of DC1 used in the mix.

Table 5.8: Two-way ANOVA results: effects of %DC1 and state condition on ITS

Two-way ANOVA results: effects of %DC1 and state conditions on ITS.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC1	12788.49	3	4262.829	1.038086	0.402	3.238872
State condition	17.33946	1	17.33946	0.004223	0.949	4.493998
Interaction	810.5592	3	270.1864	0.065796	0.977	3.238872
Within	65702.88	16	4106.43	-		
Total	79319.27	23	-			

5.3.2.3.3 Effect of DC2 on TSR

A one-way ANOVA was conducted to investigate the effect of DC2 content on TSR, which was found to be not significant (p = 0.513; Table 5.9). This confirms that the change in DC2 dosage had a negligible effect on TSR.

Table 5.9: One-way ANOVA results: effect of %DC2 on TSR

One-way ANOVA results: effect of %DC2 on TSR.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
Between groups	152.7	3	50.90154	0.8312741	0.5131437	4.0661805
Within groups	489.8	8	61.23316	-		
Total	642.5	11	-			

5.3.2.3.4 Effects of DC2 content and state condition on ITS

A two-way ANOVA was carried out to examine the effect of DC2 content and dry/wet state on ITS. Neither was significant (p = 0.265 and = 0.309, respectively). It can be concluded that the DCT2 improves the affinity between the recycled particles and bitumen without affecting the ability of these particles to mobilize high friction forces upon testing. This led the DC2-asphalt specimens to achieve, to some extent, comparable ITS values in dry and wet test conditions regardless of the dosage of DC2 used in the mix.

Table 5.10: Two-way ANOVA results: effects of %DC2 and state condition on ITS

Two-way ANOVA results: effects of %DC2 and state conditions on ITS.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC2	9329.044	3	3109.681	1.295964	0.309	3.238872
State condition	3202.578	1	3202.578	1.334679	0.265	4.493998
Interaction	6468.986	3	2156.329	0.898653	0.463	3.238872
Within	38392.19	16	2399.512			
Total	57392.8	23				

5.3.2.4 Comparison of TSR for DC1 and DC2 mixtures

Based on the results of the TSR tests presented in the above sections, a comparison between asphalt mixes produced with DC1 and DC2 was made. The following points are summarised according to TSR test results:

- DC2 mixes demonstrated higher TSD and TSW than corresponding DC1 mixes. In this regard, the Sika Tite-BE treatment is considered to be the main factor that reduced the tensile strength of DC1 mixes particularly the TSD..
- Both DCTs (DCT1 and DCT2) helped to decrease the gap between the dry and wet ITSs of asphalt mixes made with 20%, 40% and 60% DC1 and DC2 compared to that of the 40RCAs mix. The results revealed that the gap between dry and wet tensile strength was further decreased in the case of DC1 mixes.
- DC1 mixes exhibited higher TSR than their equivalent DC2 mixes. This is considered to be a result of the effects of CSP and Sika Tite-BE treatments used in DCT1.
- According to ANOVA, the dosages of DC1 and DC2 had no significant effect on the ITS of asphalt specimens containing DC1 or DC2. Furthermore, the test condition (dry or wet) was also not significant. This indicates that both DCTs provide sufficient protection against moisture and reduce the difference

between the tensile strengths in the dry and wet conditions, regardless of the percentage of DC1 and DC2 in the mix.

5.3.3 Indirect tensile stiffness modulus (ITSM) test results

ITSM tests were conducted to measure the resilient modulus of asphalt mixtures made with granite aggregates and different dosages of DC1 and DC2 (20%, 40% or 60%). In addition, a mix with 40% uncoated RCA was made for comparison purposes. Three samples were tested to obtain the mean resilient modulus of each mix. The test was carried out at two temperatures (25 °C and 40 °C). The following sections introduce the resilient modulus results and discussion.

5.3.3.1 Resilient modulus at 25 °C

The resilient modulus results at 25 °C for all asphalt mixtures are presented in Table 5.11. The results reveal that utilization of DC1 in HMA production decreases the asphalt mix's stiffness. As explained earlier in this chapter, the second treatment used in DCT1 (Sika Tite-BE treatment) reduced the friction forces mobilized by DC1 aggregates and, thus, a lower stiffness was achieved by DC1-mixes.

A different trend can be seen in the results obtained for DC2 mixes. The addition of coarse RCA coated with Sika Tite-BE and cured in an oven at 155 °C for 1½ hours increased the stiffness up to 40% granite aggregate substitution. This is expected to be related to the effect of the Sika Tite-BE coating treatment, which improved the durability and strength of coated RCAs, as explained previously in Section 3.10.2.2. Additionally, keeping the DC2-asphalt mixtures in an oven maintained at 155 °C for 1.5 hours allowed some time for uncoated areas to obstruct after absorbing some bitumen. Thus, the adhesion between these particles and bitumen could improve. At 60% granite replacement, the stiffness of DC2-asphalt mixes exhibited a decreasing trend. Therefore, the DC2 mixes are expected to demonstrate a decline in resilient modulus as the DC2 dosage increases.

Table 5.11: Resilient modulus of asphalt mixtures at 25 °C

Aggregate type	Mixture type	Sample #	Resilient modulus (MPa)	Average
Granite	0RCAs	1	5046	5160.3
		2	5095	
		3	5340	
RCA	40RCAs	1	4704	4984.7
		2	5156	
		3	5094	
DCRCA type 1	20DC1	1	4913	4933.3
		2	4575	
		3	5312	
	40DC1	1	4584	4695.7
		2	4909	
		3	4594	
	60DC1	1	4251	4194
		2	4499	
		3	3832	
DCRCA type 2	20DC2	1	5313	5430
		2	5351	
		3	5626	
	40DC2	1	5109	5628
		2	5841	
		3	5934	
	60DC2	1	4789	4834.3
		2	4909	
		3	4805	

It can be seen that the DC2 mixes always demonstrated higher resilient modulus than corresponding DC1 mixes. Also, the 40DC2 mix exhibited better stiffness than the 40DC1 and 40RCAs mixes. These results indicate that DCT2 led to better stiffness than that obtained by DCT1. According to Australian practices, a dense graded asphalt mix typically needs a resilient modulus of 3000–4000 MPa in order to be used as wearing, intermediate and base courses (Alderson, 2008). In this regard, all asphalt mixes made in this study exhibited higher resilient modulus than this (see Table 5.11).

5.3.3.2 Resilient modulus at 40 °C

In order to investigate the effects of testing temperature on the resilient modulus of asphalt mixtures, ITSM tests were also conducted at 40 °C. The results are presented in Table 5.12.

It can be seen that the DC1 mixes showed different behaviour at 40 °C than that seen at 25 °C. The resilient modulus values decreased as the DC1 dosage increased from 20% to 40% and then to 60%, but these values were higher than those obtained for control mixes with up to 40% granite aggregate substitution. This improvement in stiffness at high temperature needs further investigation. However, the DC2 mixes showed a similar trend to that exhibited at 25 °C, where the resilient modulus improved up to 40% DC2 addition. At 40 °C, the DC2 mixes still demonstrated higher stiffness than corresponding DC1 mixes, and the 40DC2 mix showed a greater resilient modulus than the 40DC1 and 40RCAs mixes. These results are in accordance with the results obtained at 25 °C.

It can be concluded, based on ITSM test results, that DCT2 obtained better stiffness results than that of DCT1. These results are considered to be related to the strength and surface texture improvements resulted by DCT1 and DCT2.

Table 5.12: Resilient modulus of asphalt mixtures at 40 °C

Aggregate type	Mixture type	Sample #	Resilient modulus, MPa	Average
Granite	0RCAs	1	1266	1266.3
		2	1257	
		3	1276	
RCA	40RCAs	1	1095	1244.7
		2	1228	
		3	1411	
DCRCA type 1	20DC1	1	1595	1430.7
		2	1335	
		3	1362	
	40DC1	1	1470	1318.3
		2	1244	
		3	1241	
	60DC1	1	1241	1106.7
		2	1053	
		3	1026	
DCRCA type 2	20DC2	1	1609	1542.0
		2	1429	
		3	1588	
	40DC2	1	1713	1643.0
		2	1576	
		3	1640	
	60DC2	1	1228	1183.3
		2	1234	
		3	1088	

5.3.3.3 ANOVA analysis results

The results of the ITSM tests were also statistically analysed using one-way and two-way ANOVA, and introduced in the following sections.

5.3.3.3.1 Effect of DC1 content on resilient modulus at 25 °C and 40 °C

Two one-way ANOVAs were carried out to examine the effect of DC1 content on the resilient modulus of DC1 mixes at 25 °C and 40 °C. The results are shown in Table 5.13 and

Table 5.14, respectively, which show that the relationships were significant (p = 0.014 and 0.045), respectively.

Table 5.13: One-way ANOVA results: effect of DC1 content on resilient modulus at 25 °C.

Effect of DC1 content on resilient modulus at 25 °C.						
Source of variation	SS	df	MS	*F*	*P*-value	F crit
Between groups	1542010	3	514003.2	6.659582	0.01444	4.066181
Within groups	617460	8	77182.5	-		
Total	2159469.7	11	-			

Table 5.14: One-way ANOVA results: effect of DC1 content on resilient modulus at 40 °C.

Effect of DC1 content on resilient modulus at 40 °C.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
Between groups	163200.3	3	54400.11	4.22547	0.045782	4.066181
Within groups	102994.7	8	12874.33	-		
Total	266195	11	-			

5.3.3.3.2 Effects of DC1 content and test temperature on resilient modulus

A two-way ANOVA was also performed to examine the effects of DC1 content and test temperature (25 °C and 40 °C) on the stiffness of DC1 mixes. Based on ANOVA results, the testing temperature was more significant (p = 1.82E-17) than DC1 content (p = 0.001).

Table 5.15: Two-way ANOVA results: effects of DC1 content and test temperature on resilient modulus

Effects of DC1 content and test temperature on resilient modulus.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC1	1201545	3	400514.9	8.894714	0.001062	3.238872
Test temperature	72051211	1	72051211	1600.128	1.82E-17	4.493998
Interaction	503665.3	3	167888.4	3.7285	0.033131	3.238872
Within	720454.7	16	45028.42	-		
Total	74476875	23	-			

5.3.3.3.3 Effect of DC2 content on resilient modulus at 25 °C and 40 °C

Two one-way ANOVA analyses were performed to examine the effect of DC2 content on the resilient modulus of asphalt mixes made with DC2 at 25 °C and 40 °C. The results are shown in Table 5.16 and Table 5.17, respectively.

Table 5.16: One-way ANOVA results: effect of DC2 content on resilient modulus at 25 °C

Effect of DC2 content on resilient modulus at 25 °C.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
Between groups	1066228	3	355409.4	5.417586	0.024981	4.066181
Within groups	524823.3	8	65602.92	-		
Total	1591052	11	-			

Table 5.17: One-way ANOVA results: effect of DC2 content on resilient modulus at 40 °C

Effect of DC2 content on resilient modulus at 40 °C.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
Between groups	431171.3	3	143723.8	26.98827	0.000155	4.066181
Within groups	42603.33	8	5325.417	-		
Total	473774.7	11	-			

As can be seen in Table 5.16 and Table 5.17, the DC2 content significantly affected the resilient modulus of DC2 mixes at 25 °C and 40 °C (p = 0.025 and 0.000), respectively.

5.3.3.3.4 Effects of DC2 content and test temperature on resilient modulus

In addition, a two-way ANOVA was carried out to study the effects of DC2 dosage and the test temperature (25 °C and 40 °C) on the resilient modulus of DC2 mixes. The two-way ANOVA showed that both investigated factors were significant at a 95% confidence level. Both p-values were less than 0.05, as shown in Table 5.18. The p-value for DC2 content equals 0.000135 and the p-value of test temperature equals 5.04E-19. Based on ANOVA results, the test temperature had a greater effect on the resilient modulus of DC2 mixes than DC2 content.

Table 5.18: Two-way ANOVA results: effects of DC2 content and test temperature on resilient modulus of DC2 mixes

Effects of DC2 content and test temperature on resilient modulus of DC2-mixes.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
DC2 content	1405712	3	468570.7	13.21251	0.000135	3.238872
Test temperature	89143022	1	89143022	2513.608	5.04E-19	4.493998
Interaction	91687.5	3	30562.5	0.861785	0.480986	3.238872
Within	567426.7	16	35464.17	-		
Total	91207848	23	-			

5.3.3.4 Comparison of DC1 and DC2 mixtures based on ITSM testing

Based on the results of ITSM testing, the following comparison between DC1 and DC2 mixes can be made:

- Asphalt mixes made with 20%, 40% and 60% DC2 always exhibited higher resilient modulus than corresponding DC1 mixes. Coating DC2 particles with Sike Tite-BE and then curing them in an oven at 155 °C for 1.5 hours improved their strength and adhesion with bitumen, thus achieving higher mix stiffness compared to that of DC1 mixes.
- The 40DC2 mix demonstrated better stiffness than the 40DC1 and 40RCAs mixes. Also, the 40DC1 mix had a lower resilient modulus than the 40RCAs mix. The second treatment in the DCT1 (i.e., Sika Tite-BE treatment) reduces the friction forces mobilized by DC1 aggregates and, therefore, a reduction in stiffness in DC1 mixes was produced.
- The DC2 mixes showed stiffness improvements up to 40% granite substitution. At 60% DC2 addition, the stiffness started to decrease. However, the resilient modulus values of DC1 mixes decreased as the DC1 dosage increased from 20% to 60%.

5.4 Stage 3: Performance test results

At this stage of research, performance characteristics of asphalt mixtures were assessed. Wheel tracking, flow number, four-point bending fatigue and dynamic modulus tests were carried out for this purpose. At this stage of research, the 40RCAs mix was excluded from evaluation. This was due to the greater amounts of material and effort required to make asphalt mixture samples for performance testing, and the considerable time and effort needed to fabricate DCRCAs (DC1 and DC2). In keeping with this, Figure 5.13 shows RCA coated with two successive coats of Sika Tite-BE. The coating with Sika Tite-BE has been done using hands, as shown in Figure 3.13 and Figure 3.14 respectively, due to the lack of tools needed for such a procedure in the laboratory.

A total of seven asphalt mixtures were made: the control mix, three mixes made with 20%, 40% or 60% DC1, and three mixes made with 20%, 40% or 60% DC2. The results of the performance tests were used to study the effects of the developed DCTs (DCT1 and DCT2) on the mechanical characteristics of Australian HMA containing RCAs. In the following sections, the results of the performance tests are presented and discussed.

Figure 5.13: Hand-coated RCAs with Sika Tite-BE for fatigue life test

5.4.1 Resistance to permanent deformation

Resistance to permanent deformation was studied through two performance tests: wheel tracking and flow number tests. The wheel tracking and flow number tests were performed as explained in Sections 4.5.2.1.1 and 4.5.2.1.2, respectively.

5.4.1.1 Wheel tracking test results

The results of the wheel tracking tests of asphalt mixtures made with 100% granite aggregate (0RCAs mix) and different dosages of DC1 and DC2 (20%, 40% and 60%) are displayed in Figure 5.14 and Figure 5.15, respectively. Each curve in Figure 5.14 represents an average of three rutting slabs tested at 60 °C. The addition of 20% DC1 to the asphalt mixture increased the rut depth from 1.8 mm to 3.2 mm. The addition of 40% and 60% DC1 to the mixture, however, seemed to improve resistance to rutting. The final rut depths of mixes made with 40% and 60% DC1 were 3 mm and 2.9 mm, respectively. All DC1 mixes showed lower resistance against rutting compared to the control mix. The results for DC1 mixes support the suggestion that the Sika Tite-BE treatment decreases the friction mobilized between DC1 aggregates. Such an effect can, in turn, reduce the ability of DC1 mixes to resist wheel tracking loads. The rutting enhancement gained by adding more DC1 to the mix might be because of adhesion improvement after Sika Tite-BE treatment. Furthermore, the results indicate that the inclusion of DC2 improves the rutting behaviour of asphalt mixtures. According to the results obtained, asphalt mixtures containing DC2 always showed better rutting resistance than corresponding DC1 mixes.

As can be seen, the addition of 20% and 40% DC2 reduced the final rut depth from 1.8 mm (control mix) to 1.5 mm and 1.6 mm, respectively. The results, however, reveal that 60% addition of DC2 produced a final rut depth of 2.2 mm, which is higher than that of the 0RCAs mix (i.e., the control mix). It should be noted that the final rut depths of all asphalt mixtures were less than 3.5 mm. Therefore, it can be said that both DC1 mixes and DC2 mixes exhibited superior rutting performance according to the Austroads Pavement Research Group (APRG), as introduced in Section 4.5.2.1.1, Table 4.9.

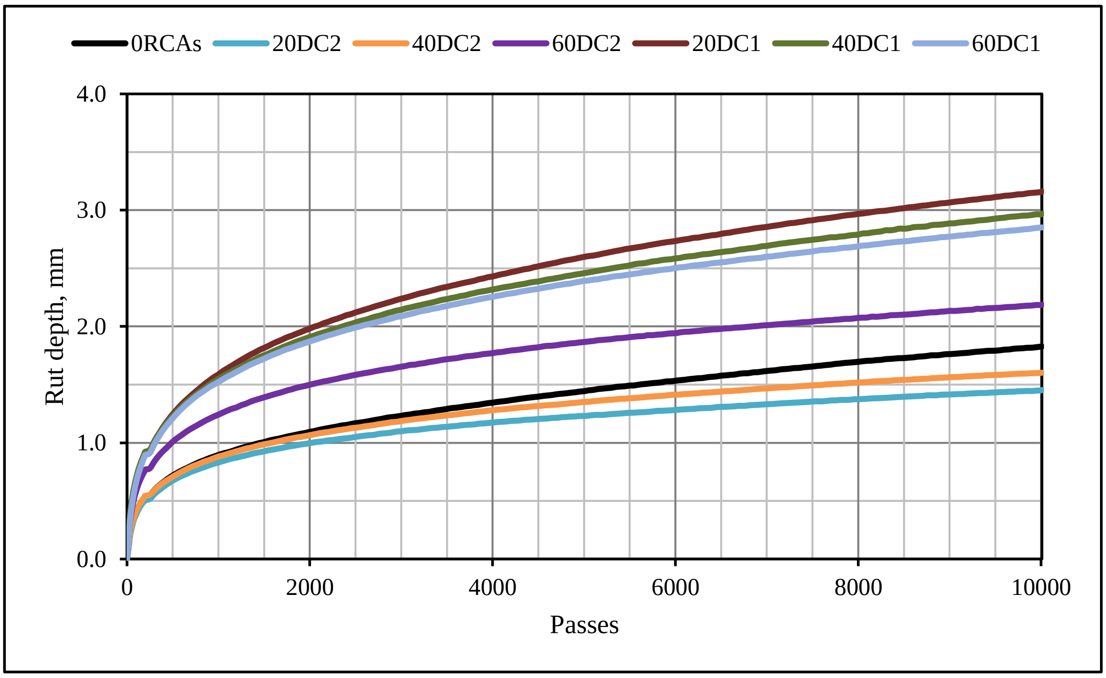


Figure 5.14: Final rut depth of asphalt mixtures made in the study

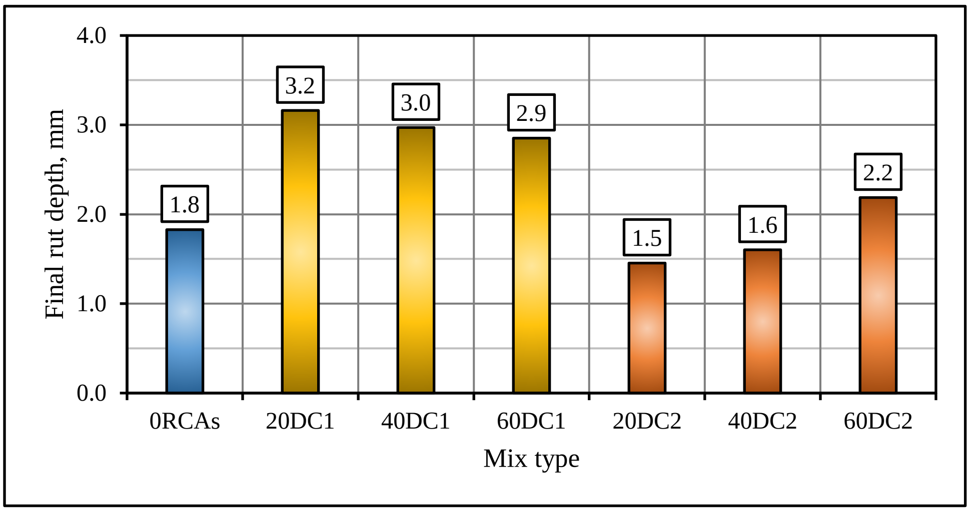


Figure 5.15: Final rut depths of asphalt mixtures

Figure 5.16 shows the dynamic stability of asphalt mixes made with DC1 and DC2. The general trend of the results shows that the addition of DC2 into HMA increases the dynamic stability of DC1 mixes, while the inclusion of DC1 decreases it. As more and more DC1 is added to the mix, the dynamic stability of the mix increases. However, as more and more DC2 is included in the mix, the dynamic stability decreases.

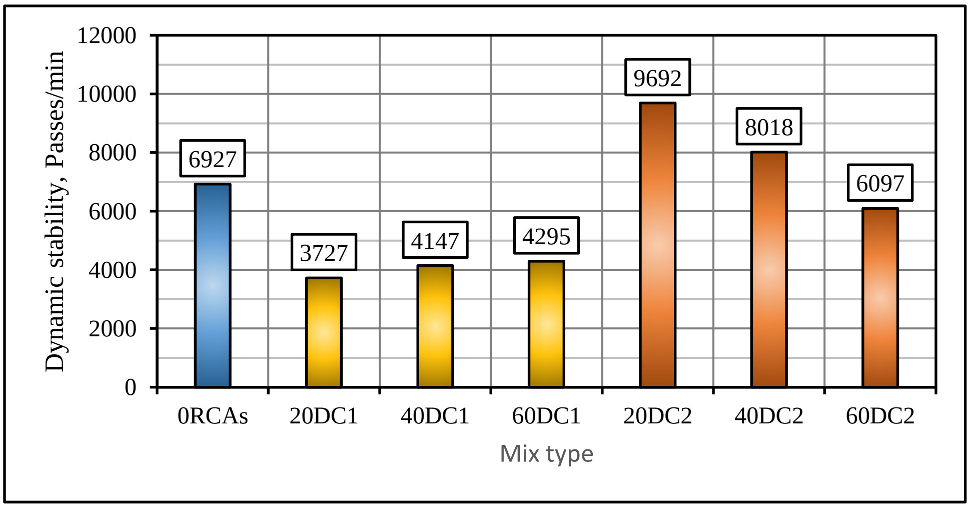


Figure 5.16: Dynamic stability of asphalt mixtures

It was mentioned previously in Section 5.2.1, the Marshall quotient (MQ) can be used as an indicator of HMA rutting performance. The results of the wheel tracking test indicate that MQ might not be an accurate indicator of rutting. The 0RCAs mix achieved the highest MQ, as shown in Figure 5.1. However, asphalt mixtures made with 20% and 40% DC2 were found to achieve lower rut depths (better rut resistance) than the 0RCAs mix.

As explained in Chapter 4, Section 4.5.2.1.1, the Wheeltracker EN software measures rut depth in accordance with the European Standard (EN 12697-22), not the Australian Standard (AG:PT/T233). Therefore, the final rut depth was measured based on European Standard requirements. It was found that the rut depths measured based on the Australian standard were the same as those calculated in accordance with European Standard. Thus, the use of the available wheel tracking machine which measures the rut depth based on European practices will not affect the accuracy of the rut depth results as per the requirements of Australian standards.

Besides the final rut depth results, resistance to rutting can also be assessed through the steady state tracking rate (SSTR; mm/kPasses), as introduced in Section 4.5.2.1.1. The results in Table 5.19 indicate that the tangential slopes of the rutting curves of DC1 mixes between 4000 and 10,000 wheel passes were not significantly affected by the addition of DC1 into the asphalt mix. However, all DC1 mixes showed greater SSTR compared with that achieved by the control (0RCAs) mix. In contrast, the addition of DC2 reduced the SSTR of asphalt mixtures compared to that of the control mix made with 100% granite aggregate. The DC2 mixes always showed better rut depth and SSTR values than those obtained for equivalent DC1 mixes.

Table 5.19: Results of ASSTR (mm/kpasses) of seven asphalt mixtures made in the present study

Results of ASSTR (mm/kpasses) of seven asphalt mixtures made in the present study.				
Mix type	Tr*at 10,000 passes	Tr at 4000 passes	Ts ˣ	SSTR, mm/kpasses
0RCAs	1.8	1.3	8.0E-05	0.080
20DC1	3.2	2.4	1.2E-04	0.121
40DC1	3.0	2.3	1.1E-04	0.108
60DC1	2.9	2.3	9.9E-05	0.099
20DC2	1.5	1.2	4.6E-05	0.046
40DC2	1.6	1.3	5.4E-05	0.054
60DC2	2.2	1.8	6.9E-05	0.069
Tr = Tracking rut, Ts = Tangential slope				

5.4.1.1.1 ANOVA analysis of wheel tracking test results

Two one-way ANOVA tests were performed to study the effects of DC1 content and DC2 content on rutting depth.

5.4.1.1.1.1 Effect of DC1 content on rut depth

The results of the one-way ANOVA analysis are presented in Table 5.20. The amount of added DC1 significantly affected the measured rut depth of DC1 mixes (p = 0.002).

Table 5.20: One-way ANOVA results: effect of DC1 content on rutting of DC1 mixes

One-way ANOVA results: effect of DC1 content on rutting of DC1-mixes.						
Source of variation	SS	df	MS	F	P-value	F crit
%DC1 vs Rut depth	3.0825	3	1.0275	12.33	0.002278	4.066181
Within groups	0.666667	8	0.083333	-		
Total	3.749167	11	-			

5.4.1.1.1.2 Effect of DC2 content on rut depth

Another one-way ANOVA analysis was carried out to investigate the effect of DC2 content on rutting depth of asphalt mixtures. As shown in Table 5.21, the dosage of DC2 can significantly affect rutting in DC2 asphalt mixtures (p = 0.019).

Table 5.21: One-way ANOVA results: effect of DC2 content on rutting in DC2 mixes

One-way ANOVA results: effect of DC2 content on rutting of DC2-mixes.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC2 vs rut depth	0.929167	3	0.309722	5.994624	0.019187	4.066181
Within groups	0.413333	8	0.051667	-		
Total	1.3425	11	-			

5.4.1.1.2 Comparison of DC1 and DC2 mixture rutting test results

According to the wheel tracking test results, the following points can summarize the differences between mixes made with DC1 and DC2:

- DC2 mixes demonstrated higher rutting resistance than that achieved by equivalent DC1 mixes. The use of DCT1 seems to reduce the total friction forces between DC1 aggregates, while DCT2 improves adhesion between DC2 and bitumen without affecting friction between DC2 aggregates.
- The more DC1 is added to a mixture, the better its rutting resistance. However, the more DC2 is added into HMA, the lower its resistance to rutting.
- The DC2 mixes exhibited lower SSTR values compared to corresponding DC2 mixes. Furthermore, all mixes made with DC2 achieved lower SSTRs than control mix, while all DC1 mixes shown higher SSTRs than control mix.
- Although both DC1 and DC2 aggregates had significant effects on rutting, the addition of the former reduced resistance to rutting while inclusion of the latter led to rutting performance enhancements with up to 40% granite replacement.

5.4.1.2 Flow number test results

The results of flow number (FN) tests are presented in Table 5.22 for mixtures made with granite and different dosages (20%, 40% or 60%) of DCRCAs (DC1 or DC2).

Based on the results of the FN tests, the addition of both types of DCRCA (DC1 and DC2) had significant effects on the rutting performance of DC1 and DC2 mixtures. The results reveal that only adding 20% of DC1 reduced FN, as compared with that achieved by the control mix. However, mixtures containing 40% and 60% DC1 exhibited higher FN than the 0RCAs mix. This performance is different than that shown by the same mixes in wheel tracking tests.

The results for DC2 mixtures showed, however, similar behaviour to those obtained in wheel tracking tests. Only one exception was found: the asphalt mixture produced with 60% DC2 demonstrated higher FN (higher rutting resistance) than the control mix. It can also be seen that the asphalt mixture made with 60% DC2 exhibited slightly lower FN (better rutting resistance) than the 60DC1 mix. These findings in some way contradict those of the wheel tracking tests, where DC2 mixtures always exhibited higher rutting resistance than corresponding DC1 mixes.

Additionally, samples made with DC1 and DC2 generally showed higher variation in FN results compared to samples of control mix (0RCAs mix). This is likely because of the heterogeneity of the RCAs used in the study.

Table 5.22: FN test results of HMA made with granite, DC1 or DC2 aggregates

FN test results of HMA made with granite, DC1 and DC2 aggregates.				
Mix type	Sample #1	Sample #2	Sample #3	Average
0RCAs	148	137	151	145.3
20DC1	116	146	144	135.3
40DC1	174	182	157	171.0
60DC1	187	197	196	193.3
20DC2	157	164	164	161.7
40DC2	162	195	185	180.7
60DC2	189	164	209	187.3
[a] = standard deviation, [b] = coefficient of variation				

5.4.1.3 ANOVA analysis of FN test results

Two one-way ANOVA tests were carried out to examine the effect of DC1 content, and DC2 content on FN test results.

5.4.1.3.1 Effect of DC1 content on FN results

The results of the one-way ANOVA are presented in Table 5.23. It can be seen that the DC1 content affected the FN of DC1 asphalt mixtures (p = 0.001).

Table 5.23: One-way ANOVA results: effect of DC1 on FN of DC1 mixes

One-way ANOVA results: effect of DC1 on FN of DC1-mixes.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
Between groups	6148.25	3	2049.417	15.49653	0.001074	4.066181
Within groups	1058	8	132.25	-		
Total	7206.25	11	-			

5.4.1.3.2 Effect of DC2 content on FN results

The results of the second one-way ANOVA analysis are tabulated in Table 5.24. According to the results presented, the content of DC2 had a great effect on the FN of DC1 mixtures (p = 0.030).

Table 5.24: One-way ANOVA results: effect of DC2 content on FN of DC2-asphalt mixtures

One-way ANOVA results: Effect of DC2 content on FN of DC2-asphalt mixtures.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
Between groups	3257.583	3	1085.861	5.019389	0.030286	4.066181
Within groups	1730.667	8	216.3333	-		
Total	4988.25	11	-			

5.4.1.4 Comparison of DC1 and DC2 mixture FN test results

The FN test results show that the addition of DC1 or DC2 can improve the rutting resistance of asphalt mixtures. The following comparison between asphalt mixtures produced with DC1 and those produced with DC2 can be made:

- In general, asphalt mixes made with DC2 demonstrated better rutting resistance (i.e., higher FN) than DC1 mixes.
- All DC1 mixes and DC2 mixes exhibited better rutting resistance than control mix (i.e., 0RCAs mix). Only one exception was seen, asphalt mixture made with 20% DC1 showed lower resistance to rutting as compared to 0RCAs mix.
- Asphalt mixes made with 20% or 40% DC2 demonstrated higher FNs (higher rutting resistance) than corresponding mixes made with DC1. However, the 60DC2 mix showed slightly lower FN (193.3) than the 60DC2 mix (187.3).

5.4.1.5 Differences between wheel tracking and FN test results

Both the wheel tracking and flow number tests aim to assess the permanent deformation resistance of asphalt mixtures. The results obtained by these performance tests are variable. Table 5.25 shows the ranking, from highest to lowest, of asphalt mixtures made with DC1 and DC2 based on their rutting and FN performance.

Table 5.25: Mixture ranking, from highest to lowest, based on rutting and FN results

Mixtures classification, from highest to lowest, based on their rutting and FN results.		
Rank	Mix (wheel tracking test)	Mix (FN test)
1	20DC2	60DC1
2	40DC2	60DC2
3	0RCAs	40DC2
4	60DC2	40DC1
5	60DC1	20DC2
6	40DC1	0RCAs
7	20DC1	20DC1

According to the wheel tracking test results, only mixes with 20% and 40% DC2 behaved better against rutting than the control mix. However, based on the FN test results, only the 20DC1 mix showed less resistance to rutting than the control mix (i.e., the 0RCAs mix). In addition, according to the wheel tracking results, the rutting resistance of mixtures made with DC2 decreased with increasing DC2 content. However, based on FN results, the more DC2 was added to the mixture, the higher the FN value. This difference could be related to the different deformation mechanisms in each test, as shown in Figure 5.17. The results highlight the importance of in situ evaluation of both mixtures to determine their actual rutting behaviour in the field.

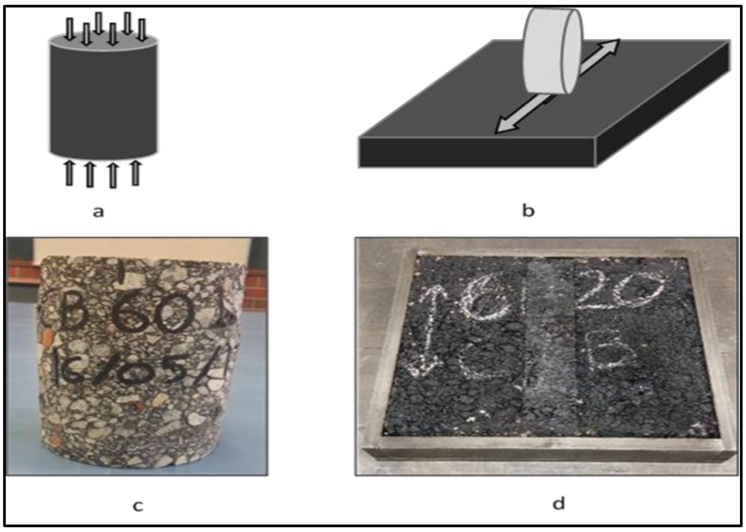


Figure 5.17: (a) Uniaxial compression load during FN testing, (b) simple shear test during wheel tracking test, (c & d) specimens after FN and rutting tests, respectively

5.4.2 Four-point bending fatigue life test results

The fatigue life of asphalt mixtures made with two types of DCRCA (DC1 and DC2) was assessed according to test method AG:PT/T233, as described in Section 4.5.2.2. In the following sub-sections, the results of fatigue life tests are presented and analysed.

5.4.2.1 Flexural stiffness of asphalt mixtures

The initial flexural stiffness and termination flexural stiffness of asphalt mixtures made with granite aggregate and different percentages (20%, 40% or 60%) of DC1 or DC2 are plotted in Figure 5.18 and Figure 5.19 respectively. Each result in these figures represents the average flexural stiffness of three tested beam fatigue samples. The initial flexural stiffness was calculated by the data acquisition software at the 50th load cycle, while the termination flexural stiffness was calculated at 50% of the initial stiffness, as described in Section 4.5.2.2.

According to the results presented, the contents of DC1 and DC2, respectively, can effect flexural stiffness measured at the beginning and termination of the tests. The effect of DC2 content, however, was more noticeable than that of DC1 content on the fatigue stiffness of Australian HMA type AC14, as shown. Based on Figure 5.19, the inclusion of 20% DC1 increased the initial flexural stiffness from 7069.9 MPa (control mix) to 7409.8 MPa. The addition of 40% DC1 to HMA led to a further increase in flexural stiffness. However, adding 60% DC1 decreased the flexural stiffness to 7088.7 MPa, as shown in Figure 5.19. The initial flexural stiffness values of asphalt mixtures made with 20%, 40% and 60% DC1 were 4.8%, 5.1% and 0.3% higher than that of the made with 100% granite aggregate (i.e., 0RCAs mix).

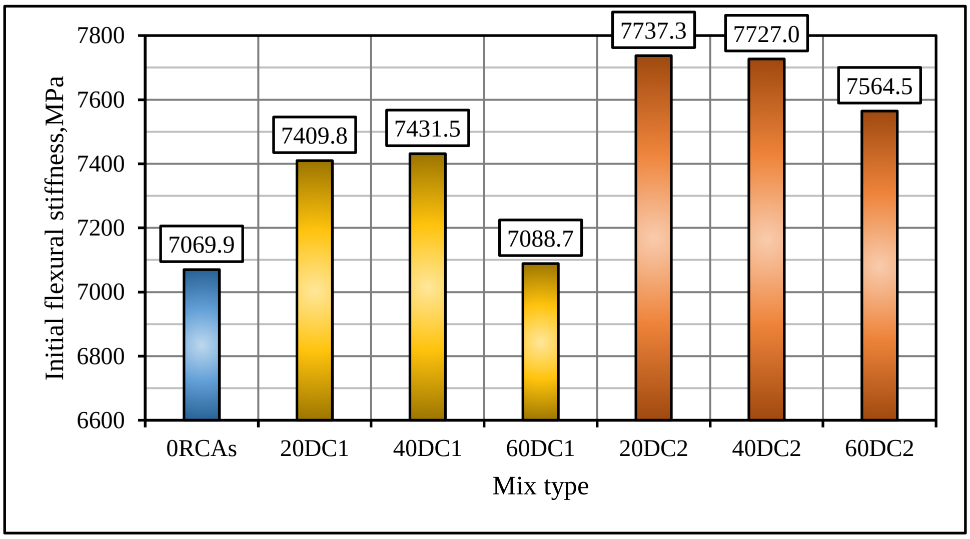


Figure 5.18: Initial flexural stiffness HMA made with different dosages of DC1 and DC2

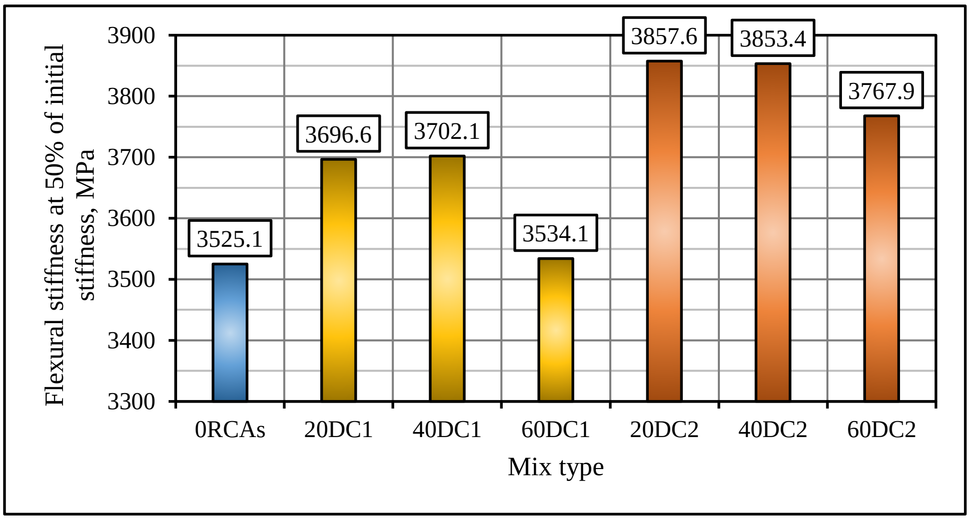


Figure 5.19: Termination stiffness of HMA made with different percentages of DC1 and DC2

The behaviour of DC2-asphalt mixtures was slightly different: the addition of DC2 further increased the initial and termination flexural stiffnesses. The addition of 20% and 40% DC2 to HMA caused the initial fatigue stiffness to increase from 7069.9 (control mix) to 7737.3 MPa and 7727 MPa, respectively. The addition of 60% DC2 decreased the initial flexural stiffness to 7564.5 MPa, as shown in Figure 5.18. The initial flexural stiffnesses of asphalt mixtures made with 20%, 40% or 60% DC2 were 9.4%, 9.3% and 6.9% higher than that of the control mix. The same story can be seen in the results of the termination stiffnesses of DC1 and DC2 mixes. These results indicate that the designed asphalt mixtures made with DC1 and DC2 showed more elastic behaviour (less viscous) than that recorded for the control mix.

5.4.2.2 Phase angle of asphalt mixtures

Figure 5.20 shows the initial phase angle of asphalt mixtures made with granite aggregate and different dosages of DC1 and DC2. The phase angle is considered an indicator of fatigue damage in asphalt mixtures. The phase angle of asphalt mixtures, therefore, increases as the number of load cycles increases during the test. It was found that the higher the dosage of DC1 or DC2, the lower the value obtained for the phase angle. This indicates that the DC1 and DC2 mixtures had less viscous (more elastic) performance than the control mix (0RCAs mix). It can be seen that the decrease in phase angle was higher in DC2-asphalt mixtures than that obtained in corresponding DC1 mixtures. The phase angles of asphalt mixtures made with 20%, 40% or 60% DC1 were 7.3%, 11% and 12.7% lower than that of the control mix, as presented in Figure 5.20. In addition, the phase angles of asphalt mixtures containing 20%, 40% or 60% DC2 were 11.4%, 12.6% and 13.1% lower than for of the asphalt mix made with 100% granite aggregate.

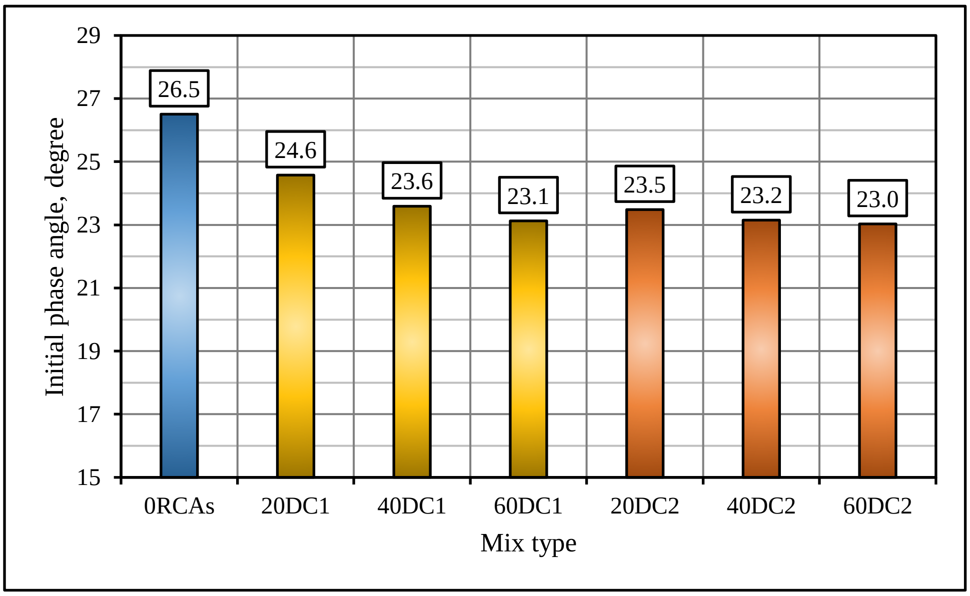


Figure 5.20: Initial phase angle of HMA made with different dosages of DC1 and DC2

The same scenario can be seen in Figure 5.21 by looking at the phase angle values calculated at failure. Only one exception can be seen, the phase angles at 60% DC1 and DC2 were slightly higher than those measured at 40% DC1 and DC2. This might be explained by the heterogeneity of the RCAs used.

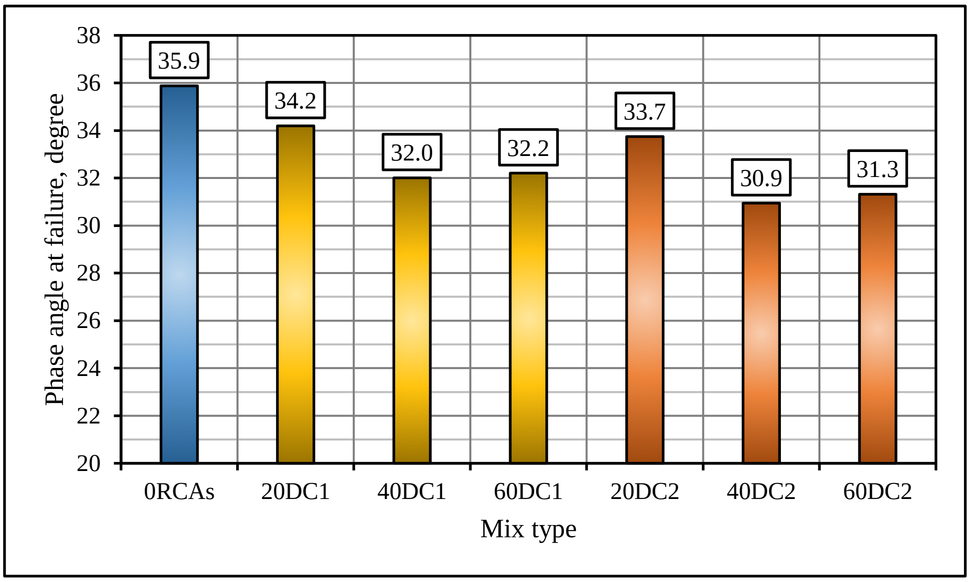


Figure 5.21: Termination phase angle of asphalt mixtures made with DC1 and DC2.

5.4.2.3 Fatigue life of asphalt mixtures made with DCRCAs

Typical results of the four-point bending fatigue test for flexural stiffness and phase angle versus the number of load cycles are plotted in Figure 5.22. It can be seen that the figure is divided into three phases. In phase 1 (adaptation phase), a rapid reduction in flexural stiffness and increase in phase angle is observed. Then, in phase 2 (quasi-stationary phase), the reduction in the stiffness modulus is almost linear while the phase angle tends to be constant. Finally, in phase 3 (failure phase), the flexural stiffness experiences a noted decrease that indicates crack initiation and propagation occurs in the specimen.

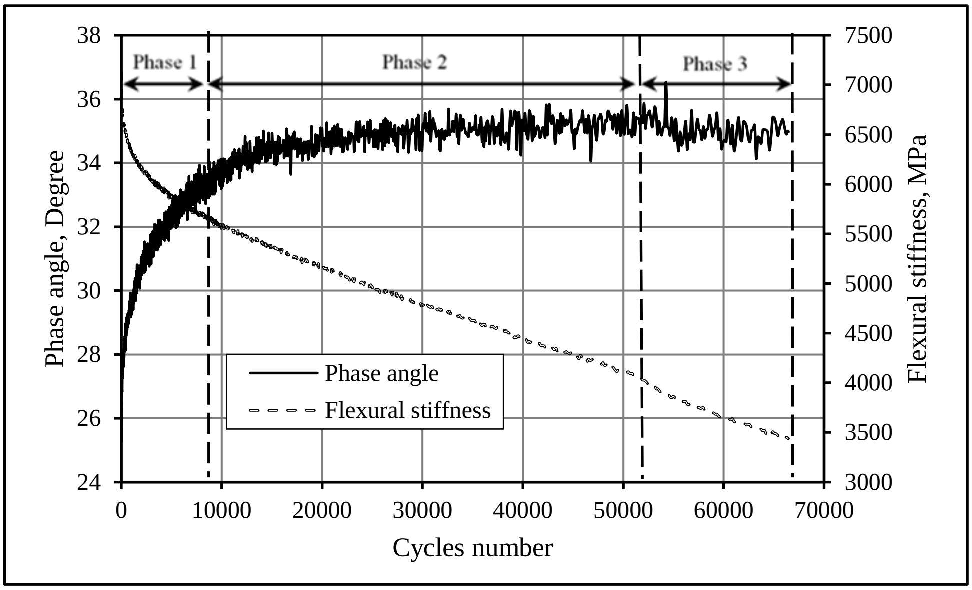


Figure 5.22: Typical progression in flexural stiffness and phase angle with load cycles in an asphalt mixture

The flexural stiffness versus the number of load cycles for asphalt mixtures made with granite aggregate, and 20%, 40% or 60% DC1 and DC2 are plotted in Figure 5.23 to Figure 5.29, respectively. The tests were terminated when asphalt mixtures reached half their initial stiffness. The beam samples were manufactured at an average air void range of 4.5–5.5% in order to limit the effects of void content on fatigue life, as introduced in Section 4.5.2.2.

It can be seen that the inclusion of both double coated recycled concrete aggregates (DCRCAs), i.e., DC1 and DC2, in asphalt mixtures can lead to an increase in the number of load cycles to failure. The more DC1 or DC2 is added to a mixture, the higher the number of load cycles to failure.

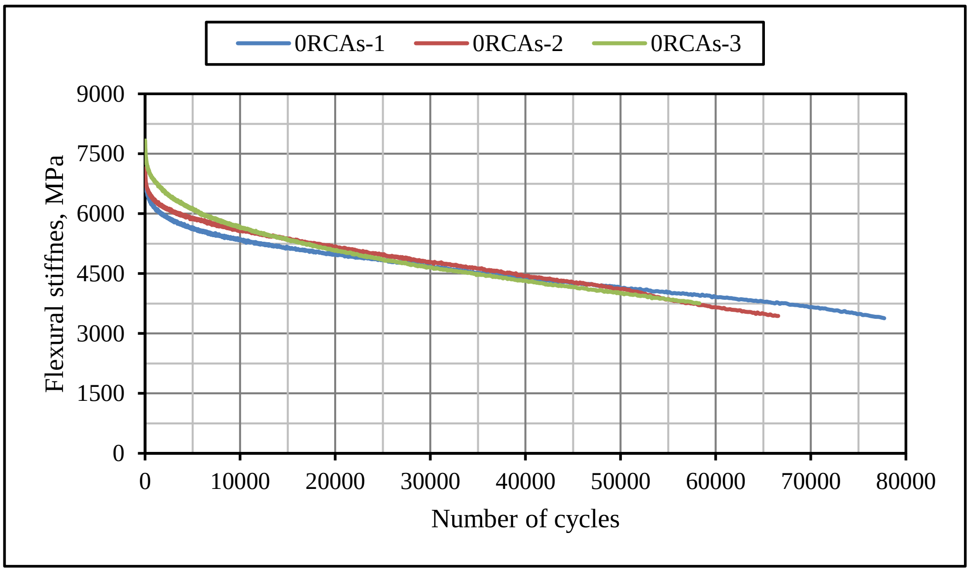


Figure 5.23: Relationship between flexural stiffness and load cycles for the control mix

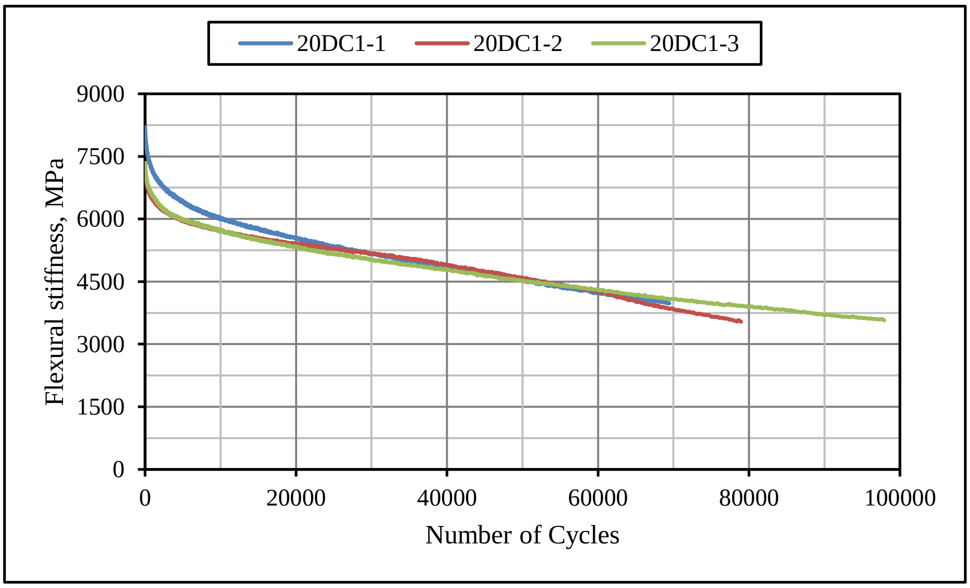


Figure 5.24: Relationship between flexural stiffness and load cycles for the 20DC1 mix

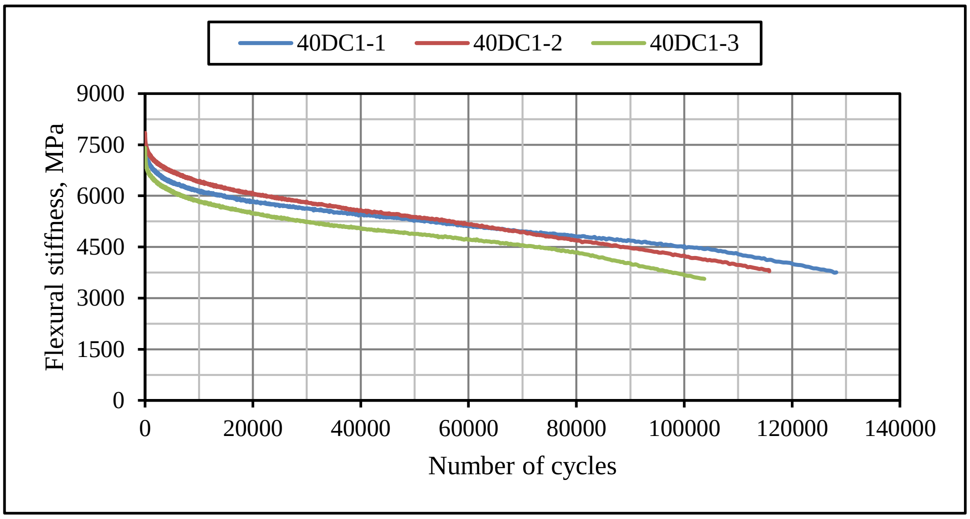


Figure 5.25: Relationship between flexural stiffness and load cycles for the 40DC1 mix

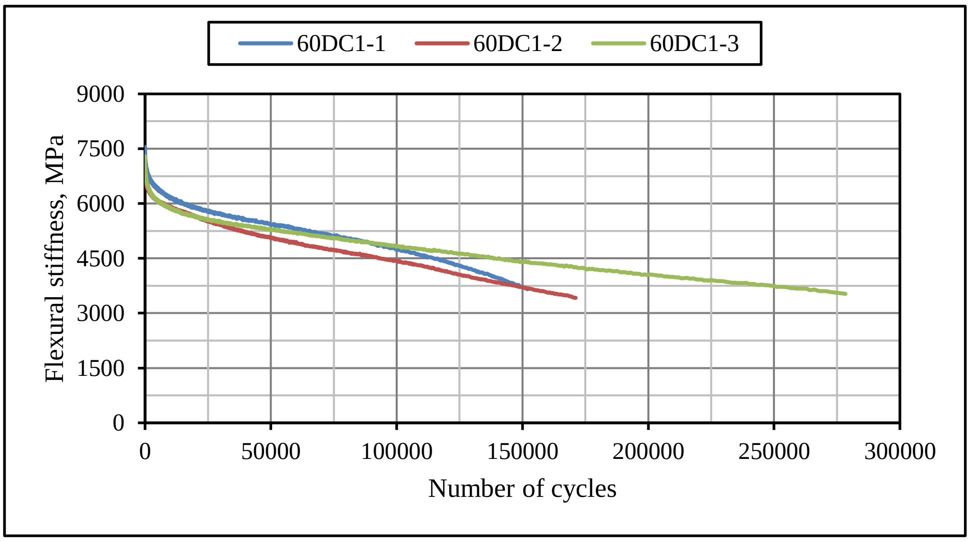


Figure 5.26: Relationship between flexural stiffness and load cycles for the 60DC1 mix

It can be seen that the beam samples made with 20%, 40% or 60% DC1 required a greater number of cycles to halve their initial stiffness and fail than corresponding samples made with DC2.

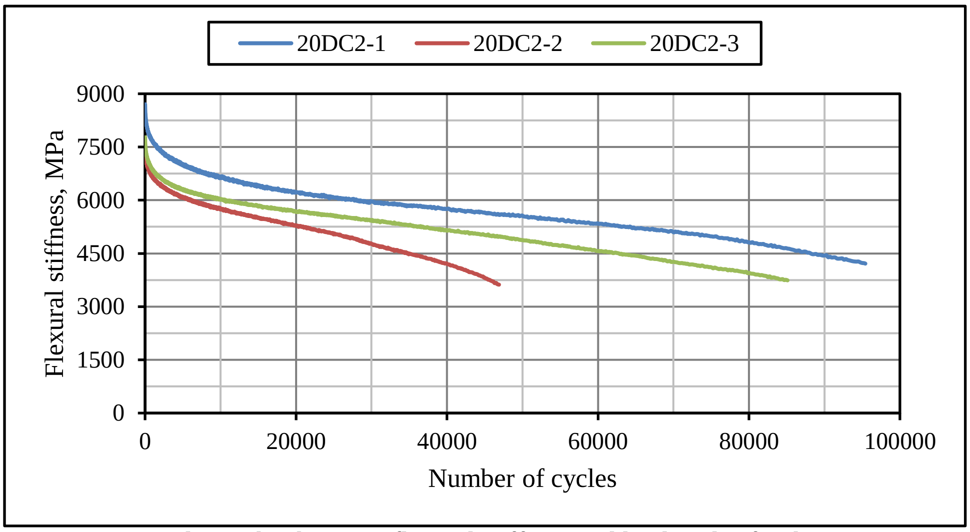


Figure 5.27: Relationship between flexural stiffness and load cycles for the 20DC2 mix

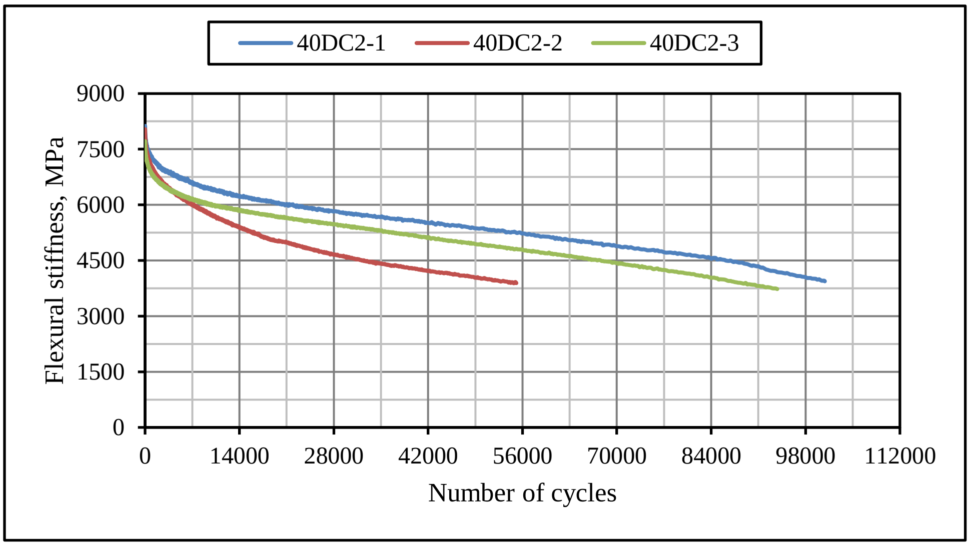


Figure 5.28: Relationship between flexural stiffness and load cycles for the 40DC2 mix

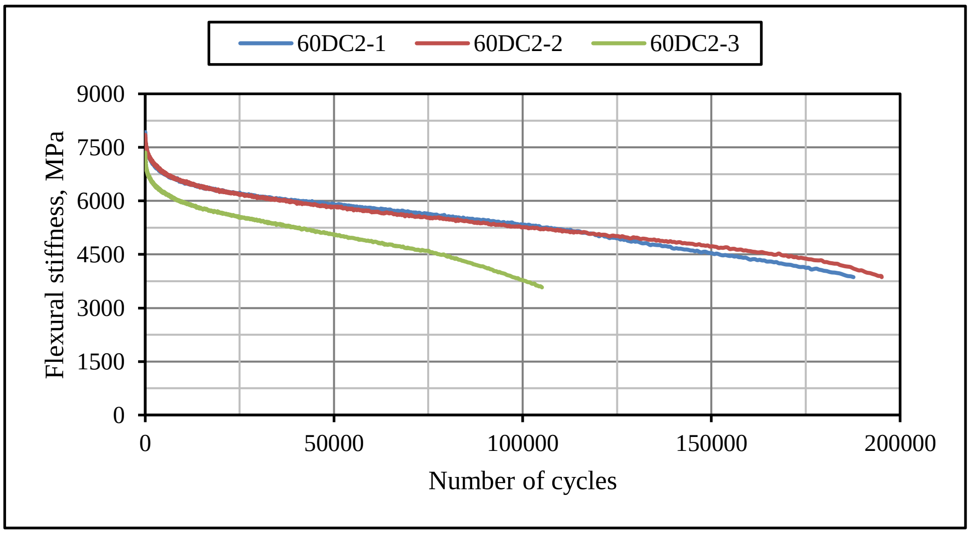


Figure 5.29: Relationship between flexural stiffness and load cycles for the 60DC2 mix

Figure 5.30 shows the number of load cycles required to halve the initial stiffness of asphalt mixtures made with granite aggregate and different percentages (20%, 40%, and 60%) of DC1 and DC2. As can be seen, the dosages of DC1 and DC2 significantly affected the number of load cycles to failure of asphalt beam samples. The addition of more DC1 and DC2 increased the number of load cycles required to halve the initial stiffness. The number of load cycles to failure continued to increase as the contents of DC1 or DC2 increased from 20% to 40% and then to 60%. The results reveal that the addition of DC1 helped achieve a higher number of load cycles than the corresponding DC2-asphalt mixtures. It can be concluded that the inclusion of DC1 and DC2 offer strong opposition to fatigue cracking and help to increase the number of load cycles required to halve the initial stiffness. The DC1-asphalt mixtures, however, were more capable of withstanding more load cycles at the same tensile strain than equivalent DC2 mixtures.

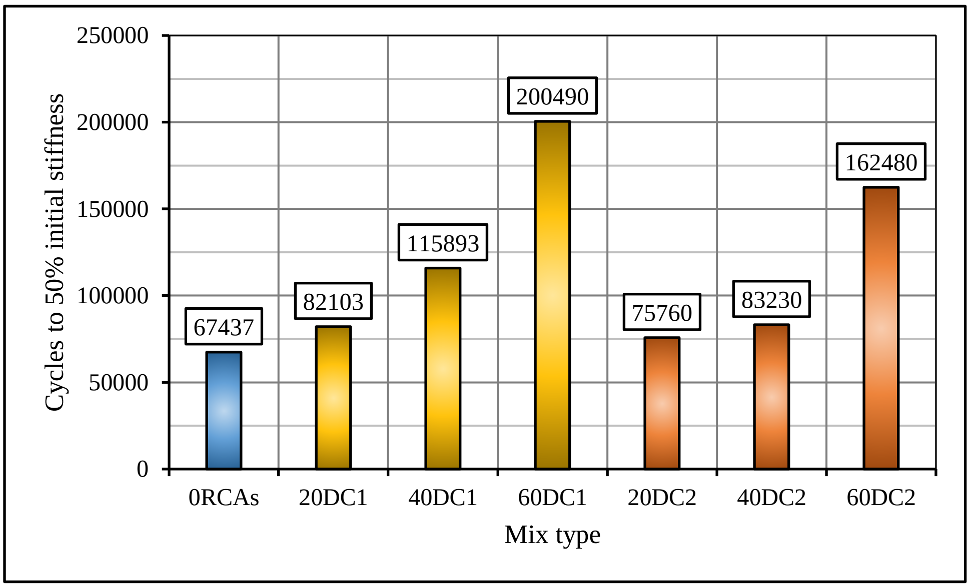


Figure 5.30: Number of cycles required to halve the initial flexural stiffness of asphalt mixtures

Figure 5.31 shows the percentage increase in load cycles-to-failure of asphalt mixtures made with 20%, 40% and 60% DC1 and DC2 compared with the control mix (0RCAs). The numbers of load cycles required to halve the initial stiffness of DC1-asphalt mixtures were 22%, 72% and 197% higher than that of the 0RCAs mix. Also, the numbers of load cycles to failure of DC2-asphalt mixtures were 12%, 23% and 141% higher than for the control mix. These findings highlight the importance of the two DCTs in improving the quality of HMA made with RCAs so that it can withstand fatigue cracking better than HMA made with 100% granite aggregates. Enhancement of the strength of RCAs, and adhesion at the bitumen-DCRCA interface, may contribute to this performance.

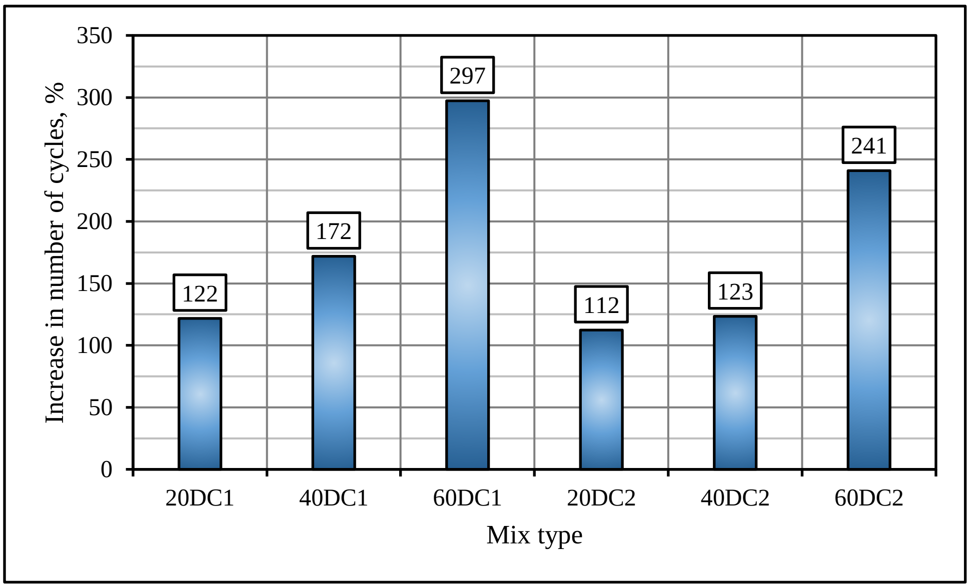


Figure 5.31: Increase in load cycles to failure of asphalt mixtures made with DC1 and DC2 relative to the control mix

5.4.2.4 ANOVA Analysis of fatigue life test results

A number of one-way ANOVA analyses were carried out to investigate the effects of DC1 and DC2 contents on the initial phase angle, termination phase angle, initial flexural stiffness, termination flexural stiffness, and load cycles to failure of asphalt mixtures.

5.4.2.4.1 Effect of DCRCAs on phase angle

5.4.2.4.1.1 Effect of DC1 content on phase angle

To examine the effect of DC1 content on the initial and termination phase angles of asphalt mixtures, two one-way ANOVAs were carried out (Table 5.26 and Table 5.27). Based on ANOVA results, the DC1 content significantly affected the initial and termination phase angles of DC1-asphalt mixtures (p = 0.000 and 0.002).

Table 5.26: One-way ANOVA results: effect of DC1 content on initial phase angle

One-way ANOVA results: effect of DC1 content on initial phase angle.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC1 vs initial phase angle	20.210	3	6.7367	16.6472	0.00084	4.06618
Within groups	3.2374	8	0.4046	-		
Total	23.447	11	-			

Table 5.27: One-way ANOVA results: effect of DC1 content on termination phase angle

One-way ANOVA results: effect of DC1 content on termination phase angle.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC1 vs termination phase	30.0358	3	10.01	12.4374	0.00221	4.06618
Within groups	6.43986	8	0.804	-		
Total	36.4756	11	-			

5.4.2.4.1.2 Effect of DC2 content on phase angle

Two one-way ANOVAs were performed to examine the effect of DC2 content on phase angle measured at the 50th load cycle and 50% initial stiffness in DC2 mixtures (Table 5.28 and Table 5.29). ANOVA shows that the content of DC2 greatly affected the phase angle calculated at the beginning and termination of fatigue testing (p = 0.004 and 0.000).

Table 5.28: One-way ANOVA results: effect of DC2 content on initial phase angle

One-way ANOVA results: effect of DC2 content on initial phase angle.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC2 vs initial phase	24.619	3	8.20649	10.6351	0.00364	4.06618
Within groups	6.1731	8	0.77164	-		
Total	30.792	11	-			

Table 5.29: One-way ANOVA results: effect of DC2 content on termination phase angle

One-way ANOVA results: effect of DC2 content on termination phase angle.						
Source of variation	SS	df	MS	F	P-value	*F* crit
%DC2 vs termination phase	47.6629	3	15.88	26.8470	0.00015	4.06618
Within groups	4.73426	8	0.591	-		
Total	52.3972	11	-			

5.4.2.4.2 Effect of DCRCAs on flexural stiffness

5.4.2.4.2.1 Effect of DC1 content on flexural stiffness

Two one-way ANOVAs were carried out to investigate the effect of DC1 content on the initial and termination flexural stiffnesses of DC1 beam samples. The results reveal that the DC1 content did not greatly affect the flexural stiffness of DC1 asphalt mixtures calculated at the 50th load cycles and when mixtures reached 50% of their initial stiffness (p = 0.493 and 0.487; Table 5.30 and Table 5.31, respectively.)

Table 5.30: One-way ANOVA results: effect of DC1 content on initial flexural stiffness

One-way ANOVA results: effect of DC1 content on initial flexural stiffness.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC1 vs initial flexural stiffness	350805	3	116935	0.87519	0.493223	4.066181
Within groups	1068888	8	133610.9	-		
Total	1419693	11	-			

Table 5.31: One-way ANOVA results: effect of DC1 content on termination flexural stiffness

One-way ANOVA results: effect of DC1 content on termination flexural stiffness.						
Source of variation	SS	df	MS	F	P-value	*F* crit
%DC1 vs termination flexural stiffness	86620.4	3	28873.47	0.889114	0.487084	4.066181
Within groups	259795.5	8	32474.44	-		
Total	346415.9	11	-			

5.4.2.4.2.2 Effect of DC2 content on flexural stiffness

Two one-way ANOVAs were performed to study the effect of DC2 content on the flexural stiffness of DC2 beam samples measured at the beginning and termination of the tests. They produced p-values higher than 0.05, as presented in Table 5.32 and Table 5.33, respectively. ANOVA results indicate that DC2 percentage did not significantly affect the flexural stiffness of asphalt mixtures.

Table 5.32: One-way ANOVA results: effect of DC2 content on initial flexural stiffness

One-way ANOVA results: effect of DC2 content on initial flexural stiffness.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC2 vs initial flexural stiffness	883717	3	294572.3	1.644536	0.254844	4.066181
Within groups	1432975	8	179121.9	-		
Total	2316692	11	-			

Table 5.33: One-way ANOVA results: effect of DC2 content on termination flexural stiffness

One-way ANOVA results: effect of DC2 content on termination flexural stiffness.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC2 vs termination flexural stiffness	219448.6	3	73149.52	1.642368	0.255292	4.066181
Within groups	356312.4	8	44539.05	-		
Total	575760.9	11	-			

5.4.2.4.3 Effect of DCRCAs on the number of load cycles to failure

5.4.2.4.3.1 Effect of DC1 content on the number of load cycles to failure

To assess the effect of DC1 content on the number of load cycles to failure of DC1 mixes, a one-way ANOVA was performed. The results are presented in Table 5.34. The content of DC1 significantly affected the number of load cycles to failure of DC1 beam samples (p = 0.008).

Table 5.34: One-way ANOVA results: effect of DC1 content on number of load cycles to failure

One-way ANOVA results: effect of DC1 content on number of load cycles.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC2 vs number of load cycles	3.19E+10	3	1.06E+10	8.357932	0.007563	4.066181
Within groups	1.02E+10	8	1.27E+09	-		
Total	4.21E+10	11	-			

5.4.2.4.3.2 Effect of DC2 content on the number of load cycles to failure

A one-way ANOVA was carried out to study the effect of DC2 content on the number of load cycles to failure of DC2 beam samples (Table 5.35). According to ANOVA, DC2 content can significantly affect the number of load cycles to failure of asphalt mixtures containing DC2 (p = 0.019).

Table 5.35: One-way ANOVA results: effect of DC2 content on number of load cycles to failure

One-way ANOVA results: effect of DC2 content on number of load cycles.						
Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
%DC2 vs number of load cycles	1.74E+10	3	5.8E+09	6.030889	0.018882	4.066181
Within groups	7.7E+09	8	9.62E+08	-		
Total	2.51E+10	11	-			

5.4.2.5 Comparison of DC1 and DC2 mixture fatigue life test results

The results of fatigue life tests indicate that asphalt mixtures containing 20%, 40% or 60% DC1 or DC2 demonstrated less viscous performance than the control mix. The fatigue life of asphalt mixtures was improved by the addition of both types of DCRCA (DC1 and DC2). Based on these results, a comparison of DC1 and DC2-asphalt mixtures can be made as follows:

- DC1-asphalt beam samples always demonstrated lower initial and termination flexural stiffness than corresponding DC2-beams.
- The DC1-samples exhibited slightly higher phase angle values compared with equivalent samples made with DC2. This was true in the case of initial and termination phase angle values.
- Asphalt mixtures containing DC1 offered more opposition to fatigue cracking (i.e., required more load cycles to reach 50% of initial stiffness) compared to corresponding asphalt mixtures made with DC2.
- Although both DCTs (DCT1 and DCT2) improved the fatigue life of asphalt mixtures, however, the mixes made based on DCT1 (coating with CSP and Sika Tite-BE) were more capable of withstanding fatigue cracking at the same tensile strain than those made based on DCT2 (coating with Sika Tite-BE and heating).

5.4.3 Dynamic modulus test results

The dynamic modulus and phase angle of asphalt samples were determined at different temperatures and loading frequencies according to AASHTO TP 79-13 (AASHTO, 2014b). The dynamic modulus samples were prepared as explained in Section 4.4.1 and compacted as detailed in Section 4.4.2.2. The tests were performed at three temperatures (4 °C, 20°C and 40°C) and four loading frequencies (0.01 Hz, 0.1 Hz, 1

Hz, and 10 Hz). The results of the dynamic modulus test are presented along with a discussion in the following sections. Firstly, the effects of temperature on dynamic modulus and phase angle are analysed. Then, the effects of loading frequency are presented. The effects of DCRCA percentage on dynamic modulus and phase angle are shown. Construction of master curves for asphalt mixtures containing granite aggregates and different dosages of DC1 and DC2 at the 20 °C reference temperature are also presented. Then, ANOVA analysis of the dynamic modulus test results is introduced.

5.4.3.1 Effect of test temperature on the dynamic modulus and phase angle of asphalt mixtures made with DCRCAs

The average dynamic modulus of three samples of each asphalt mixture measured at different test temperatures are plotted in Figure 5.32 to Figure 5.38. Each point on the curves represents the average of three readings of dynamic modulus measured at same testing conditions.

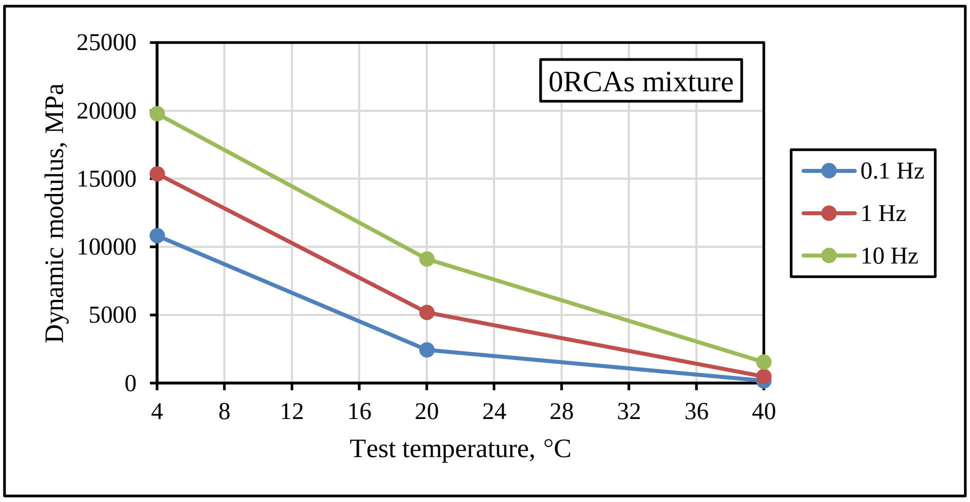


Figure 5.32: Relationship between dynamic modulus and test temperature for the 0RCAs mix

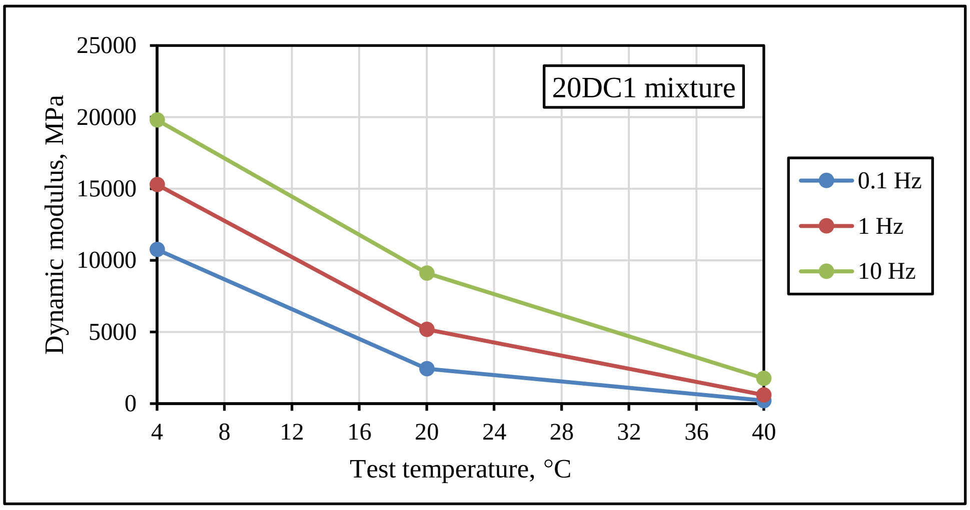
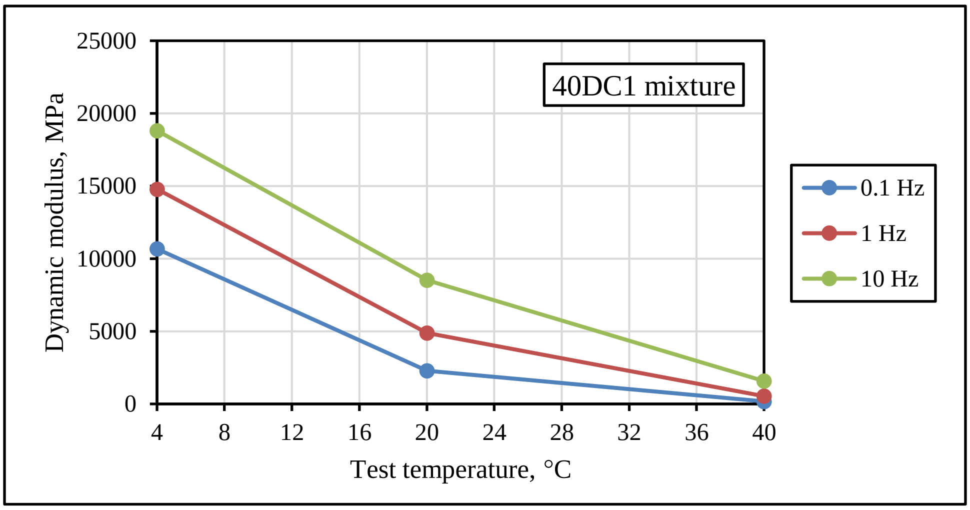


Figure 5.33: Relationship between dynamic modulus and test temperature for the 20DC1 mix



5.34: Relationship between dynamic modulus and test temperature for the 40DC1 mix

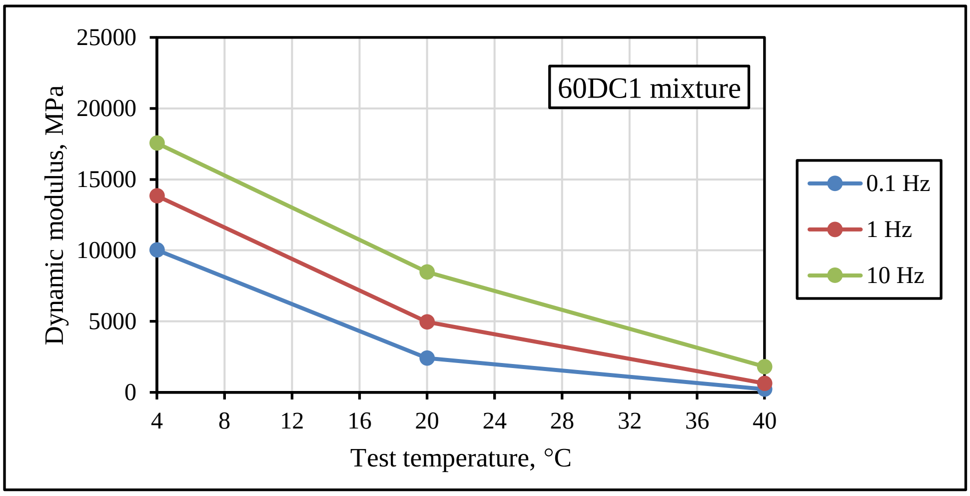


Figure 5.35: Relationship between dynamic modulus and test temperature for the 60DC1 mix

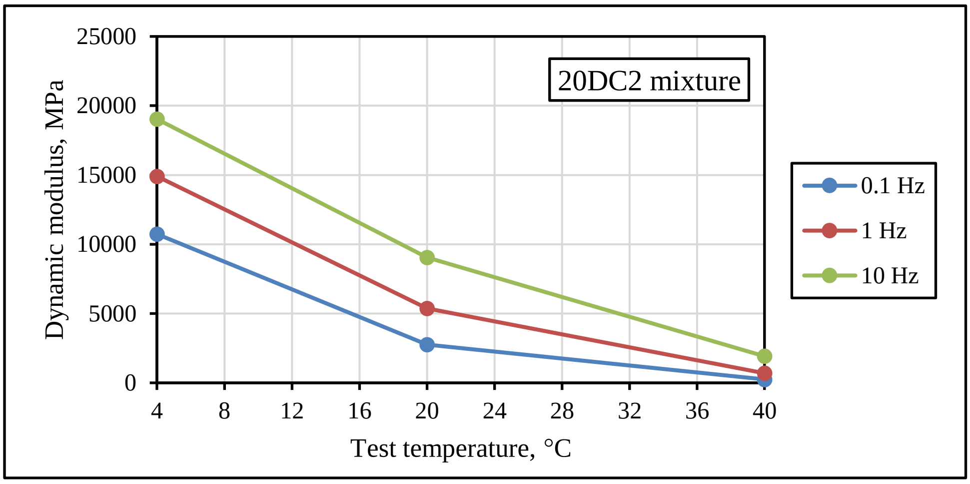


Figure 5.36: Relationship between dynamic modulus and test temperature for the 20DC2 mix

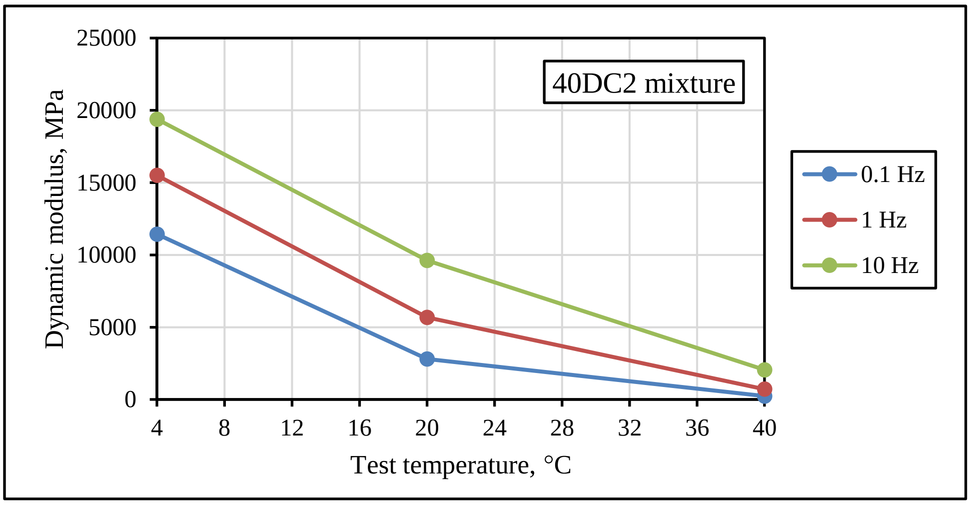


Figure 5.37: Relationship between dynamic modulus and test temperature for the 40DC2 mix

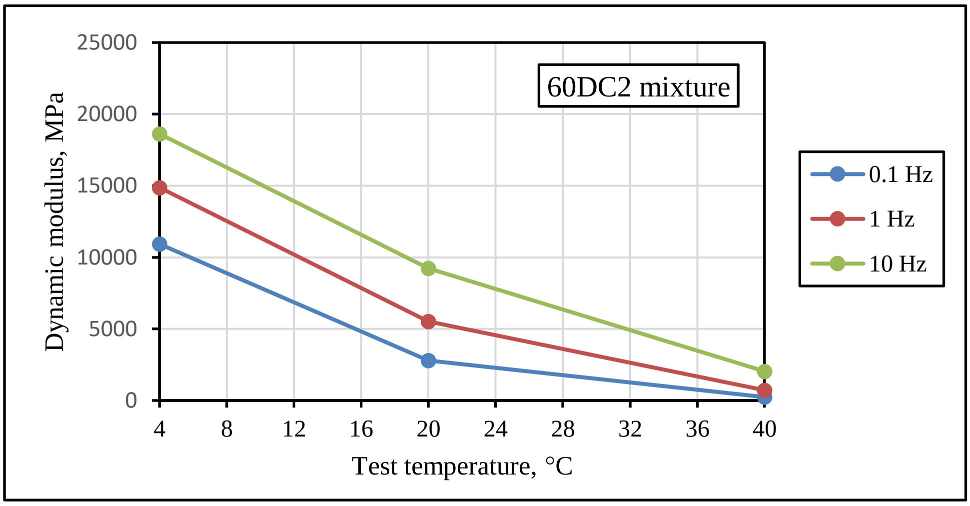


Figure 5.38: Relationship between dynamic modulus and test temperature for the 60DC2 mix

Based on the figures, the test temperature affects the dynamic modulus of asphalt mixtures made with granite aggregate and both types of DCRCAs (DC1 and DC2). The asphalt mixtures exhibited higher dynamic modulus at lower test temperatures than higher ones. At different loading frequencies, the change in dynamic modulus due to temperature was more evident at lower temperatures than higher ones. For instance, the average dynamic modulus of the 60DC2 mixture measured at 4 °C and 0.1 Hz, 1 Hz and 10 Hz were 10,926, 14,871.3 and 18,619.3 MPa, respectively. However, the dynamic modulus of the above-mentioned mixtures at 40 °C was 240, 720.2 and 2041 MPa at 0.1 Hz, 1 Hz and 10 Hz, respectively.

The average phase angle of three AMPT samples measured at 4 °C, 20 °C and 40 °C for each asphalt mixture are shown in Figure 5.39 to Figure 5.45.

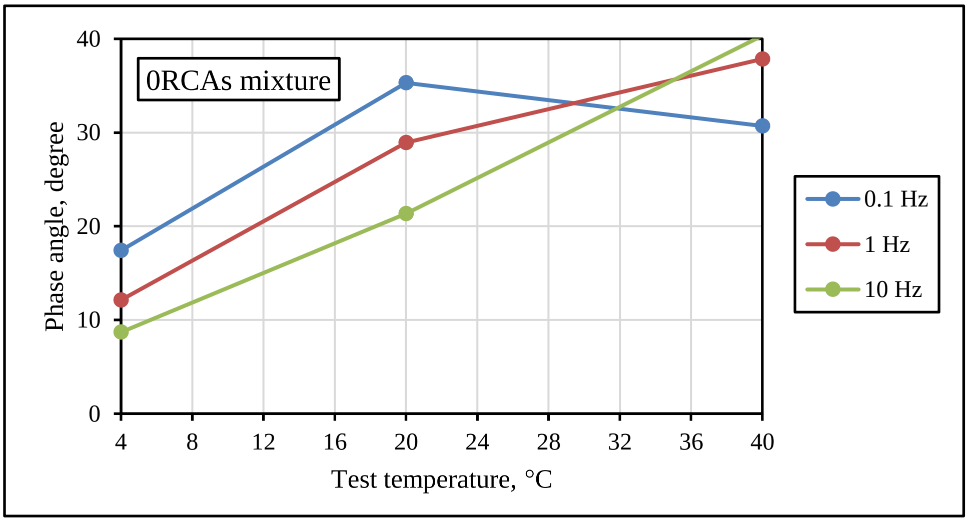


Figure 5.39: Relationship between phase angle and test temperature for the 0RCAs mix

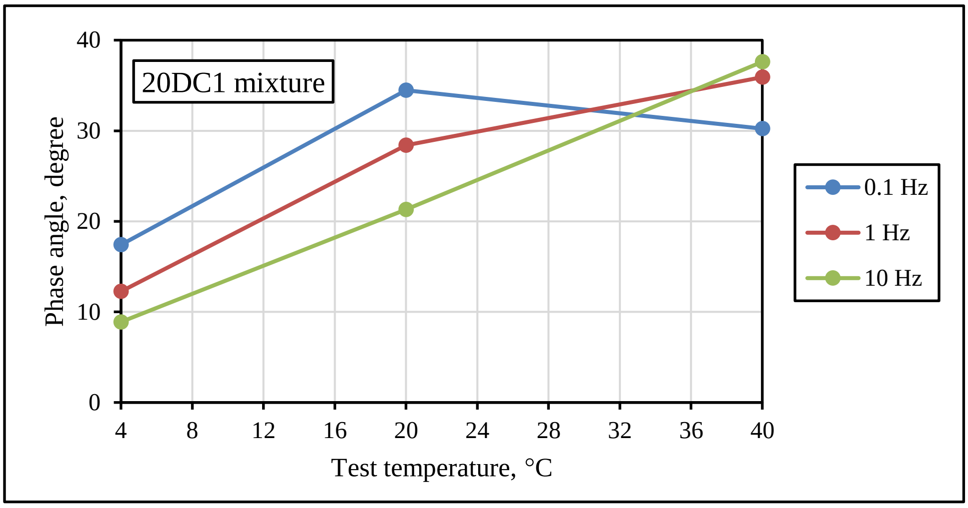


Figure 5.40: Relationship between phase angle and test temperature for the 20DC1 mix

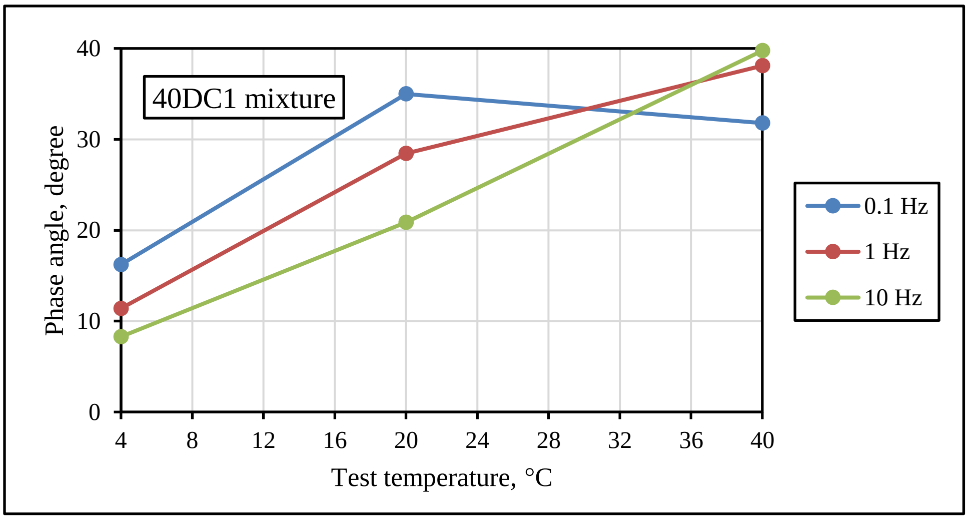


Figure 5.41: Relationship between phase angle and test temperature for the 40DC1 mix

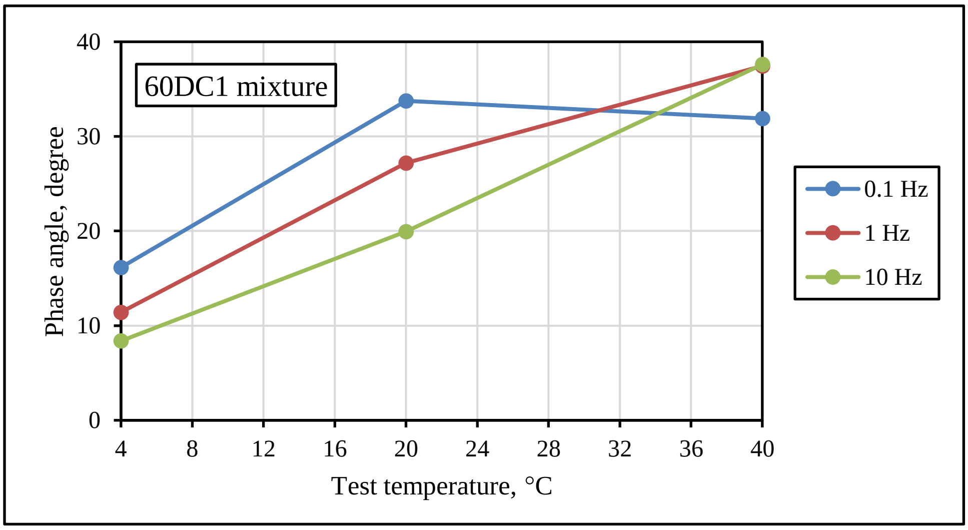


Figure 5.42: Relationship between phase angle and test temperature for the 60DC1 mix

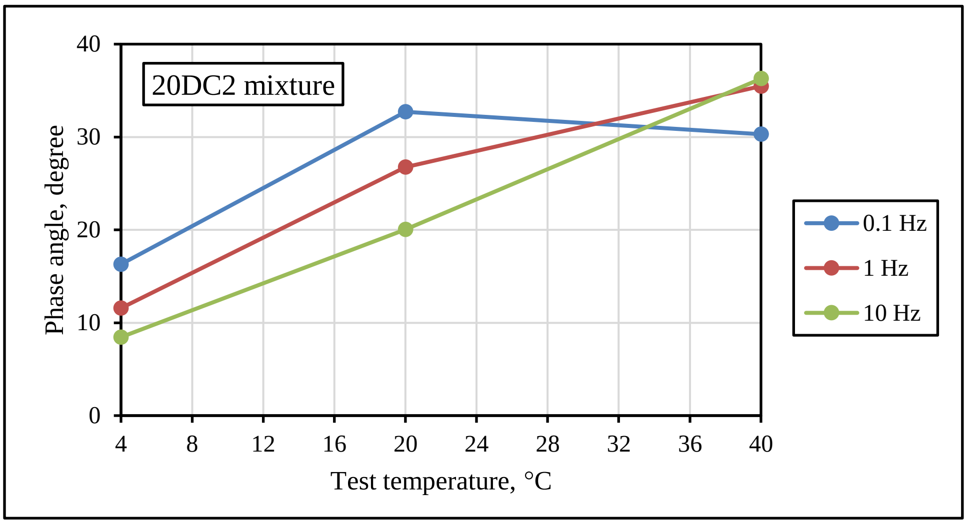


Figure 5.43: Relationship between phase angle and test temperature for the 20DC2 mix

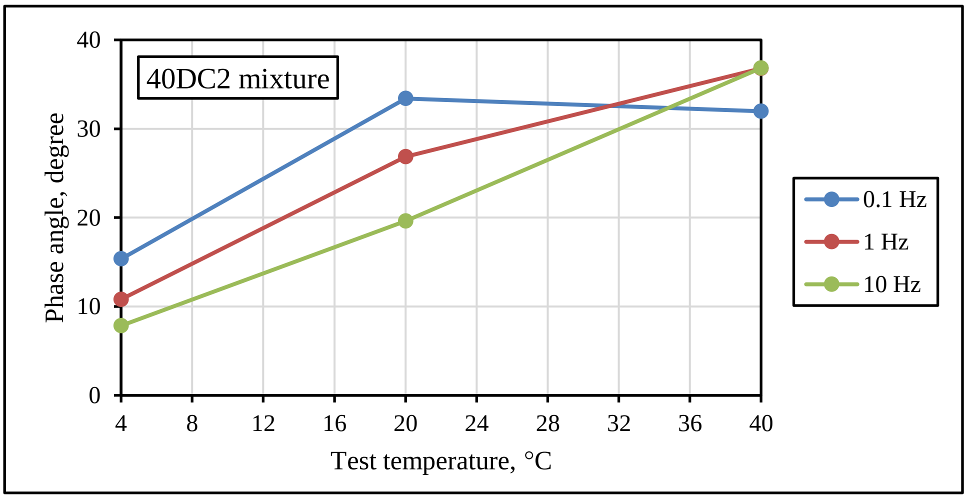


Figure 5.44: Relationship between phase angle and test temperature for the 40DC2 mix

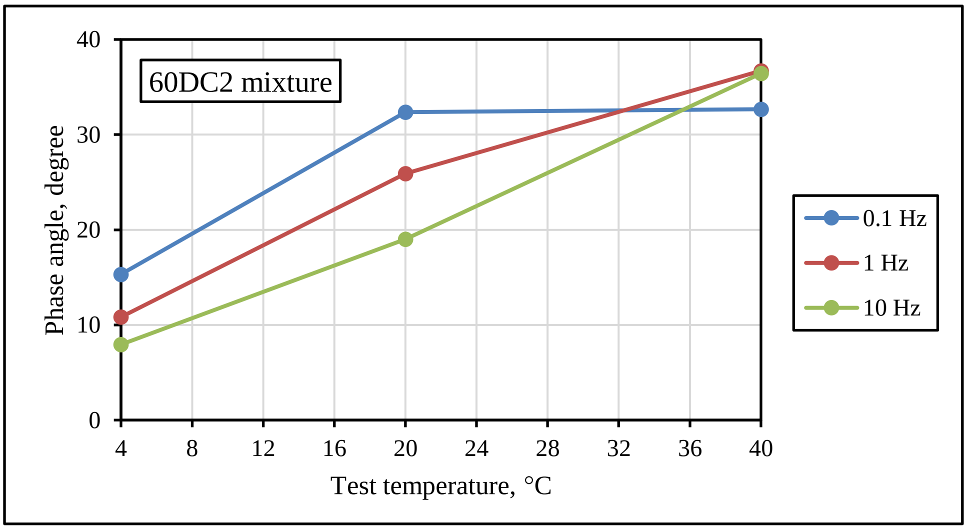


Figure 5.45: Relationship between phase angle and test temperature for the 60DC2 mix

As shown in the above figures, the phase angles of the asphalt mixtures are dependent on test temperature. Generally, the phase angle increases as the test temperature increases. This phenomenon is true for all asphalt mixtures tested at 1 Hz and 10 Hz.

However, the phase angle calculated at 0.1 Hz did not display the same trend when the tests were carried out at 40 °C. It can be seen, in Figure 5.44 for example, that the phase angle measured at 20 °C was 33.4° while that at 40 °C was 32°. There is only one exception to this trend. The 60DC2 mixture exhibited a slightly higher phase angle at 40 °C (32.7°) than at 20 °C (32.4°), as shown in Figure 5.45. This result might reflect the inhomogeneity of the RCAs used in the study.

5.4.3.2 Effect of loading frequency on dynamic modulus and phase angle of asphalt mixtures made with DCRCAs

The effects of loading frequency on the dynamic modulus of asphalt mixtures are plotted in Figure 5.46 to Figure 5.52.

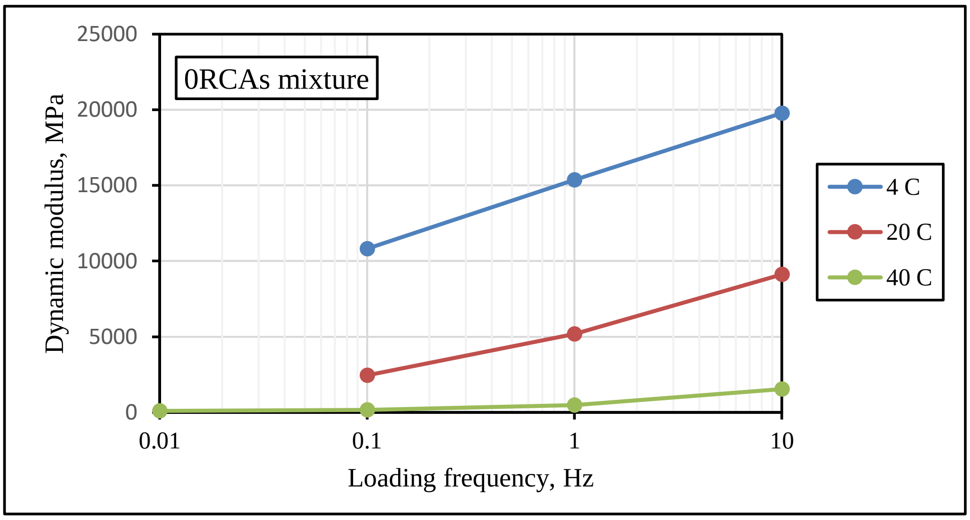


Figure 5.46: Relationship between dynamic modulus and loading frequency for the 0RCAs mix

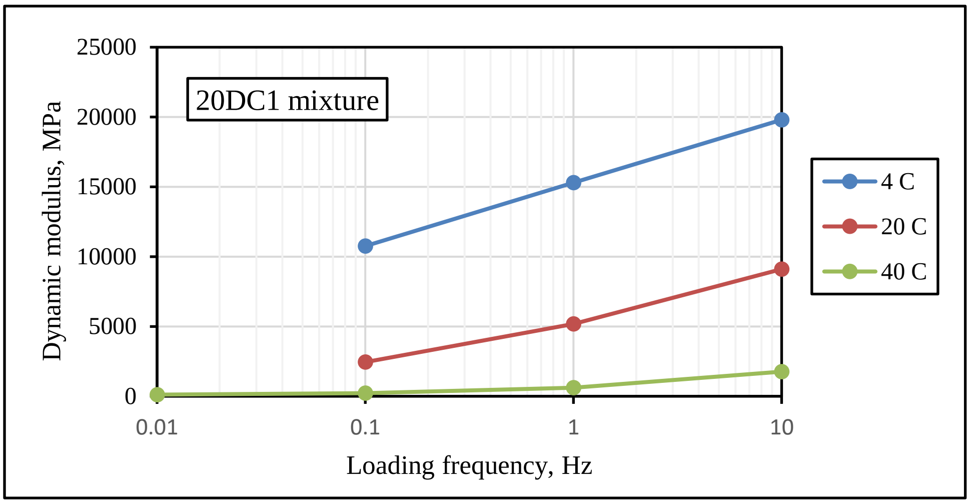


Figure 5.47: Relationship between dynamic modulus and loading frequency for the 20DC1 mix

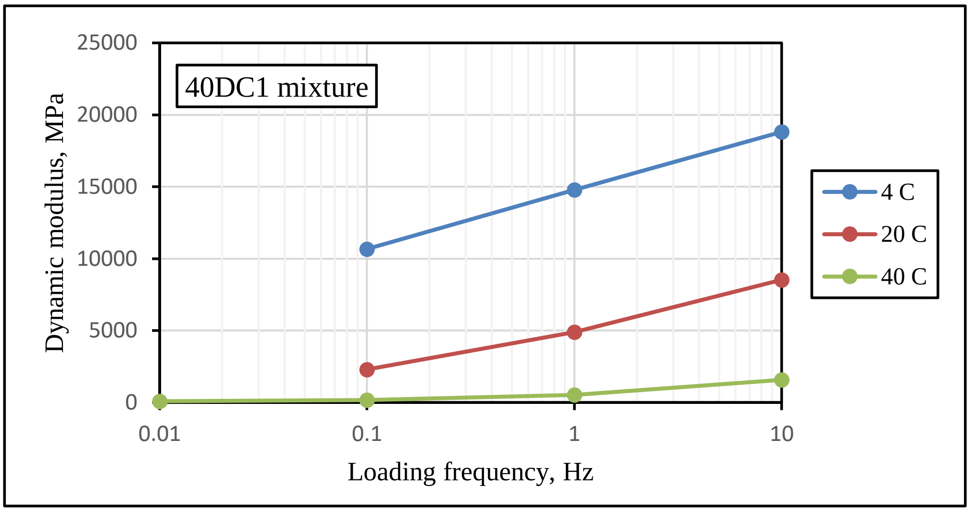


Figure 5.48: Relationship between dynamic modulus and loading frequency for the 40DC1 mix

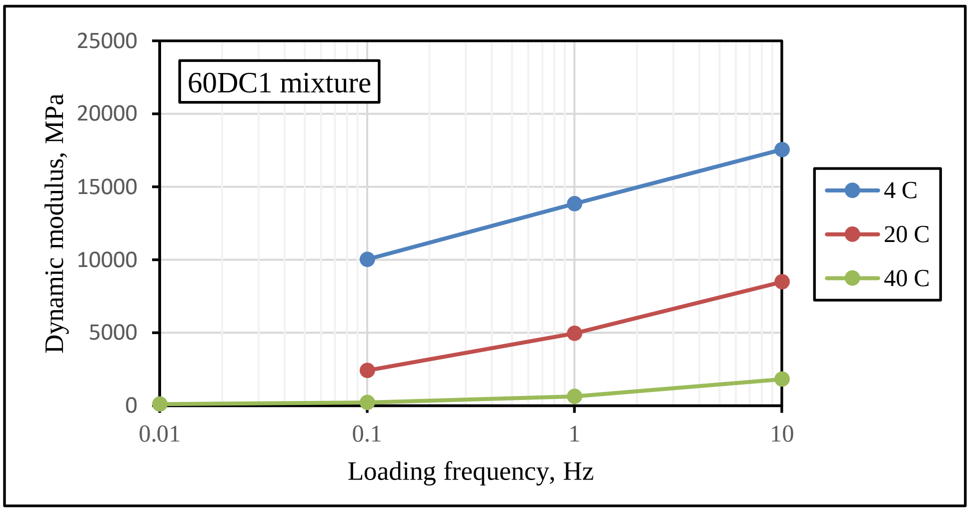


Figure 5.49: Relationship between dynamic modulus and loading frequency for the 60DC1 mix

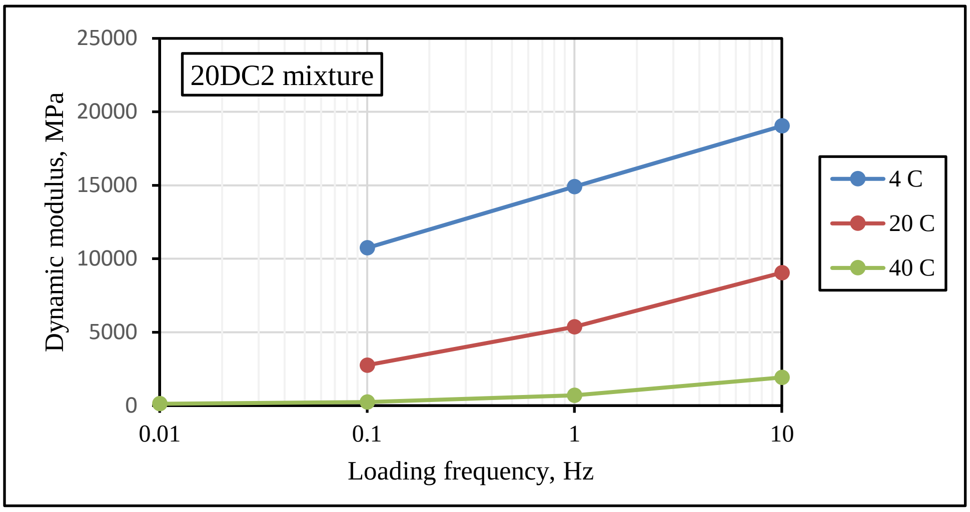


Figure 5.50: Relationship between dynamic modulus and loading frequency for the 20DC2 mix

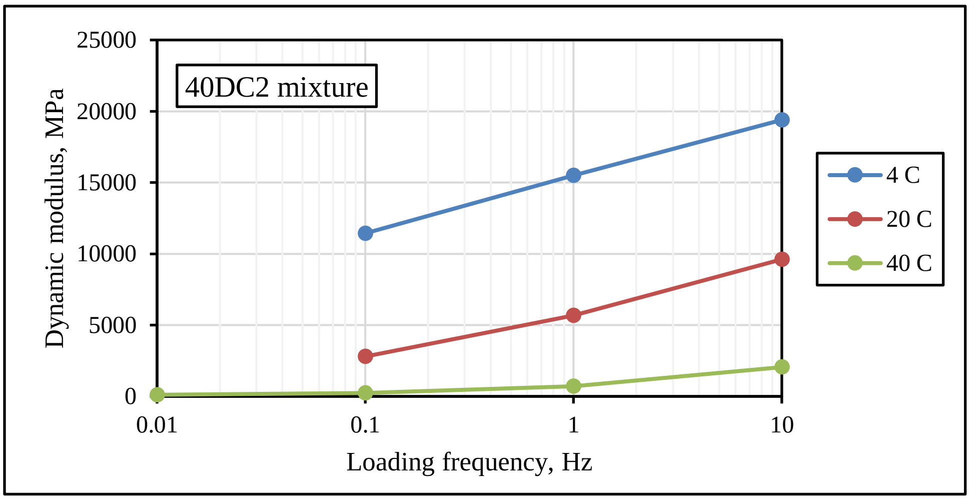


Figure 5.51: Relationship between dynamic modulus and loading frequency for the 40DC2 mix

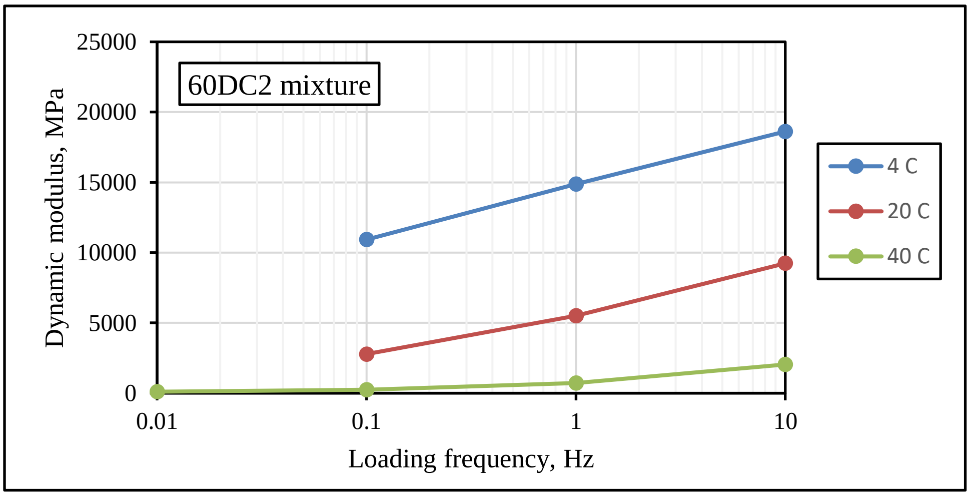


Figure 5.52: Relationship between dynamic modulus and loading frequency for the 60DC2 mix

According to the above figures, the loading frequency can affect the dynamic modulus of the control mixture and those with various dosages of DC1 and DC2. At the same test temperature, the mixtures demonstrated higher dynamic modulus values at higher loading frequencies than lower frequencies.

Additionally, the rate of change in dynamic modulus was more noticeable when the test was performed at lower testing temperatures than at higher temperatures.

The changes in the phase angles of asphalt mixtures due to changes in loading frequency are presented in Figure 5.53 to Figure 5.59.

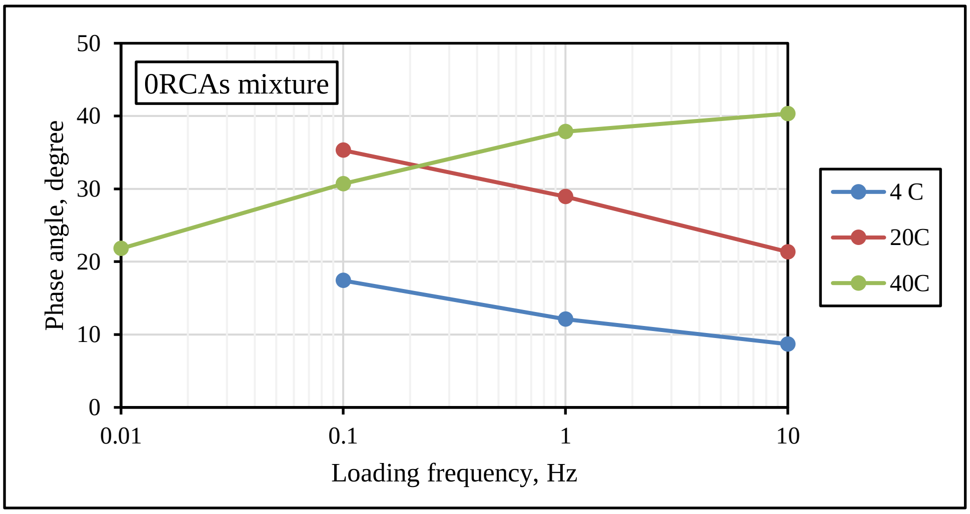


Figure 5.53: Relationship between phase angle and loading frequency, 0RCAs mix

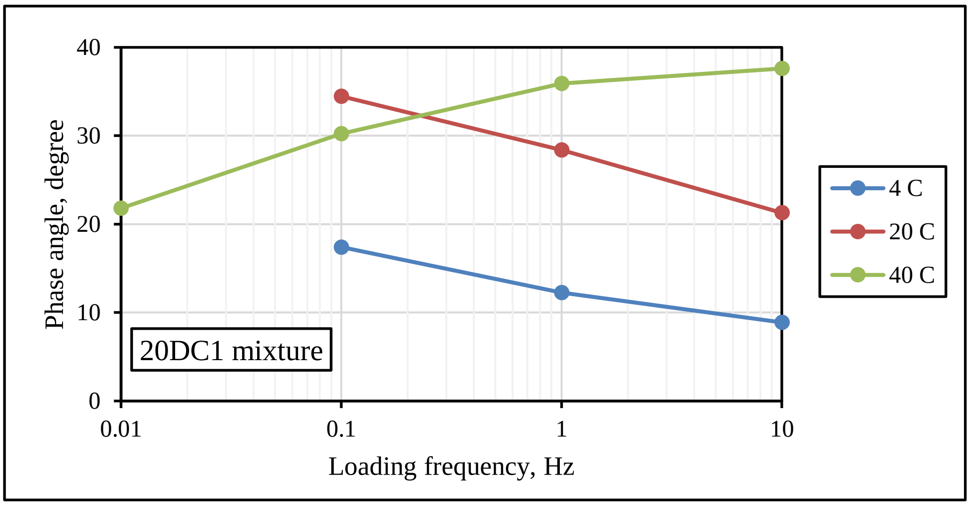


Figure 5.54: Relationship between phase angle and loading frequency, 20DC1 mix

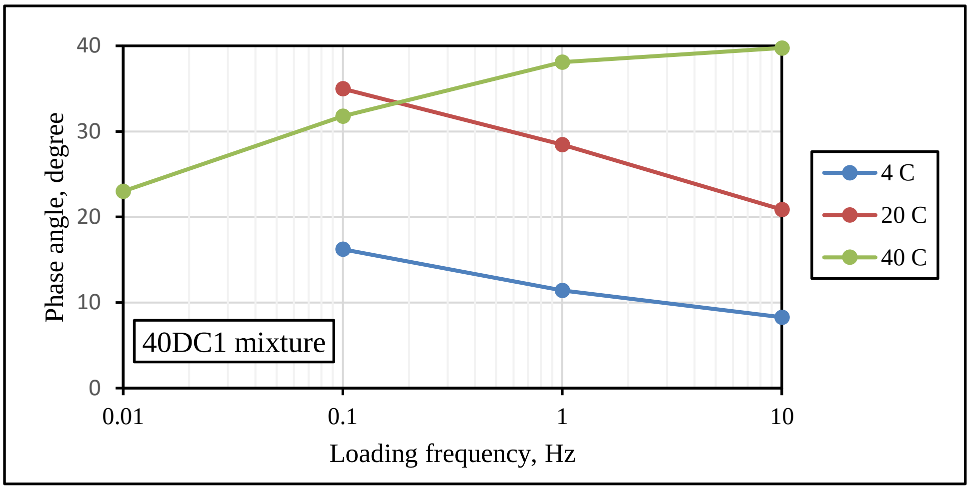


Figure 5.55: Relationship between phase angle and loading frequency, 40DC1 mix

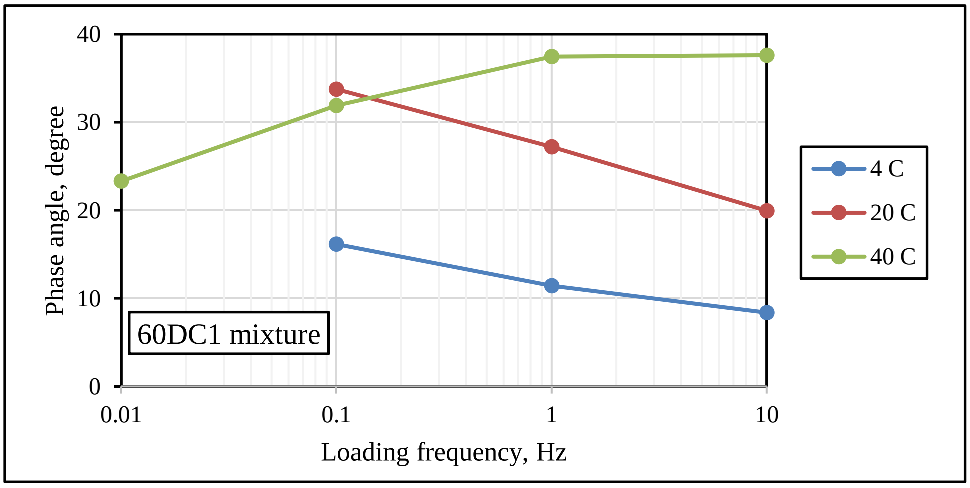


Figure 5.56: Relationship between phase angle and loading frequency, 60DC1 mix

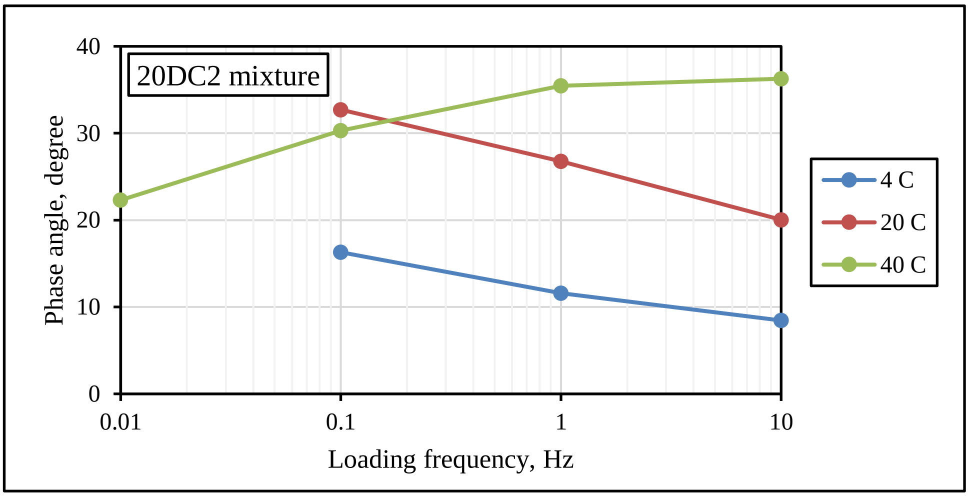


Figure 5.57: Relationship between phase angle and loading frequency, 20DC2 mix

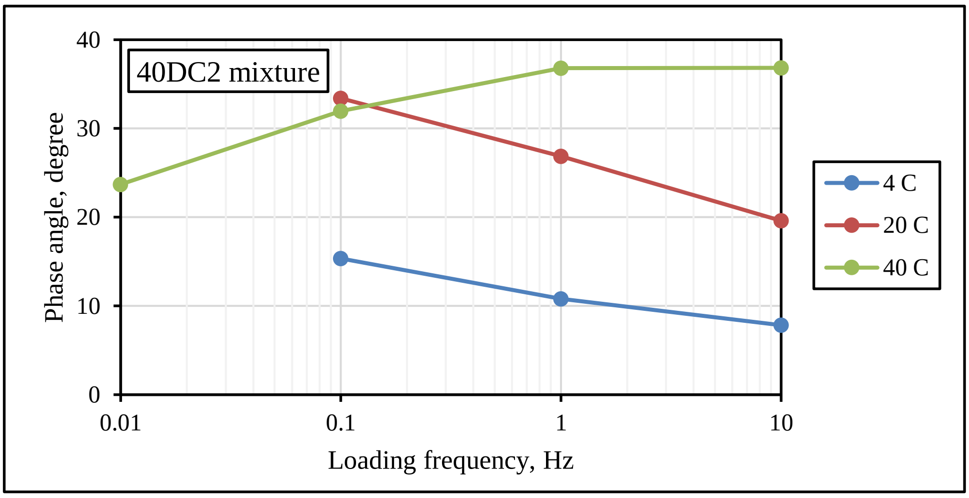


Figure 5.58: Relationship between phase angle and loading frequency, 40DC2 mix

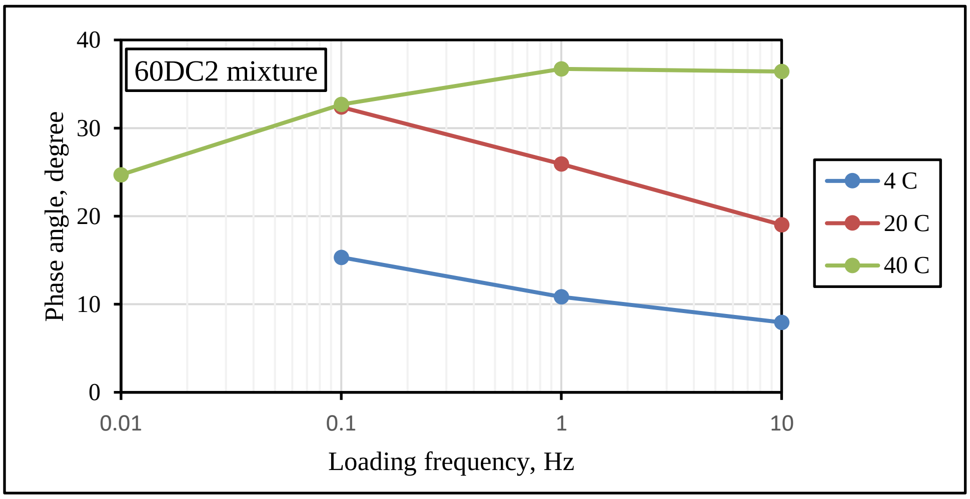


Figure 5.59: Relationship between phase angle and loading frequency, 60DC2 mix

Based on the above figures, the phase angles of asphalt mixtures tested at 4 °C and 20 °C tended to decrease as the loading frequency increased. However, this was not the case for tests performed at 40 °C. It can be seen in the figures that the phase angle continued to increase with loading frequency. There is only one exception to this trend. The 60DC2 mixture showed a slightly lower phase angle at 10 Hz (36.4°) than at 1 Hz (36.7°), as shown in Figure 5.59. This might be resulted due to heterogeneity of RCAs used in the research.

5.4.3.3 Effect of DCRCAs on the dynamic properties of asphalt mixtures

The two primary results of dynamic modulus testing are the dynamic modulus and phase angle. In the current investigation, two DCTs were developed to upgrade the performance of asphalt mixtures made with low-quality RCAs derived from C&D waste. It is, therefore, valuable to study the effects of the inclusion of different dosages of DC1 and DC2 on the dynamic characteristics of Australian HMA. In the following sections, the effects of adding DC1 and DC2 on the dynamic modulus and phase angle of asphalt mixtures are introduced and discussed.

5.4.3.3.1 Effect of DC1 addition on dynamic modulus and phase angle

The effect of DC1 content on the dynamic modulus of asphalt mixtures made with 0% DC1 (control mix) and 20%, 40% or 60% DC1 are shown in Figure 5.60, Figure 5.61, and Figure 5.62, respectively. Based on these figures, the DC1 content did not dramatically affect the dynamic modulus of DC1 mixtures. Also, the rate of change in dynamic modulus was found to depend on the DC1 dosage, test temperature and loading frequency.

It can be seen that the addition of 20% DC1 to asphalt mixtures has helped to achieve comparable dynamic modulus as the control mix at 4 °C and 20 °C. However, at 40

°C and with 40% or 60% DC1 inclusion, the DC1 mixtures demonstrated higher dynamic modulus than the control mix.

These findings agree with previous results obtained for asphalt mixtures made with 20% and 40% DC1 using ITSM testing, as explained in Section 5.3.3.2. On a different front, these results contradict those obtained by Mills-Beale and You (2010) and Bhusal and Wen (2013), where RCA-asphalt mixes showed lower dynamic modulus than a control mix at different loading frequencies and temperatures.

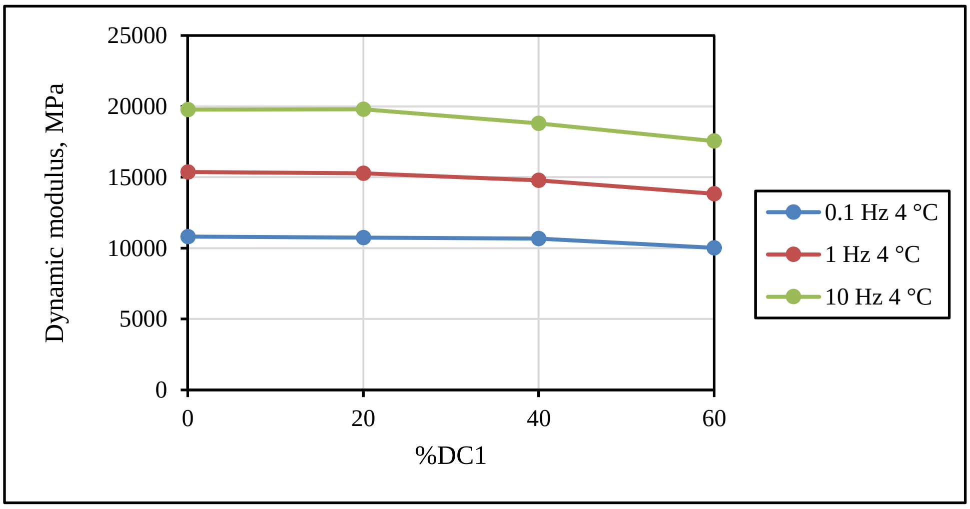


Figure 5.60: Dynamic modulus versus DC1 content at 4 °C

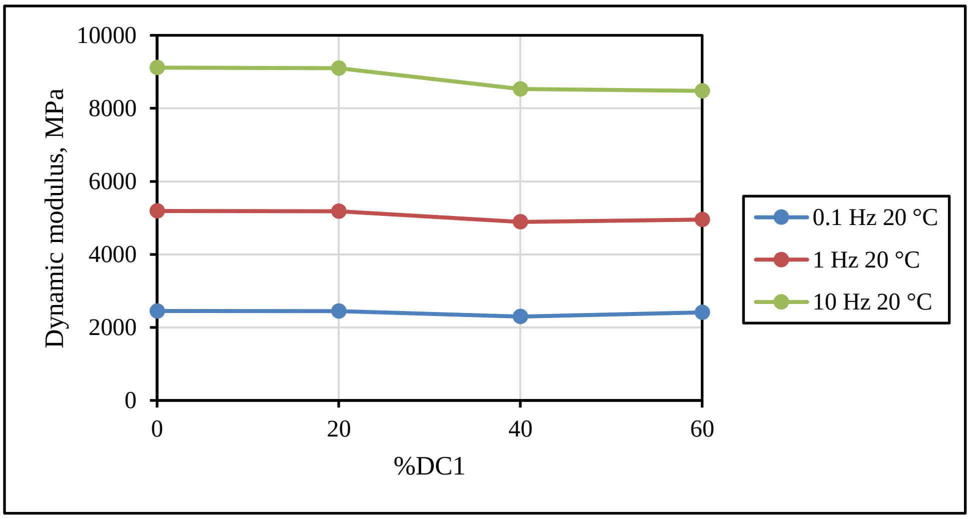


Figure 5.61: Dynamic modulus versus DC1 content at 20 °C

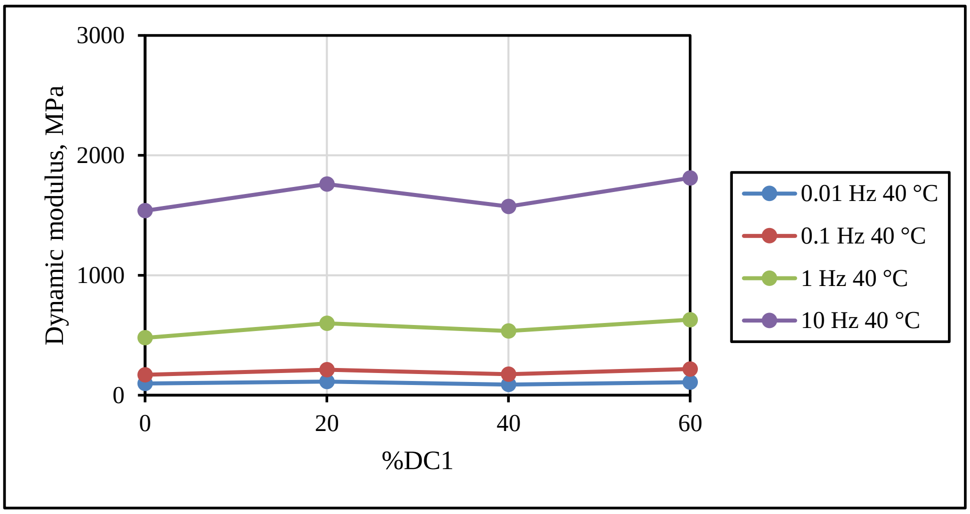


Figure 5.62: Dynamic modulus versus DC1 content at 40 °C

The effects of the DC1 content of Australian HMA on phase angles are shown in Figure 5.63, Figure 5.64, and Figure 5.65. The figures reveal that the addition of up to 60% DC1 to an asphalt mixture decreases its phase angle. Only one exception was noted, where asphalt mixtures made with 40% and 60% DC1 tended to demonstrate higher phase angles than the control mix at 40 °C and lower loading frequencies of 0.01 Hz and 0.1 Hz. The results indicate that the addition of DC1 gives the mixtures less viscous performance than the control mix at low temperatures (4 °C and 20 °C). Additionally, at 40 °C and lower frequencies (0.01 Hz and 0.1 Hz), the mixtures made with 40% and 60% DC1 exhibited more viscous behaviour than the mixture with no DC1 (control mix).

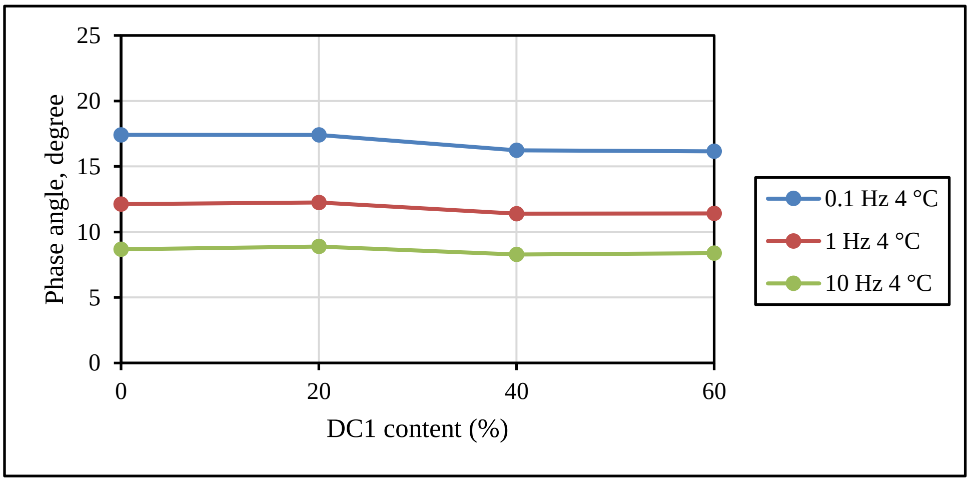


Figure 5.63: Phase angle versus DC1 content at 4 °C

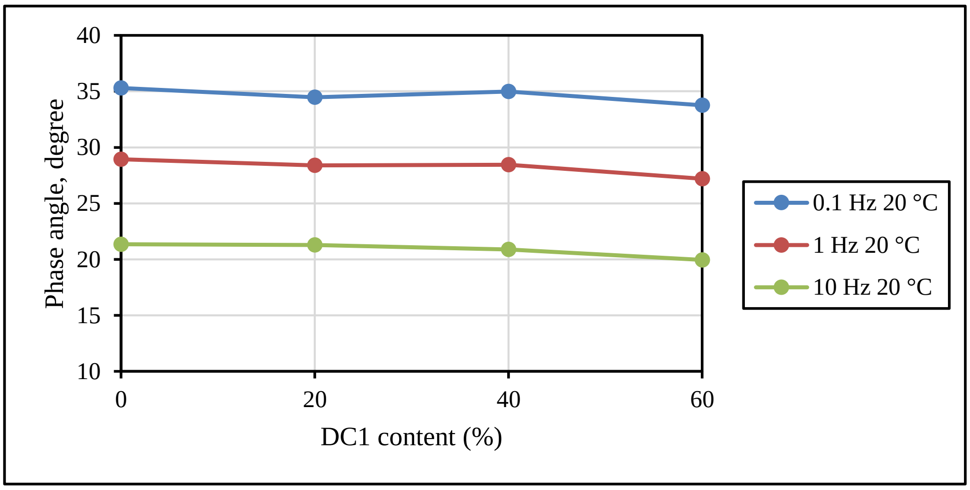


Figure 5.64: Phase angle versus DC1 content at 20 °C

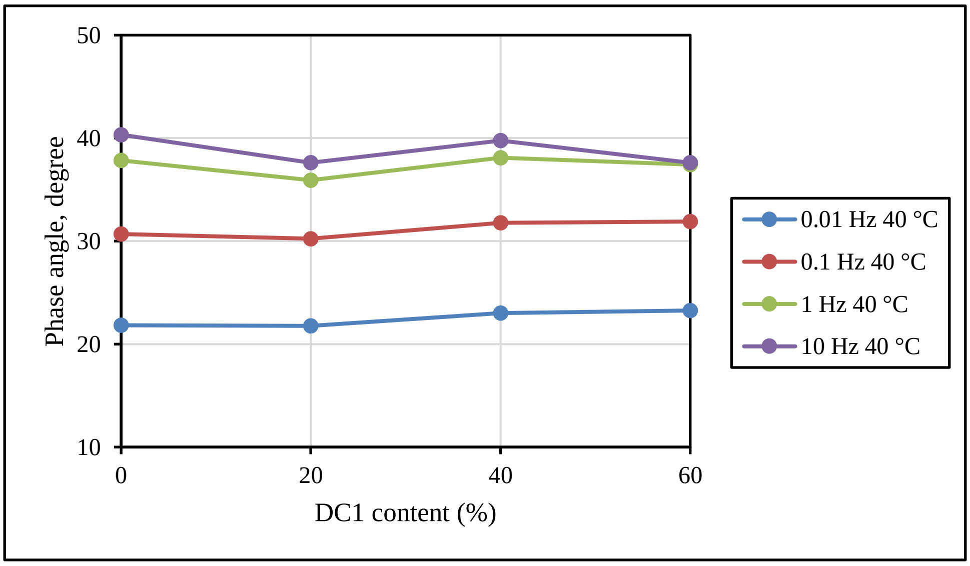


Figure 5.65: Phase angle versus DC1 content at 40 °C

An alternative way to show the effects of DC1 addition to Australian HMA is by comparing the dynamic modulus values of control mixtures and those made with different DC1 contents, as shown in Figure 5.66 to Figure 5.69. The values of dynamic modulus above the line of equality represent an improvement in the dynamic modulus of DC1 mixtures over that of control mix, and vice versa. The results of the dynamic modulus testing of DC1 mixtures, in general, show that the addition of RCAs derived from C&D waste and upgraded through DCT1 does not greatly affect the dynamic modulus of Australian HMA.

It can be seen that the addition of 20% DC1 to asphalt mixtures produced a dynamic modulus approximately 15% higher than that obtained for the control mix (Figure 5.66). However, the dynamic modulus of the 40DC1 mixture decreased by about 20% compared to the control mix (Figure 5.67). However, asphalt mixtures with 60% DC1 showed comparable dynamic modulus values as the control mix, as can be seen in Figure 5.68. These results in some way contradict those obtained by Mills-Beale and You (2010) and Bhusal and Wen (2013), where RCA-asphalt mixes were reported to have lower dynamic modulus than control mix at different loading frequencies and temperatures. The behaviour of asphalt mixtures made with DC1, particularly 20DC1 and 60DC1 mixtures, implies that there was a performance improvement gained after DCT1 (i.e., coating with CSP and Sika Tite-BE. DCT1 gave the DC1 mixtures better adhesion between aggregate particles and bitumen. Thus, asphalt mixtures made with DC1 tended to demonstrate comparable viscoelastic performance to the control mix, as shown in Figure 5.69.

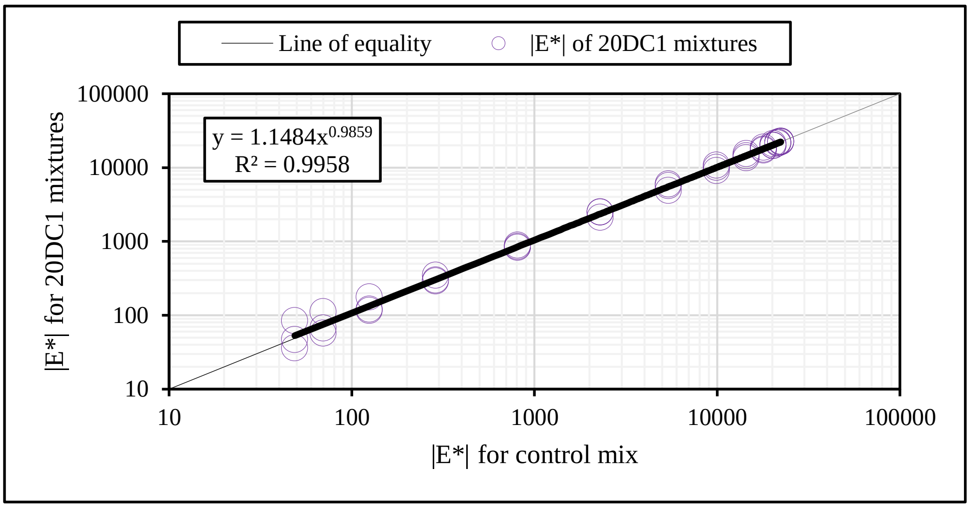


Figure 5.66: Comparison of dynamic modulus of control mix versus 20DC1 mixtures

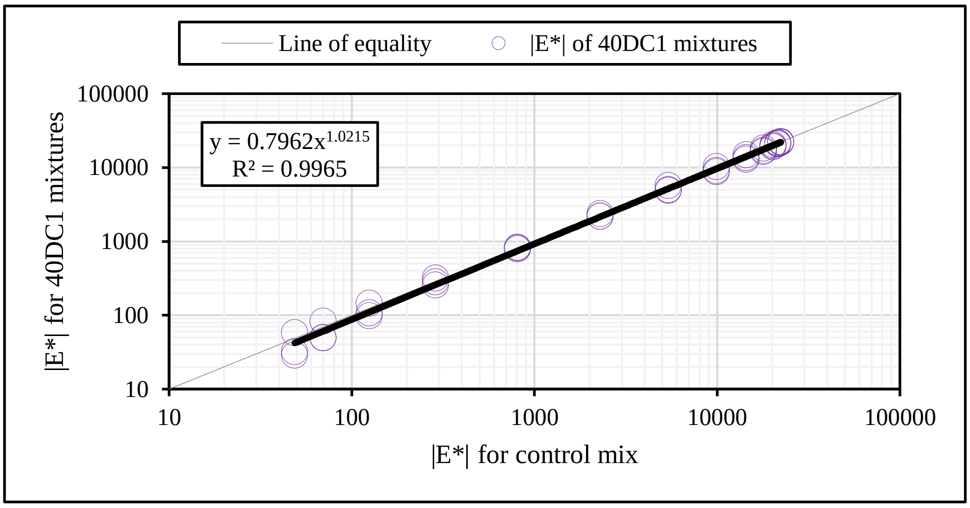


Figure 5.67: Comparison of dynamic modulus of control mix versus 40DC1 mixtures

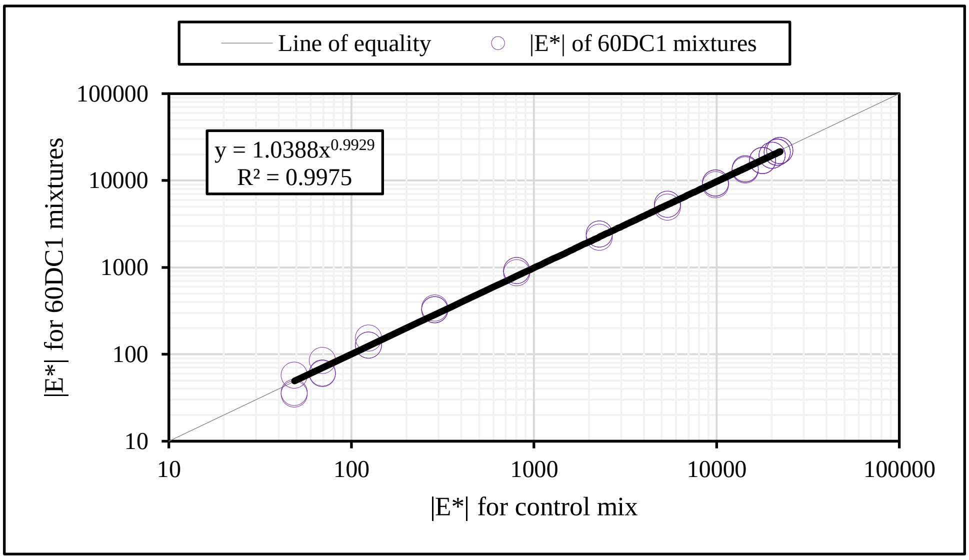


Figure 5.68: Comparison of dynamic modulus of control mix versus 60DC1 mixtures

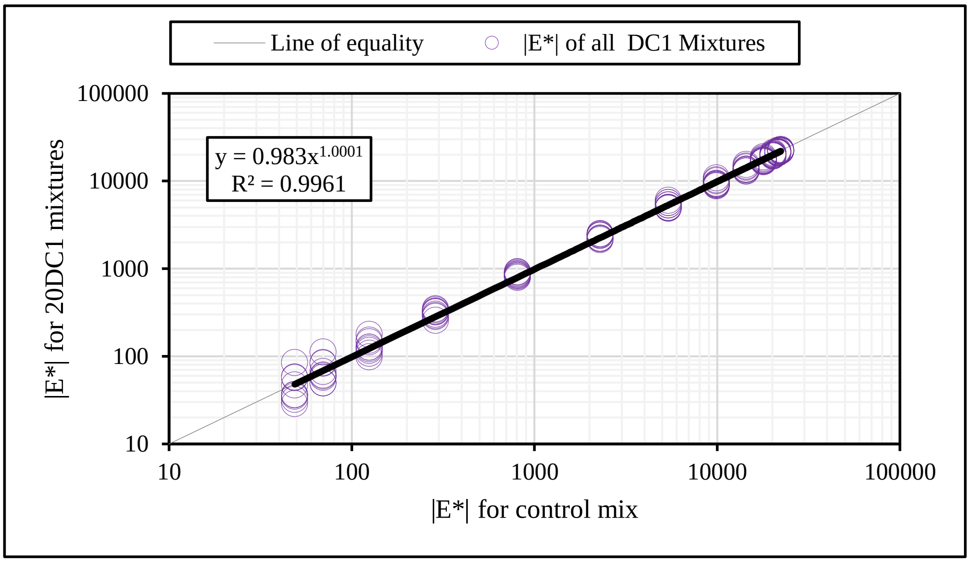


Figure 5.69: Comparison of dynamic modulus of control mix versus all DC1 mixtures

5.4.3.3.2 Effect of DC2 addition on dynamic modulus and phase angle

The results of dynamic modulus testing of Australian HMA made with 0% (control mix), 20%, 40% and 60% DC2 are shown in Figure 5.70, Figure 5.71, and Figure 5.72 respectively. Based on these figures, the addition of DC2 improved the dynamic modulus of asphalt mixtures, particularly at 20 °C (moderate) and 40 °C (high) temperatures. The dynamic modulus of asphalt mixtures containing 40% and 60% DC2 was improved when tested under conditions of low frequency (0.1 Hz) and temperature (4 °C).

It can be seen that asphalt mixtures made with 20%, 40% and 60% DC2 tended to demonstrate slightly lower dynamic modulus than the control mix (0% DC2) at 4 °C and loading frequencies of 1 Hz and 10 Hz. Furthermore, the results indicate that the dynamic modulus of DC2 mixes peak at 40% granite replacement at different temperatures and frequencies. These results are consistent with those obtained using the ITSM test, where the resilient modulus of DC2 mixtures peaked at 40% DC2, as presented in Table 5.11 and Table 5.12. It should be mentioned that these findings are dissimilar to those obtained by Mills-Beale and You (2010) and Bhusal and Wen

(2013). This shows that Australian HMA made with low-quality RCAs derived from C&D waste and upgraded through DCT2 behaves differently than mixtures made in other regions of the world, such as the USA.

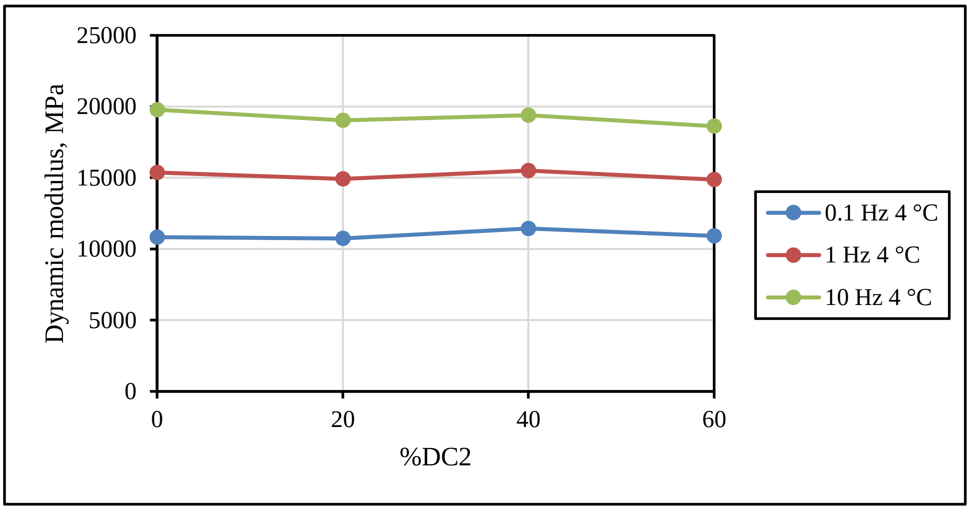


Figure 5.70: Dynamic modulus versus DC2 content at 4 °C

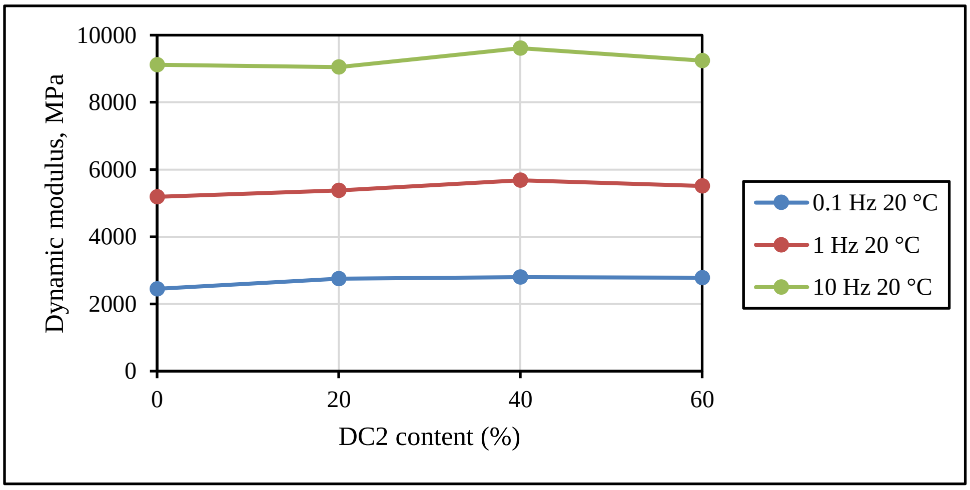


Figure 5.71: Dynamic modulus versus DC2 content at 20 °C

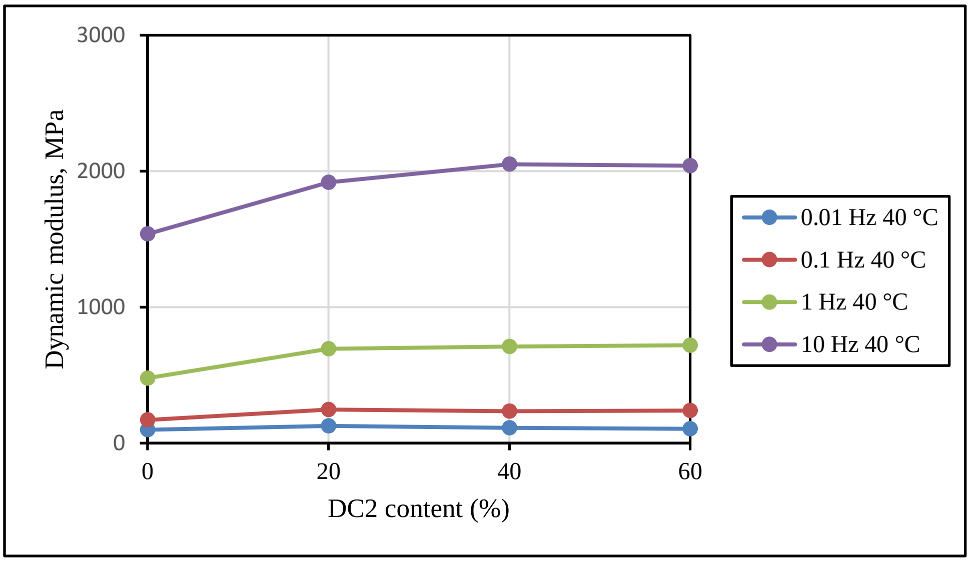


Figure 5.72: Dynamic modulus versus DC2 content at 40 °C

Figure 5.73, Figure 5.74, and Figure 5.75 show the effects of DC2 content on the phase angle of the asphalt mixtures. Based on the results presented in these figures, the phase angles decreased with addition of DC2 to HMA. Only one exception was found: the asphalt mixtures made with DC2 generally had higher phase angles than the control mix at 40 °C and lower loading frequencies (0.01 Hz and 0.1 Hz), as shown in Figure 5.75. This trend in the phase angle results is similar to that obtained for DC1 mixtures.

The results imply that asphalt mixtures containing DC2 had more elastic (less viscous) performance than the control mix. Additionally, at higher test temperatures (40 °C) and lower loading frequencies (0.01 Hz and 0.1 Hz), these mixtures tended to exhibit less elastic (more viscous) behaviour compared to the mix with no DC2.

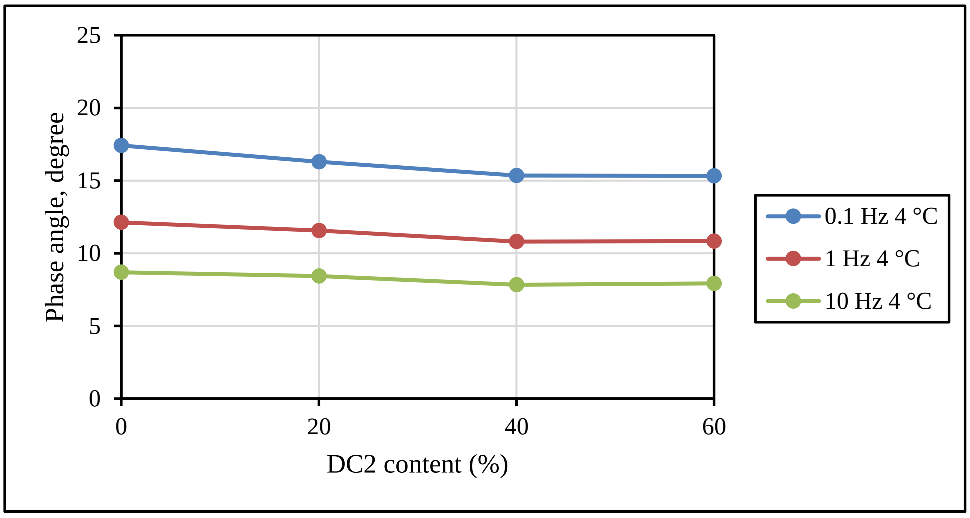


Figure 5.73: Phase angle versus DC2 content at 4 °C

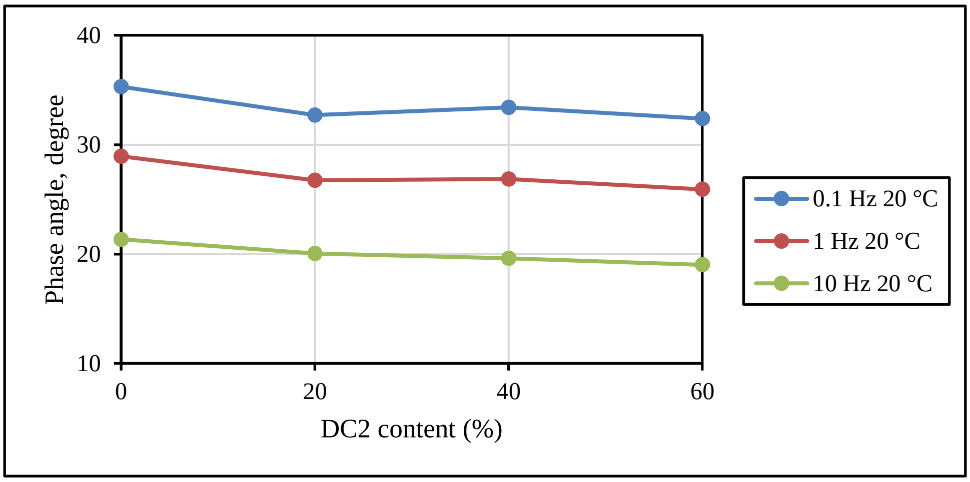


Figure 5.74: Phase angle versus DC2 content at 20 °C

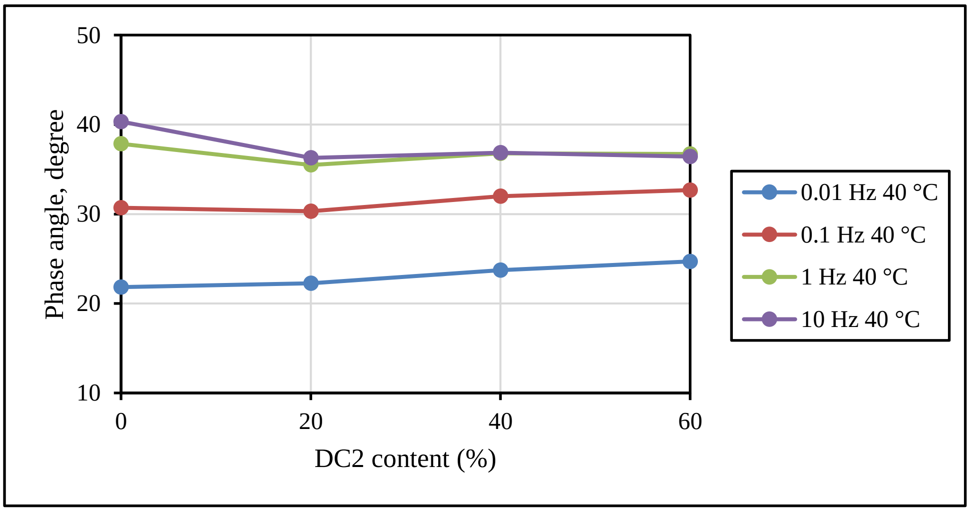


Figure 5.75: Phase angle versus DC2 content at 40 °C

An alternative way to assess the effects of DC2 addition on the dynamic modulus of Australian HMA is presented in Figure 5.76, Figure 5.77, Figure 5.78, and Figure 79 respectively. The results in these figures confirm that adding DC2 improves the viscoelastic performance of asphalt mixtures. The DC2 asphalt mixtures exhibited higher dynamic modulus than the control mix. The dynamic modulus increased by about 1.5 and 1.16 for asphalt mixtures made with 20% and 40% DC2, respectively. Also, the 60% DC2 mixture achieved a comparable dynamic modulus to that obtained for the control mix, as demonstrated in Figure 5.78.

Additionally, Figure 5.79 shows the dynamic modulus results for all DC2 asphalt mixtures as compared to the control mix. It can be seen that the addition of DC2 to Australian HMA increased the dynamic modulus by about 22%. These findings are promising and encouraging. It should be noted that the RCAs used in this research were low-quality RCAs derived from C&D waste, which is not allowed to be used in HMA production in Australia. The DCT2 seemingly improves the viscoelastic performance of asphalt mixtures made with RCAs. This is due to strength and durability improvements, as explained in Chapter 3, Section 3.11.

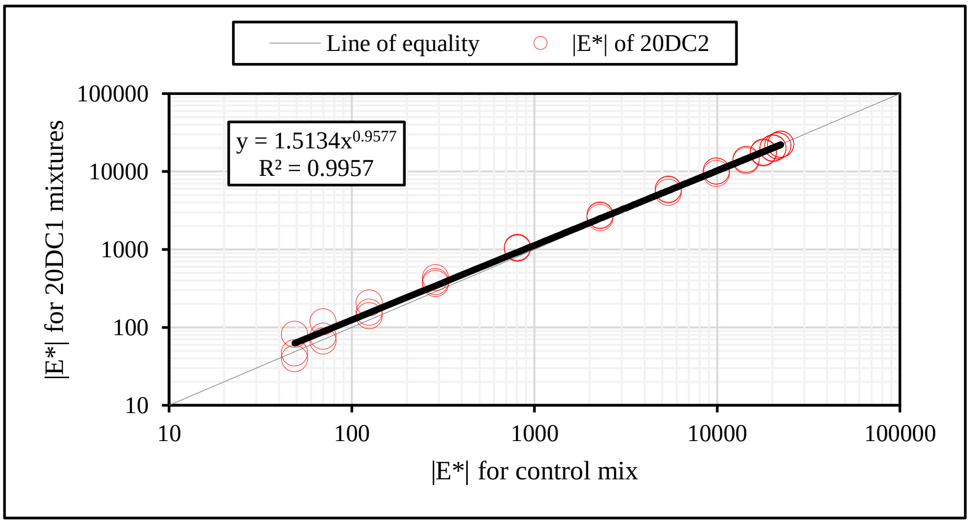


Figure 5.76: Comparison of the dynamic modulus of control mix and 20DC2 mixtures

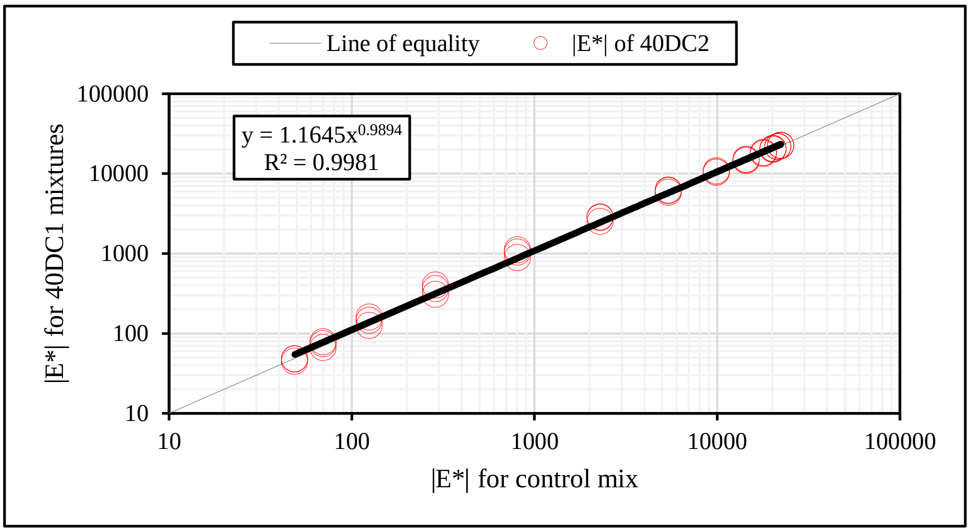


Figure 5.77: Comparison of the dynamic modulus of control mix and 40DC2 mixtures

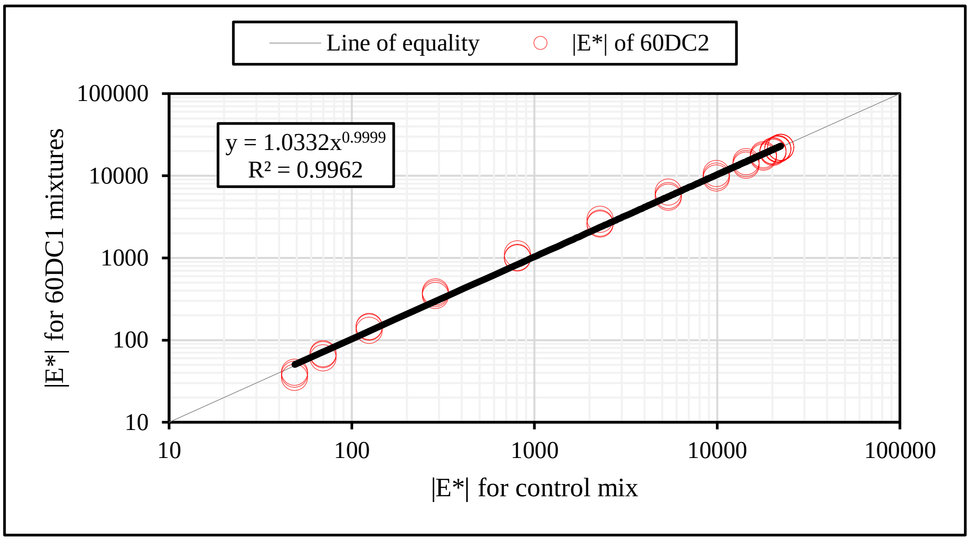


Figure 5.78: Comparison of the dynamic modulus of control mix and 60DC2 mixtures

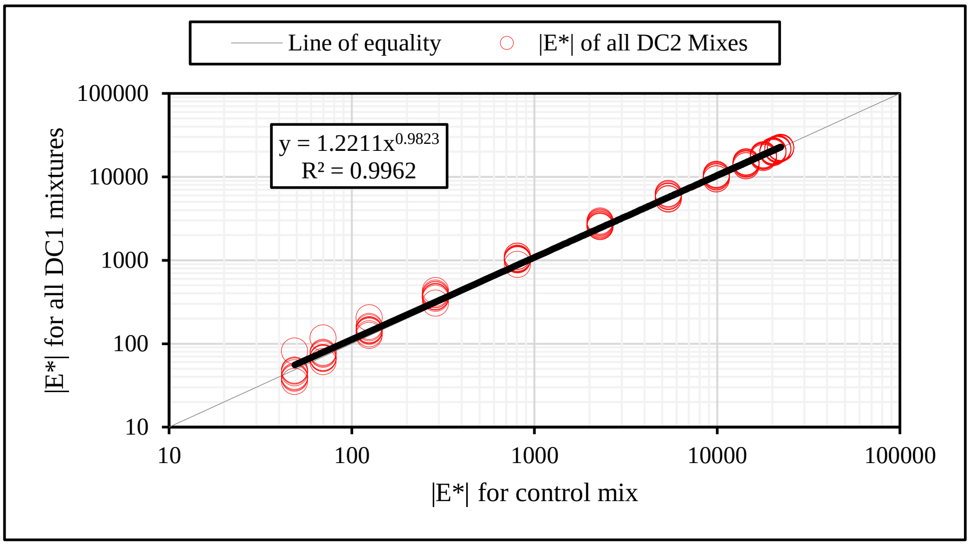


Figure 5.79: Comparison of the dynamic modulus of control mix and 20DC2 mixtures

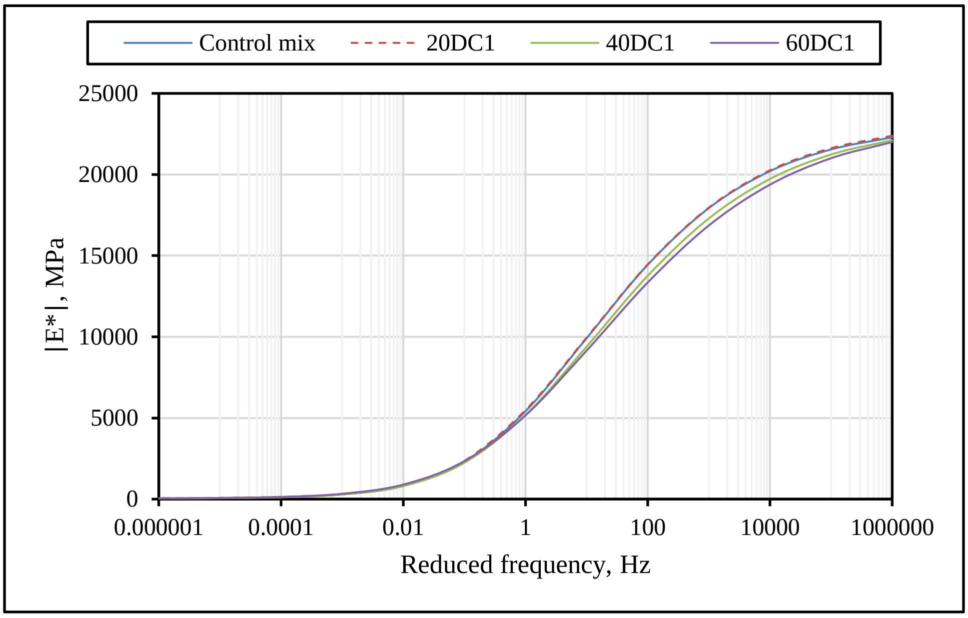
5.4.3.4 Master curves of asphalt mixtures made with DCRCAs

In the mechanistic-empirical design guide developed under NCHRP project 1-37, the dynamic modulus $|E^*|$ is considered the most important property of hot-mix asphalt (M. Witczak & Bari, 2004). The dynamic modulus of HMA can be determined at any temperature and loading frequency from a dynamic modulus master curve constructed at a reference temperature (Shaopeng Wu, Ye, Li, & Yue, 2007). The master curves of the asphalt mixtures made in this study were generated based on laboratory data collected from dynamic modulus testing and the equations mentioned in Section 4.5.2.3.1.

5.4.3.4.1 Master curves of DC1 mixtures

The dynamic modulus master curves for asphalt mixtures made with 0% (control mix), 20%, 40% and 60% of DC1 at 20 °C are shown in Figure 5.80.

As mentioned previously in Section 5.4.3.3.1, the results indicate that the addition of DC1 did not affect the dynamic modulus dramatically. It can be seen that the curves were passing each other under loading frequencies less than 1 Hz and at very high frequencies. In order to provide clarity on the role of DC1 on the dynamic modulus of Australian HMA, the data used to generate the master curves is provided in Table 5.36.



. Figure 5.80: Dynamic modulus master curves of control mix and asphalt mixtures containing 20%, 40% or 60% DC1

Table 5.36: Data for master curves of asphalt mixtures made with granite aggregate, and 20%, 40%, and 60% DC1

The data of master curves of asphalt mixtures made with granite aggregates, and				
Reduced frequency, Hz	Control mix	20DC1 mix	40DC1 mix	60DC1 mix
1000000	22290.9	22387.0	22117.4	22000.0
100000	21544.6	21621.4	21232.7	21004.8
10000	20204.9	20257.5	19723.4	19374.6
1000	17930.5	17960.1	17294.7	16859.6
100	14437.1	14459.0	13759.6	13345.8
10	9912.0	9952.9	9392.6	9148.4
1	5414.4	5483.7	5176.9	5159.0
0.1	2285.5	2355.9	2240.3	2339.4
0.01	806.9	853.1	807.5	895.0
0.001	287.9	312.2	283.5	328.8
0.0001	124.5	137.5	115.3	135.0
0.00001	69.7	77.7	59.3	68.0
0.000001	48.8	54.6	38.3	42.4

It can be seen that the replacement of 20% of the granite with DC1 increased the dynamic modulus at all loading frequencies, as shown in Table 5.36. As more and more DC1 was included in the HMA, the mixtures tended to be more sensitive to DC1 content. It can be seen that bituminous mixtures made with 40% and 60% DC1 demonstrated lower modulus than control mix at loading frequencies greater than 0.1 Hz. However, the data presented in Table 5.36 imply that 60DC1 mixtures exhibited, in general, higher dynamic modulus than control mix at frequencies lower than 1 Hz.

Furthermore, the dynamic modulus of the 40DC1 and 60DC1 mixtures were more affected at loading frequencies ranging from 10 to 10,000 Hz. However, the difference between the dynamic modulus results for these mixtures and that obtained for the control mixture decreased as the loading frequency increased. For example, at 10 Hz, the 40DC1 and 60DC1 mixtures showed dynamic modulus of 9392.6 and 9148.4 MPa, respectively, which were lower than that obtained for the control mix (9912 MPa) by only 5.2% and 7.7%. However, at 10,000 Hz, the 40DC1 and 60DC1 mixtures showed dynamic modulus of 21,232.7 and 21,004.8 MPa, which were 1.4% and 2.5% lower than that of the control mix (21,544.6 MPa).

5.4.3.4.2 Master curves of DC2 mixtures

The dynamic modulus master curves for asphalt mixtures made with 0% (control mix), 20%, 40% or 60% DC2 at the 20 °C reference temperature are shown in Figure 5.81. The data represented in each curve are the average of three AMPT test samples, as explained in Section 4.5.2.3. It can be seen that the dynamic modulus curves of different asphalt mixtures were close to each other at all test frequencies. Therefore, in order to take a closer look at the role of DC2 on the dynamic modulus of different mixtures made with different dosages (0%, 20%, 40% and 60%) of DC2, the data used to generate these curves is provided in Table 5.37.

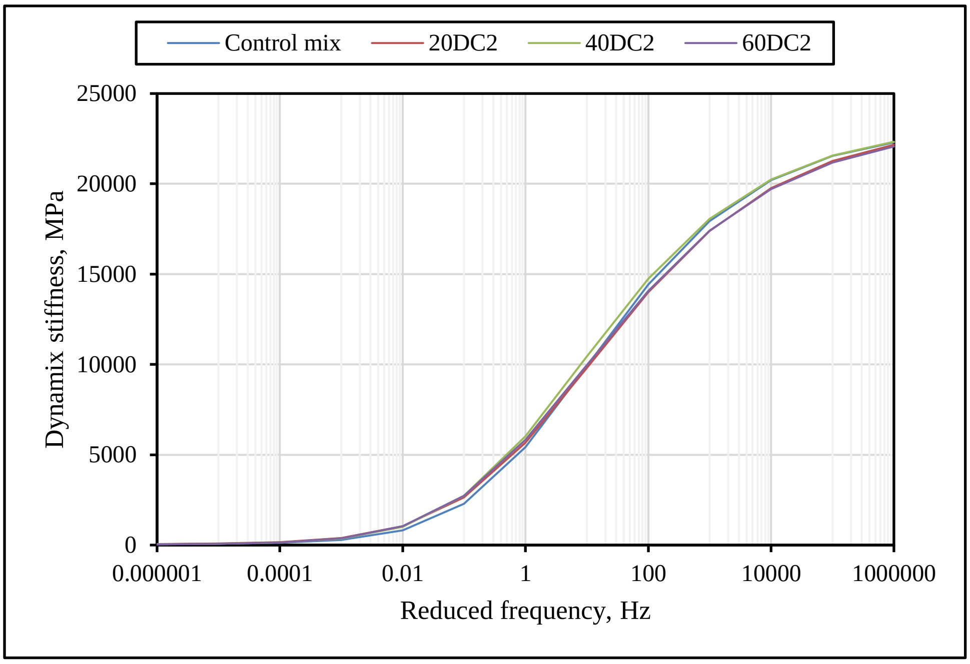


Figure 5.81: Dynamic modulus master curves of control and 20%, 40% and 60% DC2 asphalt mixtures

Table 5.37: Master curve data for asphalt mixtures containing granite aggregate and 20%, 40% and 60% DC2

The data of master curves of asphalt mixtures made granite aggregates, and 20%,				
Reduced frequency, Hz	Control mix	20DC2 mix	40DC2 mix	60DC2 mix
1000000	22290.9	22163.2	22329.4	22067.2
100000	21544.6	21264.6	21570.1	21179.3
10000	20204.9	19764.4	20246.7	19708.8
1000	17930.5	17396.8	18056.9	17399.7
100	14437.1	13997.5	14747.2	14084.8
10	9912.0	9803.3	10447.5	9962.5
1	5414.4	5670.4	6026.3	5827.5
0.1	2285.5	2640.5	2723.8	2719.6
0.01	806.9	1036.5	1008.2	1040.6
0.001	287.9	390.7	354.9	366.6
0.0001	124.5	164.9	142.5	139.7
0.00001	69.7	85.4	72.3	64.8
0.000001	48.8	54.6	46.3	37.7

According to the results, at loading frequencies lower than 10 Hz, the dynamic modulus of asphalt mixtures made with 20%, 40% or 60% DC2 were found to be higher than that obtained for control mix (0RCAs). It should, however, be noted that these mixtures tended to demonstrate lower dynamic modulus than control mix at very low loading frequencies. The results also indicate that the dynamic modulus of DC2-mixtures peaked with the addition of 40% DC2. Mixtures containing 40% DC2 showed the highest viscoelastic behaviour of all the mixtures, as shown in Table 5.37.

Under loading frequencies higher than 10 Hz, the viscoelastic performance of the 20DC2 and 60DC2 mixtures was lower than that of the control mix. It was evident that the difference in dynamic modulus between these mixtures and the control mix decreased as the loading frequency increased. For instance, at 100 Hz, 20DC2 and 60DC2 mixtures showed dynamic modulus of 13,997.5 and 14,084.8 MPa, respectively, which is 3% and 2.4% lower than that of control mix (14,437.1 MPa). However, at 1,000,000 Hz, the 40DC1 and 60DC1 mixtures showed dynamic modulus of 22,163.2, and 22,067.2 MPa respectively, which is 0.6% and 1% lower than that achieved by control mix (22,290.9 MPa). The results imply that there was a minimal change in the viscoelastic performance of asphalt mixtures made with 20% and 60% DC2.

This improvement in the dynamic modulus can be attributed to adhesion and strength enhancement after DCT2 (coating with Sika Tite-BE and heating). The DCT2 seems to strengthen the weak particles of RCAs so they display comparable and/or higher dynamic modulus than that of control mix.

5.4.3.5 ANOVA analysis of dynamic modulus results

5.4.3.5.1 Effect of DC1 on dynamic modulus and phase angle

One-way ANOVA was performed to assess the effects of DC1 and DC2 contents on dynamic modulus and phase angle. Since the dynamic modulus test was carried out at several loading frequencies and temperatures, the effect of DCRCA (i.e., DC1 and DC2), content was checked versus each combination of frequency and testing temperature. Therefore, a total of ten one-way ANOVAs were performed to examine the effects of DC1/DC2 content on dynamic modulus at different temperatures and loading frequencies. Also, a further ten one-way ANOVAs were conducted to study the effects of DC1/DC2 content on phase angle at different temperatures and frequencies.

5.4.3.5.1.1 Effect of DC1 content on dynamic modulus

The results of the ten one-way ANOVAs carried out to assess the effect of DC1 content of dynamic modulus are shown in Table 5.38. The effect of DC1 content on the viscoelastic performance of asphalt mixtures was checked at ten combinations of temperature and loading frequency. The ANOVA results reveal that the DC1 content did not significantly affect the dynamic modulus ($p > 0.05$ in most cases). It can be seen that the dosage of DC1 was only significant for the test conducted at 40 °C and 0.1 Hz ($p = 0.019$).

Table 5.38: Results of ANOVA: effect of DC1 content on dynamic modulus (DM)

Results of ANOVA: effects of DC1 content on dynamic modulus.							
#	Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
1	%DC1 vs DM*(4 °C, 0.1 Hz)	1.21E+06	3	4.0E+05	1.1	0.413	4.066
2	%DC1 vs DM (4 °C, 1 Hz)	4.47E+06	3	1.5E+06	1.8	0.219	4.066
3	%DC1 vs DM (4 °C, 10 Hz)	1.01E+07	3	3.4E+06	2.3	0.149	4.066
4	%DC1 vs DM (20 °C, 0.1 Hz)	4.52E+04	3	1.5E+04	0.7	0.603	4.066
5	%DC1 vs DM (20 °C, 1 Hz)	2.15E+05	3	7.2E+04	0.8	0.527	4.066
6	%DC1 vs DM (20 °C, 10 Hz)	1.19E+06	3	4.0E+05	1.5	0.275	4.066
7	%DC1 vs DM (40 °C, 0.01 Hz)	1.10E+03	3	3.7E+02	0.9	0.498	4.066
8	%DC1 vs DM (40 °C, 0.1 Hz)	5.51E+03	3	1.8E+03	6.0	0.019	4.066
9	%DC1 vs DM (40 °C, 1 Hz)	4.11E+04	3	1.4E+04	3.9	0.055	4.066
10	%DC1 vs DM (40 °C, 10 Hz)	1.64E+05	3	5.5E+04	1.2	0.362	4.066

The ANOVA results can be further explained by the dynamic modulus values of the DC1-mixes shown in Table 5.39. It can be seen that DC1 mixes tended to demonstrate slightly lower dynamic modulus than control mix at 4 °C and 20 °C, while they exhibited higher dynamic modulus than control mix at 40 °C. The results indicate that the addition of DC1 causes minimal change in dynamic modulus at lower and moderate temperatures. Besides, the DC1 mixtures performed better dynamic modulus than the control mix at higher temperatures (40 °C). This implies an improvement in the viscoelastic behaviour of asphalt mixtures containing DC1. In this regard, the DCT1 tends to improve the strength and durability of RCAs and enhance adhesion between DC1 and bitumen, as explained previously. As a result, an enhancement in the dynamic modulus is achieved.

Table 5.39: Dynamic modulus of asphalt mixtures made with 0% (control mix), 20%, 40% and 60% DC1

Mix type	Test conditions: (Temperature, loading frequency)									
	(4 °C, 0.1 Hz)	(4 °C, 1 Hz)	(4 °C, 10 Hz)	(20 °C, 0.1 Hz)	(20 °C, 1 Hz)	(20 °C, 10 Hz)	(40 °C, 0.01 Hz)	40 °C, 0.1 Hz)	(40 °C, 1 Hz)	(40 °C, 10 Hz)
Control mix	10818.2	15372.0	19778.3	2449.5	5188.0	9158.1	98.2	170.3	478.2	1537.8
20DC2	10753.3	15287.3	19799.7	2442.0	5180.0	9102.3	113.4	211.4	598.3	1759.7
40DC2	10671.3	14787.7	18813.7	2296.3	4889.0	8527.3	88.4	174.7	534.4	1574.0
60DC2	10023.3	13837.7	17551.0	2411.0	4951.0	8478.3	108.1	218.5	629.9	1811.7

5.4.3.5.1.2 Effect of DC1 content on phase angle

Ten one-way ANOVAs were conducted to study the effect of DC1 content on the phase angles of asphalt mixtures made with different dosages of DC1 (0%, 20%, 40% and 60%) and the results are presented in Table 5.40.

Table 5.40: Results of ANOVA: effects of DC1 content on phase angle (PA)

Results of ANOVA: Effects of DC1 content on phase angle.							
#	Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
1	%DC1 vs PA*(4 °C, 0.1 Hz)	4.48E+00	3	1.49E+00	7.6	0.010	4.066
2	%DC1 vs PA (4 °C, 1 Hz)	1.84E+00	3	6.14E-01	7.6	0.010	4.066
3	%DC1 vs PA (4 °C, 10 Hz)	6.40E-01	3	2.13E-01	4.3	0.043	4.066
4	%DC1 vs PA (20 °C, 0.1 Hz)	4.09E+00	3	1.36E+00	1.3	0.343	4.066
5	%DC1 vs PA (20 °C, 1 Hz)	4.93E+00	3	1.64E+00	3.5	0.067	4.066
6	%DC1 vs PA (20 °C, 10 Hz)	3.84E+00	3	1.28E+00	7.6	0.010	4.066
7	%DC1 vs PA (40 °C, 0.01 Hz)	5.51E+00	3	1.84E+00	0.4	0.730	4.066
8	%DC1 vs PA (40 °C, 0.1 Hz)	5.98E+00	3	1.99E+00	0.5	0.713	4.066
9	%DC1 vs PA (40 °C, 1 Hz)	8.61E+00	3	2.87E+00	1.3	0.348	4.066
10	%DC1 vs PA (40 °C, 10 Hz)	1.82E+01	3	6.08E+00	2.4	0.147	4.066

ANOVA results indicate that the inclusion of DC1 has great effect on the phase angle at 4 °C, as *p*-values were lower than 0.05, as shown in Table 5.40. Additionally, the ANOVA showed that asphalt mixtures made with DC1 exhibited comparable phase angles as those obtained for control mix at moderate and high temperatures, respectively. This is explained by the *p*-values obtained at 20 °C and 40 °C, which were higher than 0.05 in most cases.

The average phase angles of DC1 mixtures are presented in Table 5.41. As can be seen, asphalt mixtures containing DC1 demonstrated generally lower phase angles, i.e., more elastic (less viscous) performance, than control mix. However, the DC1-asphalt mixtures showed slightly higher phase angles than control mix at 40 °C and 0.01 Hz and 0.1 Hz, as shown in Table 5.41.

Table 5.41: Phase angle of asphalt mixtures made with 0% (control mix), 20%, 40% and 60% DC1

Mix type	Test conditions: (Temperature, loading frequency)									
	(4 °C, 0.1 Hz)	(4 °C, 1 Hz)	(4 °C, 10 Hz)	(20 °C, 0.1 Hz)	(20 °C, 1 Hz)	(20 °C, 10 Hz)	(40 °C, 0.01 Hz)	(40 °C, 0.1 Hz)	(40 °C, 1 Hz)	(40 °C, 10 Hz)
Control mix	17.4	12.1	8.7	35.3	28.9	21.4	21.8	30.7	37.9	40.3
20DC2	17.4	12.3	8.9	34.5	28.4	21.3	21.8	30.2	35.9	37.6
40DC2	16.2	11.4	8.3	35.0	28.5	20.9	23.0	31.8	38.1	39.8
60DC2	16.2	11.4	8.4	33.8	27.2	20.0	23.3	31.9	37.4	37.6

5.4.3.5.2 Effect of DC2 content on dynamic modulus and phase angle

5.4.3.5.2.1 Effect of DC2 content on dynamic modulus

Ten one-way ANOVAs were performed to evaluate the effects of DC2 content of the dynamic modulus of HMA. The results of these tests show that DC2 did not significantly affect the dynamic modulus value as shown in Table 5.42. This is true when the testing is carried out at low (4 °C) and moderate (20 °C) temperatures. The p-values at 40 °C, however, exhibited a different trend. ANOVA indicated that the addition of DC2 had no significant effect on the dynamic modulus measured at 40°C and 0.1 Hz or at 40 °C and 1 Hz (p = 0.017 and 0.042, respectively).

The average dynamic modulus of DC2-asphalt mixtures at each combination of temperature and loading frequency are shown in Table 5.43. The results indicate that the addition of DC2 caused minimal change in the dynamic modulus of the produced asphalt mixtures. It can be seen that the addition of DC2 produced a slightly lower dynamic modulus at low temperature but slightly higher dynamic modulus at moderate and high testing temperatures compared to the values obtained for control mix.

Table 5.42: Results of ANOVA: effects of DC2 content on dynamic modulus (DM)

Results of ANOVA: Effects of DC2 content on dynamic modulus.							
#	Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
1	%DC2 vs DM*(4 °C, 0.1 Hz)	9.0E+05	3	3.0E+05	2.1	0.185	4.066
2	%DC2 vs DM (4 °C, 1 Hz)	6.1E+05	3	2.0E+05	0.8	0.538	4.066
3	%DC2 vs DM (4 °C, 10 Hz)	2.2E+06	3	7.3E+05	2.2	0.171	4.066
4	%DC2 vs DM (20 °C, 0.1 Hz)	2.5E+05	3	8.2E+04	2.2	0.166	4.066
5	%DC2 vs DM (20 °C, 1 Hz)	3.9E+05	3	1.3E+05	1.7	0.236	4.066
6	%DC2 vs DM (20 °C, 10 Hz)	5.7E+05	3	1.9E+05	1.7	0.246	4.066
7	%DC2 vs DM (40 °C, 0.01 Hz)	1.3E+03	3	4.5E+02	3.0	0.096	4.066
8	%DC2 vs DM (40 °C, 0.1 Hz)	1.1E+04	3	3.8E+03	6.3	0.017	4.066
9	%DC2 vs DM (40 °C, 1 Hz)	1.2E+05	3	4.0E+04	4.4	0.042	4.066
10	%DC2 vs DM (40 °C, 10 Hz)	5.2E+05	3	1.7E+05	2.8	0.108	4.066

Table 5.43: Dynamic modulus of asphalt mixtures made with 0% (control mix), 20%, 40% and 60% DC2

Mix type	Test conditions (temperature, loading frequency)									
	(4 °C, 0.1 Hz)	(4 °C, 1 Hz)	(4 °C, 10 Hz)	(20 °C, 0.1 Hz)	(20 °C, 1 Hz)	(20 °C, 10 Hz)	(40 °C, 0.01 Hz)	(40 °C, 0.1 Hz)	(40 °C, 1 Hz)	(40 °C, 10 Hz)
Control mix	10,818.2	15,372.0	19,778.3	2449.5	5188.0	9114.8	98.2	170.3	478.2	1537.8
20DC2	10,736.7	14,908.7	19,038.0	2755.0	5374.0	9049.3	126.8	247.0	693.5	1917.7
40DC2	11,439.0	15,305.3	19,392.7	2796.7	5679.7	9612.7	112.5	235.0	709.3	2051.0
60DC2	10,926.0	14,871.3	18,619.3	2783.0	5514.7	9238.0	105.0	240.0	720.2	2041.0

5.4.3.5.2.2 *Effect of DC2 content on phase angle*

An additional ten one-way ANOVAs were performed to test the effect of DC2 content on the phase angles of asphalt mixtures made with different dosages of DC2 (0%, 20%, 40%, 60%). The outcomes reveal that the DC2 content greatly affected the phase angles of asphalt mixes at 4 °C and 20 °C Table 5.44. Only one exception was observed: asphalt mixtures tended to demonstrate a comparable phase angle at 20 °C and 0.1 Hz. This was verified by the *p*-value obtained, which was 0.056. The ANOVA also showed that asphalt mixtures made with DC2 exhibited comparable phase angles at high temperatures, where the *p*-values at 40 °C were > 0.05 in most cases.

Table 5.44: Results of ANOVA: effects of DC2 content on phase angle (PA)

Results of ANOVA: Effects of DC2 content on phase angle.							
#	Source of variation	SS	df	MS	*F*	*P*-value	*F* crit
1	%DC2 vs PA*(4 °C, 0.1 Hz)	8.8E+00	3	2.9E+00	13.8	0.002	4.066
2	%DC2 vs PA (4 °C, 1 Hz)	3.1E+00	3	1.0E+00	5.1	0.029	4.066
3	%DC2 vs PA (4 °C, 10 Hz)	1.5E+00	3	5.0E-01	5.0	0.030	4.066
4	%DC2 vs PA (20 °C, 0.1 Hz)	1.5E+01	3	5.1E+00	3.9	0.056	4.066
5	%DC2 vs PA (20 °C, 1 Hz)	1.5E+01	3	5.0E+00	7.2	0.011	4.066
6	%DC2 vs PA (20 °C, 10 Hz)	8.9E+00	3	3.0E+00	9.6	0.005	4.066
7	%DC2 vs PA (40 °C, 0.01 Hz)	1.6E+01	3	5.3E+00	2.3	0.150	4.066
8	%DC2 vs PA (40 °C, 0.1 Hz)	1.1E+01	3	3.6E+00	1.4	0.322	4.066
9	%DC2 vs PA (40 °C, 1 Hz)	8.5E+00	3	2.8E+00	1.5	0.290	4.066
10	%DC2 vs PA (40 °C, 10 Hz)	3.3E+01	3	1.1E+01	4.7	0.036	4.066

In order to better understand the ANOVA results, Table 5.45 shows the average phase angles of asphalt mixtures containing DC2 at each combination of temperature and loading frequency. It can be seen that DC2 mixtures had lower phase angles than control mix at low and moderate test temperatures. However, the DC2 mixes showed slightly higher phase angles than control mix at 40 °C and 0.01 Hz and 0.1 Hz, as shown in Table 5.45.

Table 5.45: Phase angles of asphalt mixtures made with 0% (control mix), 20%, 40% and 60% DC2

Mix type	Test conditions (temperature, loading frequency)									
	(4 °C, 0.1 Hz)	(4 °C, 1 Hz)	(4 °C, 10 Hz)	(20 °C, 0.1 Hz)	(20 °C, 1 Hz)	(20 °C, 10 Hz)	(40 °C, 0.01 Hz)	(40 °C, 0.1 Hz)	(40 °C, 1 Hz)	(40 °C, 10 Hz)
Control mix	17.4	12.1	8.7	35.3	28.9	21.3	21.8	30.7	37.9	40.3
20DC2	16.3	11.6	8.4	32.7	26.8	20.0	22.3	30.3	35.5	36.3
40DC2	15.3	11.0	7.8	33.4	26.9	19.6	23.7	32.0	36.8	36.8
60DC2	15.3	10.8	7.9	32.4	25.9	19.0	24.7	32.7	36.7	36.4

5.4.3.6 Comparison of DC1 and DC2 mixture dynamic modulus results

This section of the chapter aims to investigate the effect of the type of DCT used (DCT1 or DCT2) on the dynamic modulus of Australian HMA made with two types of DCRCA (DC1 and DC2). To achieve this aim, a comparison between the dynamic modulus of asphalt mixtures produced with the same dosage of DCRCA but different DCT was conducted. Thus, this section will focus on the differences in dynamic modulus between each asphalt mixture containing DC1 and their corresponding mixtures containing DC2.

Figure 5.82, Figure 5.83, and Figure 5.84 illustrate master curves for asphalt mixtures produced with the same percentage of double-coated RCA but different DCT. It can be seen that the DC2 mixes achieved better viscoelastic performance than those of their corresponding DC1 mixes. However, the master curves cross each other, as shown in Figure 5.82, or are close to each other, as presented in Figure 5.83, and Figure 5.84. Therefore, the original master curve data, along with the control mix data, are presented in order to facilitate comparison of the data in the figures. The percentage change in the dynamic modulus of asphalt mixtures due to the addition of DC1 or DC2 as compared to control mix was calculated using Equation 5.1:

$$Percentage\ change = \left(\frac{(DM\ of\ CM - DM\ of\ mixture\ made\ with\ DCRCAs)}{DM\ of\ CM}\right) \times 100 \qquad 5.1$$

Where *DM* = dynamic modulus, *CM* = control mix. If the percentage change is a negative number, then there is an increase in the dynamic modulus of asphalt mixtures compared to that of control mix and vice versa.

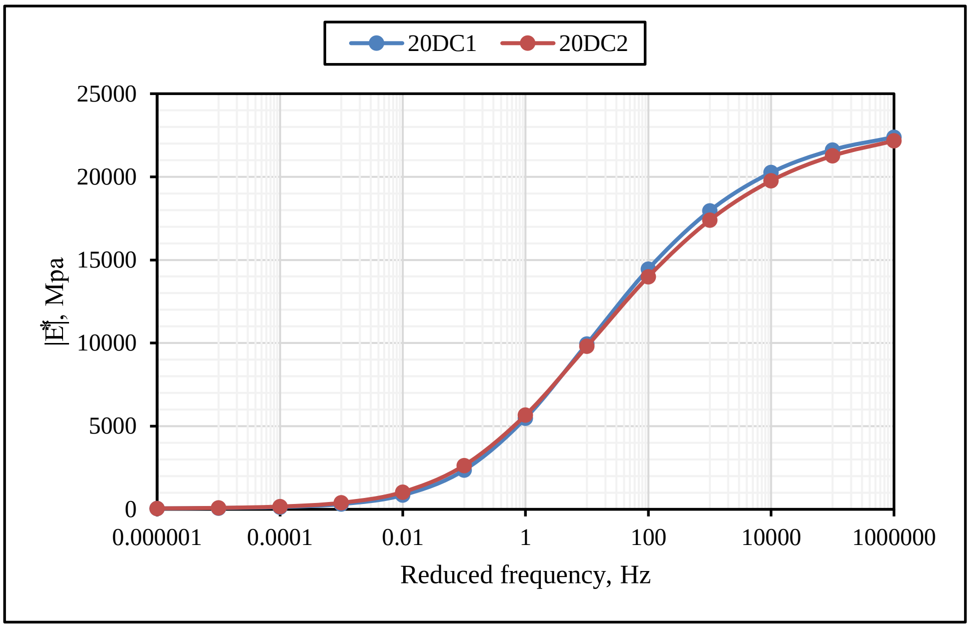


Figure 5.82: Master curves for 20DC1 and 20DC2 mixtures

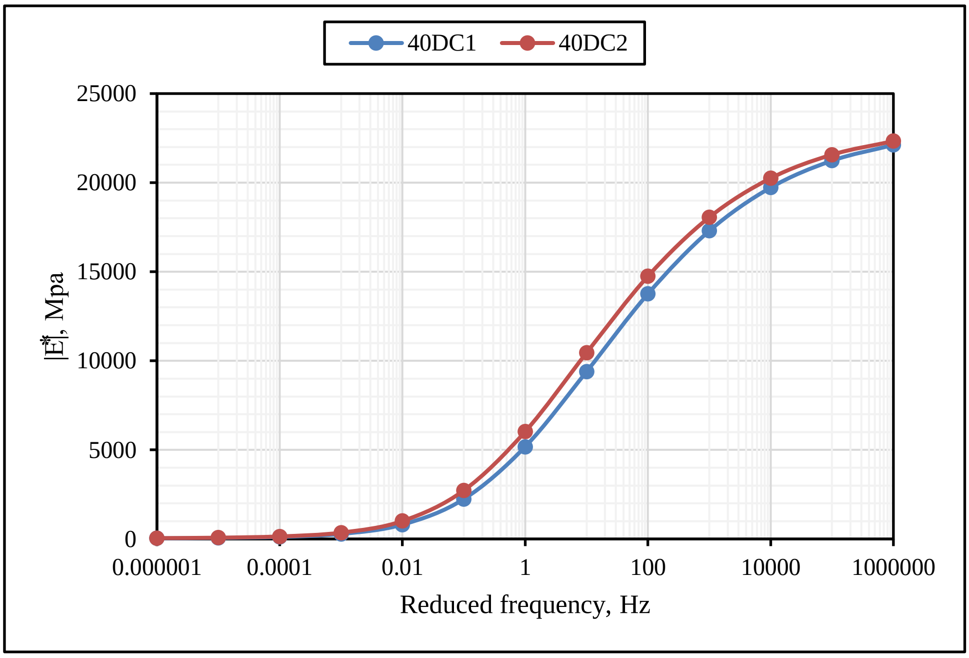


Figure 5.83: Master curves for of 40DC1 and 40DC2 mixtures

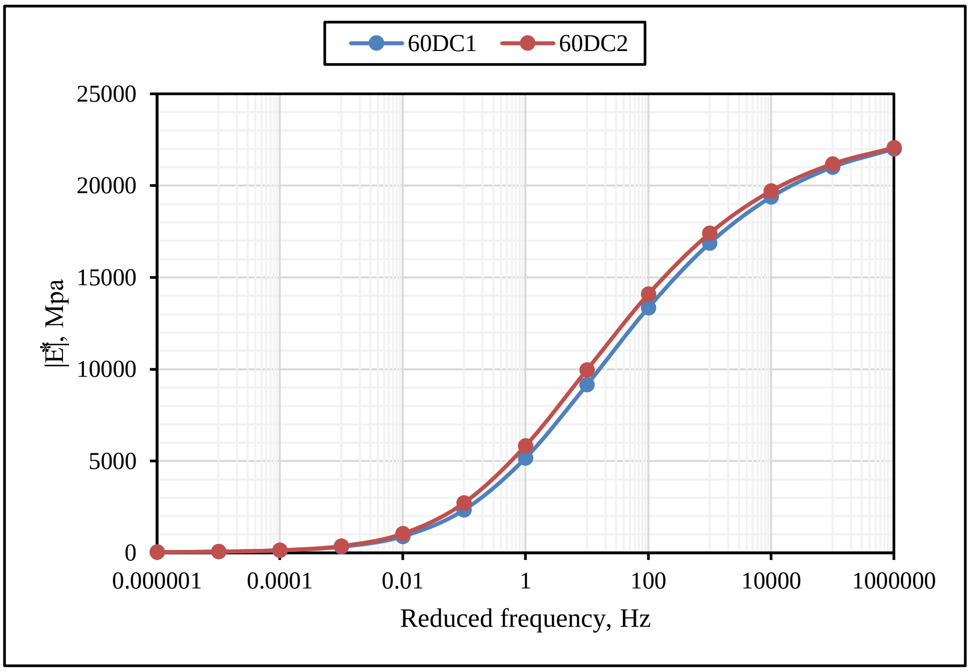


Figure 5.84: Master curves for 60DC1 and 60DC2 mixtures

Generally, asphalt mixtures made with 20%, 40% and 60% DC2 exhibited higher dynamic modulus than corresponding DC1 mixtures at all loading frequencies. However, it was noted that 20DC1 asphalt mixtures tended to demonstrate higher dynamic modulus than 20DC2 mixtures at loading frequencies higher than 10 Hz, as presented in Figure 5.82.

The percentage change in the dynamic modulus of asphalt mixtures containing DCRCAs (DC1 or DC2) was calculated according to Equation 5.1. The dynamic modulus data of asphalt mixtures made with different dosages of DC1 or DC2, in addition to their percentage changes, are presented in Table 5.46, Table 5.47 and Table 5.48, respectively. According to the results presented in these tables, Australian HMA made with double-coated RCAs behaved differently than RCA-mixes made in other parts of the world, such as USA. In this research, the addition of DCRCAs (i.e., DC1 and DC2) did not dramatically affect the viscoelastic performance of Australian HMA. Furthermore, it can be seen that asphalt mixtures made with DC2 performed better than corresponding asphalt mixtures produced with DC1, at all percentages. In most cases, the addition of DC2 into HMA considerably increased the dynamic modulus or caused a minimal decrease.

Table 5.46: The percentage change of dynamic modulus of 20DC1 and 20DC2 mixes

Reduced frequency, Hz	Dynamic modulus			Change in dynamic modulus (%)	
	Control mix	20DC1	20DC2	20DC1 mix	20DC2 mix
1000000	22,290.9	22,387.0	22,163.2	-0.4	0.6
100000	21,544.6	21,621.4	21,264.6	-0.4	1.3
10000	20,204.9	20,257.5	19,764.4	-0.3	2.2
1000	17,930.5	17,960.1	17,396.8	-0.2	3.0
100	14,437.1	14,459.0	13,997.5	-0.2	3.0
10	9,912.0	9,952.9	9,803.3	-0.4	1.1
1	5,414.4	5,483.7	5,670.4	-1.3	-4.7
0.1	2,285.5	2,355.9	2,640.5	-3.1	-15.5
0.01	806.9	853.1	1,036.5	-5.7	-28.4
0.001	287.9	312.2	390.7	-8.4	-35.7
0.0001	124.5	137.5	164.9	-10.4	-32.4
0.00001	69.7	77.7	85.4	-11.4	-22.4
0.000001	48.8	54.6	54.6	-11.8	-11.8

Table 5.47: The percentage change of dynamic modulus of 40DC1 and 40DC2 mixes

Reduced frequency, Hz	Dynamic modulus			Change in dynamic modulus (%)	
	Control mix	40DC1	40DC2	40DC1 mix	40DC2 mix
1000000	22,290.9	22,117.4	22,329.4	0.78	-0.2
100000	21,544.6	21,232.7	21,570.1	1.45	-0.1
10000	20,204.9	19,723.4	20,246.7	2.38	-0.2
1000	17,930.5	17,294.7	18,056.9	3.55	-0.7
100	14,437.1	13,759.6	14,747.2	4.69	-2.1
10	9,912.0	9,392.6	10,447.5	5.24	-5.4
1	5,414.4	5,176.9	6,026.3	4.39	-11.3
0.1	2,285.5	2,240.3	2,723.8	1.98	-19.2
0.01	806.9	807.5	1,008.2	-0.07	-24.9
0.001	287.9	283.5	354.9	1.52	-23.3
0.0001	124.5	115.3	142.5	7.42	-14.5
0.00001	69.7	59.3	72.3	14.92	-3.7
0.000001	48.8	38.3	46.3	21.43	5.2

Table 5.48: The percentage change of dynamic modulus of 60DC1 and 60DC2 mixes

Reduced frequency, Hz	Dynamic modulus			Change in dynamic modulus (%)	
	Control mix	60DC1	60DC2	60DC1 mix	60DC2 mix
1000000	22,290.9	22,000.0	22,067.2	1.3	1.0
100000	21,544.6	21,004.8	21,179.3	2.5	1.7
10000	20,204.9	19,374.6	19,708.8	4.1	2.5
1000	17,930.5	16,859.6	17,399.7	6.0	3.0
100	14,437.1	13,345.8	14,084.8	7.6	2.4
10	9,912.0	9,148.4	9,962.5	7.7	-0.5
1	5,414.4	5,159.0	5,827.5	4.7	-7.6
0.1	2,285.5	2,339.4	2,719.6	-2.4	-19.0
0.01	806.9	895.0	1,040.6	-10.9	-29.0
0.001	287.9	328.8	366.6	-14.2	-27.3
0.0001	124.5	135.0	139.7	-8.5	-12.2
0.00001	69.7	68.0	64.8	2.5	7.1
0.000001	48.8	42.4	37.7	13.1	22.8

5.4.4 Use of dynamic modulus to characterise the performance of Australian HMA made with DCRCAs

In this part of Chapter 5, an attempt was made to evaluate the relationships between dynamic modulus data and several performance characteristics of Australian HMA containing DCRCAs. Relationships between dynamic modulus and rutting, and dynamic modulus and fatigue life, are presented. These relationships help to evaluate the potential correlation between the dynamic modulus and other performance properties of Australian HMA made with double-coated RCAs. The measured and predicted results of these performance properties are then briefly compared to assess their agreement.

5.4.4.1.1 Relationship between dynamic modulus and rutting resistance of asphalt mixtures containing DCRCAs

Rutting is one of the most common distresses to asphalt pavement. In general, high traffic loadings and temperatures can lower the resistance of asphalt mixtures to rutting. In several studies, researchers have calculated the parameter |E*|/sinδ as an indicator of rutting resistance, where |E*| is the dynamic modulus and δ is the phase angle. In this regard, M. W. Witczak (2002) recommended measuring the rutting indicator parameter (|E*|/sinδ) at 5 Hz and 37.8 °C (100 °F) and 54.4 °C (130 °F). In order to simulate the worst in situ conditions of asphalt pavement, Shaopeng Wu et al. (2007) suggested calculating the rutting parameter at 10 Hz and 54.4 °C. Furthermore, Zhou, Chen, Scullion, and Bilyeu (2003) used |E*|/sinδ as an indicator of rutting resistance. Zhou et al. (2003) concluded that the rutting parameter at 40 °C and 10 Hz can be used to fairly rank the rutting potential of asphalt mixtures. Asphalt mixtures that exhibit a higher dynamic modulus and lower phase angle (δ) are expected to be less sensitive to rutting.

In the current investigation, the rutting indicator parameter (|E*|/sinδ) was calculated for all asphalt mixtures at 40 °C and 10 Hz to assess the potential of asphalt mixtures made with DC1 and DC2 to rutting. The rutting indicator results for asphalt mixtures made with DC1 and DC2 and for control mix are presented in Figure 5.85 and Figure 5.86 respectively. Higher values of |E*|/sinδ indicate better resistance to permanent deformation. The rutting parameter figures reveal that the control mix was more sensitive to rutting than those produced with DC1 and DC2. It can also be seen that asphalt mixtures made with DC2, in general, demonstrated higher |E*|/sinδ values than corresponding asphalt mixtures containing DC1. Furthermore, the rutting indicator results reveal that the addition of both types of DCRCA enhanced the rutting performance of HMA. These findings agree well with those obtained from FN testing, as introduced in Section 5.4.1.2.

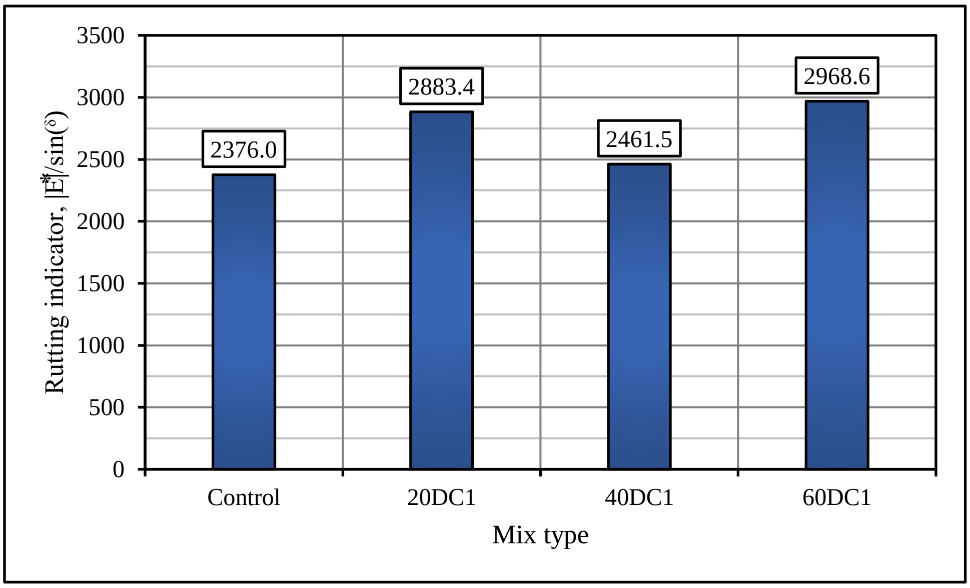


Figure 5.85: Rutting indicators for asphalt mixtures made with granite aggregate and DC1

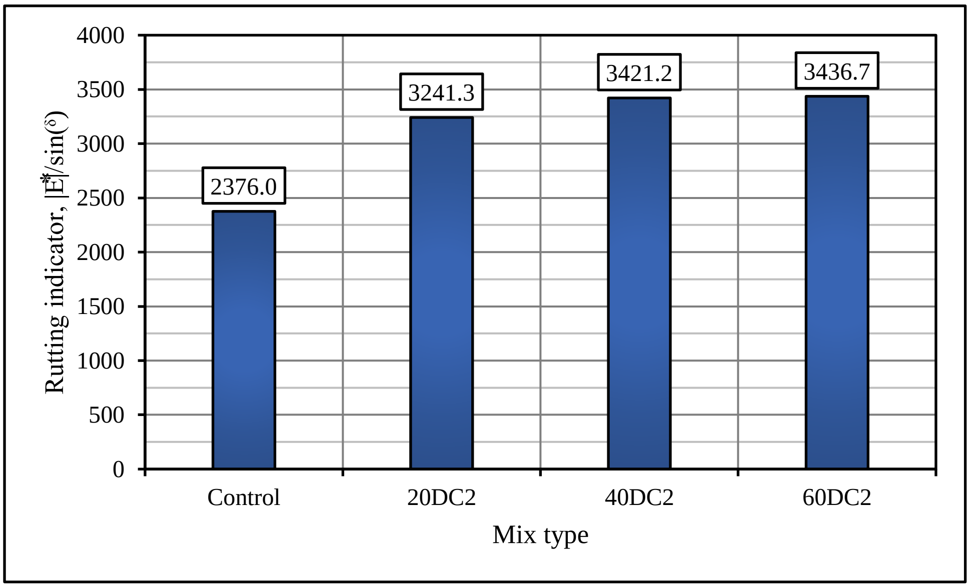


Figure 5.86: Rutting indicators for asphalt mixtures made with granite aggregate and DC2

5.4.4.1.2 *Relationship between dynamic modulus and fatigue life for asphalt mixtures made with DCRCAs*

According to the National Corporative Highway Research Program (NCHRP) project 9-19, the dynamic modulus measured at an intermediate temperature has a strong correlation to fatigue cracking (Andrei, Witczak, & Mirza, 1999). A study by Shaopeng Wu et al. (2007) investigated the effects of different fibres on the dynamic modulus characteristics of asphalt mixtures. In that study, a fatigue parameter, calculated as $|E^*| * \sin(\delta)$, was used to assess the potential of asphalt mixtures to fatigue cracking, where $|E^*|$ is the dynamic modulus and the δ is the phase angle. The researchers mentioned that the fatigue parameter can be considered as a valid parameter for evaluating the resistance of asphalt mixtures to the initiation and propagation of fatigue cracking. The lower the fatigue parameter, the higher resistance to fatigue cracking (Shaopeng Wu et al., 2007).

In the current research, fatigue life testing was carried out at 20 °C and a 10 Hz loading frequency. The fatigue parameters of asphalt mixtures containing 0%, 20%, 40% and 60% DC1 or DC2 are graphically shown in Figure 5.87 and Figure 5.88 respectively. It was confirmed that the inclusion of DC1 or DC2 in HMA can improve fatigue life compared to control mix. Moreover, the DC1 mixtures generally demonstrated lower fatigue parameter values than those of corresponding DC2 asphalt mixtures. There was only one exception: the 20DC2 mix showed a lower fatigue parameter value than 20DC2 mix. This, however, may be explained by the heterogeneity of the RCAs used in this research. The fatigue parameter results, in general, agreed well with the four-point bending fatigue life test results presented in Section 5.4.2.

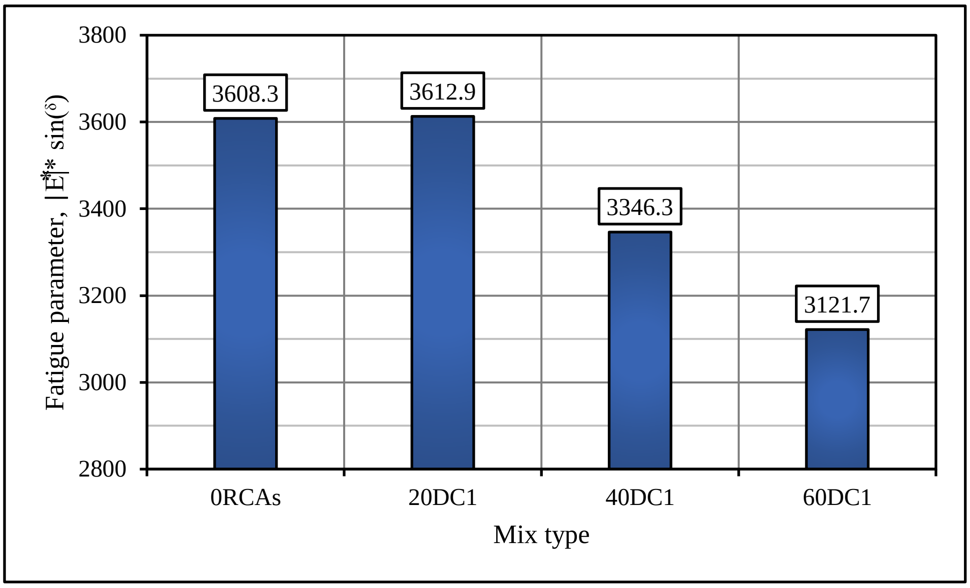


Figure 5.87: Fatigue parameters of asphalt mixtures made with granite aggregates or DC1

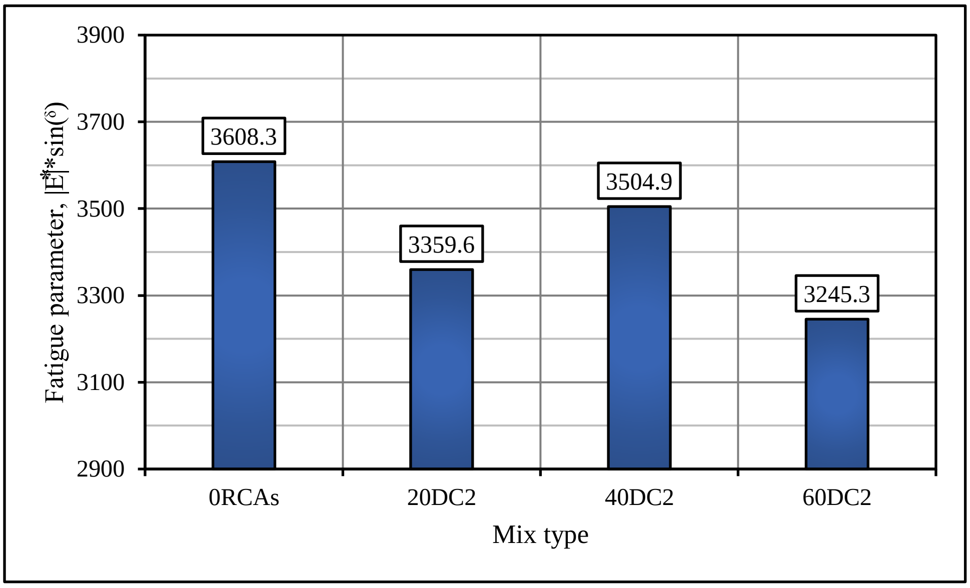


Figure 5.88: Fatigue parameters of asphalt mixtures made with granite aggregates or DC2

6 Conclusions and recommendations

6.1 General introduction

This thesis focused on development and laboratory assessment of two state-of-the-art double-coating techniques (DCTs) for RCAs used in asphalt mixtures. The two DCTs were developed in order to improve the performance of RCA-asphalt mixtures over that achieved previously using coating-based and heating-based treatments. The DCTs were designed to avoid the degradation that can occur after some treatment processes and/or remedy the RCA defects remaining after a first treatment by using a suitable second treatment. This chapter summarises the key findings at three separate phases of the DCTs: pre-development, development and assessment. In the present study, type AC14 asphalt mixtures were evaluated through a detailed experimental program. Marshal tests, indirect tensile tests, TSR tests, ITSM tests, wheel tracking tests, FN tests, four-point bending fatigue life tests and dynamic modulus tests were carried out to assess the performance of asphalt mixtures made with two types of double-coated recycled concrete aggregates (i.e., DC1 and DC2). In light of the results collected and analysed, the following conclusions and recommendations are made.

6.2 Conclusions

Based on the objectives set out for this thesis and the outcomes of the literature review and experiments, the following conclusions are drawn.

6.2.1 Conclusions of the DCT pre-development phase

In accordance with the literature review, the addition of RCAs can give asphalt mixtures inferior mechanical characteristics to those of mixes made with 100% natural aggregates. It was, therefore, suggested to use several treatments to upgrade the quality of asphalt mixtures containing RCAs, either through coating or heating procedures.

Disappointingly, it was found that the treatments improved some properties of the mixes while degrading others. For instance, although cement slag paste (CSP) and organic silicone resin (OSR) treatments are reported to enhance the abrasion resistance of coated RCAs, they increase the water and bitumen absorption rate of the final product. Additionally, curing RCA-bituminous mixes in an oven at 170 °C for 4 hours has been stated to limit the usage of asphalt mixes to hot regions. In addition, the latter treatment is documented to have a detrimental impact on the engineering properties of bitumen due to thermal and oxidative effects. It is, therefore, important to cure RCA-bituminous mixes for a period of less than 4 hours to ensure that the heating process does not affect the RCA mixes engineering properties. In keeping with this, none of the previously suggested treatments has yet been adopted for HMA production. This implies that the development of new treatments for the RCAs used in HMA is still required. Therefore, a new technique is needed to enhance the performance of asphalt mixtures made with low-quality RCAs, reduce the flow of solid RCA waste within society, and support sustainable practices.

6.2.2 Conclusions of the DCT development phase

Two DCTs (i.e., DCT1 and DCT2) were developed based on special research hypotheses and criteria. DCT1 is a combination of CSP treatment and Sika Tite-BE treatment. The second part of DCT1 (Sika Tite-BE treatment) was applied to mitigate the absorptive nature of RCAs after the CSP treatment. DCT2 was based on Sika Tite-BE coating treatment and heating treatment. The aim of the heating treatment was to seal any uncoated areas remaining after the Sika Tite-BE treatment. In order to maintain the practicality of the DCTs, the following coating and heating criteria were also set out:

a) The mixture of Sika Tite-BE and water prepared for coating should be only sufficient to shield the coarse RCA particles, no more, no less. According to the Sika Tite-BE technical data sheet, the Sika Tite-BE has to be applied in two successive coats. The percentage of Sika Tite-BE added to the mixture used for the first coat should be less than that added to the mixture for the second coat. The mixture prepared for the first coat is intended to penetrate deep into small pores and cracks, while the second coat is expected to seal any remaining pores and cracks.

b) The four-hour heat treatment was documented to affect HMA and bitumen properties. In order to achieve the objectives of the DCT2 heat treatment without affecting the RCA and asphalt mixture properties, the heating time needed to be reduced compared to that used previously (i.e., the 4 hours). In this study, the RCA mixtures were allowed to cure in an oven maintained at 155 °C for only 1½ hours. Bitumen at 155 °C is expected to be less viscous and flow through the uncoated areas present onto RCA surfaces.

Based on the results obtained during the development of DCT1 and DCT2, it was found that both DCTs can improve the strength of RCAs and reduce their permeability. After DCT1 was applied, the water absorption rate of DC1 was 12.3% and 26.1% lower than those of uncoated RCAs and CSP-coated RCAs, respectively. In addition, the ACV was 6.4% lower than that of uncoated RCAs. According to the results for Sika Tite-BE-coated RCAs (the first treatment of DCT2), such coating improves the ACV and water absorption characteristics of RCAs. It was evident that the ACV and water absorption rate of RCAs coated with Sika Tite-BE were 10.2% and 23.2%, respectively, lower than that measured for untreated RCAs. It is expected that the improvements in strength and durability are due to the RCA pores and cracks being sealed more efficiently than that offered by previous treatments.

6.2.3 Conclusions of DCT assessment phase

Assessment of the DCTs was achieved in three successive stages. In each stage, different standard tests were carried out to evaluate asphalt mixtures made with 100% granite aggregate, various contents (20%, 40%, and 60%) of two types of DCRCAs (DC1 and DC2), and various contents of RCAs. In stage 1, Marshall parameters were investigated for a total of 10 asphalt mixtures. These mixtures were the 0RCAs (control mix), 20DC1, 40DC2, 60DC1, 20DC2, 40DC2, 60DC2, 20RCAs, 40RCAs and 60RCAs mixes. In stage 2, ITS, TSR and ITSM tests were performed on eight asphalt mixtures. The mixtures evaluated in stage 2 were the control mix (0RCAs), 20DC1, 40DC2, 60DC1, 20DC2, 40DC2, 60DC2 and 40RCAs. The last mix was made for comparison purposes. In stage 3, performance testing was carried out to assess rutting, fatigue and the dynamic modulus of asphalt mixtures made with 100% granite and 20%, 40% and 60% DC1 or DC2.

6.2.3.1 Stage 1: Marshall parameters of asphalt mixtures made with DCRCAs

During this stage of the experiments, ten asphalt mixtures were designed and evaluated, as mentioned in the section above. Based on the results collected and analysed, the following conclusion can be drawn:

- Asphalt mixtures made with DC1 and DC2 tend to demonstrate higher stability than those produced with granite and RCAs. This suggests that there is an adhesion enhancement in the case of mixtures made with DC1 and DC2. The mean stability results for DC2 mixtures were higher than those for the corresponding DC1-asphalt mixtures. The Sika Tite-BE treatment of DCT1 seemingly decreases the ability of RCAs to mobilize high friction between coated aggregates particles during testing. Consequently, lower stability was achieved by the DC1 mixes.

- Both the DC1 mixes and DC2 mixes exhibited lower bulk and maximum densities than RCA samples. This is considered to be related to the densities of CSP and Sika Tite-BE, which are lower than the density of RCA.
- Asphalt mixtures containing DC1 and DC2 had lower contents of absorbed bitumen (b, %) than that measured for RCA mixes. As a result, asphalt mixtures containing DC1 and DC2 require less bitumen to reach the OBC level. This implies that the cost of the materials used in the DCTs (DCT1 and DCT2), such as cement, slag and Sika Tite-BE, can be compensated for by the lower OBC of the DC1 and DC2 mixes.
- The more DC1 or DC2 is added to a mix, the lower the VMA/VFB percentage obtained. Also, DC2 mixes demonstrated, in general, higher percentages of VMA and VFB compared with RCA mixes and DC1 mixes. The DC1-mixes, however, showed lower VMA and VFB values than corresponding RCA mixes. These findings can be explained by the b (%) and (Be (%) results of asphalt mixtures made with DC1, DC2 and RCAs. When more bitumen is absorbed by the aggregates, the VMA and VFB percentages are lower.

6.2.3.2 Stage 2: ITS, TSR and ITSM results

During stage 2 of the experiments, eight asphalt mixtures were evaluated, as detailed in Section 6.2.3. Giving the results and analyses of each test conducted in this stage, conclusions were made and are discussed in the following sub-sections.

6.2.3.2.1 Indirect tensile strength of asphalt mixtures

In light of the results obtained from ITS testing, the following points can be concluded:

- The DC2 mixtures always demonstrated higher ITS than corresponding DC1 mixtures. The combined treatment of Sika Tite-BE and heating seemingly led to increased adhesion between DC2 and bitumen and, therefore, higher ITS was achieved in case of DC2 asphalt samples. This phenomenon was true at different air void contents (5% and 8%) and testing temperatures (25 °C and 40 °C).
- Also, the 40DC1 mix exhibited lower tensile strength than the 40RCAs mix. This proves that the Sika Tite-BE treatment decreases friction among DC1 aggregate particles.

6.2.3.2.2 Moisture sensitivity of asphalt mixtures

In the view of the TSR tests performed in this investigation, the following conclusions can be drawn:

- Both DCTs (DCT1 and DCT2) helped decrease the gap between the dry and wet ITSs of asphalt samples.
- After CSP coating, the pores present on RCA surfaces were sealed and relatively smooth surface texture was produced. Then, the smoothness of RCA surfaces further increases after Sika Tite-BE coating treatment. As a result, The DC1 mixes always exhibited lower ITS in the dry and wet states than that of corresponding mixes made with DC2.
- The DC1 mixes exhibited higher TSRs than equivalent DC2 mixes. This is expected to be related to a combined effect of CSP and Sika Tite-BE treatment. The CSP coating helps to achieve smother RCA surfaces, while the Sika Tite-BE coating lubricates these surfaces, thus, significantly lower friction forces were recorded (i.e. lower TSDs) in the case of DC1-asphalt samples.

- Interestingly, the ANOVA analysis showed that the DC1 and DC2 contents did not dramatically affect the TSRs of asphalt mixtures. Moreover, the state conditions (dry or wet) had no significant effect on the ITS of asphalt mixtures containing DC1 and DC2. These findings imply that both DCTs (DCT1 and DCT2) increase the durability of asphalt mixtures, regardless of the dosage of DC1 or DC2 in the mix.

6.2.3.2.3 Resilient modulus of asphalt mixtures

Giving the results of ITSM testing presented in this thesis, the following key conclusions can be made:

- Asphalt mixtures produced with DC2 always exhibited better resilient modulus than mixes made with DC2. This can be explained by the effects of the coating and heating treatments used in DCT2. The results indicate the significance of these treatments in increasing the strength of DC2 and improving the adhesion between DC2 and bitumen.
- At 25 °C, the 40DC1 mix showed lower stiffness than the 40RCAs mix, while the 40DC2 mix exhibited a higher resilient modulus than the 40RCAs mix. However, both mixes (40DC1 and 40DC2) showed higher resilient modulus than the 40RCAs mix at the higher test temperature (40 °C). In this regard, the stiffness improvement of DC1 mixes at 40 °C warrants further investigation.
- DC2 mixes showed stiffness improvement of up to 40% with the addition of DC2 at moderate (25 °C) and high temperatures (40 °C) when compared with control mix. However, DC1 mixes did not show any stiffness improvement at lower and moderate temperatures and only displayed better resilient modulus than control mix at higher temperatures (40 °C).

6.2.3.3 Stage 3: Performance testing of DCRCA-asphalt mixtures

The asphalt mixtures evaluated during the third stage were the control mix, 20DC1, 40DC2, 60DC1, 20DC2, 40DC2 and 60DC2. Asphalt mixtures containing RCAs were excluded from testing in this stage. This was due to the greater amounts of material and effort required to make asphalt mixture samples for performance testing, and the considerable time and effort needed to fabricate DCRCAs (DC1 and DC2). According to the results and analysis, the conclusions shown in the following sub-sections were made.

6.2.3.3.1 Rutting performance of asphalt mixtures containing DCRCAs

6.2.3.3.1.1 Wheel tracking tests

- All asphalt mixtures passed the criteria for superior rutting performance based on accepted Australian practices. This is verified by the values of final rut depth of asphalt mixtures which was less than 3.5 mm in all cases.
- The addition of DC1 to asphalt mixtures resulted in higher rut depths compared to that of the control mix. However, the addition of 20% or 40% DC2 improved the resistance of DC2 mixes to rutting.
- The results of SSTR revealed that the tangential slopes of the rutting curves of DC2 mixes were always lower than that of control mix but were higher in the case of DC1 mixes.
- The results of wheel tracking tests imply that the sheer resistances of DC2 mixes with up to 40% replacement of granite aggregate were better than those of DC1 and control mixes. This finding demonstrates that the use of DCT2 enhanced the rutting behaviour in comparison with control and DC1 mixtures.

6.2.3.3.1.2 Flow number testing

- The DC2 mixes showed, in general, better FN values than those of DC1 and control mixes.
- All asphalt mixes made DC1 and DC2 exhibited better FN than control mix. There was only one exception: the asphalt mixture made with 20% DC1 showed a lower FN than the control mix.
- The greater the DCRCA (DC1 or DC2) content in the mix, the higher the resistance to permanent deformation. FN testing can describe the rutting resistance of asphalt mixtures at high temperatures. The results revealed that the addition of DC1 or DC2 into asphalt mixtures can improve the high-temperature performance of asphalt mixtures relative to that of control mix (0RCAs).

6.2.3.3.2 Fatigue life of asphalt mixtures produced with DCRCAs

In light of the fatigue life testing results and analysis, the following conclusions can be drawn:

- Asphalt mixtures made with various dosages of DC1 or DC2 demonstrated higher initial and termination flexural stiffness, and lower initial and termination phase angle, compared to those of control mix. These results indicate that the mixes with DC1 and DC2 provided more elastic behaviour (were less viscous) than control mix.
- Addition of DC1 and DC2 improves the fatigue life of asphalt mixtures. According to the results of fatigue testing, the greater the DCRCA content (DC1 or DC2), the higher the number of load cycles to failure. The fatigue cracking resistance of these mixtures was ranked as follows (from high to low):

mixes with 60% DCRCA (DC1 or DC2), 40% DCRCA, 20% DCRCAs and, finally, control mix.

- The findings imply that DC1 and DC2 mixes are more capable of withstanding fatigue cracking than control mix. This indicates that both DCTs had significant effects in increasing the fatigue life of asphalt mixtures made with low-quality RCAs.
- Asphalt mixtures made with DC1 offered higher resistance to fatigue cracking than corresponding DC2 mixes. The numbers of load cycles required to halve the initial stiffness in DC1-asphalt mixtures were 22%, 72% and 197% higher than those of control mix. In keeping with this, the numbers of load cycles to failure for DC2-asphalt mixtures were 12%, 23% and 141% higher than those of control mix. These results verify that Australian asphalt mixtures made with upgraded low-quality RCAs behave differently to RCA-asphalt mixtures made in other regions of the world, such as the USA.

6.2.3.3.3 Dynamic modulus of asphalt mixtures made with DCRCAs

According to the results of the dynamic modulus tests, the following conclusion can be made:

- The inclusion of DC1 or DC2 in Australian HMA causes minimal decreases in the dynamic modulus of asphalt mixtures. In this regard, the DCRCA-asphalt mixtures demonstrated viscoelastic performance comparable to that of control mix.
- The use of DCT2 led to better dynamic modulus than those achieved using DCT1. In most cases, the addition of DC2 aggregates increased the dynamic modulus or caused minimal decreases.

- The DC1 and DC2 mixes exhibited lower phase angles than control mix at most testing temperatures and loading frequencies.
- The findings presented in this thesis imply that Australian HMA made with low-quality RCAs behaves quite differently than RCA-asphalt mixtures from other regions of the world such as the USA.

6.4 Recommendations for future work

The work presented in this thesis provides pavement engineers in Australia and other parts of the world a first look at the performance of Australian hot-mix asphalt containing RCAs. Although the use of recycled aggregates in asphalt mixtures in Australia is not yet permitted, the results presented here are auspicious. It is evident that the behaviour of Australian asphalt mixtures made with double-coated RCAs is satisfactory. The results indicate that double-coated recycled concrete aggregates (DCRCAs) can potentially be used to replace fresh granite aggregates in HMA production. In light of the results and analyses achieved in this thesis, several recommendations for future research in this area are provided.

1. The RCA used in this study was sourced from Capital Recycling, one of the leading suppliers in the Perth region. This RCA is derived from C&D waste and, thus, is considered low quality. Investigation of high-quality RCAs that only contain crushed cement concrete waste is recommended for future studies.
2. The use of fine RCA as a partial replacement for fine granite aggregate was not considered in this thesis because of its high water absorption rate compared to that of coarse RCAs, which is expected to negatively affect bitumen absorption and mix durability. However, in view of the encouraging results obtained for asphalt mixtures made with coarse RCAs, the use of fine RCAs may be worth exploring in future investigations.

3. In this study, the low-temperature properties of asphalt mixtures made with DCRCAs were not investigated. This is due to the assumption that Australia does not have many low-temperature areas, except for some areas in Victoria. In keeping with this, there is no specific testing protocol for evaluating thermal cracking in Austroads technical report AP-T100/08 *Testing Asphalt in Accordance with the Austroads Mix Design Procedures* (Alderson, 2008). In future work, it would be useful to investigate low-temperature thermal cracking to obtain more comprehensive knowledge of all the performance indexes of Australian asphalt mixtures made with DCRCAs.
4. The addition of DCRCAs in amounts higher than 60% (e.g., 80% and 100%) could be investigated in future studies. This would be the most practical way to produce more sustainable asphalt mixtures that contain higher quantities of RCAs.
5. Field evaluation of Australian asphalt mixtures containing DCRCAs or uncoated RCAs is imperative for future research. Results obtained from field and laboratory investigations should be compared to check for discrepancies.

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Appendix A: Attribution of Authorship

Paper "**Evaluation of the double coated recycled concrete aggregates for hot mix asphalt"** Construction and Building Materials, 2018, 172, 544-552.

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<table>
<tr><td></td><td>Background /literature</td><td>experimental design/data collection</td><td>data management/ data analysis</td><td>Interpretation/ discussion</td><td>Proofread</td><td>Final approval</td></tr>
<tr><td>Professor Hamid Nikraz</td><td></td><td></td><td></td><td></td><td>✔</td><td>✔</td></tr>
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Signature: 3/09/19</td></tr>
<tr><td>Dr. Hossein Asadi</td><td></td><td></td><td></td><td></td><td>✔</td><td></td></tr>
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Paper "**Performance of hot-mix asphalt produced with double coated recycled concrete aggregates**" Construction and Building Materials, 2019, 205, 425-433.

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Professor Hamid Nikraz					✔	✔
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Dr. Hossein Asadi					✔	
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Paper "**Development of a double coating technique for recycled concrete aggregates used in hot-mix asphalt"** World Academy of Science, Engineering and Technology, International Journal of Urban and Civil Engineering, 2018, 12(6).

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Conference Paper "**Development of a double coating technique for recycled concrete aggregates used in hot-mix asphalt**" Paper presented at the 20th International Conference on Pavement Engineering and Infrastructure. June, 25-26, 2018. Paris, France.

Authors and full affiliations: **Abbaas Inaayah Kareem**[1a], and Hamid Nikraz[2]

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Conference Paper "**The effects of using double coated recycled concrete aggregates on volumetrics of hot-mix asphalt**" Paper presented at the One Curtin International Postgraduate Conference. November, 26-28, 2018. Miri, Malaysia.

Authors and full affiliations: **Abbaas Inaayah Kareem**[1a], and Hamid Nikraz[2]

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Book Chapter "**Recycled aggregates (RAs) for asphalt materials**" Advances in construction and demolition waste: Management, processing and environmental assessment. Woodhead Publishing Limited-Elsevier, 2019. **Accepted**.

Authors and full affiliations: **Abbaas Inaayah Kareem**[1a], and Hamid Nikraz[2]

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Author: Abbaas I. Kareem,Hamid Nikraz,Hossein Asadi

Publication: Construction and Building Materials

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