



**University of
Nottingham**

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AIRFIELD PAVEMENT DESIGN WITH COLD RECYCLED MATERIALS

By

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Doctor of Philosophy

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*I am among those who think that
science has great beauty.*

Marie Skłodowska-Curie

ABSTRACT

The UK has adopted the concept of sustainable development and the construction industry is playing a key role in improving the efficient use of materials. The aim is to minimise the waste generated and maximise quantities of materials reused or recycled, minimising raw material consumption.

Using Reclaimed asphalt pavement (RAP) is a rehabilitation technique which involves recycling materials from asphalt layers that have already been in service. This reduces the use of new bitumen and aggregates and avoids disposal. However, UK pavements constructed prior to 1980 or surfaced in the late 1980's may contain tar, a carcinogenic substance that cannot be reheated and, therefore, cannot be recycled into hot mix asphalt (HMA). Recycling these pavements into unbound materials is also prohibited; consequently, disposal or cold recycling are the two available options.

Cold recycling of asphalt is a proven technique that reduces material disposal and raw material and energy consumption. The reduction in energy consumption is largely achieved by avoiding aggregate drying and mixing of the material at ambient temperature. In this sense, using cold recycled bound materials (CRBMs) becomes the most economic and sustainable option. However, despite the increasingly common use of CRBM in roads, the specifications for the use of these materials in airfields are under-developed and there is no guidance to ensure that pavement design with these materials is trustworthy. This is the reason why this Thesis focuses on airfield pavement design with CRBM.

The aim of this investigation is to develop a design methodology to use CRBMs in airfield pavements. For this purpose, the objectives were to review past experience on performance of these materials, measure and analyse the effect of key variables on performance to establish material limitations and develop a design methodology, proposing design guidance for airport authorities and practitioners.

To achieve the project aims and objectives, a literature review was carried out focusing on pavement engineering, airfield pavement design and CRBM. The objective was to gain sufficient knowledge on key areas to conduct the research.

Based on this literature review it was decided to use foamed bitumen as the cold technology and Kenlayer as pavement analytical design software. It was also found that the current design methodology for using CRBM in airfields is to conservatively equate material properties to those of a HMA commonly used in airfield base course (HDM50). Therefore, this practise should be analysed to decide if it is correct or if it can be improved.

Subsequently, a laboratory programme was established to analyse CRBM mechanical properties and, therefore, understand the material's behaviour and performance under cyclic loading. RAP, fly ash, cement and foamed bitumen were used to manufacture laboratory specimens, compacted with a gyratory compactor. These specimens were tested to analyse densities, air voids, stiffness, strength, permanent deformation and fatigue.

For developing a new design methodology, Kenlayer was used to analyse strains and stresses within the airfield pavement. The first step was to ascertain Kenlayer adequacy and establish inputs related to loading, traffic and subgrade condition. For this purpose, 96 case studies were analysed with HMA, with different aircraft types, traffic and subgrade conditions. These cases were compared to those of a well-established airfield design guide, namely DMG 27. Then the software could be used to model pavements containing CRBM and with the knowledge gained in the laboratory about its behaviour, establish layer thicknesses to bear traffic during the pavement design life.

With the results obtained from the laboratory investigation it was concluded that CRBM mixes have acceptable properties for use in airfield pavements.

Resistance to permanent deformation, fatigue, temperature susceptibility and durability results show that these materials give reasonable performance; however, they differ from conventional hot mixes. Thus, current practice can be improved, justifying the need for design guidance for using CRBM in airfields.

As fatigue is one of the main failure modes in asphalt mixtures and flexible pavements, a deeper study into fatigue behaviour of CRBM was carried out using Indirect Tensile Fatigue Tests (ITFT) in strain control mode and Wheel Track Test (WTT). The results showed different failure mechanisms for CRBM from those of HMA; thus, a new failure criterion was established. In HMA the failure criterion of 50% stiffness reduction is related to the appearance of macro cracks. CRBM develops dispersed micro-cracking that lowers the mixture stiffness without producing macro cracks until late in the material's life. Macro cracks only tend to appear at 70% stiffness reduction; therefore, this was established as new failure criterion for CRBM.

Once CRBM properties were defined, the pavement structure could be modelled. The results obtained from HMA modelling showed that the software and the inputs selected were appropriate for this investigation. Then the HMA base properties were substituted with CRBM properties obtained in the laboratory. The results showed that DMG 27, Chart 7, can be used for designing airfield pavements using CRBM increasing the base thickness by 9%, with a minimum Dry Lean Concrete (DLC) layer of 150 mm.

A deterioration analysis was also carried out with the design software. In this case the aim was to analyse how strains distribute within the CRBM layer and how this affects the pavement life. With these analysis, it was highlighted how different CRBM behaves compared to HMA. Strains distributed linearly within the HMA layer; however, this does not happen with the CRBM. Moreover, this analysis showed how fatigue data can be used to obtain a more accurate pavement life taking into account different strain levels.

Nevertheless, the study carried out here is based on laboratory performance of one type of CRBM. There is need for further investigation to establish a relationship between fatigue behaviour in the laboratory and the field and confirm how micro cracking affects the bearing capacity of the CRBM layer, establishing shift factors to optimise CRBM layer thickness. Moreover, the laboratory study has been carried out analysing CRBM in the same way as HMA; therefore, further study is needed to analyse the adequacy of the testing methodology. Also, modelling has been done comparing one CRBM to one HMA, namely HDM50; therefore, further investigation is needed to open the model to other HMA. Consequently, the design guidance presented here is a first step towards an airfield pavement design guide and further study is needed to optimise it.

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DECLARATION

The research reported in this Thesis was conducted at the University of Nottingham, Department of Civil Engineering, Nottingham Transportation Engineering Centre (NTEC), between July 2014 and July 2017.

I declare that the work is my own and has not been submitted for a degree at another university.

Helena Isabel Lacalle Jiménez

The University of Nottingham.

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SECTION 1. INTRODUCTION

This Thesis presents research carried out as part of the SUP&R ITN project. This project aims to form a new generation of multidisciplinary researchers capable of conceiving, planning and executing sustainable road and railway infrastructure (SUP&R ITN, 2003). There are 12 Early Stage Researchers (ESR) and 3 experienced researchers comprising the SUP&R ITN. The author is one of the ESRs and she is developing her individual research in sustainable pavements with the project “ESR1: Pavement design with cold recycled materials”.

SCOPE AND BACKGROUND TO THE PROBLEM

The requirement of using recycled asphalt planings back into new asphalt is becoming increasingly important. The two main reasons for this are the economic and environmental benefits (Copeland, 2011). These benefits include reducing demand on finite resources, reducing the generation of waste materials and reducing embodied energy (CIRIA, 1999).

Pavement constructions in the United Kingdom may contain tar. Tar is similar in appearance to bitumen and was commonly used in asphalt layers until the mid-1970 and surface dressing works until the mid-1980. Tar contains Polycyclic Aromatic Hydrocarbons (PAH) and it is potentially harmful to the environment. In addition, PAHs are known to have carcinogenic effects and levels of these are very high in the case of tar but extremely low in bitumen. For waste disposal purposes, tar is classified as “Hazardous Waste” by the European Directives which were implemented in the UK by The Hazardous Waste Regulations 2005 (HWR) (County Surveyors Society, 2008).

Given the health and environmental concerns, pavements containing PAH present limitations for recycling:

- Bituminous materials with PAH over 25mg/kg shall not be recycled into hot mix asphalt (County Surveyors Society, 2008).

- Bituminous materials with PAH over 1000 mg/kg and benzo(a)pyrene levels above 100 mg/kg are not permitted to be recycled into unbound applications (County Surveyors Society, 2008).

The acceptable threshold values of Total PAH and Benzo(a)pyrene are 1000 mg/kg and 100 mg/kg respectively. Above these limits material is classified as hazardous waste if disposed of (Bradshaw, 2005). In addition, the Environment Agency considers that if benzo[a]pyrene is present at a concentration of 50 mg/kg or more in asphalt arising's (excluding any other waste material), the waste should be classified as hazardous and denoted by the European waste code (EWC) 17 03 01 (bituminous mixtures containing coal tar) (ADEPT, 2013).

The implications of the above are that if disposed of, pavement layers containing tar would be classified as Hazardous Waste, leading to a high transportation and disposal cost. The health and environmental concerns also limit potential for this material to be recycled into hot mix asphalt or unbound mixtures. Recycling into cold recycled bound material with bituminous binders is permitted within the United Kingdom (ADEPT, 2013) and is likely to offer a cost effective and viable engineering solution.

Cold recycling of asphalt is a proven technique that reduces energy consumption (Kazmierowski et al., 1992; Thenoux et al., 2007). This reduction is largely achieved by avoiding aggregate drying (Defence Estates, 2009) and mixing of the material at ambient temperature instead of 170°C-185°C , as required for hot mix asphalt (HMA) (Defence Estates, 2005).

This research looks at foamed asphalt with cement and this type of material is classified in the UK as a cold recycled bound material (CRBM) (Defence Estates, 2009; Merrill et al., 2004).

Despite the increasingly common use of CRBMs in low to medium volume trafficked roads (Merrill et al., 2004; West and Willis, 2014), the specifications for the use of these materials in airfields are under-developed (Defence Estates, 2009) and there is no guidance to ensure that pavement design with these materials is trustworthy (Cardno, 2012; Lacalle Jiménez et al., 2016). This is combined with a frequent occurrence of tar in the pavements of airfields which are now coming of age where reconstruction may be required. This is the reason why this Thesis focuses on airfield pavement design with CRBM.

This PhD research programme focused on:

- The definition of performance parameters required to design pavements with CRBMs.
- Providing guidance on the behaviour of CRBMs under cyclic loading.
- Providing guidance on airfield pavement design with CRBMs.

AIM AND OBJECTIVES OF THE PROJECT

The aim of this research project is to develop a design methodology to use CRBMs in airfield pavements. With this purpose, 3 objectives were established:

- O1 Review of field experience on performance of cold-mix asphalt, in-situ and off-site recycled, and pavement design methods.
- O2 Design an appropriate laboratory test programme in order to measure effects of key variables on performance and establish practical limitations on the use of cold recycled materials.

- O3 Develop a pavement design methodology. Propose appropriate specifications and design guidance for airfield authorities and practitioners.

RESEARCH METHODOLOGIES AND WORK PLAN

In this section the techniques and factors that require consideration and evaluation during the experimental research are discussed. To decide the best research methods and techniques to develop the Thesis objectives, the project was broken down into three sections, and each one was associated with the most relevant research methodology as shown in Figure 1. These three sections are directly related to the main objectives of the project.

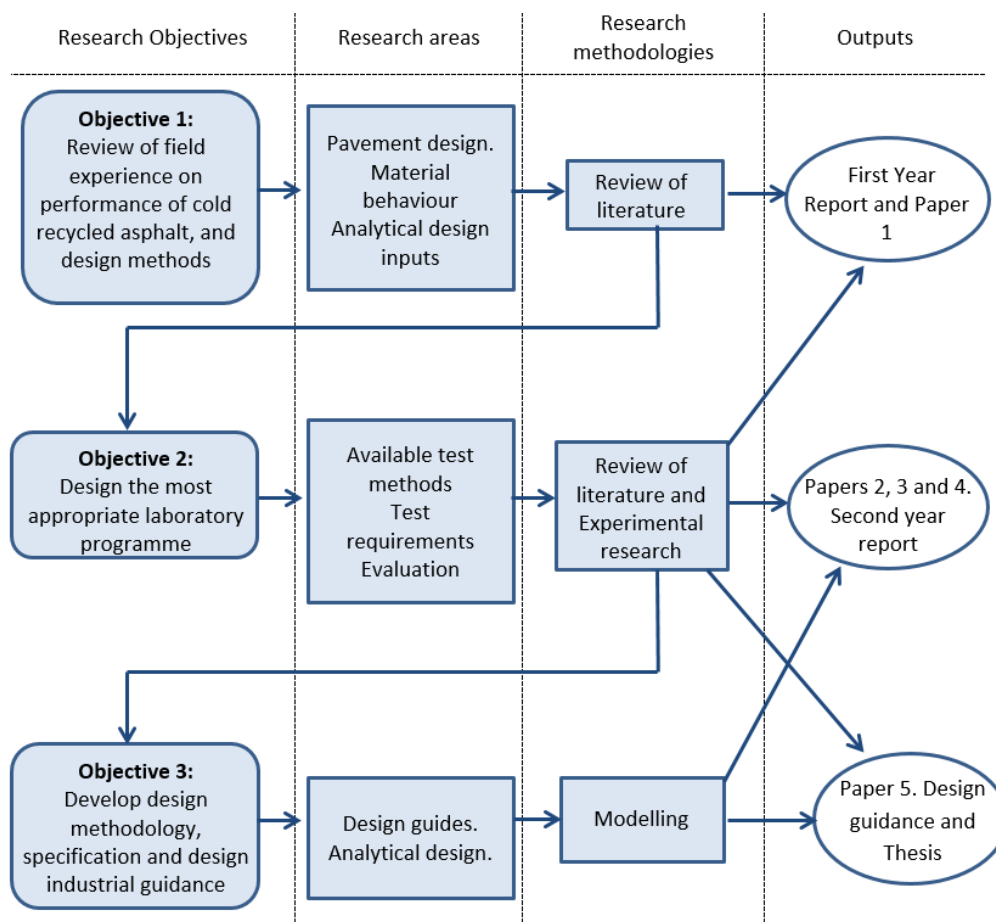


Figure 1 Research methodologies

The literature review focuses on pavement engineering principles, airfield pavement design and cold recycled materials. The aim of this literature review was to inform decisions during the experimental and analytical stage of the research.

For the experimental research a CRBM containing RAP, fly ash, cement and foamed bitumen was chosen with the appropriate characteristics for airfield pavement design. This choice is justified by the literature review carried out in Chapter 3.

The methodology comprises:

- Materials characterisation: bitumen content and recovery from RAP; binder characterisation with softening point and penetration; and RAP grading.
- Mixture manufacturing: establish an appropriate mix design, compaction and curing procedure.
 - Two compaction methods were used: gyratory compactor and roller compactor.
 - Two curing procedures were chosen: 28 days at 20°C and 3 days at 40°C.
- Mixture characterisation:
 - Binder content and recovery for characterisation with softening point and penetration.
 - Bulk density, maximum density and air voids content.
 - Mechanical properties: stiffness modulus, resistance to permanent deformation, strength, fatigue and failure mechanism.

Finally, an analytical pavement design software was chosen for modelling airfield pavements. The first stage comprises a study to understand inputs needed and adequacy of the software for airfield pavement design. In the second stage CRBM properties were input into the software, to analyse strains and stresses within the pavement layers. The results obtained are used to establish guidance on how to design airfield pavements including CRBM and achieve the final aim of the project. A deterioration analysis was also carried out to study how strains distribute within the CRBM layers and how this could affect the pavement's life.

The outputs highlighted in Figure 1 comprised the first and second year reports submitted to the university and this Thesis supported by the papers listed below and presented in the Appendix A.

Paper 1: "Laboratory trials of cold recycled foamed bitumen asphalt: RAF Waddington." The International Journal of Pavement Engineering and Asphalt Technology (PEAT) ISSN 1464-8164. Volume: 16, Issue: 1, May 2015, pp.82-95. Presented in JLM Conference February 2015, Liverpool, UK.

Paper 2: "Comparison between laboratory results for cold recycled bound materials and DBM50 used in airfield pavements." Presented in E&E Congress 2016, 6th Eurasphalt & Eurobitume Congress, 1-3 June 2016, Prague, Czech Republic.

Paper 3: "Analysis of the stiffness and fatigue resistance of cold recycled asphalt mixtures manufactured with foamed bitumen for their application to airfield pavement design." Materiales de construcción. Accepted and waiting for publication.

Paper 4: "Analysis of the failure mechanism of cold recycled bound materials evaluating fatigue performance base on dissipated energy, displacement ratio and energy stiffness ratio." Presented in TRB 2017, Washington DC, USA.

Paper 5: "Towards airfield pavement design using cold recycled bound materials". Accepted for presenting in BCRRRA conference, June 2017, Athens, Greece.

OUTLINE OF THE THESIS

The outline of the Thesis is presented in Table 1.

Table 1 *Thesis outline*

Part	Description
Section 1. Introduction	This section includes an overview of the research, background of the problem, aim and objectives, research methodology and outline of the Thesis.
Section 2. Review of literature	This is divided into 4 chapters. Chapter 1 includes principles of pavement engineering, focusing on materials and design. Chapter 2 focuses on airfield pavement design, analysing different design methodologies. Chapter 3 includes a literature review about CRBM, previous studies, performance and characterisation tests. Chapter 4 focuses on fatigue, analysing different test methods and approaches to understand how to study CRBM failure mechanisms.
Section 3. Laboratory testing	This is divided into 2 chapters. Chapter 5 includes CRBM characterisation and the behaviour comparison with standard hot mixes used in the base of airfield pavements. Chapter 6 presents a deeper study on the failure mechanism of CRBM to understand its behaviour under cyclic loading.
Section 4. Modelling	This section is divided into 2 chapters. Chapter 7 analyses how to model an airfield pavement using Kenlayer and presents the modification for pavement design using CRBM. Chapter 8 presents discussion and recommendations for using CRBM in airfield pavements.
Section 5. Conclusions and future work	In Chapter 9 the conclusions of the research are summarised and in Chapter 10 the future work is proposed.
Appendix	A: papers published and presented in conferences. B: example of pavement design with DMG27.

SECTION 2. REVIEW OF LITERATURE

This section constitutes the starting point of the research and includes a review of pertinent literature. This literature review aims to achieve Objective O1 covering the following knowledge areas:

- Pavement engineering principles; to understand pavement structures, pavement materials, applications and design (Chapter 1).
- Airfield pavement design; going through the existing design guides to understand how they work and how cold recycled materials could be introduced to being widely used throughout the industry (Chapter 2).
- Cold recycled bound materials; collating and critically reviewing the readily available information regarding recycled asphalt materials for pavement design, stabilising agents and curing procedures (Chapter 3).
- Characterisation methods; reviewing the laboratory test methods available for the characterisation of recycled materials and determination of pavement design parameters (Chapters 3 and 4).

Chapter 1

PRINCIPLES OF PAVEMENT ENGINEERING

A pavement is defined in civil engineering as the durable surfacing of a road, airstrip, or similar area, and its primary function is to transmit loads to the sub-base and underlying soil. A typical pavement structure is based on layers and these layers can consist of different materials according to their purpose (Thom, 2008).

It is believed that the Carthaginians were the first to construct and maintain a road system (Tillson, 1900); however, there is no record of road construction in the UK prior the Romans (Collins and Hart, 1936).

In general, these roads consisted of 3 or 4 layers; comprising smooth, polygonal blocks set over a base layer composed of gravel and sand. Underneath, small stones covered a large stone layer set over the subgrade (Collins and Hart, 1936; Maggiore, 2014).

Modern road construction in the U.K is attributed to Thomas Telford and John Loudon Macadam (Maggiore, 2014; Thom, 2008). Telford's pavements consisted of a 350 to 450 mm thick structure with three layers. The bottom layer comprised large stones of around 100 mm, overlaid with two layers of 65 mm stones (Collins and Hart, 1936).

John Macadam knew that angular aggregate would have a better performance; therefore, he used a sloped subgrade (to improve drainage) on which he placed angular aggregates in two layers for about 200 mm depth. The wearing course was a 50 mm thick layer with 25 mm maximum aggregate size to provide a smooth ride (Collins and Hart, 1936).

The first tar macadam pavement was placed outside Nottingham in 1848 (Collins and Hart, 1936). Coal tar was used as the binder, possibly trying to recycle waste material from coal-gas lighting. Over a long period, this material was used in UK, France and USA. In these years, there was a lack of any specification for the tar, contributing to most of these roads failing within a few years and subsequently discrediting tar as a paving material. However, some of these tar bound surfaces lasted longer. These mixes were typically manufactured with about 6% tar binder and 20% of the aggregates passing the 10 mm sieve (Pavia Systems, 2012).

Portland cement was regularly used to create a stiff base for pavements but was not used as a wearing course until after 1910 (Pavia Systems, 2012). Around 1900 bitumen started to be used as a binder in pavement construction (Thom, 2008), leading to the emergence of hot mix asphalt and ultimately the modern pavement materials used all around the world.

In the next sections, modern pavement engineering is summarised focusing on structure, materials, applications, design and maintenance.

1.1 PAVEMENT STRUCTURE AND MATERIALS

Pavements are generally classified as flexible, rigid or composite (Read, 1996; Thom, 2008). This classification is different regarding airfield pavements; however, this will be discussed in Chapter 2. The materials employed in pavements are:

- Flexible: the layers comprise bituminous mixtures (asphalt) and unbound materials.
- Rigid: the main layer is Pavement Quality Concrete (PQC).
- Composite: combination of rigid and flexible (asphalt over concrete or hydraulically bound materials).

1.1.1 Pavement structure

Figure 2 illustrates a typical pavement and pavement foundation structure. On top, the **surface course** is a high-quality and relatively expensive material. It is designed to withstand direct loading and to provide adequate skid resistance. The most commonly used surface course material in the UK is asphalt. The **binder course** can be similar to the base; however, it has a smaller aggregate size. The **base** gives the pavement most of its strength and it can be asphalt, hydraulically-bound material or granular.

The **sub-base** layer's function is to limit pavement flexure and the performance of an asphalt or concrete base depends critically on the stiffness and quality of this layer. It normally comprises hydraulically-bound or granular material. The **capping** is generally a lower cost materials and its purpose is to have a suitable layer to allow safe construction over the subgrade. In this layer, either hydraulically-bound or granular materials are commonly used. Finally, the **subgrade** is the natural (or imported) soil that has to be protected from applied loads and contamination by the previous layers (Thom, 2008).

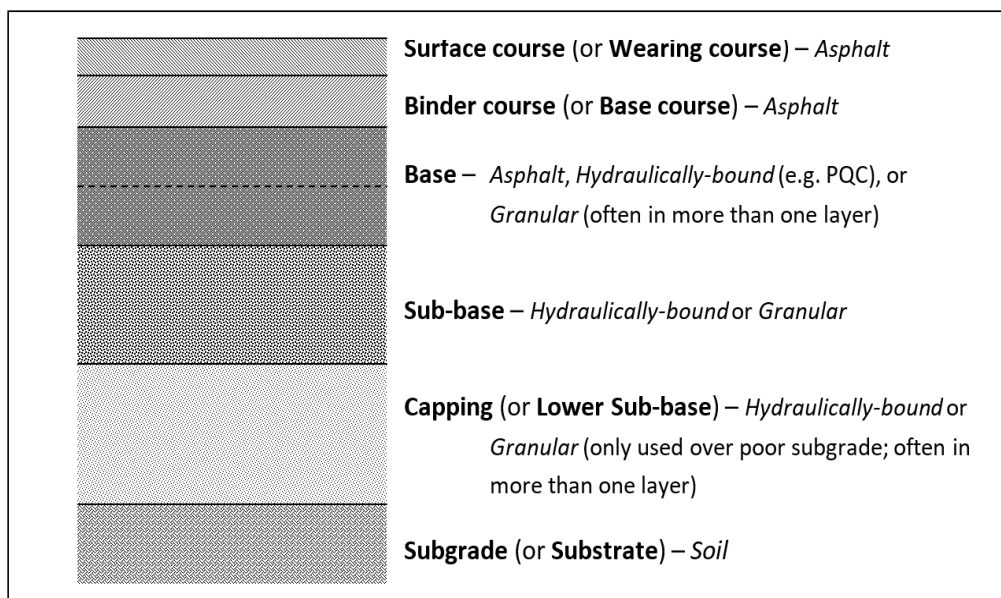


Figure 2 Typical flexible pavement structure (Thom, 2008)

1.1.2 Materials

Materials within the pavement structure can be classified as: soil, granular material (aggregates), bitumen-bound materials (asphalt), hydraulically-bound materials and other materials used to strengthen the structure or to improve its aesthetic aspect such as paving blocks, geogrids or reinforcement (steel, polymeric or glass fibres) (Thom, 2008).

In this section asphalt is more extensively explained as in this Thesis the author deals with Reclaimed Asphalt Pavement (RAP).

- Asphalt

Asphalt mainly consists of a fine aggregate (filler) and bitumen matrix with differing amounts of coarse aggregate distributed throughout the mixture (Read, 1996).

Aggregates are the basic constituents used in construction (Welsh Assembly Government, 2004). Natural aggregates are obtained from a variety of source rocks; however, their extraction is constrained by increased cost and environmental concerns (Hoyos et al., 2011).

The use of RAP in road construction has been proven to reduce raw material consumption and waste generation (CIRIA, 1999; Hoyos et al., 2011). This is because it directly substitutes virgin aggregates and potentially contributes bitumen to the asphalt mix. This contribution can then reduce the required amount of new bitumen to be added during the asphalt manufacture.

Bitumen is defined as a tarlike mixture of hydrocarbons derived from petroleum naturally or by distillation (Hunter et al., 2015). Bitumen is a very different binder from cement, it remains liquid even in service, which means that one of the most important mechanical properties is its viscosity, and this is a function of temperature (Thom, 2008).

To characterise bitumen there are practical tests such as penetration and softening point. These tests are used to assess bitumen's properties, such as viscosity, and also to understand its behaviour (Hunter et al., 2015). The most common way to describe a bitumen is referring to its penetration index. This information together with the softening point will give a practical measure of bitumen temperature susceptibility (Hunter et al., 2015; Thom, 2008).

At ambient temperature, bitumen used for paving is usually hard and very viscous; therefore, its workability should be improved making bitumen behave as a mobile liquid.

To achieve this, bitumen can be heated, dissolved, emulsified or foamed. However, solvents have disadvantages regarding health and safety considerations, and as this Thesis deals with cold recycled materials, heating is not the chosen solution. In Chapter 3, the two other technologies, emulsion and foam, are studied to choose the best option for this research.

Bitumen is affected by the presence of oxygen, ultraviolet radiation and by changes in temperature, which cause it to harden (Gamarra and Ossa, 2016; Thom, 2008). This hardening can be beneficial as it increases the stiffness of the asphalt and the load spreading capabilities of the structure (Thom, 2008). However, bitumen ageing is also one of the principal factors causing deterioration of asphalt pavements, causing failure by traffic, thermally induced cracking and ravelling (Petersen, 2009; Thom, 2008; Xiaohu and Isacsson, 2002).

This ageing occurs during the mixing and construction process as well as during long-term service (Gamarra and Ossa, 2016; Thom, 2008). The factors that affect ageing are bitumen content in the mix, nature of aggregates, particle size distribution, void content and production related factors such as temperature and time (Xiaohu and Isacsson, 2002). Early stage ageing can be called curing, and it is very different from hot mix to cold mix due to the installation temperatures. Hot mix is produced and installed at high temperatures that change bitumen characteristics while in a cold mix this process does not happen (Sunarjono, 2009; Thom, 2008).

The content of **filler** is an important property that affects asphalt behaviour. Filler is the term used to define particles passing the 0.063 mm sieve. These particles are added to the mixture to improve asphalt behaviour stiffening and strengthening the bitumen, improving the mixture's fatigue life (Thom, 2008). However, too much filler versus bitumen can dry the mixture; therefore, the amount of filler should be optimised and even if this filler is a component of the aggregate, it is normally considered as an additive.

Asphalt performance in service is significantly influenced by the rheological properties and the chemical constituents of bitumen; however, there are other factors such as aggregate nature, composition of the mixture, degree of compaction, bitumen content, etc. that influence long-term durability of asphalts (Thom, 2008).

In service, asphalt behaviour can be studied considering the surrounding circumstances of specific types of failures (Thom, 2008):

- Cracking: associated with stresses induced by traffic loads and temperature changes.
- Deformation (rutting): most these defects are related to plastic deformation of the surface course or the surfacing and often in the binder course. This can occur under moving or stationery traffic.
- Fatting-up: is a migration of the bitumen to the surface.
- Fretting: is the loss of material from the road surface(Oliveira, 2006). Affected by the content of bitumen and the degree of compaction.

The most basic property of asphalt for pavement design is its stiffness modulus (Thom, 2008). It can be measured in the laboratory with tension-compression, indirect tension and flexure tests (Thom, 2008).

As said before, one of the key design issues for an asphalt pavement is cracking; therefore, fatigue testing becomes very important (Thom, 2008). Fatigue can be measured with the same test configurations for stiffness testing. The most popular tests for fatigue are indirect tensile and flexural with four-point bending and trapezoidal providing alternative flexural testing options (Thom, 2008).

Permanent deformation is closely related to the stability of the aggregate skeleton (Thom, 2008) and/or the performance of the bitumen. There is no single property defining resistance to permanent deformation, which means that testing can only be indicative. The most usual forms of laboratory scale based tests are: repeated load axial test (RLAT), repeated load triaxial test and wheel-tracking test (Thom, 2008).

- Hydraulically bound materials

Hydraulically bound materials (HBM) are mixtures that set and harden by hydraulic reaction (British Standard, 2013a). They are the most commonly used materials for pavement sub-bases (Bateman and Nicholls, 2013).

In the United Kingdom these mixtures are classified under different grades, based on the strength of binding agent and aggregate gradation: cement bound granular mixtures, slag bound granular mixtures, fly ash bound granular mixtures, fly ash for hydraulically bound mixtures, hydraulic road binder bound granular mixtures, soil treated by cement, soil treated by lime, soil treated by slag, soil treated by hydraulic road binder and soil treated by fly ash (Bateman and Nicholls, 2013; British Standard, 2013a).

Depending on the reaction speed, there are two types of HBMs. Quick setting, based on the fast setting and hardening characteristics of cement. The addition of cement to an aggregate has the principal objective of giving cohesion to the mixture to improve mechanical properties and durability.

Slow setting HBMs use binders from industrial by-products such as fly ash or furnace slag. In pavement construction they are used to improve the strength and durability of the pavement while lowering the construction costs (Industrial Resources Council, 2008).

The most important properties for pavement engineering are stiffness and tensile strength; however, fatigue resistance, curing rate and thermal expansion ratio should also be taken into account in the construction process (CEDEX and IECA, 2009; Thom, 2008).

Concrete is a quick hydraulically-bound material formed by mixing cement, coarse and fine aggregates and water (normally drinking quality), with or without the incorporation of admixtures, additions or fibres (British Standard, 2016). There are three different types of concrete pavements: plain jointed (no steel reinforcement and transverse joints sawed across the pavement); jointed reinforced (to control transverse cracking, now rarely used) and continuously reinforced concrete (no contraction joints, low routine maintenance required) (Geopave, 1998).

Pavement quality concrete (PQC) is a high strength concrete containing Ordinary Portland Cement (OPC) with a low cement-water ratio (CEDEX and IECA, 2009; Thom, 2008).

Lean concrete is a low strength concrete with a high water content (lower concentration of cement) compared to other paving materials of this class (Department for Transport, 2016a). Lean concrete is commonly used in the base or sub-base course to provide the proper support for overlying PQC or asphalt (Kumar, 2016).

HBMs can also be classified by the compaction method. Wet-formed mixtures can be poured and compacted by means of vibration. The optimum water content should be established to avoid unused binder, strength reduction and permit sufficient workability. Roller-compacted mixtures need a high water content to allow compaction and, in general, produce weaker materials than those with wet-forming (Thom, 2008).

As in the case with all pavement materials, concrete rarely fails under a single load application. However, it can fail after multiple load applications, even if the stress is considerably less than the ultimate strength of the material. Therefore, fatigue tests are important to study concrete durability (Thom, 2008).

In the UK, over the concrete layers, asphalt can be placed to improve ride quality and reduce noise produced by traffic.

1.2 APPLICATIONS AND DESIGN

Different pavement applications lead to very different design criteria. For example, while for high-speed highways ride quality and a quiet surface are critical, a car park needs sufficient bearing capacity and acceptable surface finish, and an airfield should limit deformation, resist fuel/oil spillage and avoid foreign object damage (FOD) (Thom, 2008). Each design criterion has an appropriate material and construction process to achieve desirable performance.

To achieve a good ride quality modern paving equipment must be used. PQC joints reduce ride quality; therefore, asphalt or continuously reinforced concrete (CRC) should be used. As CRC is an expensive option it is normally only used in heavily trafficked highways (Thom, 2008).

To improve the skid resistance of a pavement, the surface requires a certain texture depth. This is most commonly achieved by asphalt (Thom, 2008).

Reducing surface noise can also be a design criterion and it is generally accepted that asphalt is less noisy than concrete (Thom, 2008).

For avoidance of rutting, there are several solutions such as designing asphalt mixtures to resist high loads, using PQC, using block paving over hydraulically-bound base or using grouted macadam (combining an asphalt skeleton with a cementitious grout) (Thom, 2008).

In all cases, low maintenance and cheap construction are desirable design criteria and they should be considered.

1.2.1 Pavement design principles

The field of pavement design is dynamic and its concepts are continually changing as new data becomes available (Mubarec, 2014; Yoder and Witczak, 1975). When designing a pavement, there will always be several different options to solve the problem. The engineer has to present a range of possibilities that not only bear in mind the technical aspects, but also the environmental cost and the true life-cycle cost (Thom, 2008).

To estimate the design life of a pavement it is necessary to carry out a structural design (Ojum, 2015; Oke, 2010). This design will optimise the layer thicknesses to give the necessary structural support to the pavement (Ojum, 2015).

A pavement should: protect the subgrade, guard against deformation and break-up of the pavement layers, provide a suitable surface and ensure maintainability (Thom, 2008). With these purposes the design involves a study of soils and paving materials, their behaviour under load and under different climate conditions (Yoder and Witczak, 1975) and the study of traffic variables, such as load magnitude, contact pressure and load groups. Durability can be affected by water; therefore, drainage should not be underestimated (Thom, 2008).

The simple, practical way to simplify real traffic loads into an input parameter for design is to convert everything into equivalent numbers of a single design load case (Defence Estates, 2011; Thom, 2008).

Most pavement design and maintenance today is carried out with reference to a document (guides with design charts) or a computer program (Thom, 2008). The challenge appears when the designer wants to use new materials that are not included in these design guides.

In the next chapter airfield pavement design is discussed to understand how to include CRBM into airfield pavement layers.

Chapter 2

AIRFIELD PAVEMENT DESIGN

To be able to establish a new airfield pavement design guide with cold recycled materials (ultimate objective of this Thesis O3), it is important to understand how actual design guides work and how new materials could fit into these guides.

In comparison to highways traffic, aircraft represent peculiarities in terms of wheel loads, arrangement and speed. In terms of loading on the runway, it is a misconception that the critical case occurs in the touchdown zone, when the aircraft is rapidly decelerating. In this case, the aircraft wings are carrying most of the load. The critical case is when the aircraft is travelling slowly on the ground with a full fuel load, prior to take off (Thom, 2008). This is an important consideration when selecting loads for airfield pavement design. In addition, airfield pavement design should focus on: limiting deformation, skid resistance, resisting fuel/oil spillage, avoiding Foreign object damage (FOD) and reducing maintenance (Thom, 2008). Avoidance of FOD to aircraft (for example ingestion of aggregate into a plane engine) is critical to the safe operation of airfields and important consideration for the selection and maintenance of pavement surfacings.

In this chapter, different airfield pavement design methods are summarised. From these methods, the most commonly used in the UK is selected and described extensively. An empirical-analytical method, namely Kenlayer, is selected to carry out the pavement analysis with CRBM; therefore, this method is also explained.

2.1 DESIGN METHODS

As already stated in Chapter 1, current design methodologies normally comprise a design guide or a computer program. In this section, different design methodologies used for airfield pavements are described.

The traditional design models can be classified into three groups: empirical, analytical or mechanistic and analytical-empirical or mechanistic-empirical.

The Empirical methods are based on the observed performance of pavements, monitoring them under traffic loading and different environmental conditions to establish the appropriate construction thicknesses. These methods are based around design charts, where the pavement thickness for a given traffic level, material type and subgrade strength can be obtained. This means that these methods are restricted to the conditions for which they were originally developed (Hakim, 2011).

Analytical pavement design uses mathematical models (multi-layer elastic system, plate on elastic foundation or finite element) and fundamental engineering properties of materials to calculate layer thickness. The stresses and strains calculated with the mathematical models are then compared with the allowable values for the materials to control fatigue cracking and pavement deformation (or other deterioration mechanisms) to address the necessary thickness of the pavement layer to have a certain design life. These methods allow non-standard materials and construction types (Hakim, 2011).

The Analytical-Empirical (or Mechanistic-Empirical) approach has two steps. The first one is based on analytical methods, calculating strains and stresses from mathematical models, and the second step is to compare these results with permissible stresses or strains obtained by observed materials performance. In all models, the analytical tool used is some version of the theory of elasticity and the damage caused by the stresses and strains is determined from empirical relations (Ullidtz, 2002).

2.1.1 Empirical methods

Empirical methods can vary from extremely simple to quite complicated. The simplest models are based on what has worked in the past and the most complex ones are based on empirical equations derived from experimentation (Pavia Systems, 2012).

2.1.1.1 Design & Maintenance Guide 27 (DMG 27) (Defence Estates, 2011)

This design guide for airfield pavements has largely grown out of the experience of pavement performance and it is based on design charts. It is the design guide typically used by UK authorities.

This guide takes into consideration the following design parameters:

- Pavement type – rigid or flexible: in the case of this study, the pavement type will be flexible as the material object of study is recycled asphalt. It must be taken into account that for airfields, a flexible pavement is what is traditionally termed composite for road construction (i.e. asphalt over HBM).
- Quality and mechanical properties of the pavement materials.
- Subgrade strength.
- Design Aircraft Classification Number (ACN): a number which expresses the relative effect of an aircraft on a pavement for a specified standard subgrade strength.
- Frequency of trafficking.
- Design life and definition of failure.
- Serviceability.

To fit CRBMs into this design method the quality and in-service mechanical performance of these materials must be at least as good as those in the pavements upon which the design method is based and have a similar behaviour. The current practise when using CRBMs as base course with this design guide is to use design chart 7. This chart is used when the pavement is designed with a bound base material, namely asphalt concrete (AC). This means that CRBM behaviour is equated with AC behaviour, simplifying the design process; therefore, material properties should be comparable. This comparison is presented as part of the author's research in Chapter 5.

2.1.1.2 California Bearing Ratio method (BAA plc Group Technical Services, 1993)

The California Beating Ratio (CBR) method of pavement design is one of the earlier pavement design approaches and has been used since the early 1940's in airfield pavement design (Taboza Pereira, 1977). In this method, the thickness design charts are developed based on observation, with reference to the subgrade CBR value for the most critical moisture condition, and a load repetition factor (α) which modifies pavement thickness to account for load repetitions and main wheel gear types. The CBR is defined as the penetration resistance of a subgrade soil relative to a standard crushed rock (Huang, 2004) and it indicated the ability of a soil to withstand wheel loads (Taboza Pereira, 1977).

As there are two failure modes for flexible pavements, two suitable design parameters must be selected to control them. The ones usually used are the horizontal strain at the base of the bituminous surfacing, and the subgrade vertical strain. These parameters are plotted in graphs based on laboratory tests and correlated against performance, or empirical evidence.

2.1.2 Analytical methods (mechanistic)

Analytical methods normally use what is termed as multi-layer elastic analysis (programs such as MPTRN, BISAR, KENLAYER, etc.) and in these methods the inputs include layer details and loads with the outputs being stresses, strains and displacements (Thom, 2008).

After the strains and stresses are calculated they should be related with the life to failure. This is normally done with an equation based on experience and/or laboratory testing. In airfields, the pavement failure criterion is based on subgrade protection and the equation used to analyse the life to failure is as shown in equation 1 (BAA plc Group Technical Services, 1993).

$$\varepsilon_v = \frac{a}{C^b} \quad [1]$$

Where ε_v is the calculated vertical strain in the subgrade, a and b are constants depending on the failure criterion and reliability and C is the number of allowable coverages (number of times a particular point on the pavement is expected to receive a maximum stress).

2.1.2.1 MPTRN

MPTRN is an in-house design software developed by Scott Wilson Ltd. based on mathematical models. The inputs in this software are:

- Number of layers and thickness.
- Material Stiffness and Poisson ratio.
- Wheel configuration: number of wheels, pressure and footprint area.
- Locations where the strains or stresses need to be computed.

The outputs are the strains, stresses and deflections of the locations specified in the inputs.

2.1.2.2 Bitumen Stress Analysis in Roads (Hunter et al., 2015)

Bitumen Stress Analysis in Roads (BISAR) was developed by Shell in 1960, and it was first called BISTRO (Bitumen Structures in Roads). With this software, the horizontal tensile strain at the bottom of an asphalt layer and the vertical compressive strain at the surface of a subgrade can be determined for various pavement configurations and these two values compared with permissible values. This software works in a very similar way to MPTRN.

To keep the method simple, two assumptions have been made: Poisson's ratio does not vary greatly and a value of 0.35 is assumed for an asphalt layer and the thickness of the subgrade is infinite. These assumptions leave only five variables for any particular system:

- The elastic modulus of an asphalt layer.
- The thickness of the asphalt layer.
- The elastic modulus of the granular sub-base layer.
- The thickness of the granular sub-base layer.
- The elastic modulus of the subgrade.

2.1.3 Analytical-empirical methods (mechanistic-empirical)

The benefit of using analytical methods is that they can include the characterisation of a material in situ, typically using a portable device (like a Falling Weight Deflectometer (FWD)) to undertake testing which replicates loadings and measures the response of the pavement (deflections). These measurements can be the input to the model equations to determine the existing pavement structural condition ("back analysis") and further assess the approximated remaining life of the pavement (Pavia Systems, 2012).

2.1.3.1 KENLAYER

This software was developed by Huang (Huang, 2004) and allows a multilayer elastic analysis. It is more versatile than MPTRN or Bisar as the failure criteria can be modified. The failure criteria are adjusted by modifying cracking and permanent deformation laws.

The outputs of the software are strains and stresses within the pavement layers and the final design life is based on the allowable number of loading repetitions defined by the failure laws.

2.1.3.2 Federal Aviation Administration method

This method is based on the CBR method for design, described under empirical methods (Muniraj, 2003). The Federal Aviation Administration (FAA) developed in 1978, through test track studies and observation of in-service pavements, pavement thicknesses necessary to protect pavement layers with various CBR values from shear failure.

The FAA design charts are based on the equivalent annual departures of a selected design aircraft. The annual departures are assumed to occur over a 20-year life.

In 2009 mechanistic design procedures were implemented in the FAA Rigid and Flexible Iterative Elastic Layer Design program (FAARFIELD), becoming an Analytical-Empirical method. The inputs for this software are:

- Traffic: type of airfield and annual departures.
- Material stiffnesses and Poisson's Ratios.
- Layer thicknesses.

With these parameters, the output will be the pavement's design life. If the design life is considered as an input, the output will be the layer thickness.

2.1.3.3 BAA (BAA plc Group Technical Services, 1993)

The BAA method is based on the CBR method described in the empirical section, with the addition of some analytical equations. This method uses graphs and charts based on specific layer properties. The pavement model used for the calculations is based on JULEA (Jacob Uzan Layered Elastic Analysis), as it was found to be suitable for both flexible and rigid airport pavements.

The BAA method assumes that the asphalt layers are one-third of the total depth of bound material and with a stiffness modulus of 1380 MPa and a Poisson's Ratio of 0.5. Depending on the traffic, the subgrade category and the reliability, the thickness of the pavement can be selected from a graph.

2.1.3.4 Mechanistic-Empirical Pavement Design Guide (Pavia Systems, 2012)

The aim of the Mechanistic-Empirical Pavement Design Guide (MEPDG) is to identify the physical causes of stresses in pavement structures and calibrate them with observed pavement performance. The MEPDG is designed to update the 1993 American Association of State Highways and Transportation Officials (AASHTO) Guide for Design of Pavement Structures, which is primarily based on empirical observation from the AASHO Road Test which began in the 1950s; therefore, this method is design for highways. In conjunction with the MEPDG research project, software was also developed to assist in organizing and performing these design calculations.

One of the significant changes is that the approach to pavement design is effectively reversed. In conventional design methods, various inputs are considered and used to produce the design requirements for the pavement structure. In mechanistic-empirical design, the design of the pavement structure is initially assumed on a trial basis, along with inputs for traffic and climate.

MEPDG software can compute how the trial design will respond to the load and environmental stresses created by these inputs. This leads to an estimate of the level of damage the pavement will sustain over time, in terms of pavement distress and deterioration of ride quality.

2.1.3.5 American Association of State Highways and Transportation Officials 2008 (Hakim, 2011)

The AASHTO Design Guide (1993) and its previous versions (1972, 1985) are based on the empirical equations developed from the AASHO road test; therefore, this method is also design for highways. The design requires specifying the reliability level and expected pavement performance (serviceability). The thickness of individual layers such as asphalt, granular base and sub-base are then calculated considering various material strengths (structural layer coefficients) to achieve the design structural number.

The new analytical-empirical design guide and software AASHTO 2008 provides several improvements. It uses a multilayer elastic system to model flexible pavements and finite element method to model rigid pavements; it considers detailed traffic load spectra rather than the concept of equivalent standard axles and it incorporates a climatic model.

2.1.3.6 Evaluation of Layer Moduli and Overlay Design (Drenth, 2006)

ELMOD is an acronym for Evaluation of Layer Moduli and Overlay Design used for the structural assessment of all kinds of pavement structures.

The latest version, ELMOD6 is able to design pavement structures by specifying the pavement model, the mechanical characteristics of all types of materials and an unlimited number of loading types.

Based on the stiffness values of each pavement layer a structural strength can be calculated using mechanistic-empirical principles expressed as a remaining life. On the basis of this, any requirement for strengthening or additional pavement thickness (overlay) can be confirmed.

2.1.4 Summary

In Table 2 a summary of the design methods discussed is presented. After different design methodologies were studied it was decided to focus on DMG 27 for this research as it is the design guide used by airport authorities in the UK. However, this design guide does not readily allow the introduction of new material properties (Lacalle Jiménez et al., 2016); therefore, a new design approach is needed. From the analytical and analytical-empirical methods described, Kenlayer was selected to analyse pavement structures as it is the most versatile software option. In this research Kenlayer will be used to analyse strains and stresses within the pavement layers and to compute the design life.

The research methodology will be explained in more detail in Chapters 7 and 8; however, in the next sections both methods are described.

Table 2 Summary of airfield pavement design methods

Type	Method	Description
Empirical	DMG 27	Design guide typically used by UK authorities based on graphs.
	CBR	Based on graphs.
Analytical	MPTRN	In house software (low availability). Calculation of strains and stresses within the pavement layers.
	BISAR	Calculation of strains and stresses within the pavement layers.
Analytical-Empirical	Kenlayer	Flexible software. Calculates strains and stresses and gives design life based on fatigue cracking and permanent deformation (laws can be adjusted).
	FAARFIELD	Calculates design life of a given structure. Materials already defined.
	MEPDG/ AASHTO 2008	More complete software, using multilayer elastic models and finite element. Materials already defined. Designed for highways, no airfields.
	BAA	Based on CBR including analytical equations. Based on graphs.
	ELMOD 6	Calculates design life of a given structure. Materials already defined.

2.2 AIRFIELD PAVEMENT DESIGN WITH DMG 27

UK airfield pavement design started in 1937. The first flexible pavements comprised layers of brick or stone overlaid with two courses of tar macadam and a sealing coat of mastic asphalt. Nevertheless, rigid pavements included either 150 mm or 200 mm thick concrete slabs laid directly on the ground, following the removal of the topsoil. However, these first pavements failed with the increasing weight of aircraft and had to be overlaid with tar macadam and rolled asphalt. Nonetheless, flexible pavements kept failing and were replaced by concrete pavements or with further overlays of tar macadam (Defence Estates, 2011).

Between 1937 and 1945 more airfields were constructed and extensive data on pavement performance, construction details, subgrade characteristics and relationship between aircraft loads or tyre pressures and damage induced to the pavements were collected (Defence Estates, 2011). In 1948 the aircraft were classified with a Load Classification Number (LCN) and in 1959 “Airfield Design and Evaluation” was published as a design guide using LCN as an aircraft loading parameter.

In 1989 a new design guide incorporating improvements to the California Bearing Ratio (CBR) method was published, based on full scale testing by the US Army Corps of Engineers. The major change in this guide was the use of the Aircraft Classification Number – Pavement Classification Number (ACN-PCN) instead of the LCN (Defence Estates, 2011).

In 2006 a 2nd edition of the guide was published incorporating findings from site investigations that had taken place since 1989 and updating aircraft and concrete strengths. In 2011 the 3rd edition was published, DMG 27, incorporating the latest performance considerations, designs and advances in construction materials (Defence Estates, 2011).

2.2.1 Materials and structure

Nowadays, airfield pavements comprise two types of construction, differing from road pavements:

- Rigid: A rigid pavement comprises either wholly or partly concrete construction which can be unreinforced, reinforced or reinforced with prestressed elements. Rigid pavements distribute the aircraft loading to the subgrade by means of its high flexural stiffness (Defence Estates, 2011).
- Flexible: A flexible pavement is composed of bound or unbound granular materials. Cement-bound granular bases beneath bituminous surfacings make pavements relatively rigid in their early years, prior to the cement-bound granular bases cracking. However, this type of construction is treated as a flexible pavement for design and evaluation purposes (Defence Estates, 2011). This is due to the change of behaviour of Dry Lean Concrete (DLC) that gradually changes to a flexible mode as its stiffness reduces after cracks propagate in the cement-bound layer.

CRBM would be used in flexible airfield pavement construction. Therefore, flexible pavements are the focus of subsequent discussion.

Asphalt is widely used in airfield pavements; however, in each section of an airfield (runway, taxiway, aprons, parking areas, etc.) different considerations apply. This can mean different types of asphalt or pavement types are selected for different pavement sections. Commonly, asphalt is used in runways and taxiways in the surface, binder and base courses (European Asphalt Pavement Association, 2003).

Defence infrastructure organisation (DIO) specifications allow the use of CRBM in the lower layers of taxiways and runways (Defence Estates, 2009); nonetheless, use of these materials is limited due to unknown long term performance and risk of failure with high rehabilitation cost and traffic delay.

The pavement structure of airfields is divided into surface, binder and base courses laying on the subgrade as shown in Figure 3, for new build and full rehabilitation designs (BAA plc Group Technical Services, 1993; Defence Estates, 2011).

The surface and binder courses have typical thicknesses of 40 mm and 60 mm respectively; therefore, the base course thickness is designed depending on traffic, design life and subgrade conditions.

The base thickness calculated is typically divided into 1/3rd of asphalt and 2/3rd of dry lean concrete (with less cement paste as compared to conventional concrete) (BAA plc Group Technical Services, 1993), with a minimum of 120 mm of asphalt base to avoid reflective cracking for high traffic scenarios (Defence Estates, 2011).

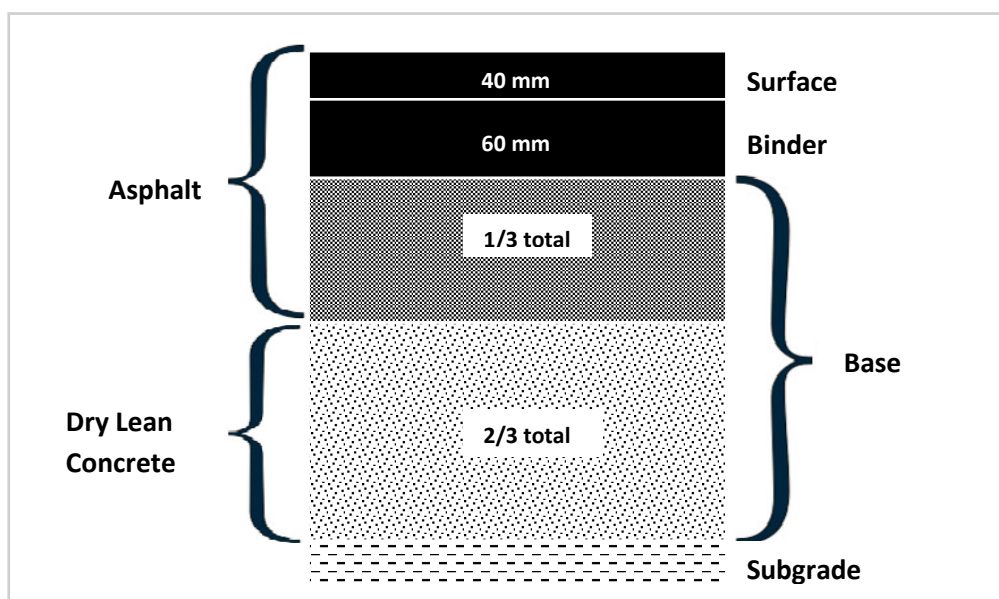


Figure 3 Airfield flexible pavement structure

2.2.2 Flexible pavement design considerations with DMG 27

To carry out an airfield pavement design with DMG 27, the following have to be taken into account: quality and mechanical performance of pavement materials, subgrade strength, design ACN, frequency of trafficking and design life (Defence Estates, 2011).

Once these parameters have been established, pavement thickness can be obtained using the graphs provided in the guide.

2.2.2.1 Pavement materials quality

Materials used for paving should be at least as good as those specified in DMG 27 (Defence Estates, 2011). This guide is specifically designed for use of hot mix asphalt (HMA); therefore, modification is needed to include CRBM.

2.2.2.2 Subgrade strength

In this guide subgrade CBR is used to establish its strength. Also other subgrade properties that should be considered are described in the guide (Defence Estates, 2011).

2.2.2.3 ACN-PCN

The ACN expresses the aircraft's relative loading severity on a pavement supported by a specific subgrade. In DMG 27 the method used to calculate the ACN for an aircraft is explained. For this Thesis aircraft were chosen from Appendix B of DMG 27 where the ACNs of several aircraft types are presented for rigid and flexible pavements and for 4 types of subgrade: 15%, 10%, 6% and 3% CBR (Defence Estates, 2011).

2.2.2.4 Traffic

For airfield pavement design, the magnitude and configuration of the wheel loads is a dominant factor; however, the fatigue caused by load repetition should also be considered. In DMG 27 graphs are plotted for 3 levels of traffic: low, medium and high. This corresponds to 10,000; 100,000 and 250,000 coverages during the design life. Coverages represent the number of times a particular point on the pavement is expected to receive a maximum stress. The coverages are calculated using the pass-to-coverage ratio, and this depends on each type of aircraft. DMG 27 provides this ratio (Defence Estates, 2011).

2.2.2.5 Design life

The structural design life for flexible pavements is 20 years (Defence Estates, 2011). However, bituminous surfacing requires maintenance after 7-10 years. DMG 27 assumes an increasing need of maintenance in the last few years of the pavement's life (Defence Estates, 2011).

2.3 PAVEMENT DESIGN WITH KENLAYER

Huang (Huang, 2004) divides the design factors into four categories: traffic and loading, environment, materials, and failure criteria.

Within the traffic and loading category, axle loads, number of repetitions, tyre-contact area and vehicle speed are considered (Huang, 2004). The environmental factors include temperature and precipitation. To determine stress, strains and displacements of the pavement, properties of materials must be specified, such as elastic modulus and Poisson's ratio. However, Poisson's ratio typically only has a relatively small effect on pavement response. In addition, Poisson's Ratio is a difficult material property to measure. Therefore, it is reasonable to adopt typical assumed values for this input (Huang, 2004).

In terms of failure criteria, three principal types of distress should be considered for flexible pavements. Fatigue cracking relates the number of allowable repetitions to the tensile strain, and it can be studied with laboratory fatigue tests. Rutting is related to the permanent deformation of the material and to control it the vertical compressive strain at the top of the layer should be limited. Finally, thermal cracking can be evaluated with the mix of stiffness and fracture strength characteristics (Huang, 2004).

The failure criterion for fatigue cracking is expressed with equation 2.

$$N_f = f_1(\varepsilon_t)^{-f_2}(E_1)^{-f_3} \quad [2]$$

Here N_f is the allowable number of load repetitions to prevent fatigue cracking, ϵ_t is the tensile strain at the bottom of the asphalt layer, E_1 is the elastic modulus of the asphalt layer and f_1 , f_2 and f_3 are coefficients determined from laboratory fatigue tests, with f_1 modified to correlate with field performance observations (Huang, 2004).

It should be determined whether this failure criterion can be also applied to CRBM. This investigation is summarised in chapter 6.

The failure criterion for permanent deformation is expressed by equation 3.

$$N_d = f_4(\epsilon_c)^{-f_5} \quad [3]$$

Here N_d is the allowable number of load repetitions to limit permanent deformation, ϵ_c is the compressive strain on the top of the subgrade, and f_4 and f_5 are coefficients determined from test pits or field performance (Huang, 2004).

The compressive strain on the top of the subgrade is used as a failure criterion as the permanent deformation is considered to be caused by the subgrade weakness rather than by the overlaying layers (Huang, 2004). However, as stated in section 2.1.2, for airfield pavements the equation applied for subgrade protection in DMG 27 is equation 1.

In this regard, Kenlayer will be used to calculate strains at the bottom of the asphalt layers and at the top of the subgrade to further calculate the pavement layer life with the corresponding fatigue and deformation law.

Once the data needed to analyse a pavement with CRBM have been established, the next step is to understand how these mixtures behave and how to characterise them. This is presented in the next section.

Chapter 3

COLD RECYCLED BOUND MATERIALS

The UK has adopted the concept of sustainable development and two key measurements of the level of sustainability achieved are the volumes of new resources consumed and the waste sent for disposal (CIRIA, 1999; Huang et al., 2007). The aim should be to minimise the waste generated and maximise quantities of materials reused or recycled.

The requirement of using recycled asphalt planings back into new asphalt is becoming increasingly important (Copeland, 2011). The main reasons for this are the economic and environmental benefits (Copeland, 2011). These benefits include reducing demand on finite resources, reducing the generation of waste materials and reducing embodied energy (CIRIA, 1999; Huang et al., 2007; Meocci et al., 2017). From an environmental point of view, in-place full-reclamation using CRBM is the most efficient technique (Betti et al., 2016).

Recycling asphalt is a pavement rehabilitation technique which involves reprocessing materials from the layers constituting a pavement that has already been in service. These materials have lost some of their original properties because of use and/or ageing. These properties include cohesion and surface texture (Al-Qadi et al., 2007; West, 2010).

The recycling processes can be divided into two major methods: hot or cold techniques. These can be further sub-divided into off-site (which includes central plant recycling on site) or in-situ recycling. Cold recycling of asphalt is a proven technique that reduces energy consumption (Kazmierowski et al., 1992; Thenoux et al., 2007). This reduction is largely achieved by avoiding aggregate drying (Defence Estates, 2009) and mixing of the material at ambient temperature instead of 170°C-185°C, as required for hot mix asphalt (HMA) (Defence Estates, 2005). This technique involves binders that can be either cementitious (hydraulic binders) and/or bituminous (Jitareekul, 2009).

The use of bituminous binders produces a flexible layer with superior fatigue performance to those with purely cementitious binders (Kearney and Huffman, 1999). Bituminous binders are typically applied as foamed or emulsified bitumen (Jitareekul, 2009). This study looks at cold recycled materials bound with bitumen classified in the UK as a cold recycled bound material (CRBM) (Defence Estates, 2009; Merrill et al., 2004).

3.1 RECLAIMED ASPHALT PAVEMENT (RAP) FOR COLD MIXES

After a certain time in service pavements need to be maintained and rehabilitated. In some cases, asphalt layers have to be replaced and for this purpose they are removed from the pavement through milling. The by-product obtained after milling is called Reclaimed Asphalt Pavement (RAP) and it is the primary recycled material of aged asphalt pavements (Al-Qadi et al., 2007; Botasso et al., 2008; West, 2010).

The main drivers for using RAP include economic savings and environmental benefits. This is because its use reduces the requirement for new materials needed in pavement construction, and it reduces the need for disposal too (Al-Qadi et al., 2007).

The existing specifications for pavement construction permit a wide range of reclaimed materials to be used in place of primary aggregate. In Table 3 a summary of recycled material applications is shown. This table, taken from highways-related advice, is only for general guidance.

Table 3 *Guidance of recycled materials applications* (The Highways Agency, 2004)

Application and Series	Pipe Bedding	Embankment and Fill	Capping	Unbound Mixtures for Subbase	Hydraulically bound Mixtures for Subbase and Base	Bitumen Bound Layers	PQ concrete
Material	500	600	600	800	800	900	1000
Blast furnace Slag	✓	✓	✓	✓	✓	✓	✓
Burnt Colliery Spoil	X	✓	✓	✓	✓	X	X
China Clay Sand/Stent	✓	✓	✓	✓	✓	✓	✓
Coal Fly Ash/Pulverised Fuel Ash (CFA/PFA)	✓	✓	✓	X	✓	✓	✓
Foundry Sand	✓	✓	✓	✓	✓	✓	✓
Furnace Bottom Ash (FBA)	✓	✓	✓	X	✓	X	X
Incinerator Bottom Ash Aggregate (IBAA)	✓	✓	✓	✓	✓	✓	✓
Phosphoric Slag	✓	✓	✓	✓	✓	✓	✓
Recycled Aggregate	✓	✓	✓	✓	✓	✓	✓
Recycled Asphalt	✓	✓	✓	✓	✓	✓	✓
Recycled Concrete	✓	✓	✓	✓	✓	✓	✓
Recycled Glass	✓	✓	✓	✓	✓	✓	X
Slate Aggregate	✓	✓	✓	✓	✓	✓	✓
Spent Oil Shale/Blaise	X	✓	✓	✓	✓	X	X
Steel Slag	✓	✓	✓	✓	✓	✓	X
Unburnt Colliery Spoil	X	✓	X	X	✓	X	X

In the specific case of asphalt recycling it has to be said that there is no restriction (if tar is not present); however, it is most often used in:

- Unbound mixtures for sub-base (Series 800) (Department for Transport, 2016b).
- Hydraulically bound mixtures for sub-base and base (Series 800) (Department for Transport, 2016b).
- Bitumen bound layers (series 900) (Department for Transport, 2008).

For the use of Recycled bound materials in Airfields the Specification 050 has to be taken into account (Defence Estates, 2009).

3.1.1 Manufacturing and Installation processes for cold recycling

There are two main manufacturing processes (European Asphalt Pavement Association, 2005):

- **Off-site recycling or Central plant recycling.** Consists of removing the material from the site to a plant located elsewhere which recycles the reclaimed asphalt in order to re-use it either on the original project or on other projects. The cold mix technology in an off-site central plant consists of the same controlled crushing and screening process as for hot mix recycling, to produce a consistent feedstock. Two types of binder, foamed and bitumen emulsion have been used with the recycled asphalt. The methods are both able to accommodate over 90% of recycled asphalt, producing material at a low energy cost with an appropriate design life.
- **In-situ recycling.** This process allows the reclaimed material to be incorporated directly back into the new asphalt pavement under construction or maintenance. The same two techniques adapted for off-site recycling were originally developed for in-situ recycling using specialised plants. The bitumen emulsion based system involves the scarification, bitumen emulsion mixing and compaction of RAP before overlaying with a new wearing course. The foamed bitumen process requires the use of an improved milling machine, which pulverises the existing pavement in a hood that also acts as a chamber in which the bitumen is foamed and mixing takes place. The recycled pavement is then graded ready for compaction and the application of a new binder layer and running surface.

The installation process involves pavers, rollers and lorries as with HMA; however, a milling machine is needed to remove the asphalt layers that are going to be recycled. In the off-site process the milling machine removes the asphalt layers and a lorry brings the material to the recycling plant. In-situ and off-site processes are shown in Figure 4 and Figure 5.

There are weather limitations in the installation process: the minimum air temperature for cold recycling with emulsions ranges from 10°C to 16°C with no temperature less than 0°C expected during the next 24 hours. In most countries, work is not allowed during rain or when fog is present (Kearney, 1997). In addition, it has limited application during night works. The reason for these limitations is that the emulsion is required to break during compaction. Low temperatures and/or excessive water all inhibit the process of the emulsion breaking resulting in an asphalt which is unlikely to achieve its desired mechanical performance.

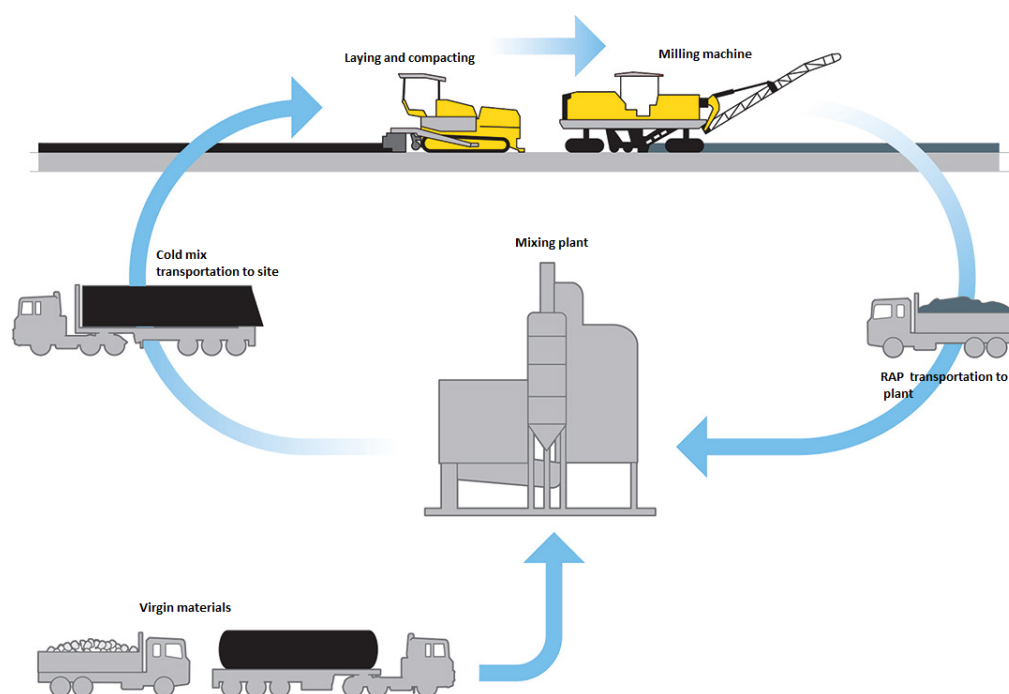


Figure 4 Cold ex-situ recycling asphalt (Atlas Copco, 2017)

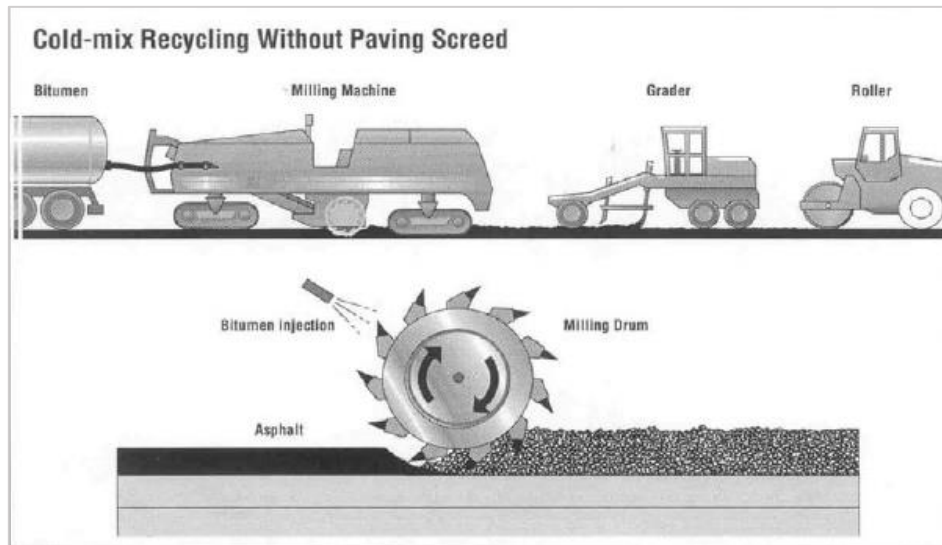


Figure 5 Cold in-situ recycling asphalt installation
(European Asphalt Pavement Association, 2005)

Since even a well-compacted cold recycled mix could have 8% to 15% air voids (Vorobieff and Preston, 2004), a surface course or seal is needed. Previous research show how these voids are small and not interconnected (Arambula et al., 2007); therefore, they should not affect mechanical performance and durability as in HMA. However, further research is needed to understand how these voids affect long-term performance of these materials (Nunn and Thom, 2002). For emulsion recycled mixes, a one to two week cure time may be necessary to allow the moisture content to decrease to acceptable levels prior to overlaying (Kearney, 1997). However, industry experience in the UK is that the mixture can be designed for immediate overlay; especially when the weather and temperature is favourable for the emulsion breaking quickly during compaction.

3.1.2 Properties and performance of RAP

The properties of RAP are largely dependent on the properties of the constituent materials and the type of asphalt mix (wearing course, binder course, etc.). There can be substantial differences between aggregate quality, size and consistency.

For example, since the aggregates in surface courses must have high resistance to abrasion to contribute to acceptable friction resistance properties, these aggregates may be of higher quality than those used in binder course applications, where polishing resistance is not of concern (Recycled Materials Resource Center, 2012).

- **Size distribution:** the aggregate degradation caused by milling and crushing the old asphalt must be borne in mind. The gradation of milled RAP is generally finer and denser than that of the virgin aggregates. Crushing does not cause as much degradation as milling; consequently, the gradation of crushed RAP is generally not as fine as milled RAP; however, it is finer than virgin aggregates crushed with the same type of equipment. The particle size distribution of milled or crushed RAP may vary to some extent, depending on the type of equipment used to produce the RAP, the type of aggregate in the pavement, and whether any underlying base or subbase aggregate has been mixed in with the reclaimed asphalt pavement material during the pavement removal. Most sources of RAP will be a well-graded coarse aggregate, comparable to, or perhaps slightly finer and more variable, than crushed natural aggregates, as milling and crushing can cause some aggregate degradation.
- **Hardening:** this depends on several factors such as mixing temperature/time, compaction, air voids and years in service. This modifies binder properties, giving higher viscosity and lower penetration values.

Despite all the benefits derived from using RAP, uptake is still limited because of the uncertainties of its performance. The reluctance to use higher contents of RAP in pavement rehabilitation lies in the possibility of premature fatigue and low temperature cracking failure due to the presence of aged bitumen (Zaumanis et al., 2014).

It is known that RAP mixtures usually show at least equivalent resistance to rutting to conventional mixtures because of the higher stiffness of the aged binder (Hajj et al., 2014). In terms of fatigue cracking, mixtures containing RAP display ambiguous performance. There are several studies in this field and it seems that depending on the criterion used, RAP can either increase or decrease fatigue life of the pavement (Shu et al., 2008).

3.1.3 Recyclability of a pavement

The criteria for ascertaining the recyclability of a pavement depend on identification of its deterioration causes, in order to develop a suitable design. Here, the recyclability of materials is not a primary concern and the focus is on if it is sensible/practical to recycle a pavement. This process typically includes an assessment of existing pavement condition, sampling and testing of base, subbase and subgrade materials and a study of the pavement construction and maintenance history and traffic (Kearney, 1997; Oke, 2010).

Most types of pavement distress can be rehabilitated by cold recycling. However, cracked pavements with structurally sound, well drained bases and subgrades are the best candidates. In Table 4 a guideline for preliminary selection of roads that are candidates for cold recycling is shown (Oke, 2010).

In some countries, with bad base, subbase and subgrade conditions, successful cold recycling has been accomplished by using Portland cement or self-cementing Class C fly ash as the stabilizing agent. However, this is not a practice in the UK.

Cold in-plant recycling is often the more cost effective where extensive base, subbase and subgrade problems are present. In this case, the failed asphalt is removed and the poor quality underlying materials can be improved. This can be on a single site or a group of sites with a centralised recycling area.

Table 4 Guideline for the preliminary selection of cold recycling candidates

Pavement distress mode/ Cause	Most appropriate  Least appropriate
Ravelling	✓
Potholes	✓
Bleeding	✓
Skid Resistance	✓
Shoulder Drop Off	✓
Rutting	✓
Rutting caused by too high asphalt content and/or fine graded aggregates	✓
Corrugations	✓
Shoving	✓
Fatigue Cracking	✓
Edge Cracking	✓
Slippage Cracking	✓
Block Cracking	✓
Longitudinal Cracking	✓
Transverse Cracking	✓
Reflection Cracking	✓
Discontinuity Cracking	✓
Swells	✓
Bumps	✓
Sags	✓
Depressions	✓
Ride Quality	✓
Strength	✓
Wet, unstable base, subbase and subgrade materials	✓
Heaving or swelling	✓
Stripping of the asphalt from the aggregate	✓

To improve cold recycled materials new aggregates, Portland cement, lime or fly ash can be added, to increase early strength and also freeze-thaw resistance.

An additional reason to recycle RAP in cold mixes is because UK pavements constructed prior to 1980, or surfaced in the late 1980's, may contain tar (County Surveyors Society, 2008). As stated in the introduction, tar is a potentially hazardous material and it is classified as "Hazardous Waste". Due to a combination of health and environmental reasons, tar cannot be recycled into unbound or HMA applications.

When materials cannot be recycled into HMA or unbound applications, cold recycling is still a viable solution due to inhibiting the risk of any leaching and or health risk associated with the use at elevated temperatures as encountered during the manufacturing of HMA.

3.1.4 Mix design and properties

After deciding which mixture type will be used in a project and specify it, the mix design is then undertaken to achieve desired properties. The basic steps for the mix design process can be outlined as (Jitareekul, 2009; Kearney, 1997):

- Obtaining representative samples from the field from reclaimed materials for use in mix design.
- Processing samples for use in mix design:
 - Evaluation of recycled pavement (bitumen content, bitumen physical properties, aggregate gradation and recycled pavement gradation).
- Selection of amount and gradation of new aggregate.
- Selection of bitumen required (penetration).
- Selection of type and amount of recycling agent.
- Mixture, compaction and testing in the laboratory.
- Establishment of job mix formula and acceptable tolerances.

Typically, the mix design is undertaken on pavement cores or bulk samples that are crushed in the laboratory to obtain a representative sample from the field. Alternatively, when time permits, samples of crushed stockpiled RAP can be sampled directly for the mix design process. In either scenario, these retrieved samples are used to establish pavement thickness, asphalt content, aggregate gradation and penetration and viscosity of the recovered asphalt. This information assists in determining the correct type and amount of bitumen and if the reclaimed asphalt needs to be blended with any filler and/or additional fine or coarse aggregate (Jitareekul, 2009; Kearney, 1997).

To determine the quantity of new bitumen that needs to be added, Emery (1993) (Jitareekul, 2009) suggested that moisture content, gradation of RAP, asphalt binder content and penetration should be determined.

Bitumen bound materials are highly sensitive to RAP grading. Milton and Earland (Jitareekul, 2009) , TRL report 611 ((Merrill et al., 2004) and the Defence Estates specification 050 (Defence Estates, 2009) recommend three grading envelopes as shown in Table 5.

Table 5 Recommended particle size distribution of aggregates for cold recycling

Sieve size (mm)	Percentage by mass passing		
	Zone A	Zone B	Zone C
40	100	100	100
31.5	100	100	86-100
20	100	100	75-100
14	85-100	85-100	52-100
10	68-100	68-100	44-100
4	38-74	38-94	26-74
2	25-58	26-84	18-58
0.5	13-38	13-64	8-38
0.25	9-28	9-51	5-28
0.063	5-21	5-38	3-21

Particle size distribution should preferably conform to zone A; however, zone B and C can be used where evidence can be provided of satisfactory performance (Merrill et al., 2004). In addition, zone C is recommended only for in situ stabilisation. Additional new aggregate may be needed to correct gradation or increase thickness of the recycled pavement layer. Aggregate gradation for this investigation was established to comply with zone A.

As mentioned in section 3.1.3, Portland cement, lime or fly ash can be added to the mixture. Typically, between 1% and 3% by dry mass of cement is added during the recycling process when the stabilising agent is used (foam or emulsion) (Lewis and Collings, 1999; Merrill et al., 2004).

The most important performance indicators of cold recycled materials include:

- Indirect tensile strength (or Marshall properties)
- Fatigue resistance
- Durability (i.e. water resistance)
- Density and air voids

Once the mix design is done the next step is manufacturing the mixture. In cold recycling, mixing is limited by the existence of asphalt granules glued together by an aged binder. This is why it is very important to have a proper mixture process (Karlsson and Isacsson, 2006).

3.1.5 Structural design

There is still no widely accepted method for structural design of cold recycled pavement available, as stated in previous sections. However, some design procedures can be applied (Jitareekul, 2009; Ojum, 2015):

- Catalogue design: based on assumed properties of the materials
- CBR design method

- Asphalt institute method
- AASHTO method
- TRL report TRL611: for road pavements (primarily based on low to medium traffic levels)
- Defence Estate guidance (Airfields)

The minimum asphalt pavement thickness to be cold recycled should be 50 mm for a CRBM with 20 mm down aggregate sizes. This is based on practical experience where thinner layers are difficult to mix and compact.

3.2 EMULSION AND FOAMED BITUMEN

An important factor of CRBM is the selection of the binders (Jitareekul, 2009) for good mixing and coating of aggregates that will provide strength and durability to the pavement (Meocci et al., 2017; Ojum, 2015). These binders can be classified into hydraulic (i.e. cement, hydrated lime or fly ash) or bituminous binders (bitumen emulsions or foamed bitumen) (Jitareekul, 2009; Ojum, 2015).

Cold recycled materials can be classified into three families depending on the binders used: hydraulic binders, visco-elastic binders and visco-elastic/hydraulic binders. In Figure 6 these families are represented in a ternary diagram based on three types of materials (apexes in the diagram): fully hydraulic bound, fully visco-elastic bound and unbound (Merrill et al., 2004).

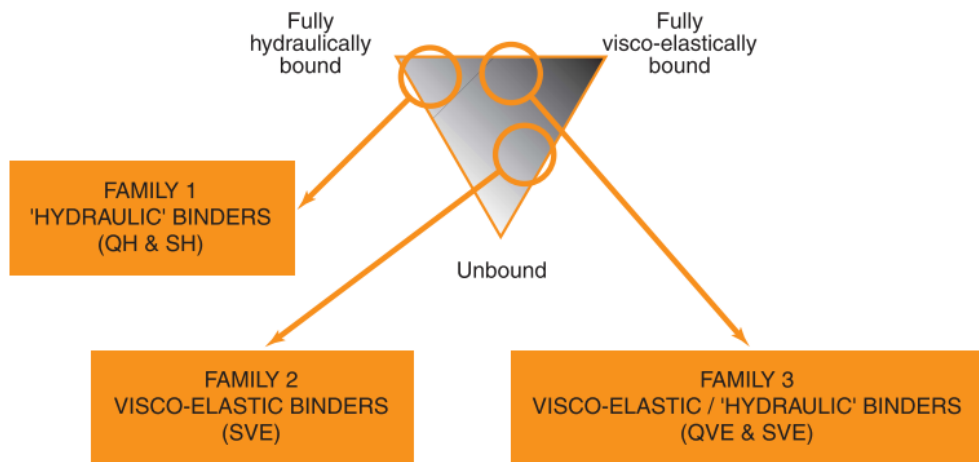


Figure 6 Cold recycled materials families (Merrill et al., 2004)

According to the binder type and the curing behaviour, cold recycled materials can be characterised by areas within this chart, having four types of materials falling into the three families (Merrill et al., 2004):

- Quick hydraulic (QH): hydraulic binder only, including cement (Family 1)
- Slow hydraulic (SH): hydraulic binder only, excluding cement (Family 1)
- Quick visco-elastic (QVE): bituminous and hydraulic binder(s) including cement (Family 3).
- Slow visco-elastic (SVE): bituminous only or bituminous and hydraulic binder(s) excluding cement (Family 2 and 3).

As stated before, CRBM can use a combination of hydraulic (lime, cement) and bituminous binders (foam or emulsion) to achieve the necessary long-term structural and durability properties (Graziani et al., 2016).

Hydraulic binders are used to disperse the foam bitumen or regulate the emulsion breaking, to accelerate curing processes and to provide the initial strength to the mixture (Meocci et al., 2017; Nivedya and Veeraragavan, 2017). Using cement-bitumen CRBM makes it possible to use up to 100% RAP. However, virgin aggregates can be added to meet specifications and achieve mechanical properties (Al-Jumaili and Issmael, 2016; Graziani et al., 2016; Meocci et al., 2017; Nivedya and Veeraragavan, 2017). In previous studies, optimum dosages of bitumen and cement were found to be between 1 and 3% and 1 to 2.5% respectively (by aggregate weight) (Meocci et al., 2017). In reality it is important to limit the addition of cement to avoid creating a HBM which would ultimately be prone to cracking, rather than a CRBM. The specification guard against this by specifying an upper limit of stiffness (Defence Estates, 2009), which particle limits cement additions to between 1 and 2% by dry aggregate weight.

In CRBM mixes, water is added to the aggregates to ensure a good dispersion of the foam/emulsion, to allow lay down and compaction and to activate any hydraulic binder (Graziani et al., 2016; Nivedya and Veeraragavan, 2017).

Curing is important as CRBM evolves over time, improving mechanical properties until reaching the long-term state (Graziani et al., 2016).

In the next section foamed bitumen and bitumen emulsions are reviewed in order to select the best stabilising agent for this research.

3.2.1 Foamed bitumen

Foamed bitumen is produced by injecting air and water droplets (1 to 6%) under high pressure (5 bars) into hot (140-180°C) liquid bitumen, resulting in bitumen taking the form of foam as schematically illustrated in Figure 7. The volume of bitumen will increase, up to 18-20 times (Nivedya and Veeraragavan, 2017), while viscosity considerably reduces (Jitareekul, 2009).

Typically between 3% and 5% of foam bitumen is added; however, when the recycled material contains a high proportion of bitumen, this can be reduced to 2% or 3% (Botasso et al., 2008; Jitareekul, 2009; Oke, 2010).

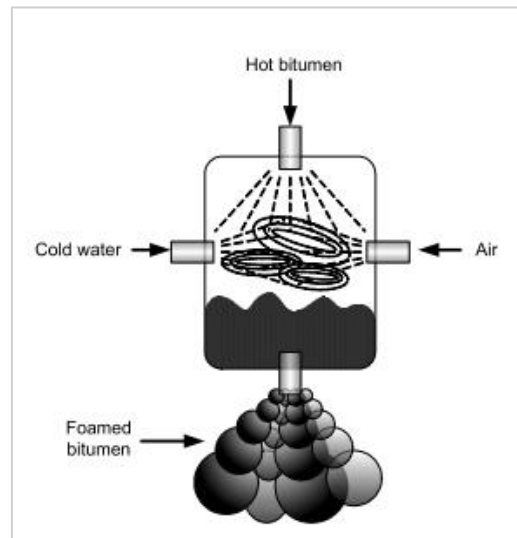


Figure 7 *Foamed bitumen production*
(Jitareekul, 2009)

Bitumen used for foaming should be grade 160/220 or harder in the UK as specified in TRL Report 611 (Merrill et al., 2004). Higher penetration grade bitumens tend to foam better (Milton and Earland, 1999), as they are softer, but they can result in low stiffness mixtures prone to permanent deformation (rutting). Lower grades produce material of higher stiffness (Milton and Earland, 1999).

The best option is to find a balance between both characteristics: coating and mechanical performance of final product. Milton and Earland (Milton and Earland, 1999) recommend a bitumen penetration grade of around 100, achieving high stiffness and reasonable coating. Bitumens suitable for foaming tend to be proprietary products sold by bitumen companies, as they come with descriptions which include suitability for foaming process.

Foamed bitumen is characterised by its expansion ratio (ER) and half-life (HL). ER is the ratio between the maximum volume the foam achieves and the final volume after the foam has decayed. HL is the time in seconds it takes the foam to collapse from the maximum volume to half of that volume (Jenkins, 2000). The greater ER and HL the better coating of particles is achieved; therefore, the more cohesion is found in the mix (Nivedya and Veeraragavan, 2017). Wirtgen manual recommends a half-life greater than six seconds and an expansion ratio greater than 8 (Wirtgen GmbH, 2012).

The optimum foaming water content (FWC) and temperature can be found through an experiment to determine which would produce the highest ER and longest HL. However, it may not necessarily produce the optimum foamed asphalt mixture. The amount of foamed bitumen should be adjusted depending on the existing pavement temperature during construction, as higher RAP temperatures may help foamed bitumen dispersion, allowing optimum behaviour with less bitumen amount (this is very important in cold in-situ recycling) (Lee and Kim, 2006). Sunarjono found optimum mixture performance using between 2% and 4% of FWC (Sunarjono, 2008). However, other investigations find different optimum FWC between 1% and 3% (Gui-ping and Wing-gun, 2007; Lee and Kim, 2006; Li et al., 2016; Yan et al., 2010). Increasing foaming temperature can reduce the optimum FWC (Kim et al., 2006) and it also depends on aggregate type, especially water absorption, and bitumen penetration grade; therefore, the foaming water content should be optimised.

There are different mix designs for CRBM using foamed bitumen depending on the country, the application and the RAP used. However there are several factors to take into account when developing this mix design: quantity of foamed bitumen, foaming water, RAP type and gradation, inclusion of virgin aggregates, active fillers, aggregate moisture content, and suitable temperatures (Hainin et al., 2014; Nivedya and Veeraragavan, 2017).

Depending on this mix design, the material will be SVE or QVE, behaving more closely to unbound materials (SVE) or to hydraulically bound materials (QVE or SVE). In Figure 8 these behaviours are represented along with their failure mechanism (rutting or fatigue).

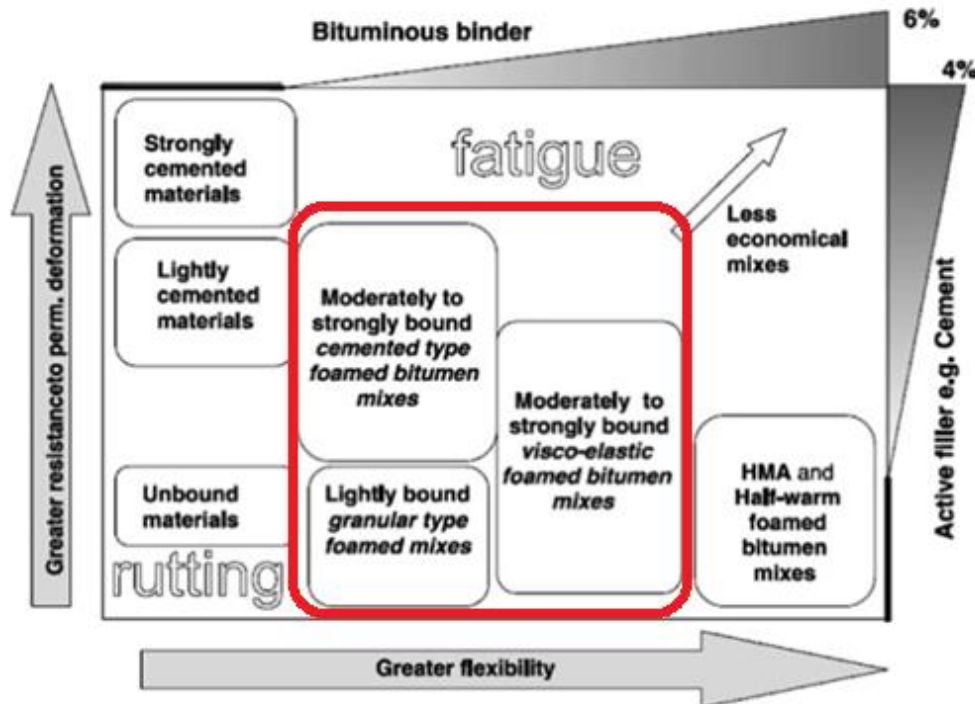


Figure 8 Foam mix behaviours (Nivedya and Veeraragavan, 2017)

The mix design can be performed using the indirect tensile test, the Marshall stability test or compaction tests. The optimum water content and foamed bitumen content can be determined from the results of these tests (Kim et al., 2006).

Valentin et al. (Valentin et al., 2016) found an optimum foamed bitumen content between 2 and 2.5% when high contents of cement are used (moderately to strongly bound cemented type foamed bitumen mixes); however, the optimum binder content is between 2-3.5% when using lower cement content (moderately to strongly bound visco-elastic foamed bitumen mixes).

RAP content plays a key role in CRBM. It has been found that high RAP contents improve permanent deformation of mixes; however, strength is reduced (Nivedya and Veeraragavan, 2017). Hainin et al. (Hainin et al., 2014) reported that this decrease of strength with the increase of RAP was due to the low fines content of the mixture. Therefore, it is recommended to adjust the grading curve adding fillers when using high RAP contents.

Previous investigations concluded that the optimum foamed bitumen content depends on the fines content (Hainin et al., 2014; Nivedya and Veeraragavan, 2017). The inclusion of fines can partly substitute cement which is advantageous (Valentin et al., 2016)

Active fillers are defined as fillers that chemically alter the mix properties. The active filler used with foamed bitumen are hydrated lime, cement, fly ash and slag (Wirtgen GmbH, 2012). These fillers are used to improve the bitumen dispersion in the mix, accelerate the strengthening of the mixture, make the mixture less sensitive to moisture content variations which affect the compactability of the mixture and accelerate the curing (Betti et al., 2016; Wirtgen GmbH, 2012).

Active fillers are reported to have greater relative influence on mix performance than bitumen content (Valentin et al., 2016). For example, the addition of 1% cement has a marked effect on curing rate and final strength which brings the stiffness of the mixture comparable to that of hot mix and improves water resistance. Therefore, the addition of a small amount of cement is then recommended (Jitareekul, 2009).

However, the use of cement can reduce the mixture flexibility going from a visco-elastic to a hydraulically bound material behaviour (Valentin et al., 2016); hence, a maximum cement content should be established (see Figure 8).

CRBM need to be allowed to cure (process explained in section 3.2.5.) (Li et al., 2016). However, foamed bitumen needs relatively less curing time than emulsions; therefore, the pavement construction is faster (Nivedya and Veeraragavan, 2017). Mechanical properties improve rapidly at the early stages of the curing period, during the first 7 days, and improve gradually after that (Hainin et al., 2014; Li et al., 2016).

The strength of CRBM with foamed bitumen is influenced by factors such as bitumen stiffness and content, cement content, moisture level, compaction (density and air voids), gradation and curing time. Assuming curing conditions are constant, strength increases with curing time and percentage of cement (Graziani et al., 2016; Li et al., 2016).

Optimum moisture content (OMC) increases with lower RAP contents due to the need of more fluid to pack the aggregates. Also, Hainin et al. (Hainin et al., 2014) found that the maximum values of strength are found at 85% of OMC and that with high contents of RAP, the variation of moisture content ($\pm 30\%$) does not affect strength significantly.

As a bitumen bonded granular material, the stiffness or resilient modulus of a foamed asphalt mix depends on both the stress state and temperature. It is important to use a test method where temperature and stress state can be controlled independently, to study the effects of the temperature sensitivity of foamed asphalt (Fu and Harvey, 2007).

In terms of fatigue performance of CRBM using foamed bitumen, contradictory results are found in different researches. Laboratory studies using Indirect tensile fatigue testing shown that foamed bitumen mixtures have lower fatigue life compared with HMA (Jitareekul, 2009; Sunarjono, 2012; Twagira et al., 2006).

However, Sunarjono reveal that foamed CRBM initiate cracking faster than HMA but the crack propagation is slower (Sunarjono, 2012). A full-scale trial carried out by the Transport Research Laboratory (TRL) showed how after several months of curing, the fatigue characteristics of the CRBM was similar to those of a conventional HMA. This highlights the need of further research in this area.

Foamed bitumen has been used with success in roads (Merrill et al., 2004; West and Willis, 2014) and some airfields (Cardno, 2012; Lacalle Jiménez and Tuck, 2015), having comparable strength and stiffness to conventional HMA after a period of curing (Lacalle Jiménez et al., 2016). Therefore, giving the pavement the necessary structural performance to bear the expected traffic for a given design life (Sunarjono, 2008). However, there are still gaps of understanding how CRBM behave.

To summarise, foaming water content or foamed bitumen content should always be optimised as different researches show different results on mixture performance when varying these parameters. RAP content is also a key point of discussion. Some researches show how using high contents of RAP can decrease the mixture performance; however, other researches show how this issue could be avoided by adjusting the grading curve and adding fines to the mixture. The use of cement (or active fillers) has shown improvement in mixture performance and curing time; nevertheless, high cement contents reduce mixture flexibility increasing the risk of cracking. Therefore, a maximum cement content should be established (from field experience at 2%). Finally, fatigue investigation should be improved, as different researches give very different results.

3.2.2 Bitumen emulsions

Bitumen emulsion is a dispersion of bitumen droplets in water (Fang et al., 2015) and it is manufactured by emulsifying bitumen (typically 60%) and water (typically 40%), using emulsifying agents (Lewis and Collings, 1999). This system is design to be unstable and emulsion breakage and bitumen coalescence is typically designed to occur during compaction of the materials (Fang et al., 2015). This process can be accelerated with the evaporation of water, heating, pH change or addition of solid particles (Fang et al., 2015; Graziani et al., 2016). Bitumen emulsions are liquid at ambient temperature; therefore, they can be easily dispersed in the mix (Al-Jumaili and Issmael, 2016)

There are three types of bitumen emulsion: anionic, cationic and non-ionic. Cationic emulsions have better adhesion properties and coat the aggregates better; therefore, this type of emulsion is preferred for asphalt mixtures (Dubai, 2014).

Emulsions are characterised by the binder content, breaking behaviour (rapid, medium or slow), viscosity, penetration power and storage stability among others (British Standard, 2012a). The optimum bitumen emulsion content in the mix is reported to be between 2.5 and 3% by Valentin et al. (Valentin et al., 2016). However, other studies report the use of 5% or 6.5% bitumen emulsion (Ojum, 2015; Oke et al., 2014). This highlight the importance of carrying out an optimisation to adjust the bitumen emulsion content in the mix.

Bitumen emulsion compatibility with aggregate depends on pre-wetting water content and optimum total fluid content (OTFC). These are important parameters that must be ascertained for cold bituminous emulsion mixtures to achieve a successful cold mix design (Oke, 2010). This parameter has to be optimised too.

The Indirect Tensile Stiffness Modulus (ITSM) test and Indirect Tensile Strength (ITS) test are useful indicators for predicting CRBM with bitumen emulsions performance as they measure tensile properties and are easy to conduct (Al-Jumaili and Issmael, 2016; Oke, 2010). With the ITSM and ITS, materials can be tested for water susceptibility too (Department for Transport, 2008; Oke, 2010).

Mixtures manufactured with RAP and bitumen emulsion tend to have low early-stage strength and low resistance to permanent deformation (Al-Jumaili and Issmael, 2016; Taherkhani et al., 2016) delaying opening to traffic. However, Grave emulsions (densely graded mixes) can be trafficked immediately after compaction due to the high internal friction and cohesion which exists within the system in early life (Needham, 1996). In addition, Al-Jumaili and Issmael (Al-Jumaili and Issmael, 2016) found acceptable moisture susceptibility for high RAP contents in early life.

There are several studies on how to improve the performance of asphalt mixes with bitumen emulsions using additives and fillers (Nassar et al., 2016). Cost, performance and climate conditions should be addressed to choose the best type of additive (Taherkhani et al., 2016).

It has been proved that cement can improve early stage mechanical performance (Al-Jumaili and Issmael, 2016; Oke, 2010), resistance to permanent deformation, moisture susceptibility and strength (Fang et al., 2015; Lewis and Collings, 1999; Niazi and Jalili, 2009; Taherkhani et al., 2016). In many cases adding between 1% and 3% of cement to the mix improves mechanical behaviour and accelerates the curing process (emulsion breaking) (Fang et al., 2015; Lewis and Collings, 1999; Nassar et al., 2016; Taherkhani et al., 2016). However, adding cement to the mixture can reduce the flexibility and, therefore, increase the susceptibility to cracking of the mixture as with foamed bitumen mixtures (Taherkhani et al., 2016). This means that also for bitumen emulsion mixture, a maximum cement content should be established.

There is still a lack of understanding on how the use of hydrated lime, fly ash or other chemical materials affects the bitumen emulsion in the mixture (Al-Jumaili and Issmael, 2016; Nassar et al., 2016). However, limestone has been known to be good at inhibiting stripping (due to its adhesion with bitumen), can increase Marshall Stability, resilient modulus, tensile strength, resistance to moisture damage and resistance to permanent deformation (Fang et al., 2015; Nassar et al., 2016; Niazi and Jalili, 2009). Nonetheless, Fang et al. found the influence of cement in the coalescence and flocculation of bitumen emulsions, but limestone did not show any such effect (Fang et al., 2015).

Other studies had found that granite filler gave a better dry stiffness property compared to limestone (Niazi and Jalili, 2009). The use of fly ash improves the mechanical properties and water sensitivity of the mixture (Nassar et al., 2016).

For improving durability, cracking resistance and moisture damage susceptibility the use of cement and polymer modified bitumen emulsion has been studied (Taherkhani et al., 2016).

The presence of RAP has been proved not to affect the compactability of the mixture (Grilli et al., 2016) but inadequate mixing time can significantly reduce compactability and stiffness response (Oke, 2010).

Regarding the curing process, it has been proved that bitumen emulsions need longer curing times (Al-Jumaili and Issmael, 2016). This curing time will depend on the mix design (type of aggregates and additives), weather conditions, type and amount of emulsifier and water content (Ojum, 2015).

The air void content depends on the moisture content, packing, filler, compaction (effort and technique) and mixing and compaction temperature (higher temperature gives lower air void content) (Oke, 2010). CRBM with bitumen emulsion have higher air voids than foamed bitumen mixtures; however the reason why is not clear in the literature (Yan et al., 2010).

Summarising, in regards to bitumen emulsions, optimum water content and bitumen emulsion content should be optimised to obtain the best mixture performance. The addition of cement to the mixture improves mechanical performance; however, a maximum cement content should be established to assure the visco-elastic behaviour of the mix. There is still lack of understanding on how other types of fillers affect the mixture properties. Good results are obtained when using emulsion with RAP and curing time and air voids content are higher in this type of mixtures than those with foamed bitumen.

3.2.3 Summary of the findings related to foamed and emulsion bitumen

In Table 6 and Table 7 a summary of foamed bitumen and bitumen emulsion findings is presented. It can be concluded that both foamed and emulsion bitumen mixtures performance is improved by the use of cement and other additives or fillers. Both technologies need an optimisation process in the mix design (optimum water content, optimum binder content, grading and compactability). In the case of bituminous emulsions, researchers are concerned about the early-stage behaviour of the mix and the longer curing period.

Table 6 *Key points for foamed bitumen*

Recommendations
The use of RAP can reduce the added bitumen content to 2-3%
The ER and HL have to be checked to optimise foaming process
Optimum water content should be optimised for each mixture
ITSM, ITS and Marshall stabilisation tests can be used for optimising the mix design
RAP improves permanent deformation
A small amount of cement improves performance of the mix
High amounts of cement can increase the brittleness of the mix
Relatively shorter curing times compared with emulsions
Test temperature influences results

Table 7 Key points for bitumen emulsions

Recommendations
Cationic emulsions preferred for asphalt mixtures
Optimum bitumen emulsion content between 2.5-3%
Need to optimise OTFC and pre-wetting water content
High contents of RAP improve moisture susceptibility
Typically, higher in situ voids than foamed
ITSM and ITS used to predict mechanical performance of the mix
Cement improves mechanical properties
High contents of cement increase brittleness
Other fillers such as fly ash, lime or granite can improve properties, but more research is needed
Polymer modified bitumen used to avoid cracking and moisture susceptibility
Longer curing period than for foamed bitumen
In the early stage, low strength and inappropriate moisture susceptibility and permanent deformation found

3.2.4 Choice of binder: foamed bitumen versus bitumen emulsion

The selection of the most effective stabilising agent for a particular application depends on (Jitareekul, 2009; Lewis and Collings, 1999): price, availability, acceptability and material type.

In Table 8 the advantages and disadvantages of foamed bitumen and emulsion bitumen are summarised.

Table 8 Choice of binder in CRBM

Agent	Advantage	Disadvantages
Bitumen emulsion	Easy to apply, fatigue and water resistance	Emulsifiers are expensive, problems with high moisture content, longer curing period, low early-stage performance
Emulsion + cement	Higher strength, quicker curing and more water resistance	More expensive than emulsion alone, the emulsion can be broken too early
Foamed bitumen	Easy application, good fatigue properties, water resistant, rapid strength gain	Requires supply of hot bitumen (180°C), more restricted grading
Foam + cement	Improved strength and better water resistance than just foam	More expensive than foamed bitumen alone. It has the same temperature and aggregate requirements as foam.

As emulsifiers are relatively expensive (due to the cost of production in a high shear mixer and transportation of at least 40% water), the curing period is longer and the moisture content and the early-stage performance can be a problem, this investigation will focus on foamed bitumen, the most commonly used technique for CRBM in the UK. Cement will be added to the mixture to improve early strength as this is an important property in airfield pavements.

3.2.5 Curing

It is generally accepted that HMA reaches its mature state, i.e. its mechanical properties reach the long term value, after a relatively short time (Serfass et al., 2004). This means that its characteristics can be measured almost immediately after installation (Serfass et al., 2004); however, this is not the case for CRBM mixes (Serfass et al., 2004). Early life CRBM mix mechanical properties change over time (Graziani et al., 2016; Serfass et al., 2004). This phenomenon, during which the cohesion between the binder and the aggregates increases as the mixture loses water, is known as curing (Graziani et al., 2016; Jenkins, 2000; Ojum, 2015). The curing phase, during the construction process, influences the opening to traffic and long term performance of CRBM layers (Li et al., 2016; Ojum et al., 2014). However, as explained in section 3.2.2., some mixtures are able to be trafficked immediately after compaction (Needham, 1996)

Curing involves several mechanisms. For mixtures using foam the curing involves water expulsion and cement hydration. In the case of emulsions, the curing is more complex, involving also emulsion breaking, flocculation and coalescence of bitumen (Graziani et al., 2016).

The objective of curing specimens in the laboratory is to simulate pavement conditions as experienced in the field (Merrill et al., 2004). However, this is not always achievable, since the curing requires a prediction of the variables on site.

It is known that the mechanical and performance properties of CRBM are intimately related to the curing condition (Batista and Antunes, 2003; Li et al., 2016). There are several studies on curing procedures and industrial guidance notes (Department for Transport, 2008; Merrill et al., 2004); however, no standard curing procedure has been established for CRBM and particular curing processes (specially at elevated temperatures) can lead to overestimation of CRBM performance in pavements (Ojum et al., 2014). In several researches it was observed how mechanical properties improve rapidly at the early stages of the curing period, during the first 7 days, and improve gradually after that (Graziani et al., 2016; Hainin et al., 2014; Li et al., 2016)

Moreover, adding active fillers (lime, cement) can improve CRBM properties in both stages accelerating curing by reducing the amount of water to evaporate (Graziani et al., 2016; Li et al., 2016).

These studies highlighted the importance of curing temperature and time, and how moisture loss is not the only mechanism involved in curing. It has been proved that curing specimens fully wrapped (sealed to prevent water loss) at 20°C for 28 days is an appropriately conservative practice to understand CRBM actual performance on site, where a surface course is applied 7 days after construction of the base layer (cold mix) with trafficking commencing 14 days later (Merrill et al., 2004; Ojum et al., 2014).

In addition, sealed curing at 40°C for 3 days is a method that aims to approximate 6 months of curing in the field (Jenkins, 2000). These two procedures will be used in this research.

3.3 COLD RECYCLED MATERIALS CHARACTERISATION: LABORATORY TESTS

In this section, the laboratory tests carried out to characterise the CRBM studied in this Thesis are summarised.

3.3.1 Mixture volumetrics

This is used for analysis of asphalt mixtures and comprises the calculation of:

- Bulk density to BS EN 12697-6 (British Standard, 2012b).
- Maximum density to BS EN 12697-5 (British Standard, 2009).
- Air void content to BS EN 12697-8 (British Standard, 2003a).

These three concepts are related to the compactability of the mixture. The higher the density, the lower the percentage of air voids, which means a greater compaction of the specimen. The parts of the BS EN standards referenced were specifically developed for HMA; however, they do not require any modification for use with CRBM other than the curing period prior testing.

3.3.2 RAP Binder properties

To characterise the binder (bitumen) the first step is to recover it from the RAP or the mixture (BS EN 12697-3 (British Standard, 2013b)), and then two tests are carried out: softening point (BS EN 1427-2007 (British Standard, 2007)) and penetration (BS EN 1426-2007 (British Standard, 2006a)) (see section 1.1.2.).

The softening point (Ring and Ball method) is defined as the mean of the temperature at which two disks of bitumen soften sufficiently to allow a 3.5g steel ball to fall, enveloped in bitumen, for 25 mm.

The needle penetration test measures the depth to which a standard needle penetrates a bitumen sample under specific conditions of load, temperature and time (100 g, 25°C and 5 seconds).

3.4.3 Indirect Tensile Stiffness Modulus (ITSM)

Asphalt material stiffness relates to its load spreading ability and temperature susceptibility, parameters used to assess pavement structural condition. In a structural layer, high stiffness indicates good load-spreading ability.

The determination of the ITSM is carried out as specified in BS EN 12697-26:2004 Annex C (British Standards, 2012).

3.4.4 Repeated Load Axial Test (RLAT)

The Repeated Load Axial Test without confinement (RLAT) to BS DD 226 (British Standard, 1996) assesses the resistance to deformation under dynamic loading.

It should be noted that although there is not a clearly defined threshold value available to interpret the results, this laboratory method is employed to rank different asphalt materials on the basis of resistance to permanent deformation (i.e. % permanent strain). From AECOM's experience, benchmarking against in situ performance, good deformation resistance for dense asphalt materials has been found for materials with measured permanent strain values below 2% for tests carried out at 40°C (Widyatmoko, 2008).

3.4.5 Indirect Tensile Strength Test (ITST)

Determination of Indirect Tensile Strength (ITS) to BS EN 12697-23:2003 (British Standard, 2003b) is used to determine tensile strength, i.e. the strength shown by a specimen subject to tension. This parameter is related to the crack propagation expected in the material.

3.4.6 Indirect Tensile Fatigue Test (ITFT)

Fatigue performance of a material is one of the key parameters used to assess a pavement's expected life, as it is directly related to the crack initiation in the asphalt. With this test the fatigue life can be calculated and plotted. This type of testing is generally not done on standard HMA materials, unless as part of a research or development of specialist products. This is due to the relatively complexity and cost of the testing versus measurement of stiffness or strength following BS EN standards.

The indirect tensile fatigue test is performed to the British Board of Agrément, Guidelines Document for the assessment and certification of thin surfacing systems for highways, App. A.13, BBA, HAPAS SG3 (British Board of Agreement, 2013). This test is carried out in stress control mode and gives information about the number of load repetitions that the material can allow at a certain stress. However; this test does not give information about the failure mechanism or the stiffness evolution during the test; therefore, it cannot be used to obtain the fatigue law necessary to carry out a pavement analysis with Kenlayer.

However, ITFT can be carried out also in strain control mode, giving the allowable number of cycles for a defined strain level. This test provides the stiffness evolution needed to obtain the fatigue law necessary to carry out a pavement analysis with Kenlayer. In addition, this test allows a deeper study of the failure mechanism and crack propagation, useful information to understand CRBM behaviour. The next chapter details how this information is used to understand CRBM behaviour.

3.4.7 Durability

The durability of asphalt mixtures can be defined as the loss of strength due to the impact of climatic conditions and repeated traffic loads (Čížková et al., 2014; Nassar et al., 2016; Wirtgen GmbH, 2012).

There are several tests to evaluate the moisture sensibility of bituminous mixtures. Generally, these tests have a conditioning and an evaluation (typically mechanical testing) phase (Airey and Choi, 2002).

For HMA, the European standard establishes that the measurement of the durability should be assessed by measuring the water sensitivity to BS EN 12697-12:2008 (British Standard, 2012d) where the durability is related to the ratio of ITS of conditioned specimens (soaked in water) to those of unconditioned specimens, expressed in % (Nassar et al., 2016).

For recycled materials for airfields, the moisture sensitivity is analysed by measuring the ITSM (British Standards, 2012) of specimens after 7 days soaked in water at 20°C and relate it to the ITSM of unsoaked specimens (Defence Estates, 2009).

Chapter 4

FAILURE MECHANISM STUDY

As already stated, fatigue is one of the main failure mechanisms in flexible pavements (Moreno-Navarro, 2013; Taherkhani et al., 2016; Twagira et al., 2006), thus, an understanding on CRBM fatigue behaviour is fundamental to developing a trustworthy design methodology. Fatigue behaviour of hot mix asphalt (HMA) has been extensively studied and reported; however, this is not the case for CRBM (Twagira et al., 2006).

There are three generic approaches to obtain fatigue performance of materials: the phenomenological approach, the fracture mechanics approach and the dissipated energy approach (Maggiore, 2014). These are explained below:

- Phenomenological approach: in this case the fatigue is expressed as the relationship between the initial stress/strain and the number of load repetitions (Maggiore, 2014) and the expression is usually not derived from a theoretical analysis (Hamed, 2010).
- Fracture mechanics: relates fatigue life to the three stages of cracking: initiation, propagation and failure. A well-known method for predicting crack propagation is the Paris law (Maggiore, 2014; Paris and Erdogan, 1962).
- Dissipated energy: as asphalt is a viscoelastic material it dissipates energy under mechanical work. It is believed that there is a strong correlation between the change of dissipated energy and the fatigue damage, and this approach also takes into account the evolution of the material during the test (Ghuzlan and Carpenter, 2000; Maggiore, 2014; Shen, 2007).

In the phenomenological approach, researchers tend to use strain control mode since results are often found to be independent of temperature (Maggiore, 2014). A simple relationship between the tensile strain at the bottom of the asphalt layer and the number of load applications was established by Pell (Pell, 1967); however, many researchers have stated that in strain control laboratory tests, the stiffness modulus should be taken into account, establishing equation 2 (see section 2.3.) (Monismith, 1985).

The dissipated energy method reduces the gap between research in the laboratory and field behaviour of the material (Maggiore, 2014). When applying a load to a material energy is being input into that material (Shen, 2007). This energy can be represented as the area under the stress-strain curve (Moreno-Navarro, 2016; Shen, 2007). During the loading-unloading process the energy can be recovered. If the loading curve and the unloading curve are the same, all the energy has been recovered (typical of elastic materials) (Moreno-Navarro, 2013); however, if the curves are different, an energy loss has occurred (typical of visco-elastic materials) (Moreno-Navarro, 2013). This loss means that part of the energy is dissipated in the form of mechanical work, heat generation or damage (Moreno-Navarro, 2013; Shen, 2007). This phenomenon is called “Hysteresis” and the energy change in each load cycle is the dissipated energy. The dissipated energy in each cycle is the area within the hysteresis loop as shown in Figure 9 and can be calculated with equation 4 (Maggiore et al., 2012; Moreno-Navarro, 2013; Shen, 2007).

$$\omega_i = \pi \cdot \sigma_i \cdot \varepsilon_i \cdot \sin\varphi_i \quad [4]$$

Where: ω_i is the dissipated energy (J/m³) in cycle i , σ_i is the stress level (Pa) in cycle i , ε_i is the strain level (m/m) in cycle i , and φ_i is the phase angle in cycle i .

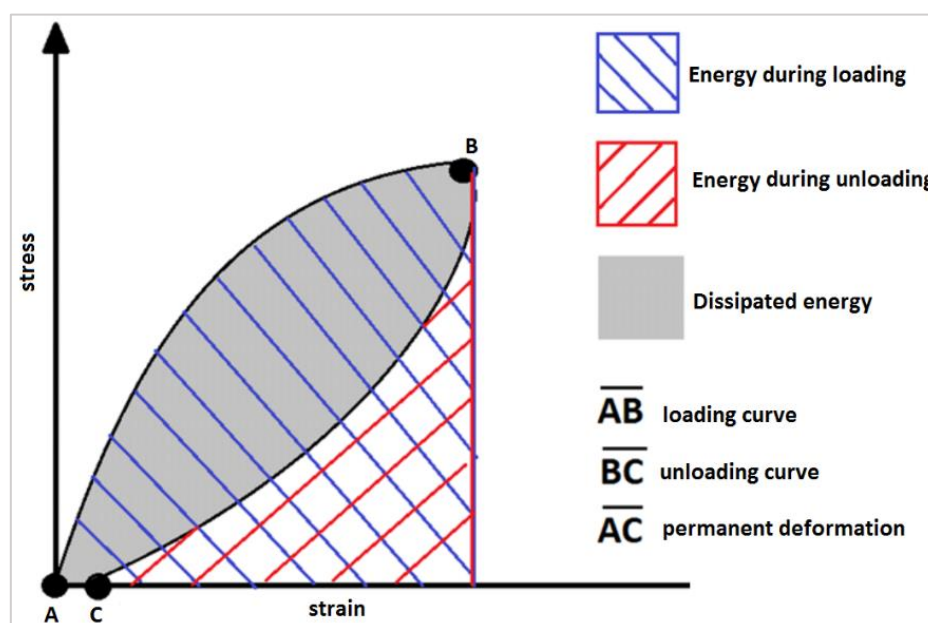


Figure 9 Hysteresis loop

The Indirect Tensile Fatigue Test (ITFT) in strain control mode was selected to obtain the necessary data to carry out this study. The strain control mode was selected as stress control tests are not well-suited for fatigue analysis (Isailović et al., 2016). This test was developed at the University of Nottingham (Maggiore, 2014). It is a relatively simple test and suitable for cylindrical specimens; therefore, the stress distribution is relatively uniform during testing and specimen manufacture can be undertaken using the gyratory compactor which realistically simulates site compaction (Tarefder and Ahmad, 2016). Other tests such as 2-point bending, 4-point bending or flexural beam are common for dissipated energy study or failure criteria (Abojaradeh, 2013; Maggiore et al., 2014); however, the use of ITFT in strain control mode avoids the requirement to cut specimens from a slab, which removes consideration of any micro cracking/coring induced damage to the CRBM specimens, prior to them being tested.

In the literature, several dissipated energy based methods, and related analysis methods, are available to analyse the data obtained from the laboratory tests (Maggiore, 2014). Some are summarised below:

- Initial dissipated energy (IDE) approach: the dissipated energy at the 50th cycle is considered the IDE, as the previous 49 cycles are considered as conditioning cycles (Rowe, 1993). However, this method is not appropriate for fatigue tests for the whole loading range (Carpenter and Shen, 2006; Maggiore, 2014; Shen, 2007) and it doesn't account for the effect of healing (Carpenter and Shen, 2006; Shen, 2007).
- Cumulative dissipated energy (CDE) approach: CDE is the summation of the dissipated energy experienced by the material during the fatigue test (Shen, 2007). It is not affected by the loading mode, frequency or occurrence of rest periods (Van Dijk and Visser, 1977). However, this parameter was found to be mix and temperature dependent (Maggiore, 2014; Shen, 2007); therefore, is not a good parameter to describe the fatigue phenomenon.
- Dissipated energy ratio approach (DER): Pronk (Pronk, 1995) suggested a fatigue life based on the difference between the energy needed for the creation of micro cracks and the energy needed for the growth of these cracks (Maggiore, 2014).
- Work ratio approach: it can be used to predict the fatigue life to crack initiation (Maggiore, 2014; Rowe, 1993; Shen, 2007; Van Dijk and Visser, 1977)
- Ratio of dissipated energy change (RDEC): this approach is an improvement on the DER approach. It is able to eliminate other forms of dissipated energy due to mechanical work or heat generation; therefore, it can be considered as a good parameter to describe the fatigue process (Maggiore, 2014; Moreno-Navarro, 2013; Moreno-Navarro and Rubio-Gómez, 2013; Shen, 2007). It is valid for the whole range of loading, different mixture types, loading modes and loading conditions (Shen, 2007).

- Mean damage parameter: way of quantifying the evolution of cracking in the material during the test (Moreno-Navarro and Rubio-Gómez, 2013).
- Displacement ratio: relates the dissipated energy with the material deformation, allowing the identification of micro-crack initiation and propagation (Moreno-Navarro, 2016).
- The energy stiffness ratio at failure represents material behaviour at which damage accumulation has produced an inability of the mix to resist further damage, and it is independent of the loading mode (Abojaradeh, 2013).

Taking the above discussion into account it was decided to focus on the DER, RDEC, displacement ratio and energy stiffness ratio approaches. However, these analyses can also be complemented with a visual test to confirm analytical results. In this regard, the wheel track test (WTT) was also used to analyse the failure mechanism and crack propagation.

4.1 DER APPROACH

With this approach the crack propagation can be studied (Moreno-Navarro, 2013; Pronk, 1995; Shen, 2007). The dissipated energy ratio can be calculated with equation 5.

$$DER_i = \frac{n_i \cdot \omega_0}{\omega_i} \quad [5]$$

Where DER_i is the energy ratio in cycle i , n_i is the cycle number, ω_0 is the dissipated energy in the first cycle and ω_i is the dissipated energy in cycle i (Pronk, 1995).

With a strain control fatigue test, when representing the DER against the number of cycles, the curve shown in Figure 10 can be obtained. This curve has an initial linear part where the growth of dissipated energy ratio is constant versus the number of cycles. When permanent deformation occurs (cracking) the line develops a rapid increase. N_i represents the initiation of micro crack development in a bituminous material. When this curve arrives to the inflexion point, the macro cracks are starting to develop (N_M) until complete failure (N_f) (Maggiore, 2014; Moreno-Navarro, 2016). It can be seen that the appearance of macro cracks in a HMA sample is related to the failure criterion of 50% stiffness reduction (N_{50} in Figure 10) (Ghuzlan and Carpenter, 2006; Maggiore, 2014).

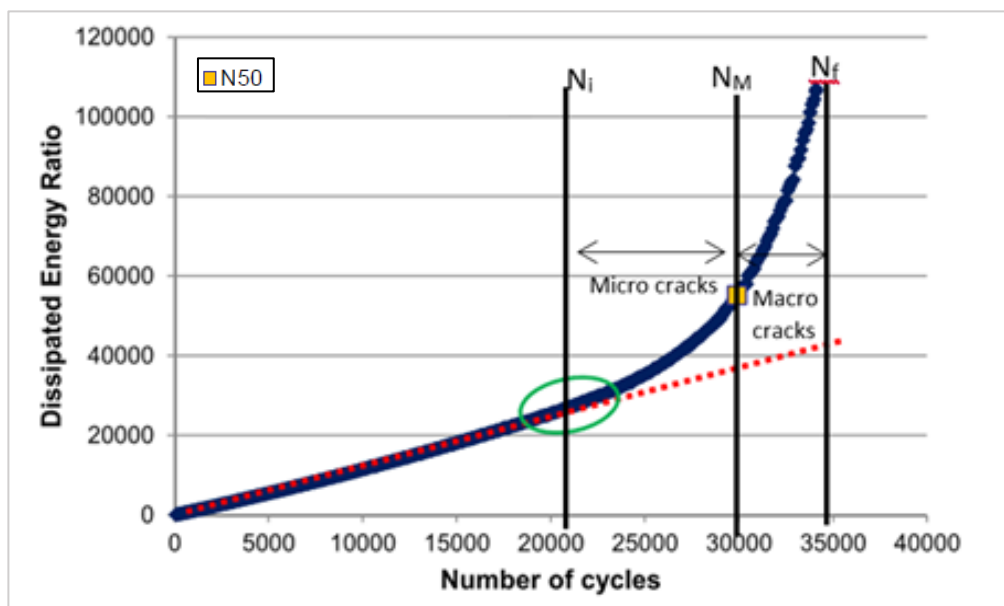


Figure 10 Evolution of DER in strain control fatigue (Maggiore 2014)

4.2 RDEC APPROACH

The RDEC is calculated with equation 6 where RDEC is the ratio of the dissipated energy change per load cycle, ω_a is the dissipated energy produced in load cycle a, ω_b is the dissipated energy produced in load cycle b and a and b are load cycles (Carpenter and Shen, 2006; Shen, 2007).

$$RDEC = \frac{\omega_a - \omega_b}{\omega_a(b - a)} \quad [6]$$

As shown in Figure 11, the RDEC presents 3 stages when represented against number of cycles. After an initial period, Stage I, the RDEC presents a plateau period, Stage II, where the percentage of input energy being turned into damage is constant. Finally, in Stage III, there is a dramatic increase in RDEC, which gives a sign of fatigue failure (Carpenter and Shen, 2006; Shen, 2007) related to the appearance of macro cracking (Maggiore, 2014; Moreno-Navarro, 2013). Once the macro crack appears there is damage incurred with each subsequent loading cycle (Moreno-Navarro, 2013). In this approach, the 50% stiffness reduction occurs at the end of stage II.

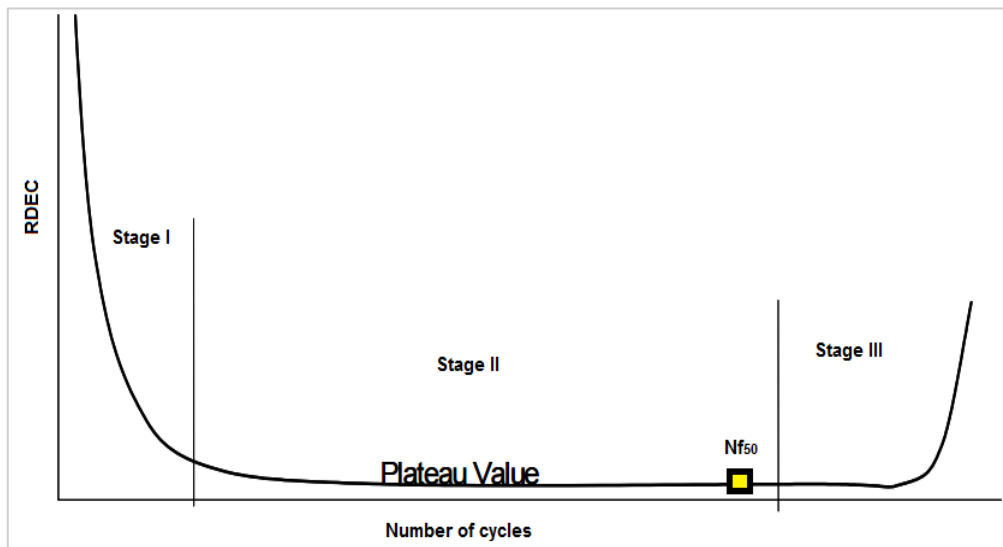


Figure 11 RDEC evolution (Carpenter and Shen, 2006)

4.3 DISPLACEMENT RATIO APPROACH

The displacement ratio represents the relationship between horizontal and vertical displacements, representing the changes produced in the geometry of the material in each load cycle (Moreno-Navarro, 2016).

It is calculated with equation 7, where δ_n is the displacement ratio in cycle n , δ_{hn} is the horizontal displacement in cycle n and δ_{vn} is the vertical displacement in cycle n . With this method, the phases of crack propagation can be studied, as in the example shown in Figure 12. Here, the first stage is where plastic deformation occurs, so no damage is induced to the material. Then crack propagation begins, starting with micro cracks (N_i) and then macro cracks (N_M) until complete failure (N_f).

$$\delta_n = \frac{\delta_{hn}}{\delta_{vn}} \quad [7]$$

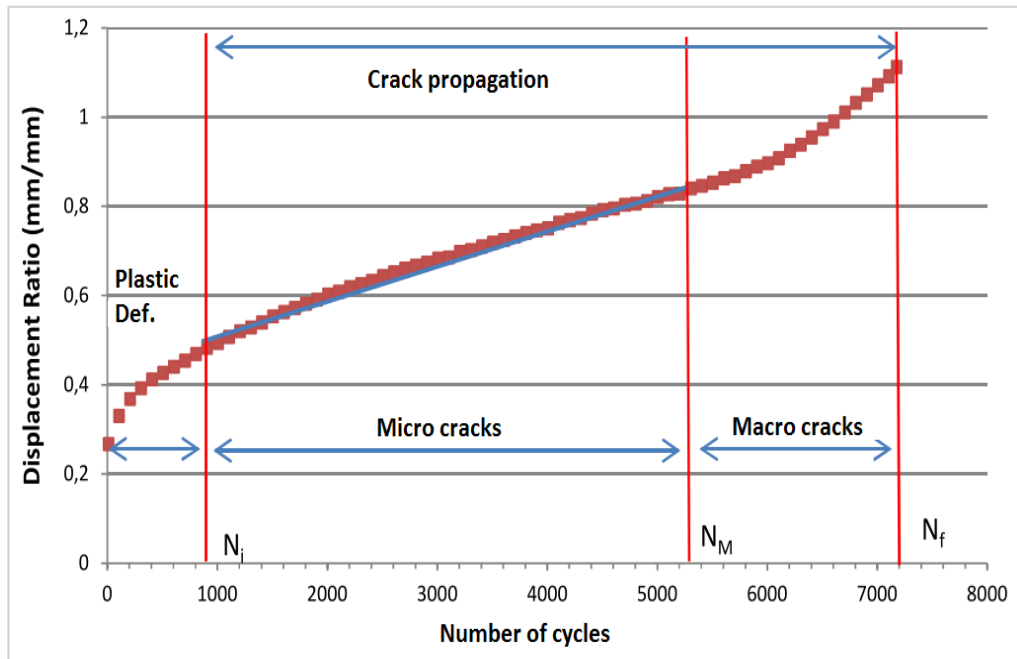


Figure 12 Crack propagation study with displacement ratio approach
(Moreno-Navarro, 2013)

4.4 ENERGY STIFFNESS RATIO

For analysing the accuracy of the failure criterion used for HMA, the energy stiffness ratio concept was defined by Abojaradeh (Abojaradeh, 2013) and it is described in equation 8. Here N_i is the cycle number, S_i is the stiffness at cycle i and S_0 is the initial stiffness taken at cycle 50 (Abojaradeh, 2013).

$$\text{Energy stiffness ratio} = \frac{N_i S_i}{S_0} \quad [8]$$

With this approach, failure is defined as the number of cycles at the peak value of the curve, when the energy stiffness ratio is plotted against the number of cycles (Figure 13) (Abojaradeh, 2013). Using this approach, Abojaradeh (Abojaradeh, 2013) confirms that the failure with this method for HMA can reliably be defined at 50% stiffness reduction.

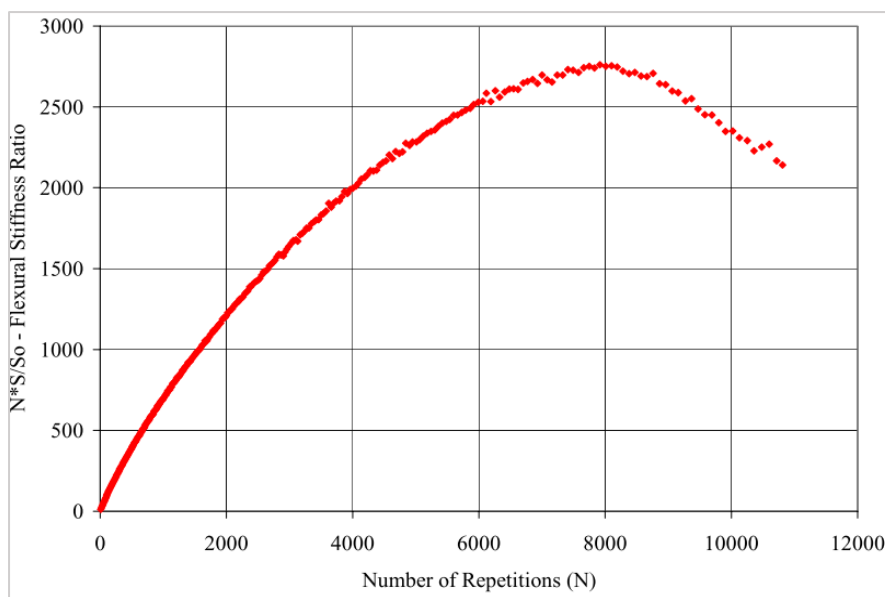


Figure 13 Energy stiffness ratio (Abojaradeh, 2013)

4.5 WHEEL TRACK TEST (WTT)

The WTT is a widely accepted test used to reproduce the effects of traffic (Ojum, 2015). It is used to assess material behaviour during repeated passes of a loaded wheel at a constant temperature (British Standard, 2003c). This test is typically used to analyse bituminous material susceptibility to deformation, assessing the rut formed under the wheel (British Standard, 2003c). However, with the right configuration, it can be used to evaluate failure mechanisms under a controlled environment, reproducing road conditions as closely as possible (Ojum, 2015).

LITERATURE REVIEW

CONCLUSIONS

In this first part of the Thesis the pavement design principles have been set out. This helps in the understanding of the role of every layer in a pavement and the use of different materials in each one.

The design methods for airfields have been reviewed to understand how they work and how a new material could be included.

Finally, a summary of cold recycled material characteristics and performance developed in past investigations has been presented.

With this literature review, the following conclusions can be drawn:

- Chapter 1:
 - Pavements are classified as flexible, rigid or composite depending on the materials used.
 - Asphalt consists of aggregates and bitumen. Bitumen gets aged with time, traffic and environmental phenomena and this aging makes the pavement fail more easily.
 - Asphalt is mainly characterised by stiffness modulus, but as cracking is a key design issue, fatigue becomes very important.
 - A pavement should protect the subgrade, guard against deformation and break-up of the pavement layers and provide a suitable surface for traffic.
- Chapter 2:
 - The need for an investigation into airfield pavement design with cold recycled materials is justified, as there is no specific guide to carry out works with these materials in airfields.

- DMG 27 was selected as the reference design guide as it is typically used by authorities in the UK. However, this guide does not readily allow the introduction of new material properties. Therefore, a new design methodology needs to be implemented.
 - For the new design methodology Kenlayer was selected as the structural model. This software is relatively versatile allowing the user to adjust were material properties and the failure criteria.
 - For analysing a pavement with Kenlayer, material properties need to be analysed: stiffness, Poisson's ratio, fatigue law. However, Poisson's ratio can be assumed.
 - Kenlayer will be used as the analytical software. The strains and stresses will be calculated at the bottom of the asphalt layer and at the top of the subgrade, to then calculate the layer life with fatigue equations for asphalt and equation 1 (BAA) for subgrade. Therefore, BAA analytical design will be used with the Kenlayer model.
- Chapter 3:
- Asphalt pavements require rehabilitation as bitumen suffers an aging process that changes its characteristics and leads to unacceptable performance on site (i.e. cracking and rutting) in relation to operational requirements.
 - RAP is obtained after planing asphalt pavements that need rehabilitation, and it is the primary recycled material derived from aged asphalt pavements.
 - The performance of pavements which incorporate RAP depends on RAP characteristics and the material into which it is used. Investigation in this subject is ongoing, trying to understand how mixtures with RAP behave and how to further increase/optimize RAP contents.

- The presence of tar in old asphalt layers makes the use of cold recycled materials an attractive solution for pavement rehabilitation due to the relatively high cost associated with materials disposal and limitations on recycling into other material options (i.e. HMA or unbound mixtures).
- On the basis of UK experience and research, aggregates gradings for CRBM should fall within the grading limits shown in Table 9 (Zone A from Table 5).

Table 9 *Grading to be used*

Sieve size (mm)	Specification (min. - max.)	
31.5	100	100
20	100	100
14	85	100
10	68	100
4	38	74
2	25	58
0.5	13	38
0.25	9	28
0.063	5	21

- There are two main technologies used for cold recycled materials: bitumen emulsion or foamed bitumen. Foamed bitumen was selected for this research as it is a more economical solution and is the most commonly used technology in the UK.
- Bitumen used for foaming should be harder than 160/220 penetration grade and suitable for foaming.
- Curing is important as cold mixes are evolutive, i.e., they improve properties with time. Thus, for cold mixes, the curing procedure in the laboratory is important as, ideally, it would simulate site conditions. However, there is no standard procedure and further investigation is needed.

- For this first stage of the research, two curing processes have been selected: 28 days at 20°C, that will simulate field conditions just before starting trafficking the pavement; and 3 days at 40°C that simulates the pavement after 6 months of performance.
 - Characterisation tests have been identified as: mixture volumetrics, binder properties, ITSM, RLAT, ITST and ITFT.
 - ITFT in strain control mode can be used to obtain the fatigue law necessary to carry out an analysis with Kenlayer.
- Chapter 4
- Fatigue is one of the main failure mechanisms in flexible pavements; therefore, it is a research priority to develop a reliable design methodology for CRBM.
 - The dissipated energy method is considered to reduce the gap between laboratory and field performance, giving information about failure mode and crack propagation.
 - Four methods are chosen to study fatigue behaviour of CRBM: DER, RDEC, displacement ratio and energy stiffness ratio approaches.
 - WTT can be used to verify results from the failure mechanism study with a more realistic configuration.

In the next section the investigation carried out in the laboratory to characterise CRBM is presented. The results obtained are then inputted into Kenlayer for modelling pavement structures containing CRBM (section 4).

SECTION 3. LABORATORY TESTING

This section presents the research carried out in the laboratory to achieve objective O2. A laboratory programme was established to characterise a CRBM, and understand its behaviour and adequacy for airfield pavement design.

Chapter 5

COLD MIX CHARACTERISATION

Cold recycled materials are relatively widely used in UK road construction; however, their use in airfield pavements is less common due to the uncertainties about long term performance and lack of developed specifications. As stated in Chapter 3, the UK has adopted the concept of sustainable development and cold recycling asphalt into foamed bitumen or bitumen emulsion bound materials minimises the waste generation and reduces the consumption of finite resources. In addition, this technique has the added benefit of readily incorporating recycled asphalts which contain tar. Many historic airfields in the UK include tar at depth in their construction. This material is costly to dispose of into landfill; therefore, cold recycling is often an attractive option for the rehabilitation of airfield pavements. The challenge is that, as mentioned in Chapter 2, there is no specific pavement design guidance for the use of CRBM and the current practice is to conservatively equate this materials' performance to an assumed performance of a hot asphalt mixture. The objective of this Chapter is to carry out an initial characterisation of cold mixes. The next stage is to verify if the current practice is accurate; therefore, CRBM performance was compared to a typically HMA used for this type of application, namely HDM50 (Heavy Duty Macadam with a 40/60 penetration grade bitumen).

5.1 MATERIALS

To undertake this investigation, site RAP was used. The RAP was selected from RAF Waddington works, as it was the airport where foam mix was first used in the UK (Lacalle Jiménez and Tuck, 2015). For CRBM specimens manufactured with foamed bitumen, the following materials were employed:

- 0-10 mm RAP and 10-20 mm RAP to comply with specifications (see Chapter 3).
- Fly Ash (CEM I, BS EN 450-1, Fineness Category S) and Cement.
- Bitumen 100/150. This bitumen was selected as it has the recommended penetration (as specified in Chapter 3) and it was proven to give good half-life and expansion ratio (Lacalle Jiménez and Tuck, 2015) in accordance with the foamed asphalt plant manufacturers guidance (Wirtgen GmbH, 2012).

5.1.1 Materials characterization

The first step was to characterize the materials used in the investigation. With this objective, the following tests were undertaken:

- RAP grading (British Standard, 2012e).
- Bitumen 100/150 characterization: penetration grade (British Standard, 2006a) and softening point (British Standard, 2007).
- Binder content (British Standard, 2003d) from RAP.
- Binder recovery from RAP (British Standard, 2012f) and posterior binder characterisation: penetration grade (British Standard, 2006a) and softening point (British Standard, 2007).

5.1.2 Results

The RAP grading (British Standard, 2012e) is presented in Table 10. It can be appreciated that the original RAP contained less fines than the specification demands; therefore, fly ash was added and the final grading is presented also in Table 10 and in Figure 14.

Table 10 RAP grading

Sieve size (mm)	% Passing. Original	% Passing. Final	Specification (Jitareekul, 2009; Merrill et al., 2004)	
			(min.	- max.)
31.5	100	100	100	100
20	100	100	100	100
14	100	85	85	100
10	88	72	68	100
4	6	41.8	38	74
2	0.3	28.7	26	58
0.5	0.2	14.6	13	38
0.25	0.1	11	9	28
0.063	0	8	5	21

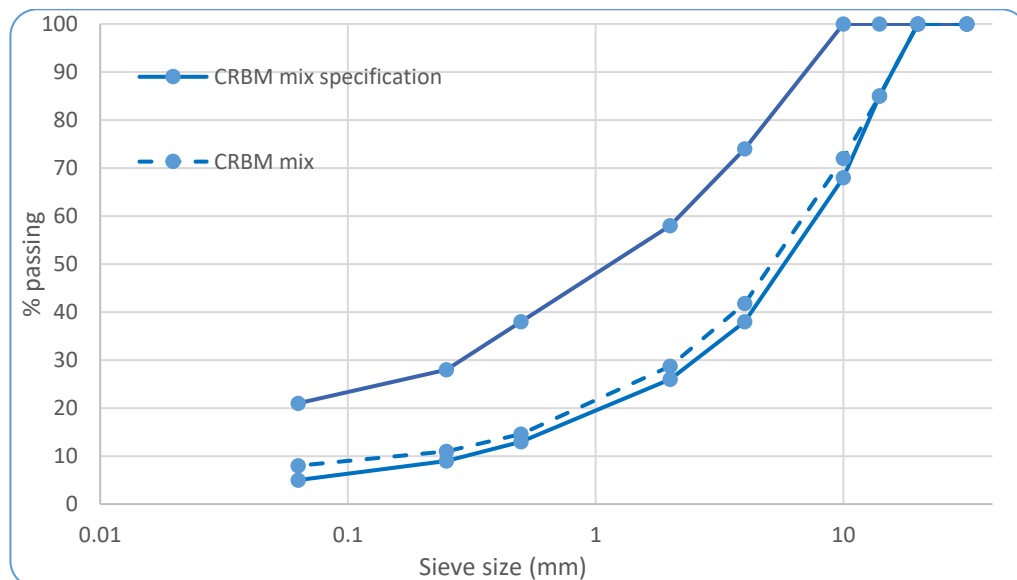


Figure 14 RAP final grading

The binders' characterisation is presented in Table 11.

Table 11 Binders characterisation

Bitumen	Binder content	Penetration (25°C,1/10 mm)	Softening point (°C)
100/150	NA	107	44.2
RAP 0-10 mm	7.2	30	58.6
RAP 10-20 mm	4.4	32	55.0

The final CRBM mixture had to meet specifications for its use in airfields. There are different approaches to analyse the blend design of RAP binders (Jiménez del Barco Carrión et al., 2015). The European blending models are based on the penetration grade of the RAP recovered binder and the virgin binder added to the mixture. For high contents of RAP, equation 9 is used (Jiménez del Barco Carrión et al., 2015).

$$a \log pen_1 + b \log pen_2 = (a + b) \log pen_{mix} \quad [9]$$

where,

pen_1 = penetration of the binder recovered from the RAP

pen_2 = penetration of the added virgin binder

pen_{mix} = penetration of the binder in the mixture

a, b = ratios by mass of the binder from the RAP and of the virgin binder respectively

With this equation and data from Table 11 it can be established that, for obtaining a mixture comprising binder with penetration grade 40/60 (binder commonly used in airfields (Defence Estates, 2010)), between 3% and 4% of virgin binder 100/150 should be added. It has to be taken into consideration that the above equation is typically used for HMA where temperatures during mixing are higher and blending is more likely to occur. However, in CRBM full blending is unlikely to happen; therefore, here this calculation is only used for initial approximation and the mixture is tested for mechanical performance to confirm if the amount and type of bitumen is suitable or should be adjusted.

5.2 SAMPLE PREPARATION

The sample preparation was undertaken to reproduce site conditions.

5.2.1 Mix design

RAF Waddington airport needed rehabilitation and reprofiling in 2014. AECOM (URS at that time) carried out a study to apply, for the first time on a UK airport, CRBM with foamed bitumen. This was considered the best available option and this project was found to be an opportunity to undertake further research. In this study, an optimum mix design was developed analysing different binder content and aggregates (Lacalle Jiménez and Tuck, 2015).

This mix design was optimised on site and adopted in this research. This was found to be the most appropriate option to bring forward the research as the RAP used in this investigation was from RAF Waddington. The future opportunity to analyse the behaviour of the mixture under real conditions was interesting for further research.

In Table 12 and Table 13 the mix design used is presented.

Table 12 *Aggregates*

Aggregates	Proportion by dry mass (%)
0-10 mm RAP plannings	48.1
10-20 mm RAP plannings	43.2
Fly ash	6.9
Cement	1.8

Table 13 *Mix design*

Ingredient	Proportion by dry mass (%)
Aggregates	90.5
Foamed Bitumen	3
Total Water content	6.5

5.2.2 Foaming

The foamed bitumen was produced in the laboratory using a Wirtgen mobile foaming plant type WLB 10 such as the one shown in Figure 15.



Figure 15 Wirtgen WLB 10

An investigation into the effect of water content on half-life and expansion ratio was carried out to determine the optimum water content required to achieve a half-life greater than six seconds and an expansion ratio greater than 8 (Wirtgen GmbH, 2012). Figure 16 provides a summary of the results from this investigation and shows that the optimum water content in the bitumen is between 1 and 1.6% water.

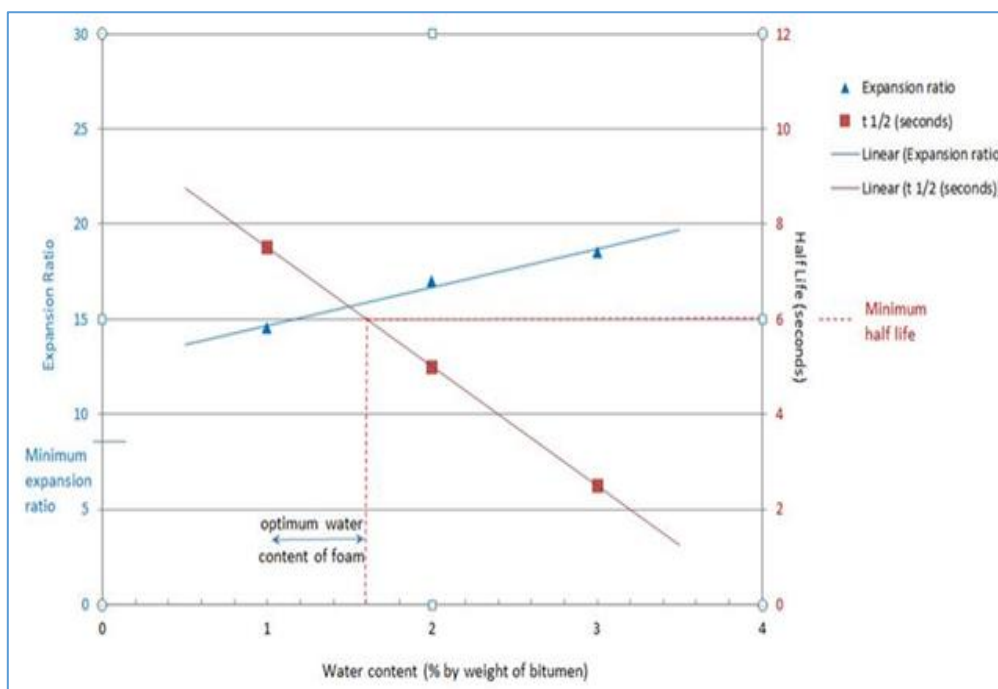


Figure 16 Influence of water content on bitumen foaming properties

The foaming conditions are presented in Table 14.

Table 14 *Foaming conditions*

Water Pressure	4 bar
Air pressure	5 bar
Bitumen type	100/150
Bitumen temperature	170°C
Water addition	1%

With these conditions , the half-life of the bitumen was measured using a Wirtgen measurement instrument (dipstick), and the results showed a half-life greater than the recommended minimum value of 6 seconds (Wirtgen GmbH, 2012).

Before starting the Wirtgen WLB 10, all the components needed for manufacturing the samples had to be batched to target the mix design specified in Table 12 and Table 13. The bitumen needed to be heated at the set temperature (170°C) between 3 and 5 hours before using the equipment.

Once the materials were ready and the parameters set in the equipment, the foam mix batch could be manufactured using a mixer coupled to the foaming plant, to avoid losing material. Water and aggregates were mixed for 1 minute before adding the foam and then another 3 minutes with the foam.

5.2.3 Manufacture of foam mix specimens

Specimens were compacted using a Coopers Gyrotory Compactor in accordance with BS EN 12697-31, at a constant pressure of 600 kPa, a speed of 30 rpm and an angle of 1.25°. The calculated amount of mixture was placed in the mould and compacted to a target density (2280 kg/m³ suggested by Lafarge) to replicate site conditions. This target density provided specimens with a maximum density of 2400 kg/m³ and 5% air voids content. The cores produced had dimensions of 100 mm diameter and were 60 mm high. After 24 hours, specimens were de-moulded, sealed and stored under the required curing conditions

5.3 CURING

For curing, test specimens were double wrapped in cling-film (poly vinyl chloride (PVC)) using two separate sheets, as shown in Figure 17. This was undertaken to avoid excessive water loss/drying out of the samples during curing. Once wrapped the specimens were stored in air at the selected curing temperature for a period of time. Two curing procedures were selected:

- Accelerated curing at 40°C for 3 days: approximating 6 months of curing in the field (Jenkins et al., 2004).
- Standard curing at 20°C for 28 days: conservative approximation to simulate actual performance of CRBM on site (Ojum et al., 2014),(Defence Estates, 2009).



Figure 17 *Double wrapped specimens for curing*

After this period, all specimens were unwrapped, conditioned and tested as described in the next section. Specimens after curing are presented in Figure 18.



Figure 18 *Specimens after curing*

5.4 TESTING METHODOLOGY

The testing methodology comprised: bulk and maximum densities, air voids content, stiffness, strength, resistance to permanent deformation and fatigue. In the next sections the tests carried out and the results are presented.

5.4.1 Binder content and binder characterisation

Binder content (British Standard, 2003d) and binder characterisation (penetration (British Standard, 2006a) and softening point (British Standard, 2007)) from the mixture was established to ascertain the mix design.

5.4.2 Bulk density, max density and air voids

Bulk density was measured in accordance with BS EN 12697-6:2012, procedure C (British Standard, 2012b). The specimens had to be weighed dry and then sealed in such a way that no water could penetrate, the internal voids were not penetrated and no extra voids were included between the seal and the specimen. In Figure 19 a sealed specimen is shown. After this procedure, sealed specimens were weighed in water with the equipment shown in Figure 20.



Figure 19 *Sealed specimen*



Figure 20 *Equipment for weighing in water*

Maximum density was measured to BS 12697-5:2009, method A (British Standard, 2009). Specimens were broken down and the material loosened and separated into coarse particles as shown in Figure 21.



Figure 21 *Material for Maximum density*

Once this was done, maximum density could be determined by a volumetric procedure using a pycnometer and a vacuum bomb to eliminate all the air that could be held between particles. The equipment used is shown in Figure 22.



Figure 22 *Maximum density equipment*

After both densities were measured, the air voids content was calculated to BS EN 12697-8:2003 (British Standard, 2003a), using equation 10.

$$V_m = \frac{m * b}{m} * 100 \% (v/v) \quad [10]$$

Where V_m is the air voids content of the mixture, in 0.1 percent (v/v), m is the maximum density of the mixture, in kg/m^3 and b is the bulk density of the specimen, in kg/m^3 .

5.4.3 Stiffness Modulus

The stiffness modulus was determined to BS EN 12697-26:2004 Annex C (British Standard, 2012c), to assess the load spreading ability and the durability at three different temperatures: 10°C, 20°C and 30°C. To evaluate the moisture sensitivity, the specimens were immersed in water at 20°C for 7 days and then the stiffness was measured (Defence Estates, 2009).

To assess durability, these specimens were again immersed for 7 days and stiffness measured, repeating this procedure to see if the stiffness value dropped.

Stiffness modulus measurements were determined using a servo-pneumatic machine, by applying indirect tension to the cylindrical specimens as shown in Figure 23. A rise time of 124ms and a target horizontal deformation of 5µm were selected. Ten conditioning pulses were applied followed by five test pulses. The measurements were repeated along two diameters and the average stiffness calculated.



Figure 23 *ITSM test*

5.4.4 Resistance to permanent deformation under dynamic loading

The resistance to permanent deformation was assessed by Repeated Load Axial Test without confinement (RLAT) to BS DD 226 (British Standard, 1996) and the configuration shown in Figure 24. The test was conducted at 40°C, with a conditioning stress of 10 kPa for 600s and a test stress of 100 kPa during 1800 pulses.

5.4.5 Strength

The indirect tensile strength (ITS) of the specimens was determined to BS EN 12697-23:2003 (British Standard, 2003e). This test involved applying compressive loads to the sample between two loading strips which creates tensile stress along the vertical diametric plane causing a splitting failure (Ojum et al., 2014). The test was carried out at 10°C and 20°C using the INSTRON test equipment as shown in Figure 25. For retained strength calculation, the test was carried out also on specimens that were soaked in water at 20°C for 24 hours. Retained strength is a durability test as detailed in section 3.4.7.



Figure 24 *RLAT*



Figure 25 *ITST*

5.4.6 Fatigue

Fatigue was determined by Indirect Tensile Fatigue Test (ITFT) to Annex 13, Fatigue characteristics of bituminous mixtures using indirect tensile fatigue (British Board of Agreement, 2008) with the configuration shown in Figure 26. Fatigue performance of a material is one of the key parameters used to assess a pavement's expected life. The fatigue test was undertaken at several stress conditions, with the number of loading cycles to failure recorded. This enabled a predicted 'fatigue life' to be calculated and plotted.



Figure 26 ITFT

5.5 RESULTS AND DISCUSSION

In following sections, the results from the test described above for CRBM characterization are presented.

5.5.1 Binder content and characterisation

Table 15 *Binder characterisation*

Bitumen	Binder content	Penetration (25°C,1/10 mm)	Softening point (°C)
Recovered from mixture	7.5	46	52.4

With equation 9, that relates binder content with binder penetration grade, the binder characterisation from the mixture, presented in Table 15, was ascertained. The results obtained were considered appropriate; therefore, the mixture and manufacturing process had been carried out in the correct way.

5.5.2 Cold mix characterization

The results from the tests described in section 5.4 are presented in Table 16.

Table 16 Initial CRBM characterisation results

Properties	Test Conditions	Previous conditioning	Foam mix		Target/ Spec (Defence Estates, 2009)
			Accelerated curing 40°C 3days	Normal curing 20°C 28 days	
Bulk density (Kg/m ³)	NA		2153 (2128-2179)	2146 (2108-2191)	2280
Max. density (Kg/m ³)			2515 (2498-2525)		2400
Air voids (%)			14.38 (13.32-15.36)	14.93 (14.40-15.60)	5
ITSM (MPa)	10°C	At least 4h in air at test temperature before testing	5719 (5227-6297)	5817 (4550-6786)	NA
	20°C		3870 (3429-4388)	3925 (3353-4854)	3100-6500
	30°C		2567 (2263-2798)	2793 (2128-3710)	NA
	20°C	7 days under water at 20°C	4876 (3844-5198)	4991 (4755-5112)	NA
Moisture sensitivity	NA		1.3 (1.1-1.4)	1.2 (1.1-1.3)	>0.80
ITS (GPa)	10°C	2h at testing temperature	$1.006 \cdot 10^{-3}$ (0.963-1.038·10 ⁻³)	$1.127 \cdot 10^{-3}$ (1.094-1.156·10 ⁻³)	>0.225
		After 24h in water at 20°C	$0.966 \cdot 10^{-3}$ (0.800-1.072·10 ⁻³)	$1.086 \cdot 10^{-3}$ (0.982-1.247·10 ⁻³)	
	20°C	2h at testing temperature	$0.677 \cdot 10^{-3}$ (0.422-0.852·10 ⁻³)	$0.756 \cdot 10^{-3}$ (0.723-0.790·10 ⁻³)	
		After 24h in water at 20°C	$0.649 \cdot 10^{-3}$ (0.586-0.684·10 ⁻³)	$0.696 \cdot 10^{-3}$ (0.575-0.817·10 ⁻³)	
Tensile Strength Retained (%)	10°C	NA	96.0	96.4	>90
	20°C		95.9	92.0	>90
RLAT. Strain (%)	40°C	Overnight in air at test temperature	0.1 (0.08-0.14)	0.5 (0.40-0.51)	<2
ITFT	30°C	At least 4h in air at test temperature	logE = 3.11-0.25 logN R ² = 0.93	logE = 3.34-0.31 logN R ² = 0.98	NA

Note: Range of values shown in parenthesis. NA: not applicable

The bulk density was under the target fixed for the compaction. This change in the compaction properties made the maximum density go below the value expected and in consequence the air voids content was higher; however similar air void content had been reported in other foam mix studies as explained in the literature review (section 3.1.1.) (Vorobieff and Preston, 2004).

Inspection of a broken specimen in Figure 27 revealed that these voids are not readily apparent. It may therefore be concluded that they are in the form of 'micro-voids' within the binder-filler-fines mortar.

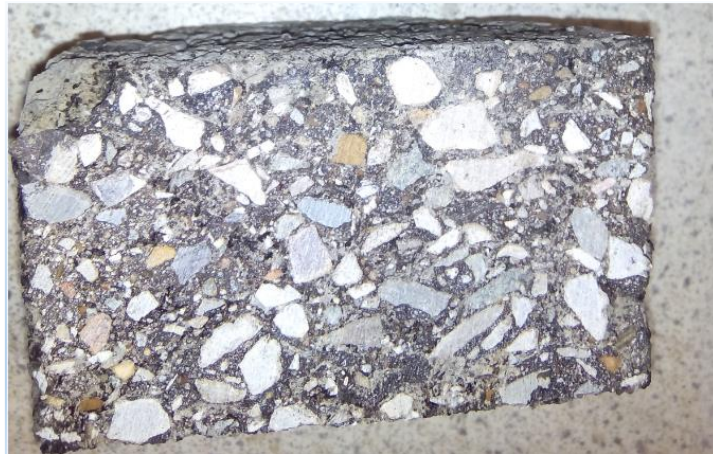


Figure 27 *Cut specimen*

Nunn and Thom (Nunn and Thom, 2002) found it difficult to achieve less than 9% air voids for foamed stabilised materials and agreed that more investigation is needed to determine if this is a crucial factor for the performance of these materials. It can also be appreciated how both curing procedures give very similar results in terms of mixture volumetrics.

The stiffness and strength values obtained were within specification 050 tolerances (Defence Estates, 2009). Comparing results with both curing procedures, Figure 28 and Figure 29, it can be said that stiffness values are comparable; however, strength was more conservative in the accelerated curing.

Strength and stiffness results after wet conditioning were obtained to assess mixture durability. It could be considered surprising that CRBM stiffness after 7 days soaking was greater than before, obtaining a retained stiffness greater than 100%; however, this was likely to be due to the content of cement, which was still active and which used the moisture to gain strength, making the specimens stiffer than before. With these results, it could be considered that CRBM after both curing procedures has good durability.

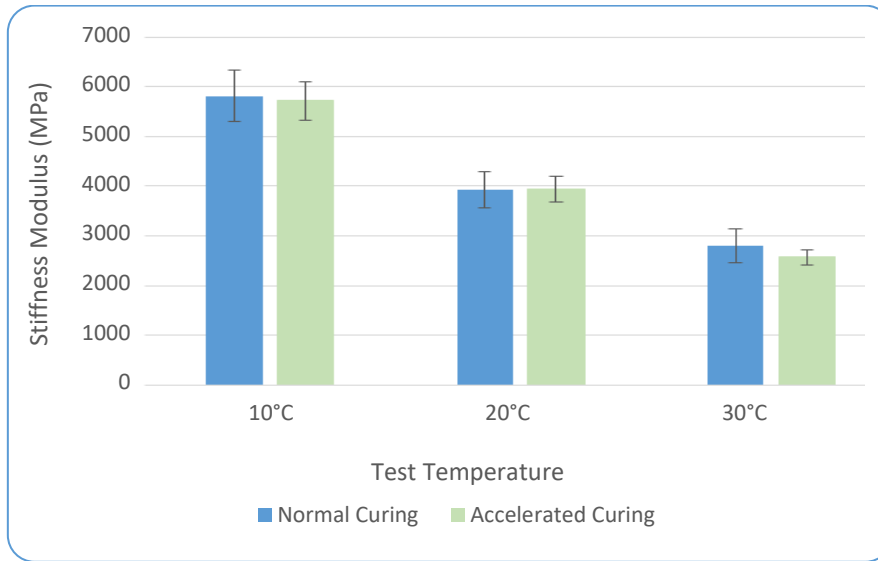


Figure 28 CRBM ITSM values for different curing procedure

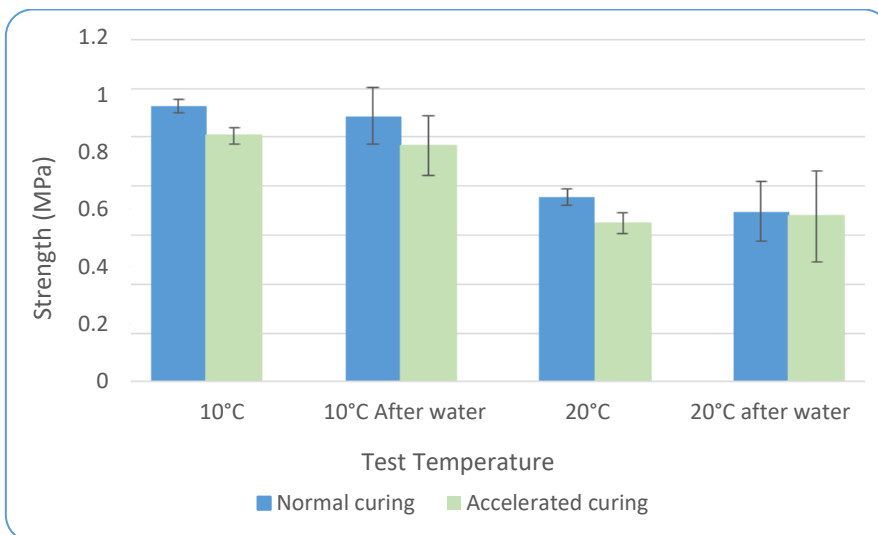


Figure 29 CRBM ITS values for different curing procedures

In terms of permanent deformation, it should be noted that although there is not a clearly defined threshold value available to interpret the results, this laboratory method is employed to rank different asphalt materials based on their resistance to permanent deformation, i.e. % permanent strain. This means that this value is useful to compare materials. Nevertheless, from the UK highways requirements (below 2% strain (Department for Transport, 2008; Widyatmoko, 2008)), good deformation resistance has been found for the CRBM with both curing procedures.

Fatigue lines were obtained for CRBM with both curing procedures. In Figure 30 both fatigue lines are presented. For 100 microstrain (typical highway strain levels) (Cooper and Pell, 1974), the fatigue life is a third greater for the accelerated curing; while for 400 microstrain (more typical of airfields), the accelerated curing is more conservative (half of the life).

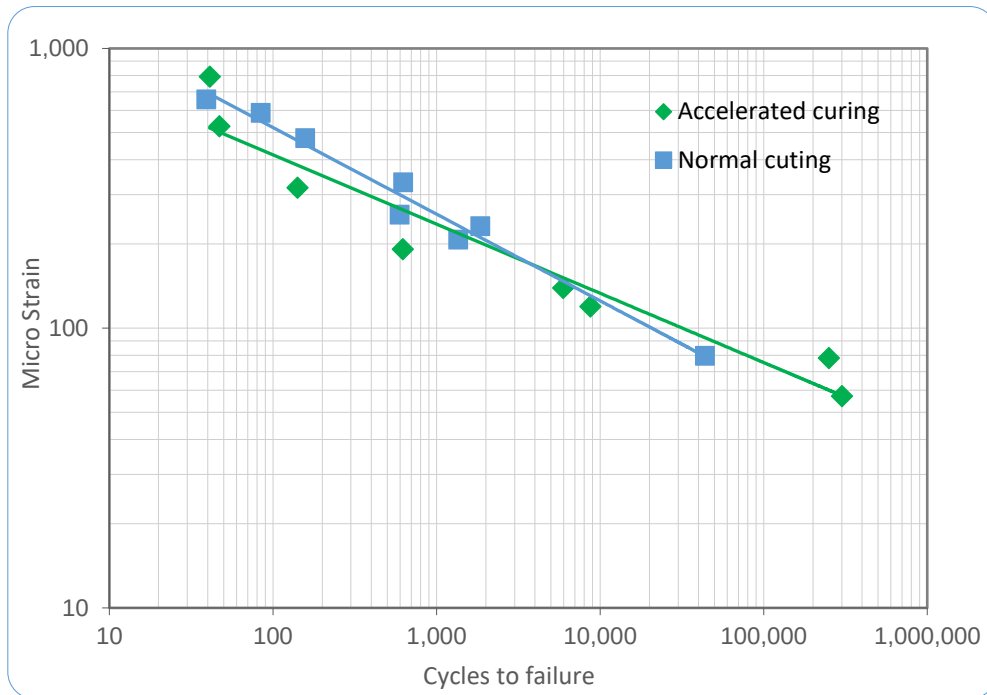


Figure 30 *Fatigue for different curing procedure*

To understand better the durability of the specimens they were immersed in water at 20°C and then the stiffness was measured in 7 days' intervals. The results are plotted in Figure 31. It can be concluded that CRBM has good durability, as after being soaked and repeatedly tested over a year, the stiffness did not decrease.

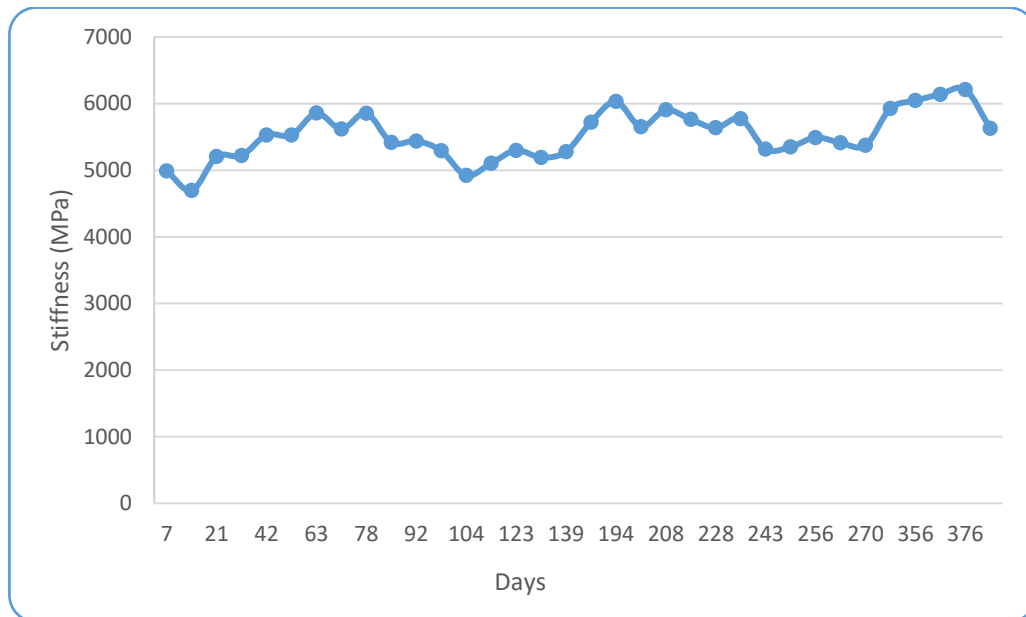


Figure 31 CRBM durability

5.6 COLD MIX COMPARISON WITH HOT MIX MATERIALS

As discussed previously, current practice for airfield pavement design with CRBM is to assume comparable behaviour to a HMA, with additional restrictions on where it can be used versus HMA options. To analyse if this practice is appropriate or if it could be improved, in this section the CRBM characteristics obtained previously are compared to a HMA commonly used in airfields.

For this purpose, it was decided to use data from previous research carried out in AECOM. In this research an HDM50 was manufactured and characterised in AECOM's laboratory following the standards. This material is commonly used in airfield base layers and as the test methodologies were the same as the ones used for the CRBM and following standards, it was concluded that the data were suitable for comparison purposes.

5.6.1 HDM50 characterisation

The manufacturing process and the results from the HDM50 were presented by Eric Draper (Draper, 2013) and are summarised in this section.

The process for hot mix sample manufacturing comprised:

- Perform Particle Size Distributions on the virgin aggregates.
- Perform Penetration Tests on the bitumen.
- Perform Softening Point Tests on the bitumen.
- Create an Aggregate Blend to fit within the specification envelope.
- Use prior knowledge of the aggregates and bitumen to estimate the Binder Content (2-4%) and to comply with (British Standard, 2006b).
- Perform Maximum Density Tests on trial mixes to confirm that the targeted volumetrics can be achieved.
- Create a Mixture Composition based on the above.
- Manufacture slabs using a Laboratory Roller Compactor.
- Core 100 mm and 200 mm diameter specimens using a diamond tipped asphalt coring rig.

5.6.3.1 Mix design

The mix design selected to achieve the desired air void content is presented in Table 17.

Table 17 *HDM50 mix design*

Constituents	Proportion by mass (%)
31.5 mm agg.	10.5
20 mm agg.	19.2
10 mm agg.	19.2
Crushed rock fines	43.1
Filler	3.8
Bitumen 40/60	4.2

5.6.3.2 Testing methodology

After the materials had been characterised, and the specimens manufactured, the following tests were carried out.

- Bitumen 40/60 characterization: penetration (British Standard, 2006a) and softening point (British Standard, 2007).
- Bulk density BS EN 12697-6:2012, procedure C (British Standard, 2012b), max density to BS 12697-5:2009, method A (British Standard, 2009) and air voids to BS EN 12697-8:2003 (British Standard, 2003a).
- Determination of indirect tensile stiffness modulus (ITSM) to BS EN 12697-26:2004 Annex C, and durability after wet conditioning, immersed at 20°C for 24 hours.
- Resistance to permanent deformation under dynamic loading was assessed by repeated load axial test without confinement (RLAT) to BS DD 226.
- Determination of the indirect tensile strength (ITS) to BS EN 12697-23:2003 before and after wet conditioning.
- Determination of indirect tensile fatigue (ITF) (British Board of Agreement, 2013).

5.6.2 Results and discussion

In Table 18 the bitumen characterisation is presented. It can be noticed that both bitumens have expected properties.

Table 18 *Bitumen characterisation comparison*

Binder	Penetration grade (tenths of mm)	Softening point (°C)	Purpose
40/60	47	72.1	HDM50
100/150	107	44.2	CRBM mix

In Table 19 the mixture volumetrics are compared. As already discussed in section 5.5, CRBM's air voids are higher due to 'micro-voids' within the binder-filler-fines mortar.

Table 19 Mixture volumetrics comparison

Mixture	Bulk density (Kg/m ³)	Max density (Kg/m ³)	Air voids (%)
CRBM	2146	2515	15
HDM50	2442	2551	4.3

Figure 32 shows that the CRBM mixture was less temperature susceptible than HDM50, behaving more consistently in terms of stiffness. This was likely to be due to the action of cement within the mixture and it has been observed in previous research (Betti et al., 2016).

Regarding the stiffness value used for design (at 20°C), it can be noticed that the difference between the two mixtures is around 30% (4000 MPa vs 5300 MPa). This difference is not a problem if it is taken into account that the HDM50 evaluated has an averaged stiffness value. HDM50 typical stiffness values at 20°C rank between 3100 MPa and 6700 MPa (Widyatmoko, 2002).

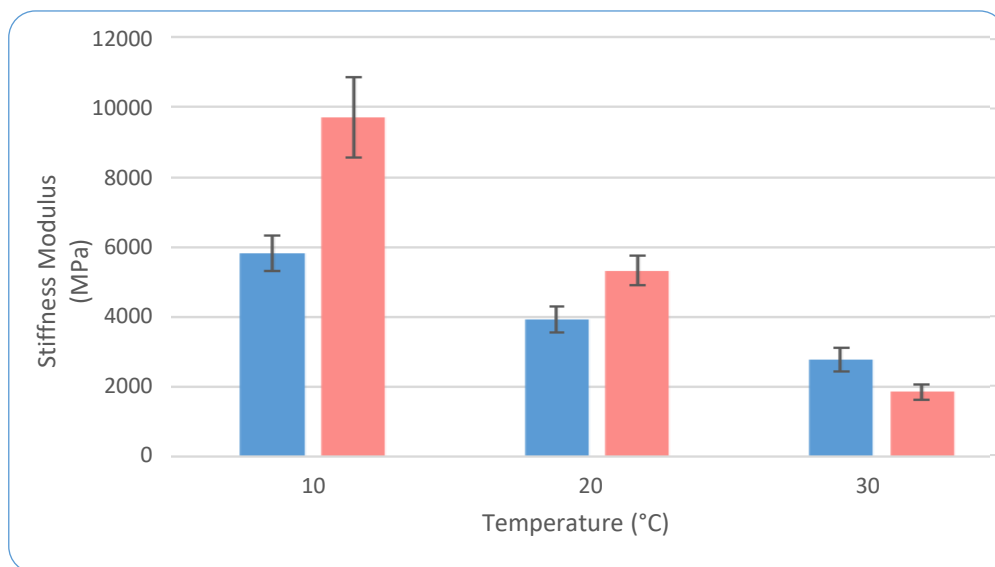


Figure 32 Stiffness comparison CRBM vs HDM50

Strength is presented in Figure 33. Here HDM50's strength is higher than CRBM's; however, as stated in section 5.5, CRBM's is within specification and in accordance with previous research (Valentin et al., 2016).

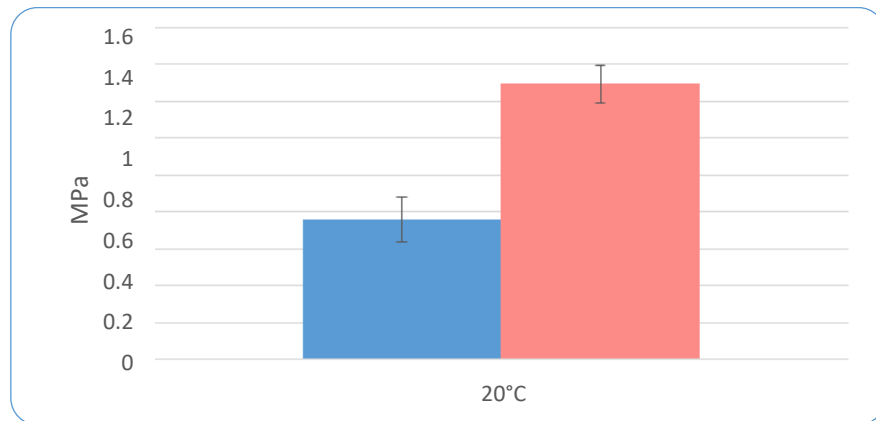


Figure 33 Strength comparison CRBM vs HDM50

In Table 20 the retained stiffnesses of HDM50 and CRBM are presented. HDM50 has a retained stiffness within typically specified value (need to be in excess of 80%) (Defence Estates, 2009) . As stated in section 5.5, the CRBM retained stiffness is greater than 100%. This apparent difference is most likely linked to using a hydraulic binder (cement) in the CRBM.

Table 20 Moisture sensitivity comparison CRBM vs HDM50

Mixture	Retained stiffness (%)
CRBM	120
HDM50	92

In Figure 34, it can be appreciated how both mixtures have much better rut resistance than the requirement of 2% suggested by the UK Department for Transport (Department for Transport, 2008). High rut resistance in very heavily stressed sites is an important characteristic, and is therefore important for airfield pavements.

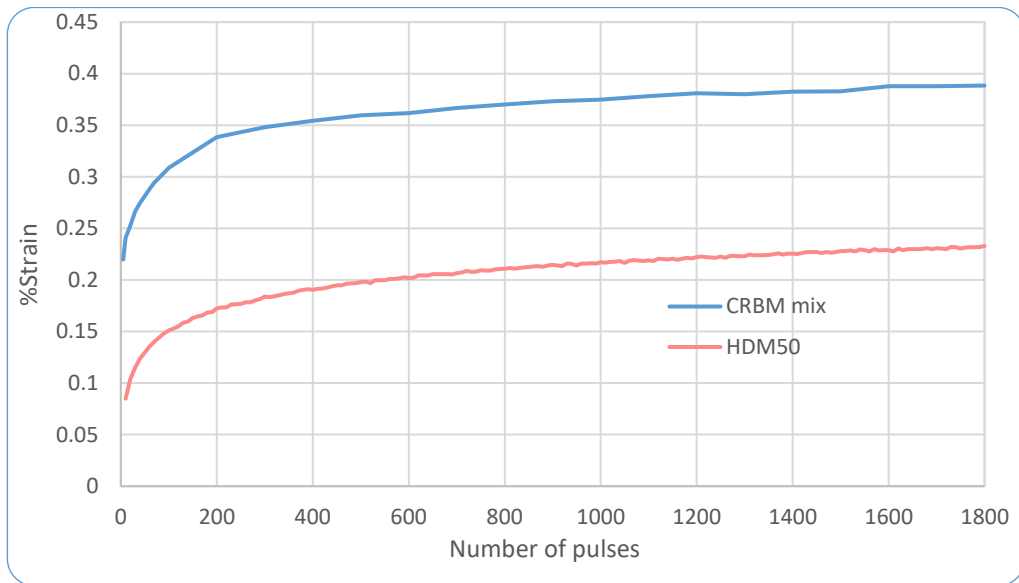


Figure 34 *Permanent deformation comparison CRBM vs HDM50*

Finally, the fatigue results in Figure 35 show a difference amounting to a factor of about four at 100 microstrain (typical highway strain levels) (Cooper and Pell, 1974), rising to about eight at 400 microstrain (more typical of airfields).

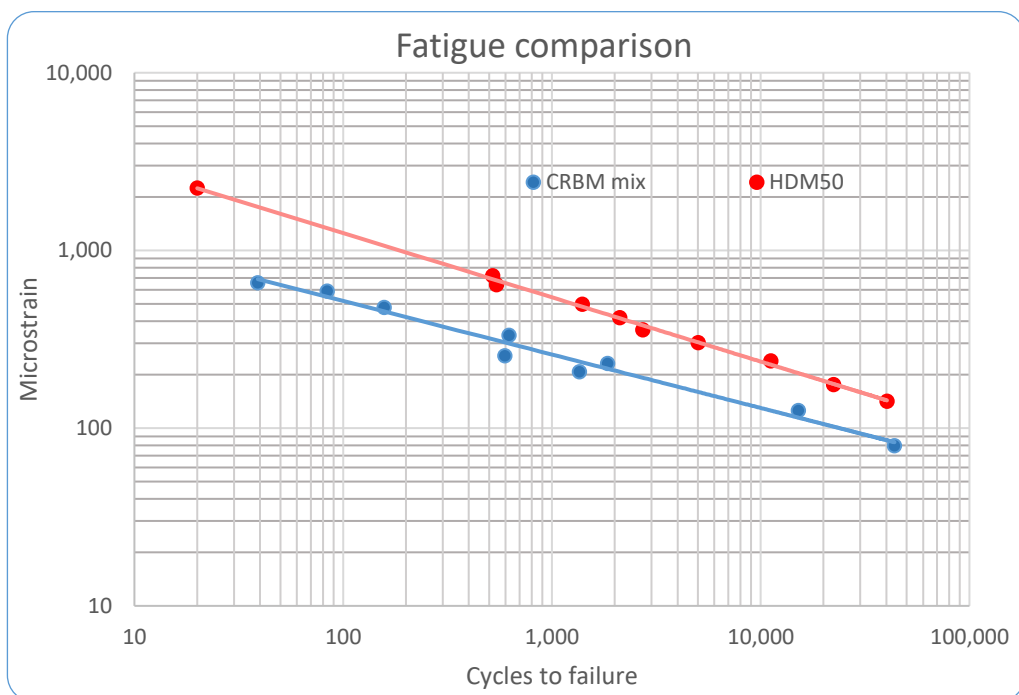


Figure 35 *Fatigue comparison CRBM vs HDM50*

5.7 CONCLUSION

The first stage of the laboratory research has been to characterise CRBM with foamed bitumen. With this purpose, samples were manufactured in the laboratory, cured and tested.

Two curing procedures were used to assess the performance of the mixture. Curing the specimens fully wrapped for 28 days at 20°C aimed to reproduce site conditions to acknowledge the mixture properties before trafficking. The second procedure, 40°C for 3 days, aimed to give information about the expected performance after 6 months in the field.

With the results obtained from the laboratory investigation it was concluded that CRBM mixes have acceptable properties for use in airfield pavements. Resistance to permanent deformation, fatigue, temperature susceptibility and durability results show that these materials give reasonable performance; however, they differ from conventional hot mixes.

This means that, for the design approach with CRBM, actual design guides should be optimised and calibrated to include performance of CRBM.

Regarding the curing, it was proved that both procedures give samples with similar characteristics and performance; therefore, the curing at 40°C for 3 days could be used in the laboratory as an accelerated curing, to reduce time lags between manufacturing and testing. However, curing samples in the laboratory is an area of ongoing research and further investigation is needed to establish an appropriate procedure.

The second stage of this laboratory research comprised a comparison of CRBM mechanical properties with a HDM50. This comparison highlighted the differences between both materials. CRBM air voids content is higher than in HDM50 as CRBM develop micro voids within the binder-filler-fines mortar.

In the CRBM, the cement helps in terms of temperature and moisture susceptibility. Strength and resistance to permanent deformation results show better performance for HDM50; however, the CRBM values are within specifications.

In terms of fatigue, it is noticed how at high strain levels, the difference between both materials is maximised reducing CRBM life to a factor of 8 compared to HDM50.

Chapter 6

CRBM FAILURE MECHANISM

As already stated in the literature review, fatigue is one of the main failure mechanisms in flexible pavements (Moreno-Navarro, 2013; Twagira et al., 2006); thus, its study is fundamental to develop a design methodology which fully take this into account.

In this section, a fatigue study on CRBM is presented to understand CRBM behaviour under cyclic loading. The results presented herein will then be used to inform pavement modelling using CRBM.

6.1 MATERIALS AND METHODOLOGY

The methodology comprises fatigue tests: ITFT in strain control mode to BS EN 12697-24:2012 Annex E (British Standard, 2012g) and Wheel Track Test (WTT). As stated in the literature review, the ITFT in strain control mode was selected as stress control tests are not appropriate for this study (Isailović et al., 2016) and other strain control tests (Abojaradeh, 2013; Maggiore et al., 2014) require specimen cutting. The WTT was selected to reproduce as closely as possible road conditions. The CRBM mixture used is the same as described in Chapter 5.

6.1.1 ITFT in strain control mode

This test method was developed at the University of Nottingham (Maggiore, 2014) and monitors the stiffness variation for a specified repeated strain value and the number of cycles until failure occurs. These parameters were needed to determine the fatigue coefficients in equation 2 (see section 2.3.).

This is a relatively simple test and suitable for cylindrical specimens; therefore, the manufacture of test specimens was straight-forward, saving materials and using the same compaction method as in Chapter 5, gyratory compaction. This also avoided the need to cut specimens from a slab, a process that can affect CRBM behaviour (Taherkhani et al., 2016; Yan et al., 2010).

The strains selected for the ITFT were between 150 and 300 $\mu\epsilon$. Tests carried out at less than 150 $\mu\epsilon$ took too much time and were impractical; and for samples containing cement, strains higher than 300 $\mu\epsilon$ produce quick cracking (Kavussi and Modarres, 2010), making the test too short to have enough data to analyse. The loading frequency was 2 Hz (established in the method) and the test was performed at 20°C, the temperature recommended for ITFT (British Board of Agreement, 2008). The test was initially carried out with the standard failure criterion of 50% stiffness reduction (Ghuzlan and Carpenter, 2000); however, after analysing the data obtained, it was decided to carry out more tests until complete failure of the specimens. This was undertaken to further understand the micro and macro crack propagation in CRBM.

The equipment used for ITFT in strain control mode was the same as for the ITFT in stress control mode; however, the configuration was different as in ITFT in stress control mode the horizontal deformation is not measured; therefore, the strain is not controlled.

The first step was to identify the position where two deformation measurement strips had to be located. To make sure both are placed on opposite sides of the horizontal diameter of the specimen the template shown in Figure 36 was used.



Figure 36 *Template to position the deformation measurement strips*

For gluing the deformation measurement strips in the positions marked with the template, a position rig with docks was used as indicated in Figure 37. This was done to ensure that the measurement strips were glued properly. When the glue was dry, the docks were removed and the specimen was placed in the pneumatic actuator in a central position between two loading strips. Two LVDTs were located horizontally held by the strips, in order to record the horizontal deformations, as shown in Figure 38.



Figure 37 *Deformation measurement strips glued to the specimens using a position rig*



Figure 38 *ITFT in strain control mode configuration*

In strain control mode, the specimen does not reach complete fracture along the vertical diameter at failure; instead, failure is typically characterised by localised deformations at the edge of loading strips with visible cracks observed along the vertical plane of the specimen, as shown in Figure 39. Fatigue life is determined as the number of pulses that causes a 50% reduction of stiffness from its initial value (Maggiore, 2014).

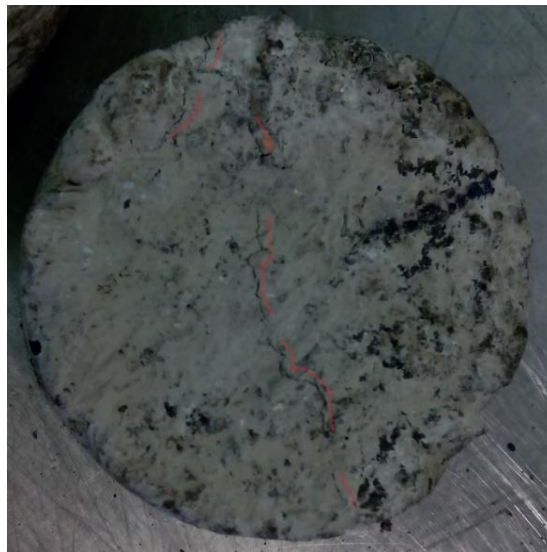


Figure 39 *Failure in strain control mode*

With these tests the stiffness variation for a specific strain value could be monitored and the number of cycles to failure recorded. These parameters were needed for calculating the fatigue coefficients from equation 2. Also, data about the dissipated energy and deformation per cycle was given, information needed to carry out the failure mechanism study.

6.1.2 WTT

For this test, slabs and beams were manufactured with 3 different configurations:

- Mix slab: HMA over CRBM
- Cold mix slab: full CRBM
- Hot mix slab: full HMA.

The aim of the test is to analyse failure mechanism; therefore, strain gauges were used to analyse strains at the bottom of the slab and a camera was employed to record crack propagation in beams.

6.1.2.1 Materials

The CRBM used in this test was also the same as in Chapter 5. For the HMA, a standard mixture with a paving grade bitumen (British Standard, 2006b) was chosen from the laboratory. The aim of this test is to compare behaviour; therefore, it was important to maintain the same mixture in all configurations and the availability of materials was the main cause for choosing the mix design, Table 21.

Table 21 *HMA mix design*

Constituents	Proportion by mass (%)
10 mm agg.	25.6
6 mm agg.	28.5
Dust	40.9
Bitumen 40/60	5

6.1.2.2 Sample preparation

The CRBM was produced as detailed in Chapter 5, using the foam rig, and then compacted with a roller compactor at room temperature in a mould with dimensions 305 mm x 305 mm. Accelerated curing followed: 3 days at 40°C.

After curing the CRBM of the mix slabs, a tack coat (bitumen emulsion) was applied 5 hours before the hot mix was overlaid as shown in Figure 40. Approximately 80 g of emulsion were applied.



Figure 40 *Tack coat application*

For the hot mix, the aggregates were heated for at least 8 hours at 150°C and the binder was heated at 150°C for 4 hours before manufacturing. Afterwards the aggregates and the binder were mixed in an asphalt mixer at 150°C and the slab compacted over the cold mix with a roller compactor at 140°C. The final slab is shown in Figure 41.



Figure 41 *Slab: 30 mm cold mix + 20 mm hot mix*

The beams were obtained by carefully cutting the slab into sections of 295 mm x 45 mm. This was undertaken 24 hours after the hot mix was compacted or after the curing period. When the beams were dry, around 5 hours after cutting, a white paint was applied to the cut side to make cracks more visible (Figure 42). The paint used was a multi-surface spray paint (PlastiKote) easy to apply and available in the laboratory. Then beams were placed for conditioning at least 5 hours before testing.

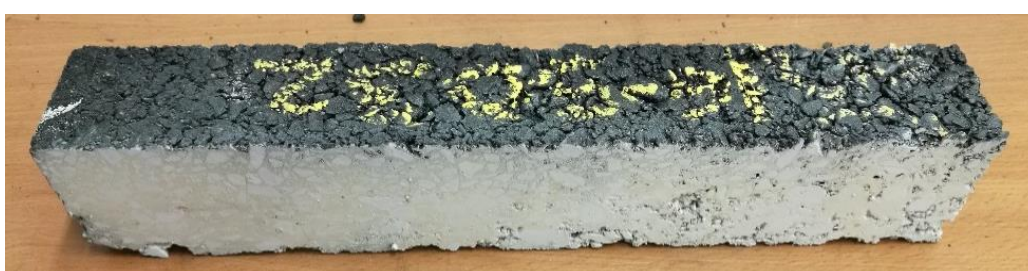


Figure 42 *Beam with sides painted*

The subgrade was simulated using a rubber mat suitable to simulate an elastic foundation, as established in previous studies (Moses, 2011; Ojum, 2015). When slabs were used, this mat was bored to allow volumetric compression as shown in Figure 43. When the hot mix was cooled down (24h after manufacturing) or the curing period was finished, the slab was de-moulded and one corner was removed to allow the cables to connect the strain gauges with the data acquisition system. Then the strain gauges were glued to the bottom of the slab. The gauges were placed in the centre of the slab (trafficked area) where the stress and strain was higher.

As they were glued to an uncut surface an epoxy resin was used to allow coverage of the voids in the slab. Early attempts with a normal glue failed.



Figure 43 *Rubber mat*

Once the resin was dry, the cables were welded to the strain gauges as shown in Figure 44 and the slab, ready for testing, was placed in the conditioning room at the test temperature for at least 5 hours before testing.



Figure 44 *Strain gauges with connections*

6.1.2.3 Testing methodology

The WTT was carried out in a temperature controlled room. The wheel moved forward and backwards with a frequency of 0.8 Hz under the loaded wheel. The configuration is shown in Figure 45.

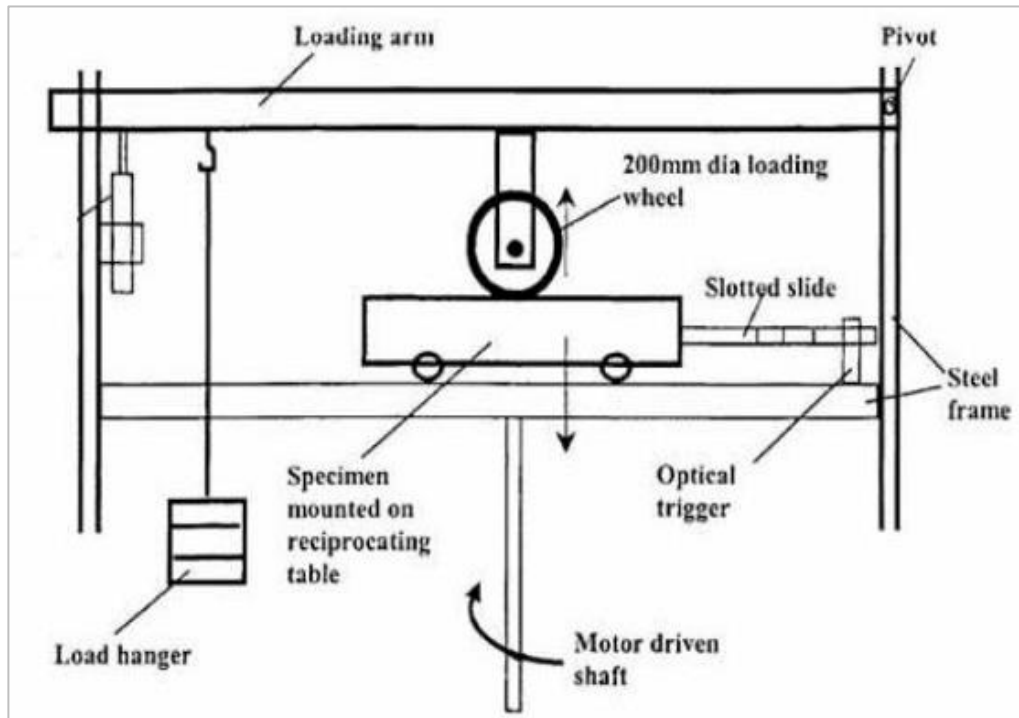


Figure 45 Schematic of the WTT (Ojum, 2015)

- Slab testing

When slabs were tested, the chosen load was 2.3kN, which for the wheel tracker wheel, closely reproduces aircraft tyre pressure (Moses, 2011). The manufactured slab with the gauges was placed over the rubber and the gauges were linked to the data acquisition system.

The aim of this test was to study the failure mechanism of the CRBM. With this purpose, in addition to the information provided by the gauges, crack propagation was studied. Different approaches were considered:

- Use of the CT-scan to see how the cracks propagate. However, the available equipment could not analyse such big specimens. The possibility of using other CT-scans was taken into account but discarded as bigger CT-scans do not have the resolution needed and travelling with the sample could produce cracks.

- Fixing cracks with fluorescent epoxy and then cutting the slab to analyse the cracks. A trial was made but it was not possible to distinguish the cracks properly.
- Use of high frequency ground penetrating radar (GPR).
- Use of Light Weight Deflectometer (LWD) to analyse stiffness before and after test.

However, after carrying out several trials, it was concluded that the test configuration (Figure 46) was not appropriate to analyse the failure mechanism as the confinement of the slab made it very difficult to crack. Therefore, it was decided to analyse beams with a semi-confined system.



Figure 46 *WTT configuration with slabs*

- Beam testing

The aim of this test is to crack the beams instead of rutting them; therefore, an adapted method was used. The beams were placed in a specially designed mould, manufactured with a solid Perspex panel to allow visibility of the asphalt beam, whilst providing lateral support to the sample, as shown in Figure 47.



Figure 47 *Beam configuration*

In these tests the load selected was lower, 1kN, to avoid excessive loading that could damage the system. A camera was used to take a picture of the beam every minute during the test. These photos facilitated the crack propagation study by capturing visual images of how the cracks formed and grew.

6.1.2.4 Experimental program

The experimental program comprised the configurations shown in Table 22.

Table 22 WTT experimental plan

*Material	Layers (mm)	Curing	Temperature (°C)	Load (N)
Hot mix + cold mix	20+30	40°C 3 days	5	1000
			10	1000
			20	1000
			10	800
Cold mix only	50	40°C 3 days	10	1000
Hot mix only	50	NA	10	1000

*At least 2 beams were tested with each configuration.

6.2 RESULTS AND DISCUSSION

Here the results from the failure mechanism study are presented. The first step was to obtain the fatigue coefficients from equation 2. However, after analysing the data from the ITFT in stain control mode, it was concluded that the failure mechanism of CRBM differs from HMA; therefore, further study was carried out to establish an appropriate failure criterion. After this study, a new fatigue law was established for CRBM.

6.2.1 Fatigue coefficients

In Table 23 the results obtained from the ITFT in strain control mode for CRBM, with 50% stiffness reduction as failure criterion, are presented.

To obtain the coefficients, the difference between the N_f obtained in the laboratory and the N_f calculated with equation 2 were minimised, thus determining the required values for f_1 , f_2 and f_3 . The fatigue curves obtained from laboratory tests and from calculation are presented in Figure 48.

Table 23 Experimental data from strain control fatigue tests for CRBM

N_f	ϵ_t	E_1 (MPa)
184279	0.00015	3426
202363	0.00015	3081
165563	0.00018	2219
78554	0.00018	2330
123526	0.0002	2343
64123	0.0002	1660
88723	0.0002	2895
65933	0.00022	1996
90173	0.00025	1711
36413	0.00025	2179
82223	0.00027	1390
59043	0.00027	1343
15183	0.0003	1432
21673	0.0003	1646

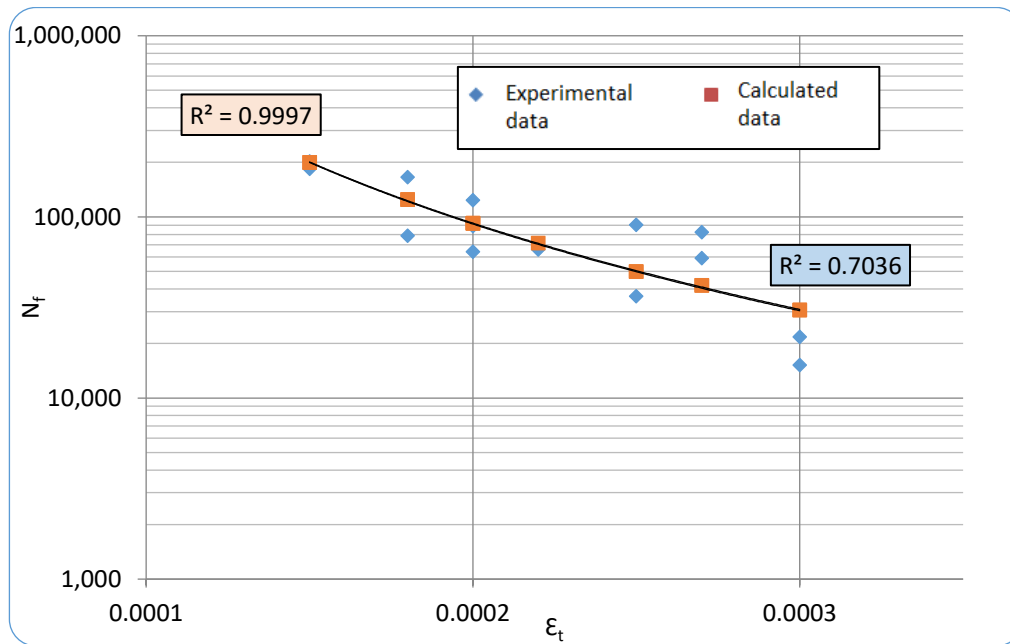


Figure 48 CRBM fatigue law calculation

Maggiore's data (Maggiore, 2014), shown in Table 24, were also used to check the adequacy of this fatigue test method, and the values of f_1 , f_2 and f_3 compared to those for HMA calculated by the Asphalt Institute and Shell (Huang, 2004; Shook et al., 1982). These results and CRBM fatigue coefficients are presented in Table 25.

Table 24 Experimental data from strain control fatigue tests for HMA

N_f	ϵ_t	E_1 (MPa)
149243	0.000125	10900
126500	0.000135	10329
88923	0.000145	11081
42613	0.000155	10231
49383	0.000165	10220
22393	0.000175	10582
30963	0.000185	9828
18683	0.0002	9928
17773	0.00022	9245

Table 25 *Fatigue coefficients*

	f1	f2	f3
Calculated HMA	0,074	4.842	3.109
Shell factors	0.0685	5.671	2.363
The Asphalt institute factors	0.0796	3.291	0.854
Calculated CRMB	$7.61 \cdot 10^{-6}$	2.826	0.110

It can be appreciated that the values calculated with Maggiore's HMA data and the values proposed by Shell are comparable. Thus, it can be deduced that the ITFT in strain control mode is a suitable test for fatigue coefficient calculation.

Previous researchers report various values for these coefficients, with the typical range of values for f_2 being between 3 and 6 (Barenberg and Thompson, 1992); however, f_1 varies by several orders of magnitude. This is in accordance with the values obtained for HMA.

With the new coefficients obtained in Table 25, the fatigue law for CRBM can be expressed as in equation 11.

$$N_f = 7.61 \cdot 10^{-6} (\epsilon_t)^{-2.826} (E_1)^{-0.11} \quad [11]$$

It is also interesting to study the material behaviour in terms of stress evolution during the test to analyse if both materials fail in the same way. In Figure 49 it can be appreciated that the stress evolution is different for HMA and CRBM mixes. HMA has a constant stress at the beginning and then it drops. For CRBM the stress starts dropping from the beginning with a smaller slope. This highlights the importance of studying the failure mode of CRBM, as it is distinct from HMA.

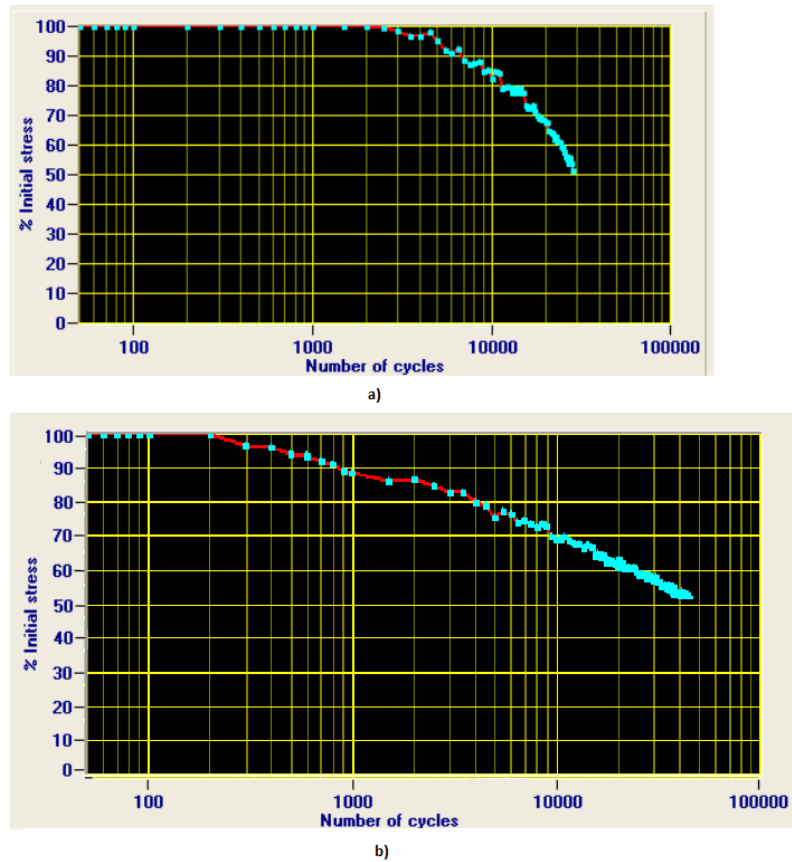


Figure 49 *a) HMA fatigue behaviour (Maggiore, 2014)*
b) CRBM mix fatigue behaviour

6.2.2 Failure mechanism study

The dissipated energy method is used to understand CRBM behaviour under cyclic loading. As stated in the literature review, three approaches will be studied: DER, RDEC and displacement ratio. Then the failure will be analysed using the energy stiffness ratio concept.

With a traditional method the stress and strain are applied with a sinusoidal waveform but in ITFT a haversine waveform is applied, thus the dissipated energy looks like half of a typical hysteresis loop (Figure 50) (Maggiore, 2014).

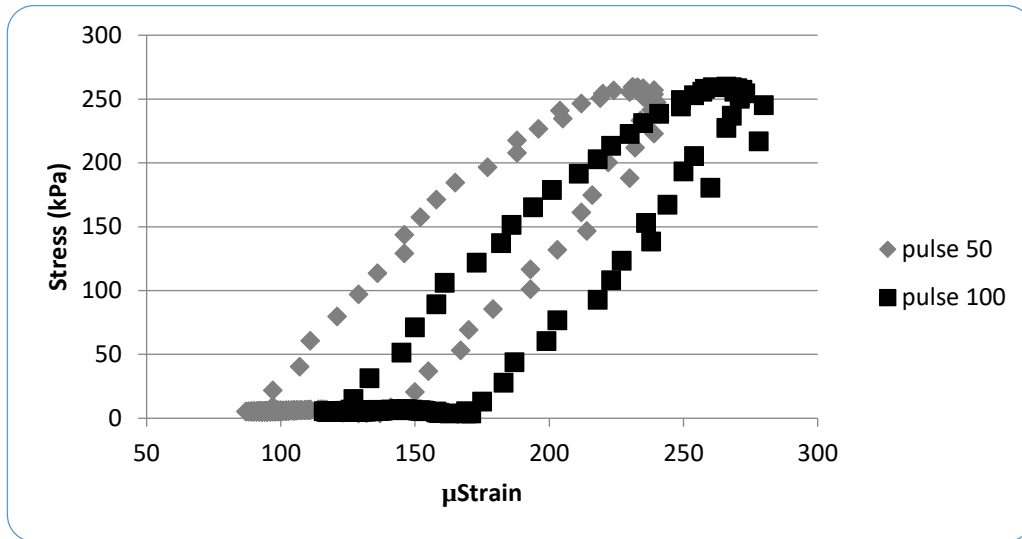


Figure 50 Hysteresis loop with haversine waveform obtained with ITFT in strain control mode

Following, the different approaches studied in this section with the ITFT results from CRBM are presented. In Figure 51 the DER results from the CRBM analysis are compared to those of a HMA. It can be appreciated how the 50% stiffness reduction, represented by the yellow square, is only related to the appearance of macro cracks in the HMA, while in the CRBM macro cracks tend to appear later, around 75% reduction (in this example).

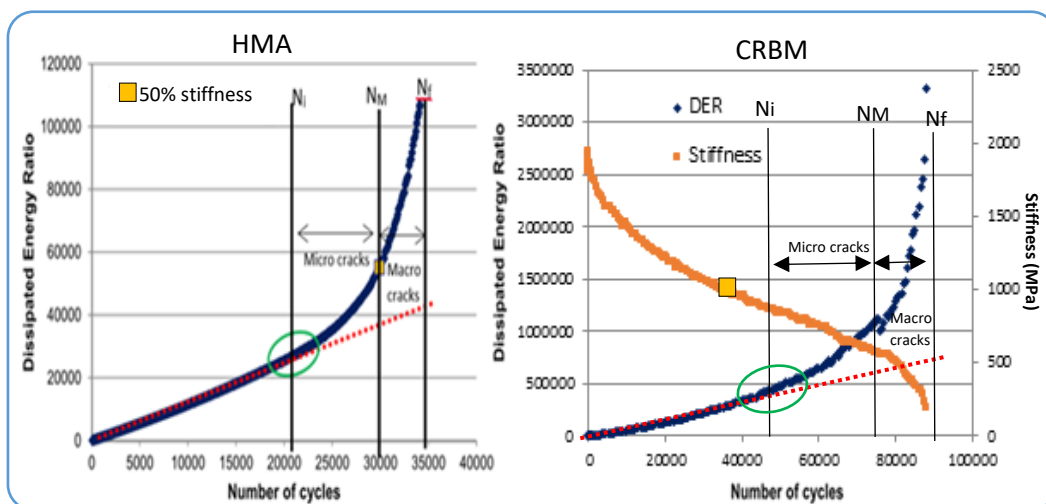


Figure 51 DER HMA vs CRBM

In Figure 52 and Figure 53 it can be observed how the RDEC and the displacement ratio analysis give similar results to the DER in terms of macro cracks initiation. However, the micro cracking initiation differs from DER. In this example, the micro cracking starts around 45000 cycles with the DER approach; however, per the RDEC and displacement ratio approaches the micro cracks start before 10000 cycles. This discrepancy is related to the type of micro cracks (Moreno-Navarro, 2013). N_i in the DER represents the widening of hairline cracks and the creation of a crack network (Maggiore, 2014); therefore if the cracks are dispersed in the specimen the DER approach will not detect them as early as in the displacement ratio approach (Moreno-Navarro, 2013). With this information, it is noted that for this CRBM there is a long period (between 10000 and 45000 cycles in this example) where dispersed micro cracks are being developed.

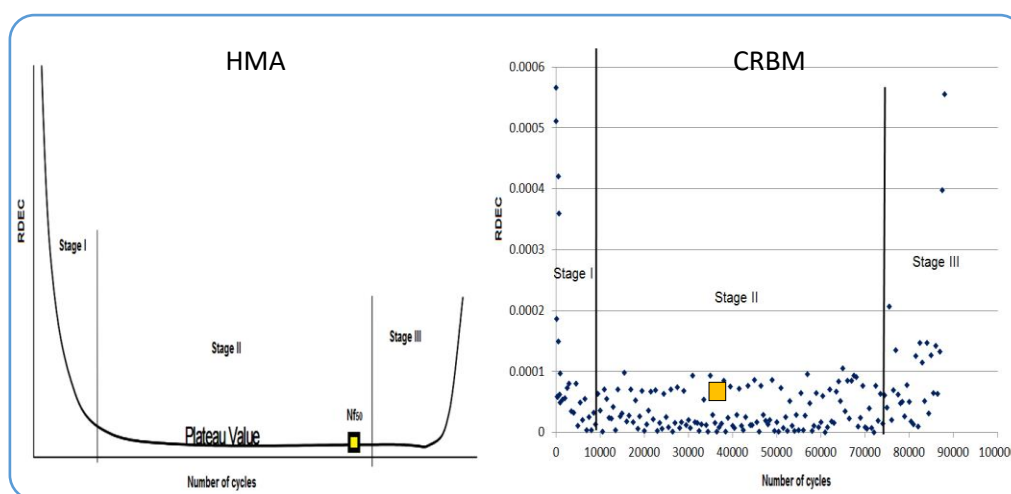


Figure 52 RDEC HMA vs CRBM

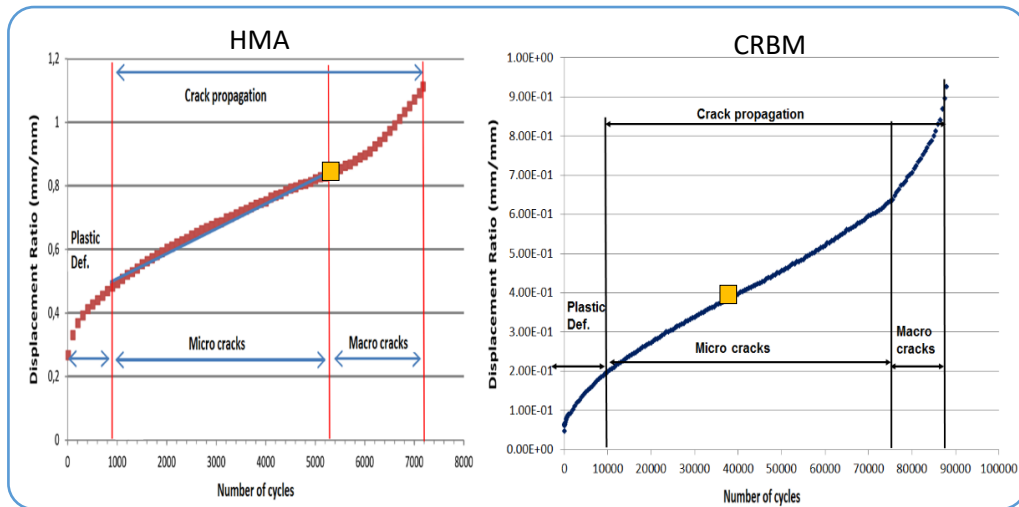


Figure 53 Displacement ratio HMA vs CRBM

Applying the energy stiffness ratio concept, as shown in Figure 54, it is noted that the failure related to the appearance of macro cracks occurs after the 50% stiffness reduction, in accordance with the other analysis approaches.

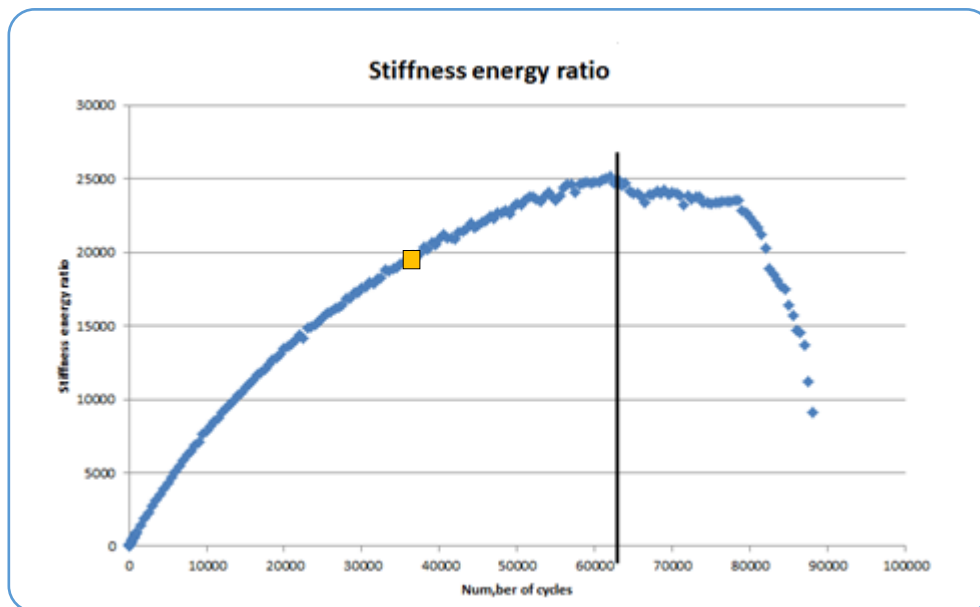


Figure 54 Stiffness energy ratio for CRBM

In Table 26 a summary of the results obtained with the four approaches and the tests carried out until complete failure of the specimens are presented.

Table 26 Failure mechanism results summary

	DER			RDEC		Displacement ratio			Energy stiffness ratio peak value (failure)
	N _i	N _M	N _f	Stage II	Stage III	N _i	N _M	N _f	
Mean	37.3	22.4	13.1	69.9	18.4	62.5	20.7	12.9	27.7
Max.	47.5	30.5	22.0	80.0	30.5	76.0	30.5	22.0	40.0
Min.	24.0	14.5	10.0	58.0	10.5	40.4	15.0	10.0	18.5
Standard deviation	6.6	5.5	4.1	7.5	6.3	11.0	5.1	4.2	7.0

It can be noticed that the appearance of macro cracks occurs between 20.7% and 27.7%, depending on the method used. This highlights the different performance of CRBM in comparison with HMA. However, further study is still needed in order to establish a failure criterion for CRBM, analysing different mix designs, temperatures, tests and field performance.

With these results, it can be confirmed that this CRBM fails in a different way from HMA; therefore, a different failure criterion should be established.

Studies of crack propagation show that, unlike conventional HMA, micro cracking still predominates at the 50% stiffness, and macro cracking only tends to occur at around 20-28% of the initial stiffness.

The study shows a clear fundamental difference in failure mechanism between this CRBM and HMA. The implications of these findings are that basing fatigue models for CRBM on established practice for HMA is likely to be overly conservative. This is in accordance with Nunn and Thom (Nunn and Thom, 2002), who found that the concept of fatigue for CRBM needed to be reconsidered.

In this Thesis, a wheel tracking device was used to reproduce CRBM behaviour under cyclic loading in a more realistic way than with the ITFT and to further investigate the failure mechanism. This method was chosen because WTT reproduces road conditions in a control environment and in a laboratory scale, avoiding time and material consumption.

6.2.3 Wheel track tests (WTT)

The wheel track test device is used to analyse CRBM behaviour under repeated passes of a loaded wheel. With the attached camera, it was possible to analyse crack propagation and compare CRBM behaviour under different conditions (temperature and loading) and with a HMA.

For analysing the crack propagation, the photographs taken by the camera were studied. In Figure 55 an example of crack propagation in a beam is presented. The time when the crack appears and its length are obtained to analyse how the cracks evolve with the different configurations.

In Figure 56 the crack length versus the relative number of cycles is presented to show the crack evolution. The difference in behaviour with different conditions can be noticed. Hot mixes have a quick crack propagation; this means that once the crack appears it grows fast. Cold mixes have the opposite behaviour; the crack evolves slowly. Comparing mix slabs (hot mix over cold mix) both temperature and load have influence on crack evolution. At 5°C the material is more brittle; therefore, the crack growth is faster. With less load, crack evolution slows down as expected.

It has to be taken into account that the camera takes a photograph every minute; therefore, the appearance of the crack cannot be detected immediately. For this reason, in some of the cases, a 40-50% crack appears as the first point. This doesn't mean the crack appeared suddenly, but it could not be detected before in the photographs. This data is used for comparison between conditions only.

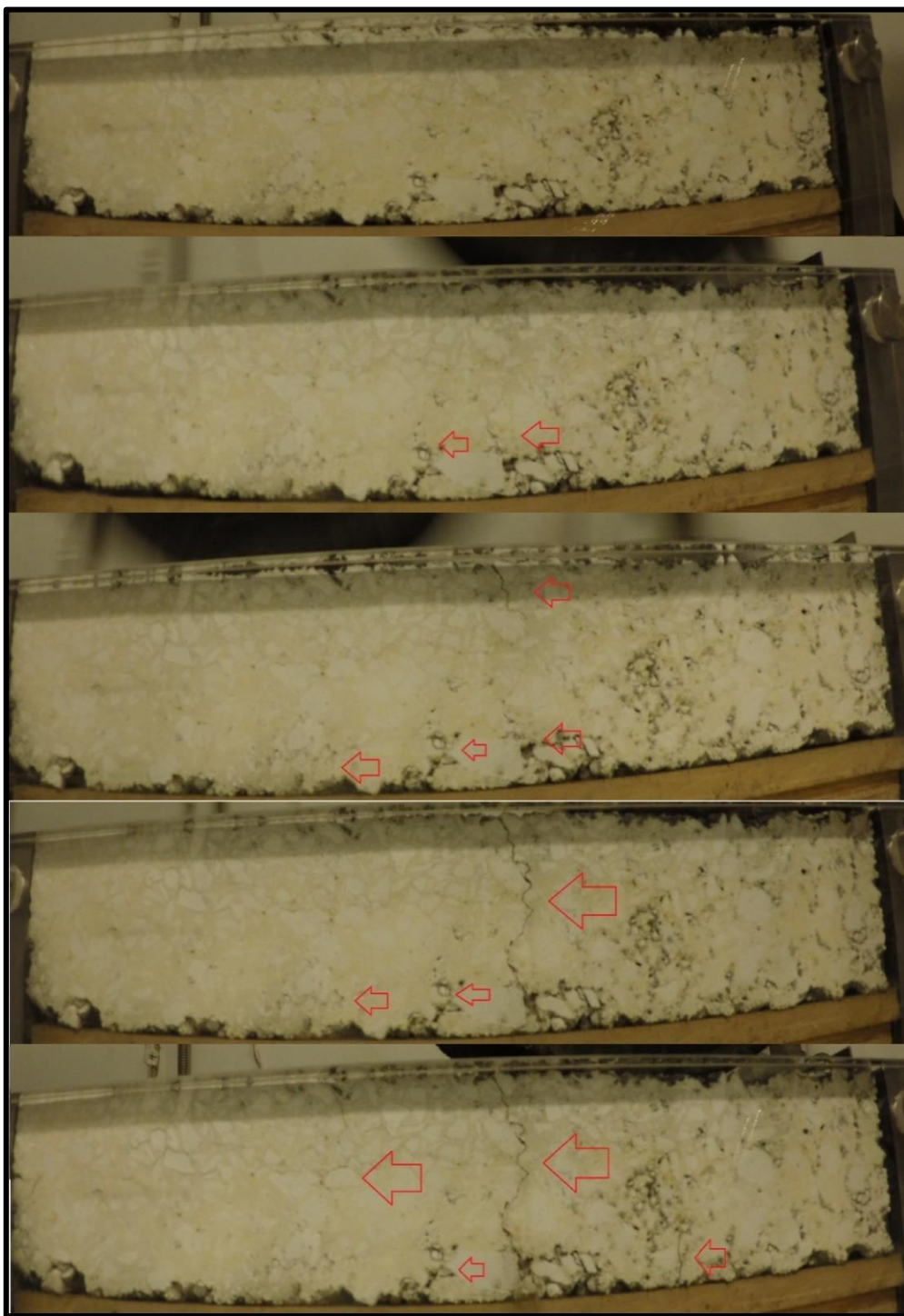


Figure 55 *Crack evolution*

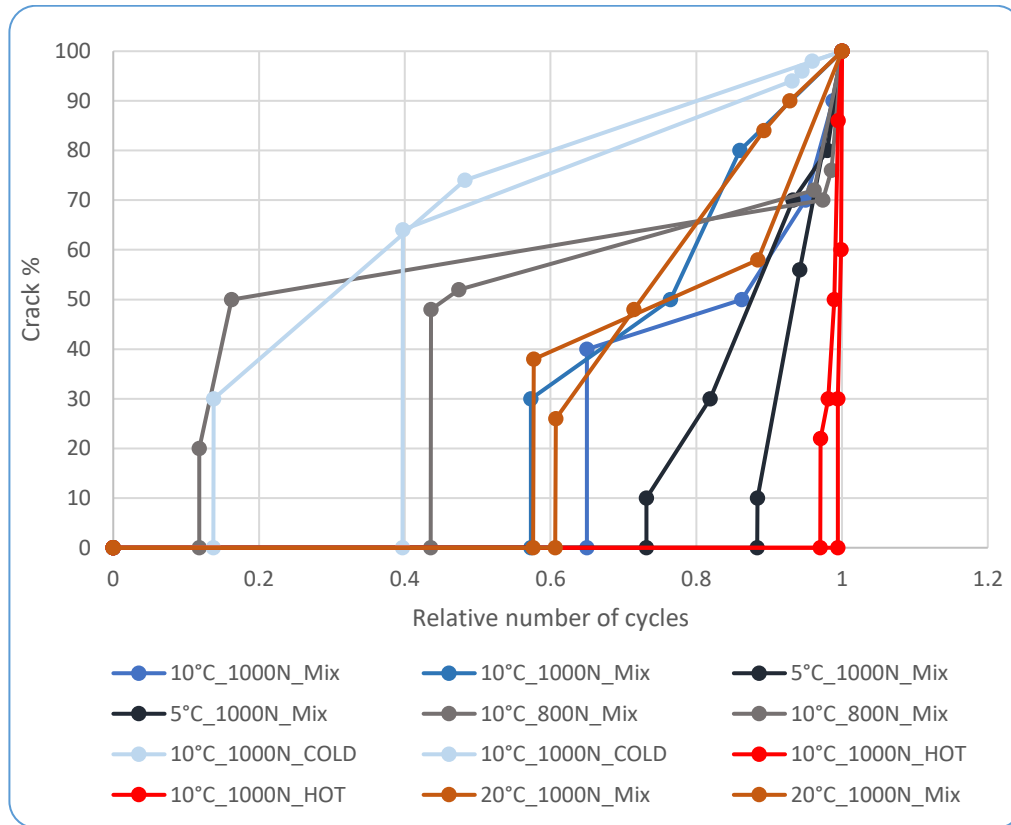


Figure 56 Crack evolution in different conditions

In Figure 57 an example of the final cracks for all the configurations is shown. The main difference is found with the HMA beam. It can be noticed how CRBM presents several cracks in all conditions while HMA develops only one; therefore, the difference in failure behaviour observed in the previous section is highlighted.

The development of several cracks in the CRBM beam can be due to the fact that more micro-cracks appear in early life, reducing the material stiffness. This is also in accordance with the ITFT in strain control results, where micro-cracking predominates when the stiffness is reduced to a 50%. This means that, unlike HMA, CRBM present disperse micro-cracks that evolve to macro-cracks and, therefore, even if the mixture stiffness is reduced significantly, the macro-cracks do not appear.

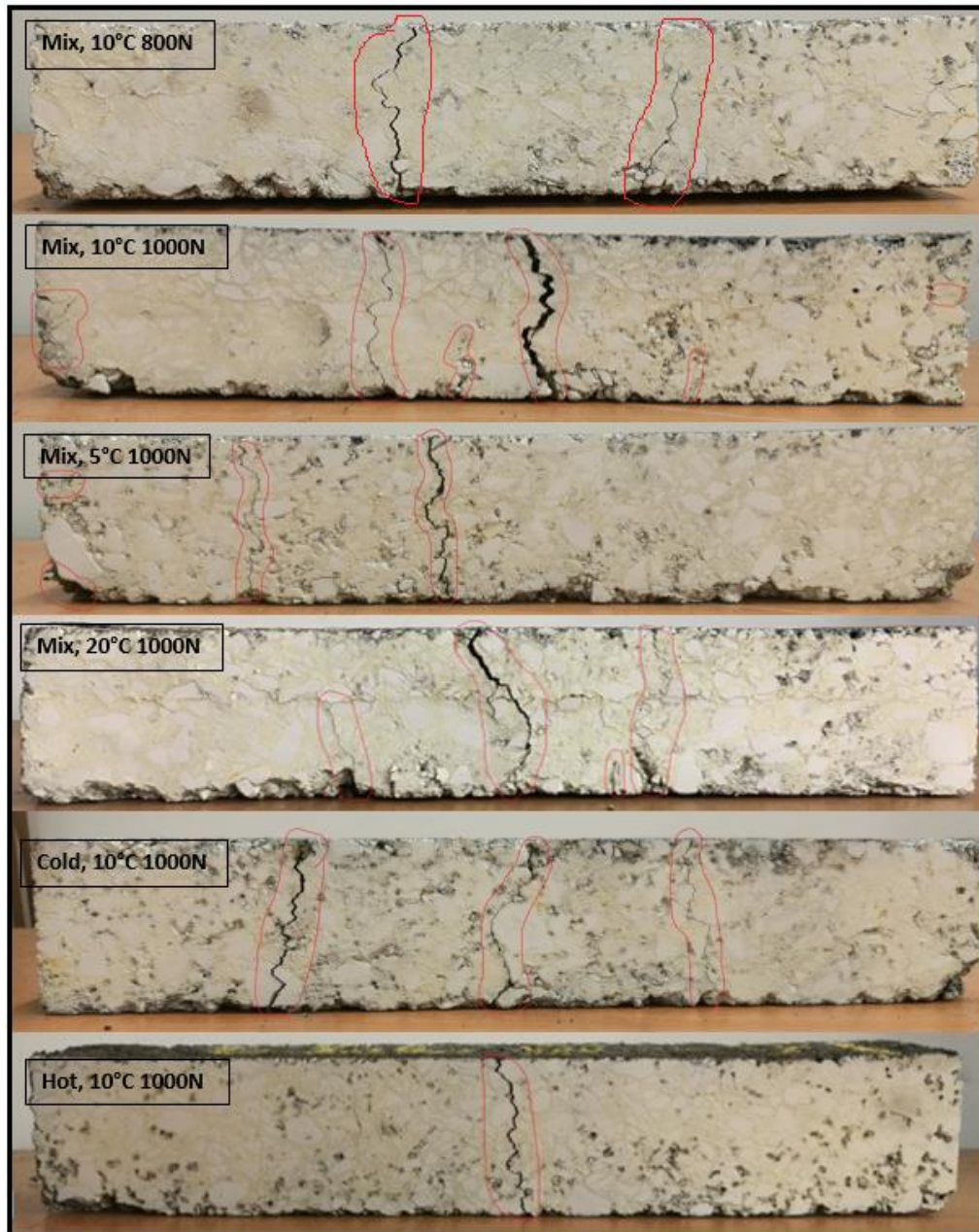


Figure 57 Beams final cracks

6.2.4 New fatigue law for CRBM

With the results presented in the previous sections, it can be concluded that the failure criterion for CRBM could be established as the stiffness reduction to 30%, instead of the 50% established for HMA. Taking this into account a new fatigue law can be established and, as in section 6.2.1, new fatigue coefficients calculated. In Table 27 and Figure 58 the data are presented.

Table 27 Experimental fatigue data for 70% stiffness reduction as failure criterion

N_f	ε_t	E_1 (MPa)
879500	0.00015	4920
872500	0.00015	3551
266000	0.00020	4006
119000	0.00020	3793
208000	0.00025	2418
88500	0.00025	3470
28000	0.00030	3793
96500	0.00030	3754
73500	0.00030	2634

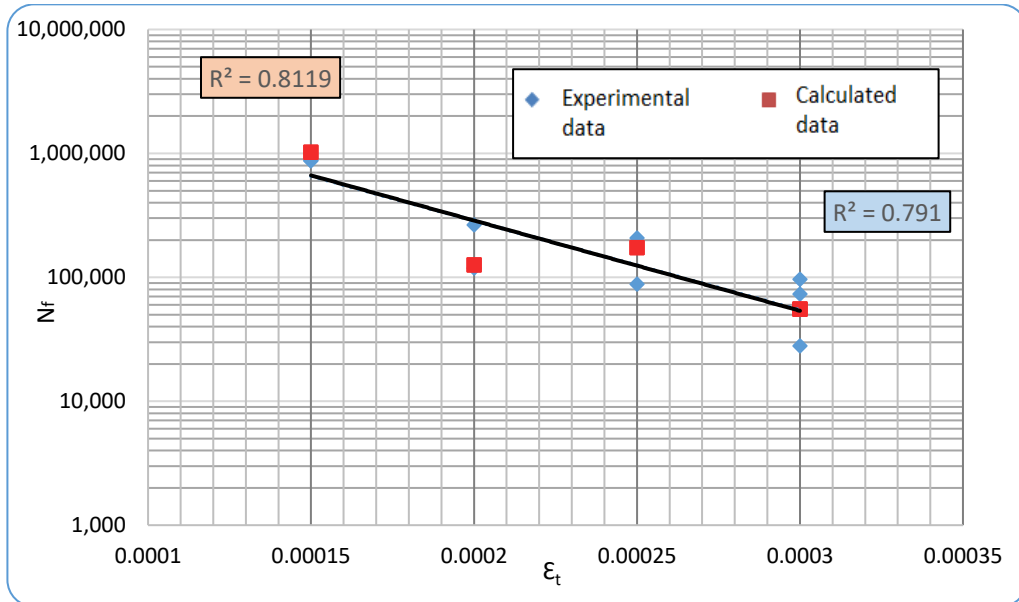


Figure 58 CRBM fatigue law calculation with 70% stiffness reduction as failure criterion

The new fatigue law is presented in equation 12.

$$N_f = 6.45(\varepsilon_t)^{-9.85}(E_1)^{-8.96} \quad [12]$$

The fundamental material input variables for CRBM assessed with the Kenlayer model were identified as stiffness and fatigue with permanent deformation being dependent on the subgrade. These variables are summarised in Table 28.

Table 28 Kenlayer inputs

Input		Value
Poisson's Ratio		0.3
Stiffness Modulus	20°C	3500 MPa
Fatigue factors	f_1	6.45
	f_2	9.85
	f_3	8.96

However, it is important to evaluate how the material deteriorates under cyclic loading. With this aim the evolution of the stiffness monitored during the fatigue tests was analysed. The results are presented in Figure 59 and the same analysis was carried out for HMA for comparison, Figure 60.

The difference between the HMA behaviour and the CRBM can also be appreciated here. In HMA the stiffness reduces with an exponential trend, while the CRBM has a more complex behaviour, having a very fast stiffness reduction in early life, but maintaining a 40% stiffness for a long period, before failing.

These results will be used in the next section to model the pavement deterioration and analyse how this affects the pavement design.

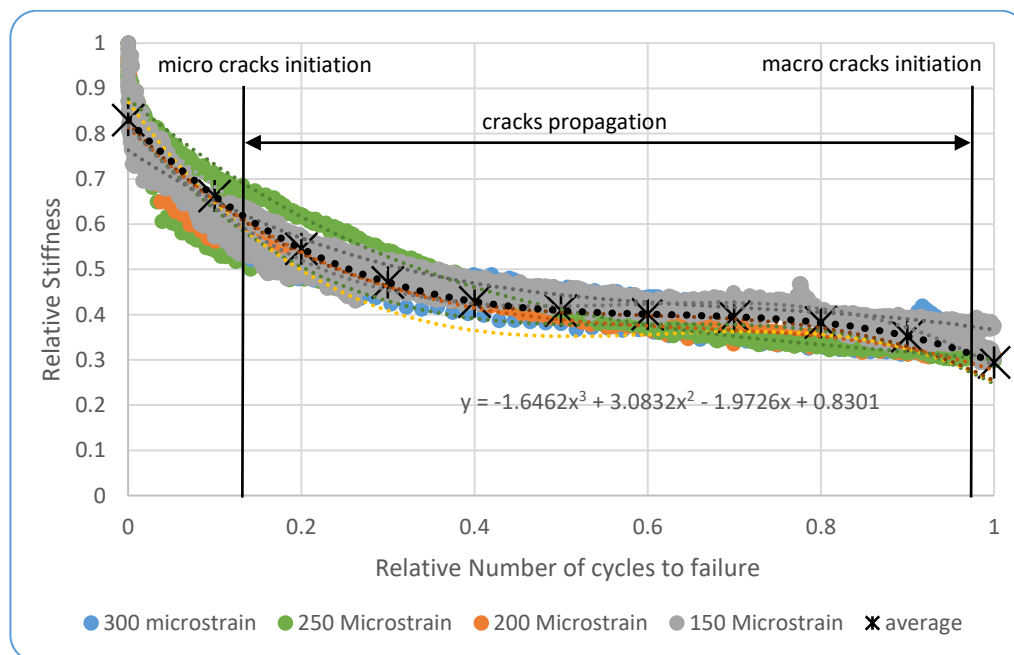


Figure 59 Stiffness evolution during fatigue tests for CRBM

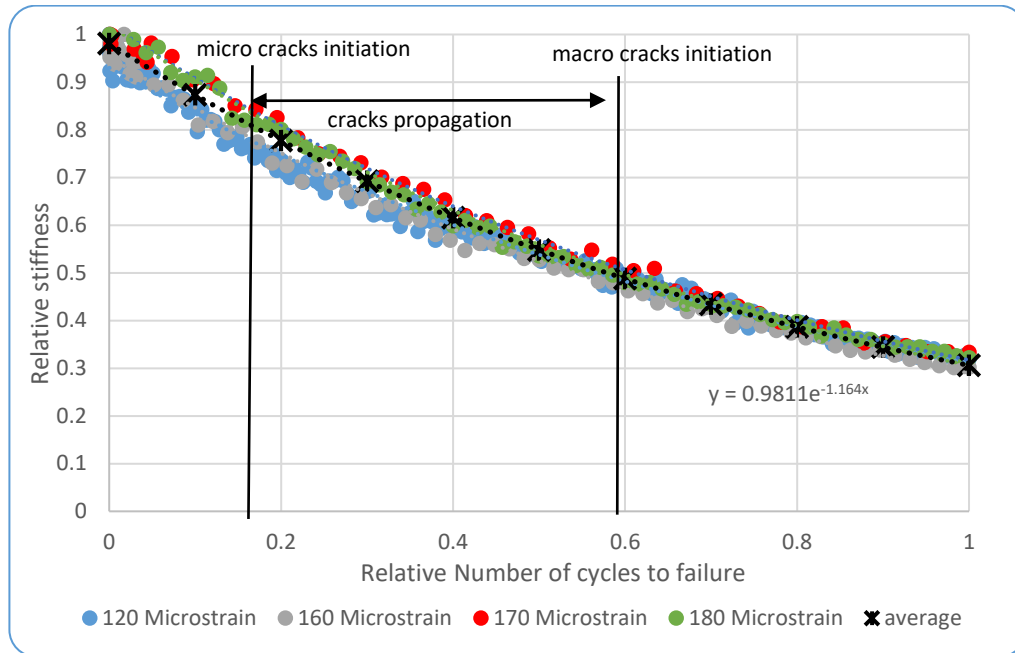


Figure 60 Stiffness evolution during fatigue tests for HMA

6.3 CONCLUSIONS

In this chapter the failure mechanism of CRBM has been analysed. With this aim two testing methodologies were established: ITFT in strain control mode and a Wheel track test.

With the ITFT a fatigue law was obtained; however, it was noticed how CRBM behaviour was different from HMA in terms of failure mechanism. Therefore, further study was carried out to analyse failure of CRBM using dissipated energy concepts. With this study, it was concluded that CRBM develops dispersed micro-cracking that lowers the mixture stiffness without producing macro cracks for a large number of cycles. For HMA, the appearance of macro cracks is related to stiffness reduction to 50%, and this is the established failure criterion. Nevertheless, for CRBM the appearance of macro cracks occurs at 70% stiffness reduction; consequently, it was decided to establish this value as the failure criterion.

To complement these results, the WTT was employed. With these tests the differences in behaviour between hot mix and cold mix, and different test conditions were assessed. Dispersed macro cracks were observed in the cold mix, agreeing with the previous study. Therefore, it was concluded that the new failure criterion is appropriate for this mixture and a new fatigue law could be established.

The effect of temperature and load were also considered and the results show how low temperatures and high load makes the material more brittle, having a relatively fast crack growth.

Finally, the deterioration mechanism of the mixture was analysed. This will be used in the next section to model pavements with CRBM and establish an appropriate design methodology.

However, further investigation into the suitability of tests and failure criteria is needed. CRBM should be studied at full scale to confirm how micro cracking affects the bearing capacity of the CRBM layer and to properly develop a design methodology for this material.

SECTION 4. MODELLING

Section 4 covers the modelling of airfield pavements using CRBM with a view to developing guidance on the design of airfield pavements. This is aligned with objective O3 of this research.

Chapter 7

MODIFICATION OF PAVEMENT DESIGN FOR AIRFIELDS WITH CRBM

In this chapter, a new design methodology is discussed and developed. As stated in Chapter 2, the BAA analytical design approach is used to calculate airfield pavement life; therefore, in this investigation, Kenlayer (software chosen in chapter 2) will be used to calculate stresses and strains within the pavement layers and equation 1 for calculating the pavement life.

7.1 SUITABILITY OF KENLAYER SOFTWARE FOR AIRFIELD PAVEMENT STRUCTURE CALCULATION

For analysing the suitability of the analytical approach and establishing the variables, a range of conventional flexible pavements were analysed for different traffic levels and foundation conditions and the results were compared with those from a conventional well established design guide. As stated in Chapter 2, DMG 27 (Defence Estates 2011) was selected as the reference design guide because it is the guide used by most UK authorities for airfield pavement design.

In the following sections analyses of different scenarios with DMG 27 and Kenlayer are presented. The aim is to establish the design variables that will be constant for further analysis with CRBM, namely those related to subgrade condition, loading and traffic.

7.1.1 DMG 27 pavement analysis

For performing pavement design with DMG 27, the first step was to establish the Aircraft Classification Number (ACN) for each subgrade condition, as detailed in Chapter 2. The ACN of an aircraft expresses the relative loading severity on a pavement supported by a subgrade of a particular strength (Defence Estates, 2011).

For this study, four different subgrade conditions were chosen: 3, 6, 10 and 15% CBR. These are the four standard subgrade categories for which the ACN is quoted by aircraft manufacturers (Defence Estates 2011). 15% CBR is considered high while 3% is considered ultra-low. Subgrades with lower CBR than 3% are considered not to provide appropriate support for airfields due to their non-uniformity. These subgrades should be improved prior to construction.

Eight different aircraft were chosen with variable ACN values between 16 and 130. This is believed to present a wide representative sample of case scenarios to be analysed.

In Table 29 the calculated ACN's for the different aircraft and types of subgrade are presented.

Table 29 *Summary of ACN range*

Aircraft	Gear type	Subgrade types			
		15% CBR	10% CBR	6% CBR	3%CBR
Bae 146-100	Dual	16.3	18.0	20.2	23.9
A320-200	Dual	40.5	42.4	46.8	52.6
A380-800	Dual Tandem	58.4	63.7	75.3	105.5
A318	Dual	32.7	33.8	37.4	43.1
A340-300	Dual Tandem	62.3	65.6	74.2	89.2
A340-500	Dual Tandem	75.3	82.2	97.8	129.8
B767-300	Dual Tandem	39.5	43.3	51.1	69.9
B777-300	Tridem	52.5	58.6	72.3	100.1

These scenarios have been studied at 3 levels of traffic. These are the traffic levels represented in DMG 27:

- Low: 10,000 coverages
- Medium: 100,000 coverages
- High: 250,000 coverages

As explained in Chapter 2, coverages represent the number of times a particular point in the pavement is expected to receive a maximum stress as a result of a given number of aircraft passes. The relationship between passes and coverages depends on several factors including the type of aircraft. Knowing the number of passes of an aircraft (traffic), the coverages can be calculated with a ratio given by DMG 27, namely the Pass-to-Coverage Ratio (Defence Estates, 2011).

For calculating the pavement thickness needed in each case scenario, Chart 7 of DMG 27 for flexible pavement design, using conventional hot mix asphalt concrete base, is used, as it is the chart currently used when designing pavements with CRBM.

The pavement structure was explained in Chapter 2. The surface and binder course have fixed thicknesses of 40 and 60 mm respectively; therefore, the base course thickness varies depending on traffic and subgrade conditions. The base thickness derived from chart 7 is commonly divided into 1/3 of asphalt concrete and 2/3 of Dry Lean concrete (BAA plc Group Technical Services, 1993), with a minimum of 120 mm of asphalt base to mitigate the risk of reflective cracking over the service-life of the pavement (Defence Estates, 2011). With this analysis, the thicknesses of all the scenarios studied here are calculated. The results are presented and compared to those obtained with Kenlayer in section 7.1.3. and an example is presented in Appendix B.

7.1.2 Proposed analytical design method

The first step for analysing the stresses and strains in a pavement structure with Kenlayer is to define the types and properties of materials that are going to be modelled.

Analytical models used for the design of flexible pavements are based on linear elastic theories which assume that each layer is homogenous, linear elastic with a constant modulus of elasticity, isotropic and that the surface layer is free from shearing and normal stresses (Ojum 2015). In addition, the UK design method uses a multi-layer, linear elastic response model (Nunn, 2004). For these reasons the type of material selected in the software was 'linear'.

The load inputs in the software are: gear type, contact tyre radius, tyre pressure, wheel coordinates and the critical locations where the software will calculate the strains and stresses. All these variables are linked into the type of aircraft loading the pavement and are presented in Table 30 for each aircraft selected.

The stiffness moduli underpinning BAA (1993) are 1380 MPa for conventional asphalt and 1207 MPa for DLC. These are design stiffnesses based on field experience, not laboratory stiffnesses. The Poisson's ratios have relatively small effects on pavement response (Huang, 2004); therefore, their values are assumed as standard materials: 0.3 for asphalt, 0.2 for DLC and 0.4 for subgrade (Nunn, 2004; Xu, n.d.).

Once all the variables are defined for each case scenario, they can be input into the software. To determine the design pavement structure, the strain at the top of the subgrade was used to calculate the pavement life based on equation 1, as stated previously in this chapter. The pavement structure is then modified (layer thicknesses) to reach the desired design life. The results are discussed in the next section.

Table 30 Load inputs

Aircraft Used	Gear Type	AUM (kg) MTOW	Load on One Main Gear Leg (%)	Tyre Pressure (kPa)	No.of wheels in each main gear	Contact Tyre Radius (cm)	Wheel Coordinates (m)												Critical Locations to Check (m)											
							1		2		3		4		5		6		1		2		3		4		5		6	
							X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
Bae 146-100	Dual	37308	46	965	2	16.66	0	0	0	0.7							0	0	0	0.3										
A320-200	Dual	77400	46.6	1440	2	19.78	0	0	0	0.9							0	0	0	0.5										
Airbus A380-800	Dual Tandem	562000	19	1500	4	23.57	0	0	0	1.4	1.8	0	1.8	1.4			0	0	0	0.7	0.9	0	0.9	0.7						
A318	Dual	68400	44.5	1020	2	21.59	0	0	0	0.9							0	0	0	0.5										
A340-300	Dual Tandem	275900	37.5	1420	4	23.85	0	0	0	1.4	1.981	0	2	1.4			0	0	0	0.7	1	0	1	0.7						
Airbus A340-500	Dual Tandem	369200	36.5	1610	4	25.56	0	0	0	1.4	1.98	0	2	1.4			0	0	0	0.7	1	0	1	0.7						
B767-300	Dual Tandem	159665	46.1	1340	4	20.71	0	0	0	1.1	1.422	0	1.4	1.1			0	0	0	0.6	0.7	0	0.7	0.6						
B777-300	Tridem	300280	47.4	1480	6	22.37	0	0	0	1.4	1.45	0	1.4	1.4	2.9	0	2.9	1.4	0	0	0	0.7	0.7	0	0.7	0.7	1.45	0	1.45	0.7

7.1.3 Results

The analytical pavement design thickness results using Kenlayer and BAA stiffnesses/performance model are plotted in Figure 61 against the DMG 27 derived thicknesses.

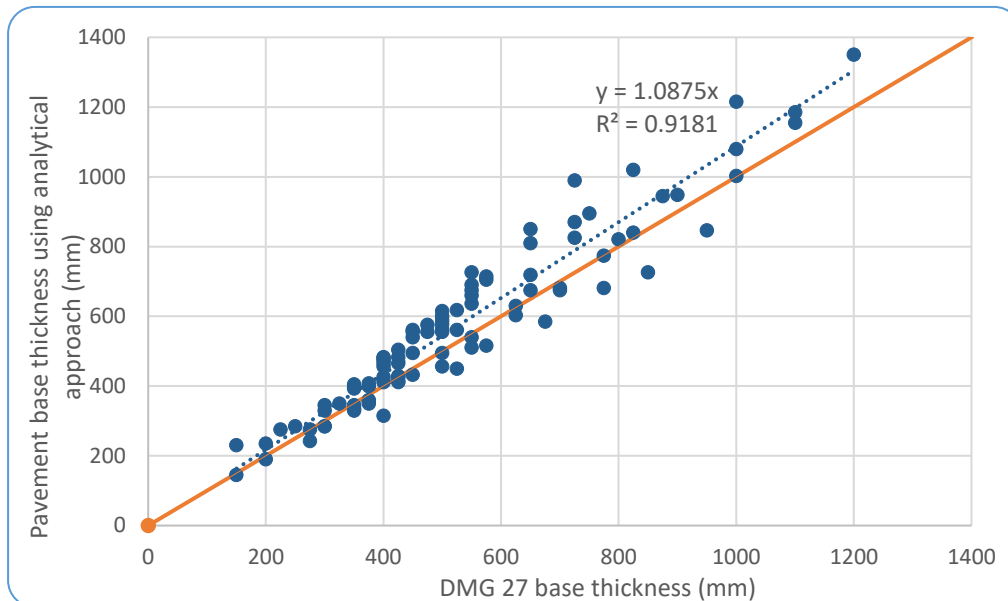


Figure 61 Comparison between DMG 27 derived base pavement thickness using asphalt concrete and thicknesses calculated using the proposed analytical approach

The results showed on average 8% thicker pavement structures calculated using the analytical approach. R^2 has a value of 0.92; therefore, the match is considered acceptable and it can be concluded that the proposed analytical approach is calibrated against a well-established empirical design method, namely DMG 27. Therefore, the variables used are appropriate for airfield pavement analysis and they can be used for analysing the pavement with CRBM.

7.2 KENLAYER ANALYSIS OF THE PAVEMENT STRUCTURE WITH CRBM

Once the design variables had been established (type of material, load inputs, stiffnesses and Poisson's ratios), the analysis with CRBM was undertaken. It has been proven in previous studies that the use of CRBM delays reflective cracking (Dhakal et al., 2016); therefore, in this analysis the minimum 120 mm of asphalt in the base layer was not imposed. Consequently, and to analyse the effect of the asphalt in the base, it was decided to fix a thickness of 150 mm for DLC and vary the CRBM thickness only.

As stated in Chapter 2, for carrying out an analysis with Kenlayer, stiffness and Poisson's ratio are needed. For design purposes, the 15th percentile of the laboratory stiffness values presented in section 5.6 is selected as a conservative approach (Table 31). In previous studies the Poisson's ratio for cold mixes was selected as 0.35 (Ojum, 2015). This value is considered appropriate as Poisson's ratio has low effect on calculations, as stated in Chapter 2. However, a sensibility analysis was carried out to confirm this statement. The results showed that for an increase of 50% in Poisson's ratio, the maximum change in strains is less than 5%.

Table 31 Statistical results for stiffness moduli at 20°C (MPa)

	CRBM	HDM50
Mean	3925	5330
Min	3353	5020
Max	4854	5890
Standard deviation	368	213
15 th percentile	3567	5117
Stiffness used in Kenlayer	3500	5100

The BAA asphalt stiffness used in the previous analysis (1380 MPa) is a design stiffness based on field experience, not a laboratory stiffness. Unlike for road pavements, there is a lack of information regarding this design stiffness for CRBM in airfields.

As the loading scenarios and pavement structures found on highways are different from those on airfields, the design stiffness developed for roads cannot be used in this study. Thus, two different analyses were carried out. The results of both analyses are presented in the next section.

7.2.1 Results

In the first analysis, it was assumed that the CRBM deteriorates in the same way as HMA. A ‘laboratory to design’ stiffness ratio was calculated for HDM50 using laboratory stiffness and design stiffness. Then this ratio was used to calculate the CRBM design stiffness from the laboratory stiffness (see Table 32). This extrapolation assumes the same behaviour for both mixes. In this research, it was proven that this is not true; however, there is no long-term information to calculate the design stiffness of CRBM in airfields. Using the same ratio for CRBM as that for HMA presents a first approximation to a design stiffness value for CRBM.

Both design stiffnesses for CRBM and HMA are used in the analytical design approach and pavement base thicknesses are compared with the DMG 27 derived thicknesses in Figure 62.

Table 32 *Design stiffnesses*

	HDM50	CRBM
Lab. Stiffness (MPa)	5100	3500
Design stiffness (MPa)	1380	947
Ratio	0.27	0.27

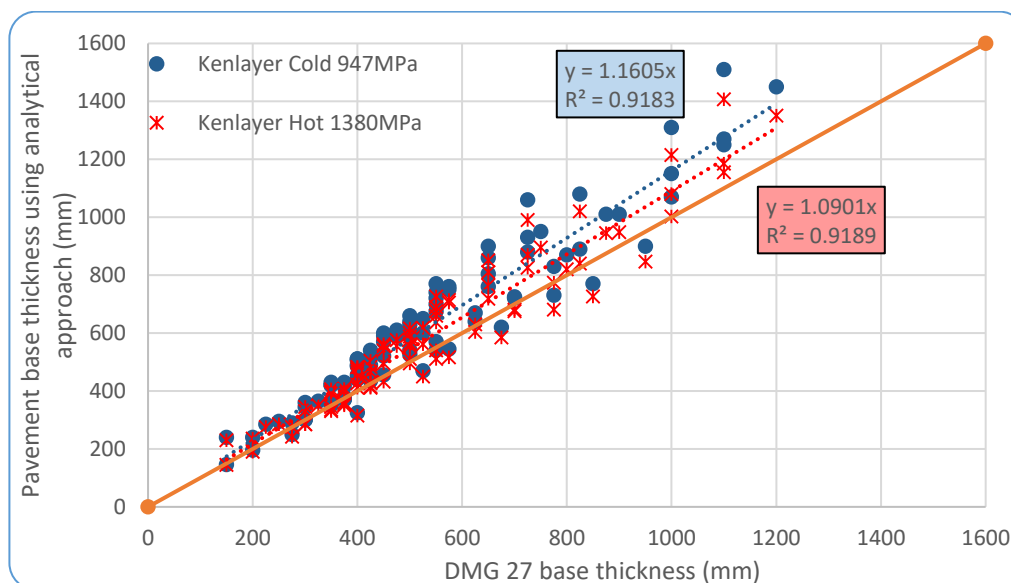


Figure 62 Comparison between DMG 27 derived base pavement thicknesses and analytical approach using hot and cold asphalt design stiffnesses

It can be appreciated that the base thickness using CRBM is 7% thicker on average than with HMA. Analysing point by point, the maximum difference is found to be 8.5%.

The second analysis was carried out using the laboratory stiffnesses of HDM50 and CRBM in the analytical approach (Figure 63).

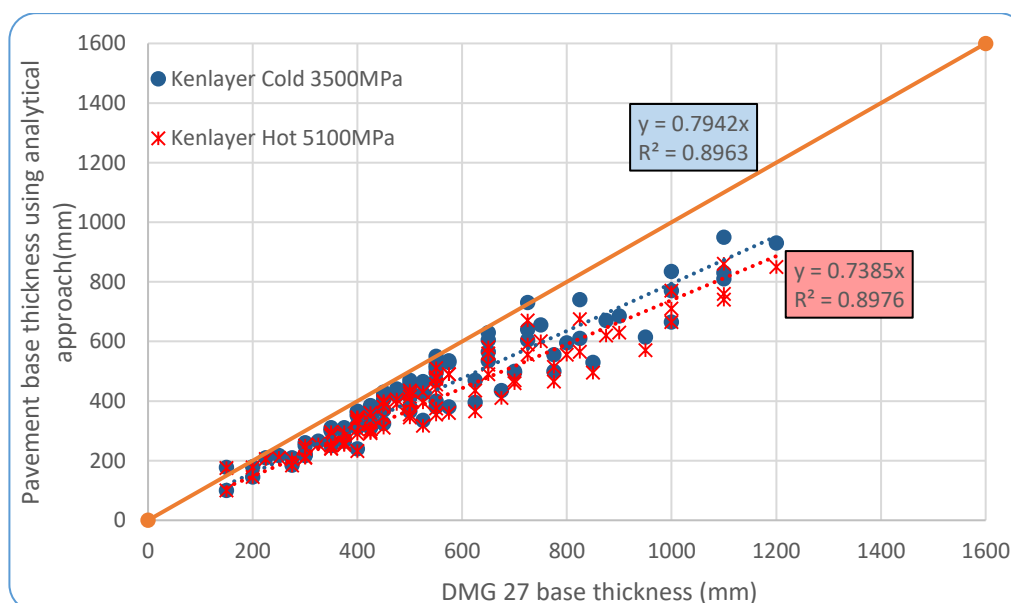


Figure 63 Comparison between DMG 27 derived base pavement thicknesses and analytical approach using laboratory stiffnesses

In this case, it can be appreciated that the average difference between analyses is 5.6%, having a maximum difference between points of 10.5%.

However, this analysis is based only on subgrade strain and it doesn't consider the CRBM performance in terms of cracking. Nunn (Nunn, 2004) highlighted that the asphalt base layer must not crack, and this should be controlled by the horizontal tensile stress at the bottom of the layer. Therefore, further analysis is needed to understand if the CRBM will bear the load for the design life. In this regard, the strain at the bottom of the CRBM layer was analysed for all the scenarios with the laboratory stiffnesses. This approach was chosen instead of the previous one with design stiffnesses as it gave the least favourable results. The fatigue line calculated in Chapter 6 was used to analyse the life of the CRBM layer.

In 20 scenarios, the life of the CRBM was lower than 20 years (pavement expected life, as stated in Chapter 2). Therefore, the thickness of the layer needed to be greater. In Figure 64 the results of the pavement thicknesses considering the fatigue life of the CRBM layer and the subgrade strain are presented. Here, the average difference between analyses is 7.8%, having a peak difference of 19.5%.

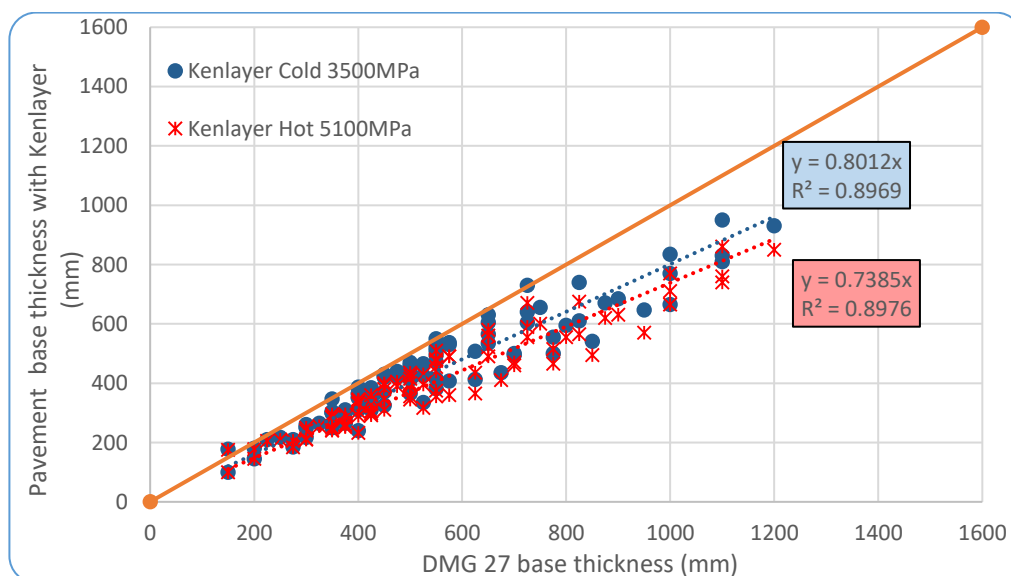


Figure 64 Comparison between DMG 27 derived base pavement thicknesses and analytical approach using laboratory stiffnesses considering CRBM fatigue life

7.3 DETERIORATION ANALYSIS

Laboratory fatigue tests are carried out in either stress or strain control, maintaining one of these variables fixed during the test, as explained in section 3.4.6. However, this does not reflect pavement conditions under traffic, as neither strains nor stress are constant.

The aim of this section is to analyse pavement structures with CRBM and HMA considering that strains vary during the pavement life. With this aim, fifteen case scenarios were built for both materials with 3 different aircraft, 3 subgrade stiffnesses, and 3 base thicknesses, giving a total of 30 analyses as presented in Table 33.

Table 33 Case scenarios for deterioration analysis

Aircraft \ Thickness	Subgrade CBR	100 mm	286 mm	500 mm
A320-200	6%	• •	• •	• •
	10%	• •	• •	• •
	15%	• •	• •	• •
A-380-800	6%		• •	
	10%		• •	
	15%		• •	
B777-300	6%		• •	
	10%		• •	
	15%		• •	

The damage that occurred within the CRBM or HMA base layer was analysed using Kenlayer, fatigue equation 12 (also with HMA coefficients from Table 25) and the damage analysis from section 6.2.4 (Figure 59). The diagram representing these cases is presented in Figure 65. The base layer with CRBM or HMA is divided into 4 section in Kenlayer to analyse how strains distribute within the layer.

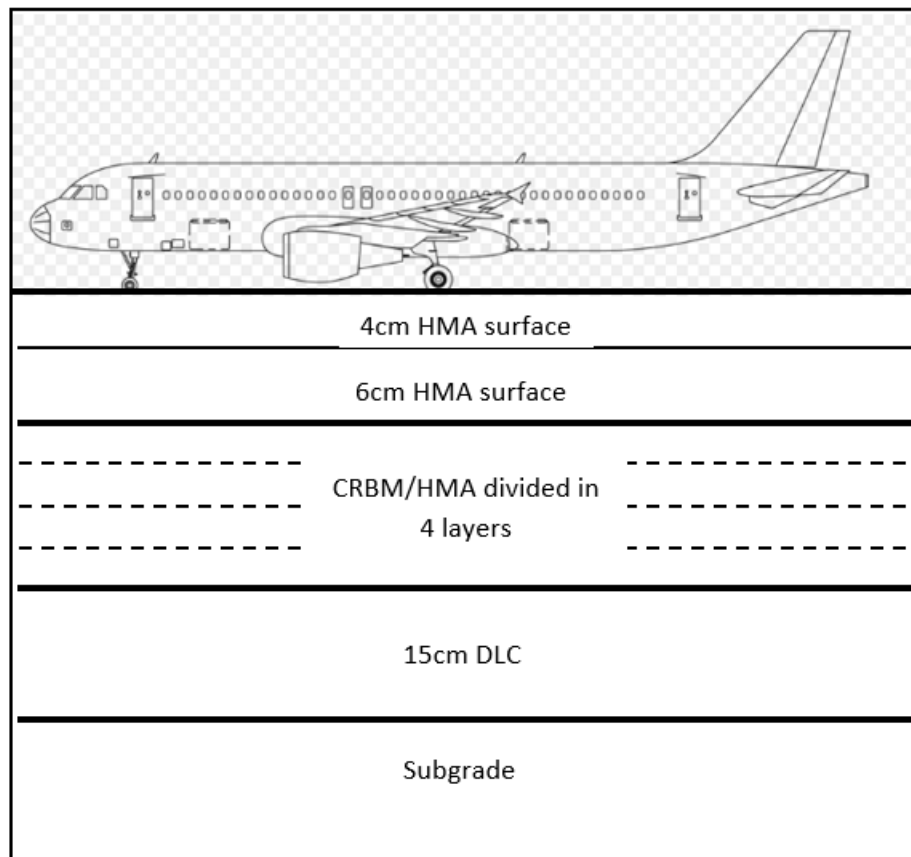


Figure 65 Case scenario for damage analysis

For this analysis, an iterative process was developed.

- The first step was to obtain the strains at the bottom of each layer (ϵ_0) using the initial stiffnesses (E_0) with the multilayer elastic analysis software, namely Kenlayer. Using these parameters, the life of each layer was calculated with the fatigue equation (N_0). Here, failure criterion is 50% stiffness reduction for HMA and 70% for CRBM, as established in Chapter 6. This represents the life of the layers if the strain was constant during the whole pavement life; however, as already stated, this is not a real scenario.
- The second step was to select an iteration range (I). In this analysis, an arbitrary 20% damage of the CRBM or HMA base layer was selected. This percentage, which corresponds to 5 iterations, was adopted to limit the computational effort. However, by increasing the number of iterations a more accurate result can be obtained.

- Next, the layer with least life was selected (usually the lowermost HMA or CRBM layer) and the life interval calculated by multiplying N_0 by 0.2 (I_0). This interval represents the life consumed in the first iteration (after 20% damage has occurred in one of the layers).
- With Figure 59 and Figure 60, new stiffnesses (E_1) of the individual CRBM and HMA layers were calculated after 0.2 N_0 load applications.
- Using Kenlayer and E_1 , new strains were obtained (ϵ_1). These strains were higher than those calculated previously as the pavement had been damaged.
- To proceed with the second iteration (the next 20% of life), the life of the pavement should be recalculated. However, the fatigue equation cannot be applied directly with E_1 and ϵ_1 as this does not consider the damage that has already taken place.
- Hence, another value of stiffness ($E_{1,0}$) which corresponds to the new calculated strain (ϵ_1) must be input into the fatigue equation. This can be obtained from the fatigue analysis which provided a wide range of initial strains and corresponding initial stiffnesses, as in Figure 66.

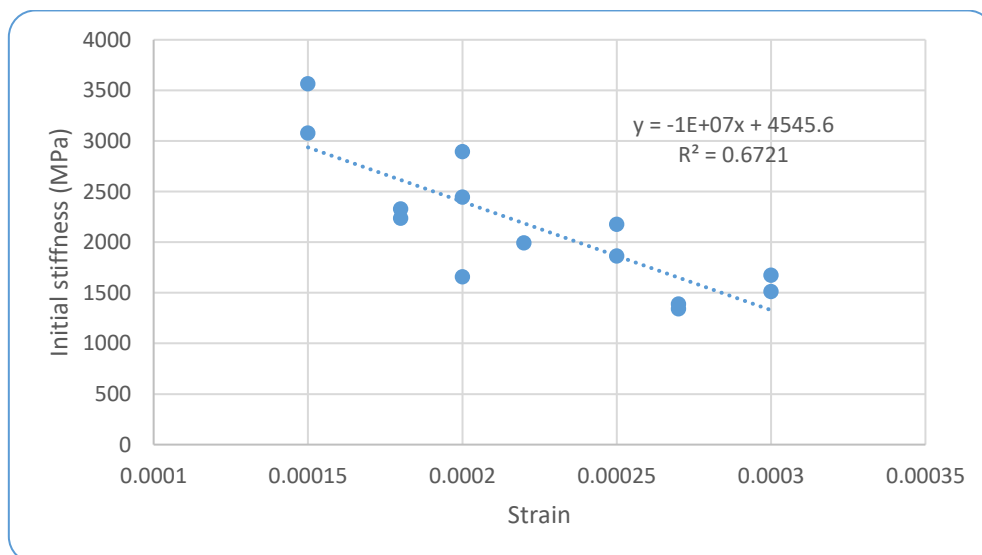


Figure 66 Relationship between strain and initial stiffness

- Using ϵ_1 and $E_{1,0}$ in the fatigue equation, the life of the pavement can be recalculated (N_1) and the new interval obtained ($I_1 = N_1 \cdot 0.2$).

- The following iteration starts by calculating the stiffness after damage (E_2) using Figure 59 and Figure 60.
- This process is repeated 5 times, until 100% of life is reached. The sum of all intervals ($\sum I_i$) gives the total life of the pavement, N .

An example is presented in Table 34.

With these results, two analyses are carried out: a study of the strain evolution within the CRBM or HMA base and a comparison between the life calculated with the fatigue equation (N_0) and with the damage analysis (N).

From Figure 68 to Figure 73, the first analysis is presented, comparing CRBM scenarios with HMA. The strains were measured at the bottom of each layer represented in Figure 65 and the dashed lines show the top and bottom of the base layer (see Figure 67). It can be noticed how the strain distribution is very different between a HMA and a CRBM. In HMA, the strain distribution is linear from the bottom to the top of the layer; however, with CRBM, this phenomenon is not observed.

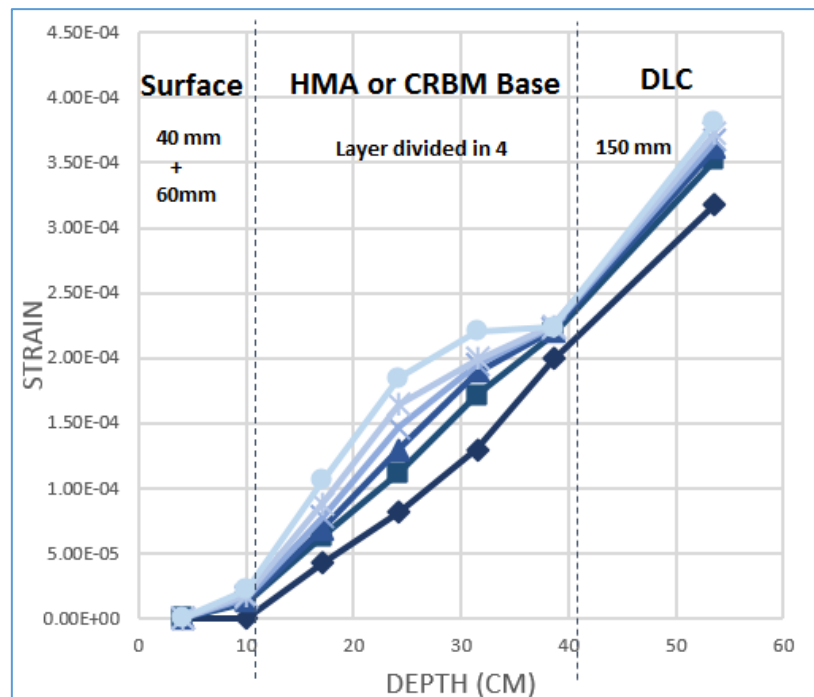


Figure 67 Example of structure represented in deterioration analysis

In Figure 68 and Figure 69, the influence of subgrade stiffness is analysed. It can be observed how for HMA, the strain distribution is similar regardless of the subgrade. However, this changes for CRBM. The greatest difference is found with 15% CBR, where the strain at the bottom of the middle layer is greater than at the bottom of the lowest layer. This phenomenon was also observed in other similar investigations with CRBM (Ebels, 2008; Ojum, 2015; Oke, 2010).

In Figure 70 and Figure 71 it can be observed how the different loading does not affect the strain distribution within the CRBM or HMA layer.

In Figure 72 and Figure 73 the influence of base thickness is presented. In these scenarios, the pavement with HMA base still presents a linear strain distribution, but again the CRBM behaviour is different. In this case, the scenario where the difference is greatest is when a 286 mm base is analysed.

This analysis highlights the different behaviour between both mixes and the necessity of studying CRBM in a different way from HMA.

Section 4. Modelling

Table 34 Iteration example for damage analysis

Iteration	Layer	Stiffness input software	Strain obtained software	$E_{0,i}$	N_i	I_i	ΣI_i	Damage	Stiffness reduction
1	3	3500	4.31E-05	5436	21,978,493,391	25,815	25,815	0.00	0.83
	4	3500	8.20E-05	5047	75,532,806	25,815		0.00	0.83
	5	3500	1.30E-04	4565	1,959,849	25,815		0.01	0.80
	6	3500	1.99E-04	3875	129,076	25,815		0.20	0.55
				Minimum life:	129,076				
2	3	2905	6.35E-05	5232	680,964,331	16,512	42,327	0.00	0.83
	4	2903	1.13E-04	4741	5,838,159	16,512		0.00	0.82
	5	2816	1.71E-04	4154	306,039	16,512		0.07	0.71
	6	1910	2.18E-04	3685	82,560	16,512		0.40	0.43
				Minimum life:	82,560				
3	3	2905	6.98E-05	5169	300,815,952	15,469	57,796	0.00	0.83
	4	2884	1.30E-04	4565	1,959,849	15,469		0.01	0.81
	5	2489	1.89E-04	3973	169,614	15,469		0.16	0.59
	6	1502	2.21E-04	3653	77,345	15,469		0.60	0.40
				Minimum life:	77,345				
4	3	2905	7.87E-05	5080	106,763,400	14,814	72,610	0.00	0.83
	4	2830	1.48E-04	4388	795,869	14,814		0.03	0.77
	5	2060	1.95E-04	3913	142,976	14,814		0.26	0.50
	6	1403	2.24E-04	3631	74,070	14,814		0.80	0.38
				Minimum life:	74,070				
5	3	2904	8.92E-05	4974	37,475,835	14,476	87,087	0.00	0.83
	4	2710	1.64E-04	4227	402,053	14,476	N	0.07	0.71
	5	1734	1.99E-04	3879	130,446	14,476		0.37	0.44
	6	1338	2.25E-04	3619	72,381	14,476		1.00	0.29
				Minimum life:	72,381				

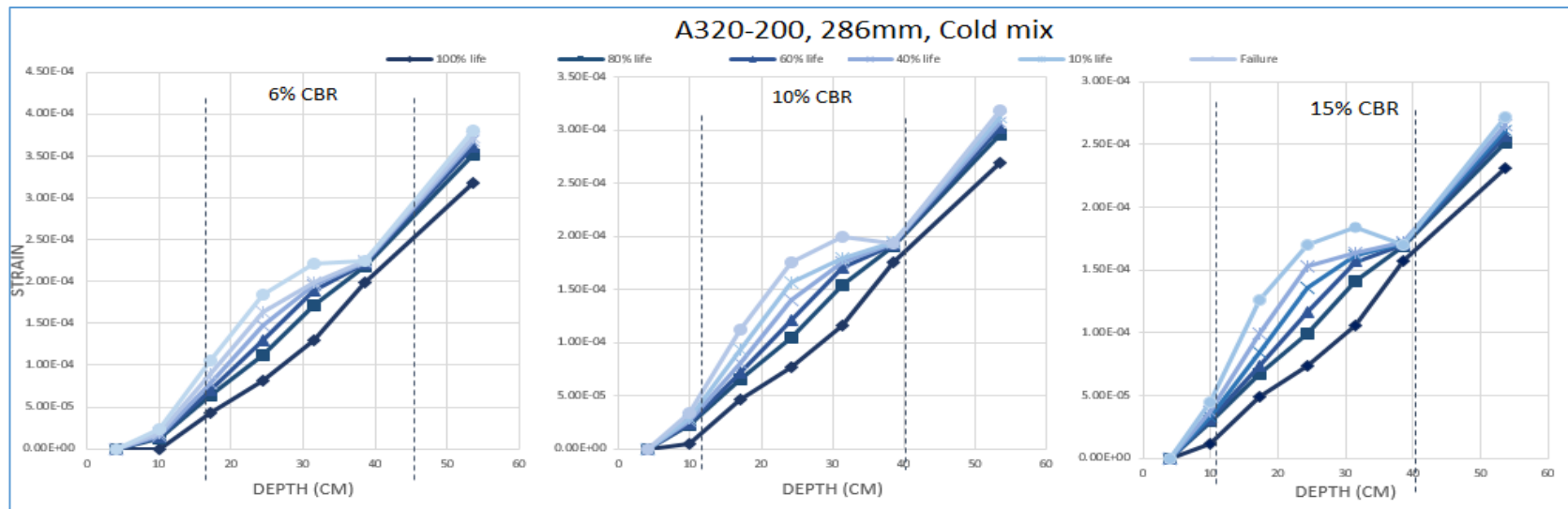


Figure 68 Strain evolution with different subgrade stiffnesses for CRBM

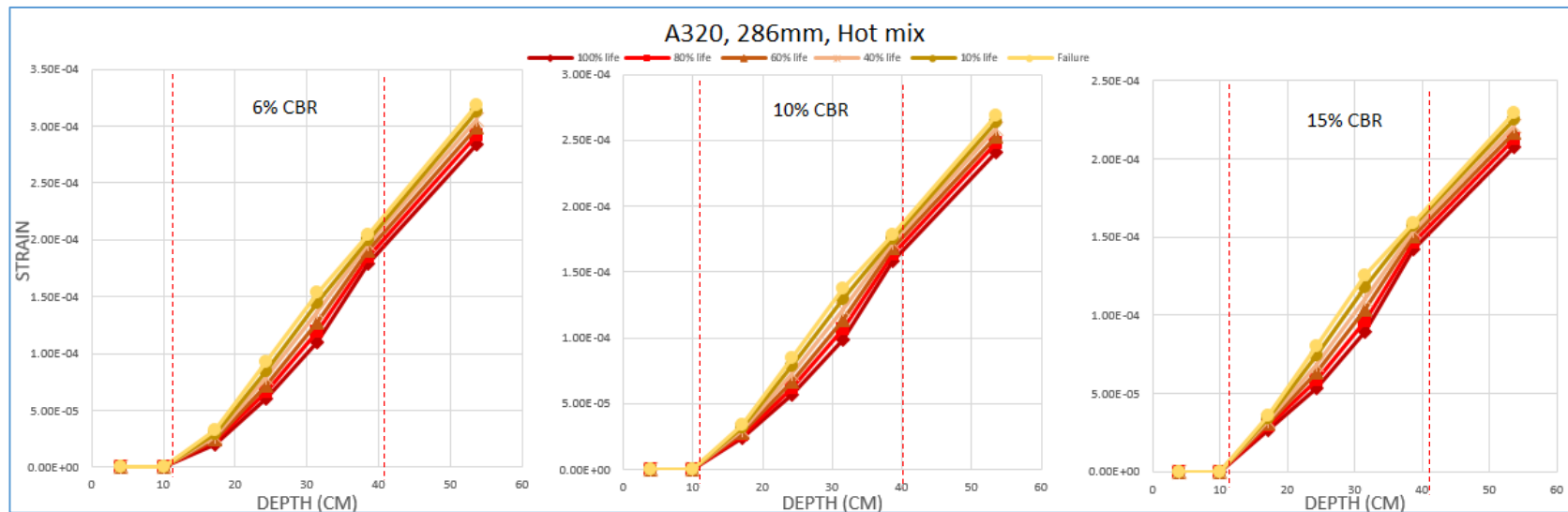


Figure 69 Strain evolution with different subgrade stiffnesses for HMA

Section 4. Modelling

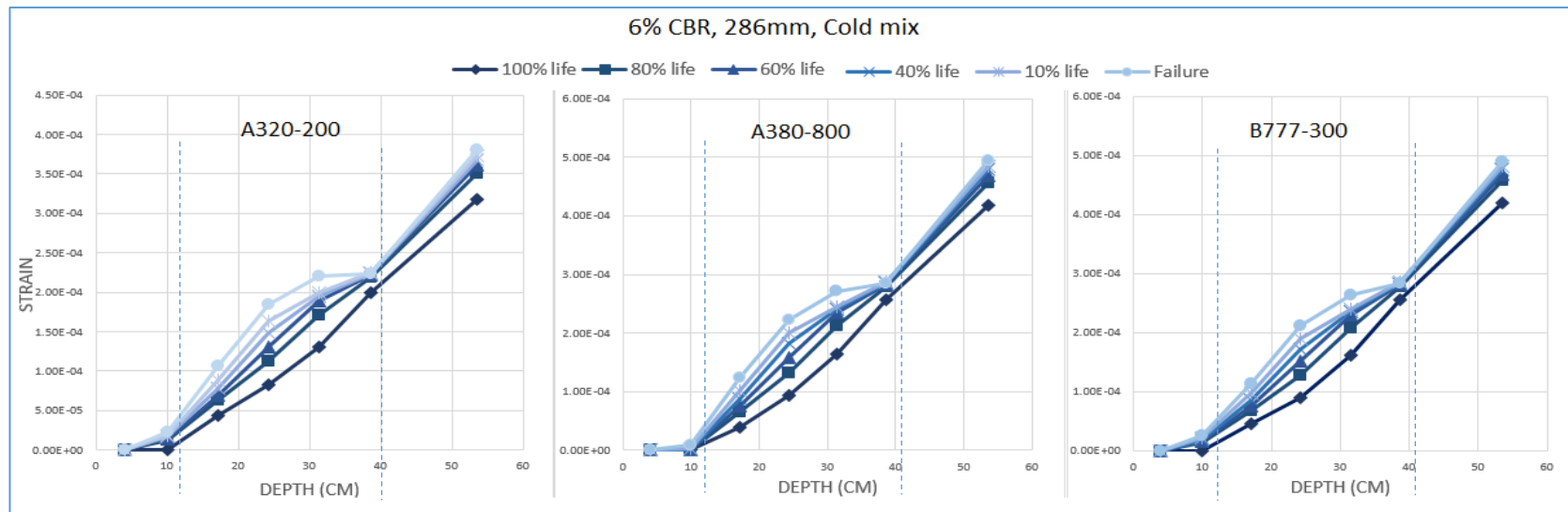


Figure 70 Strain evolution with different aircraft for CRBM

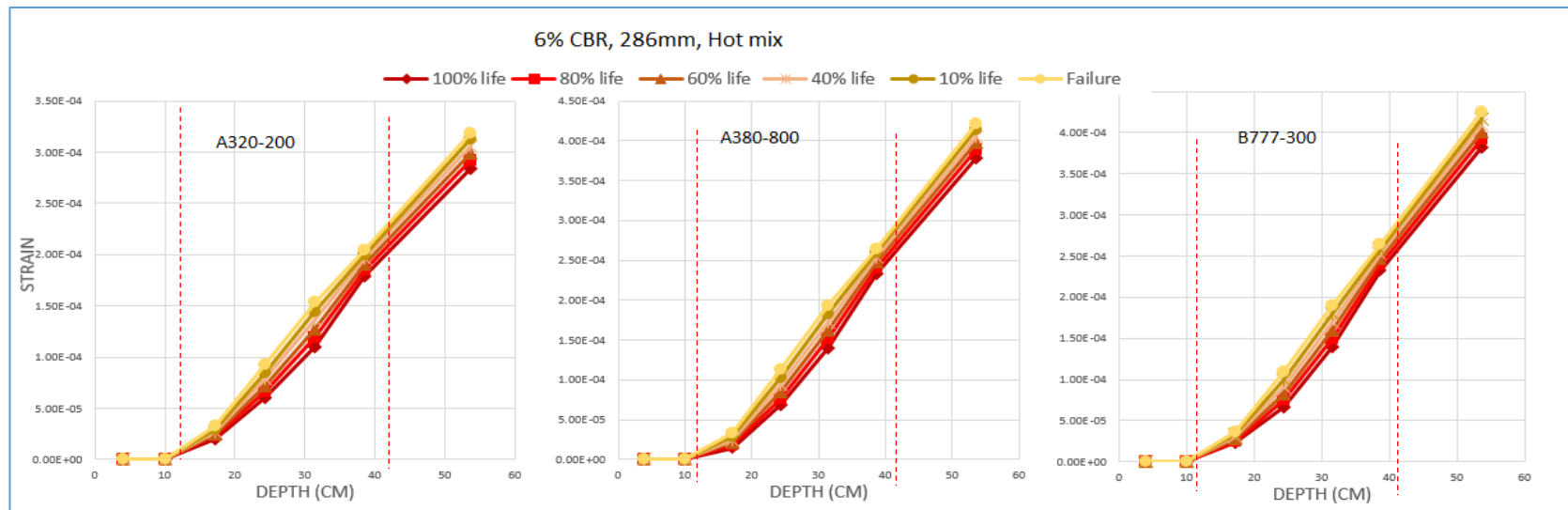
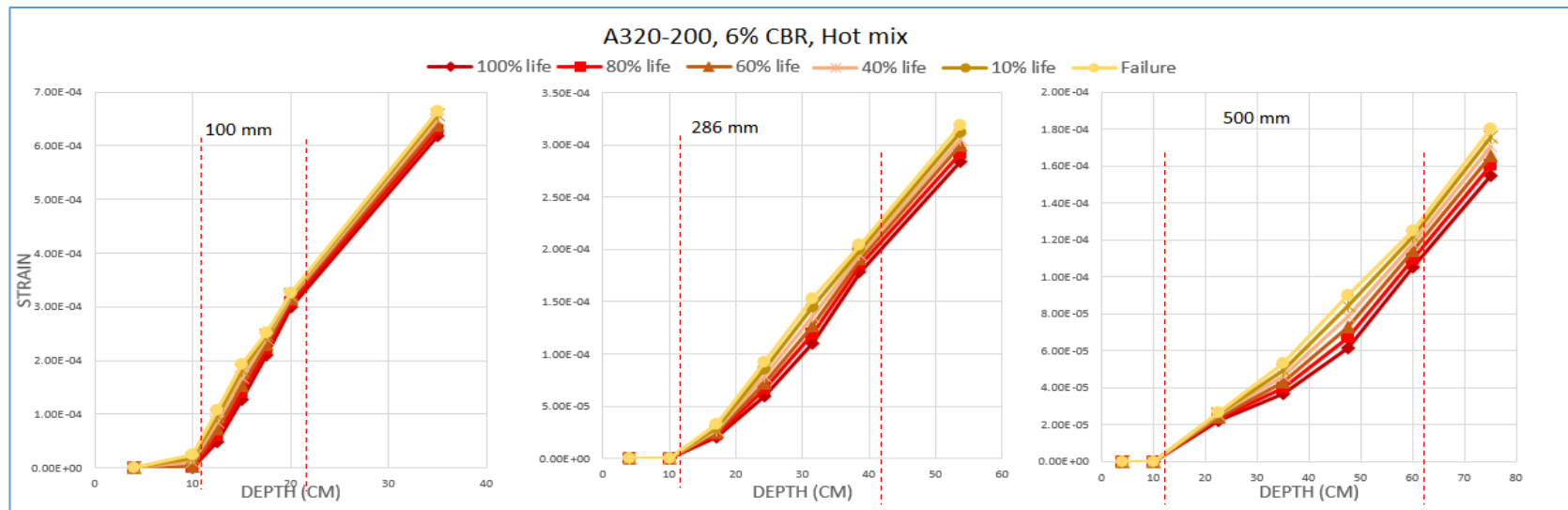
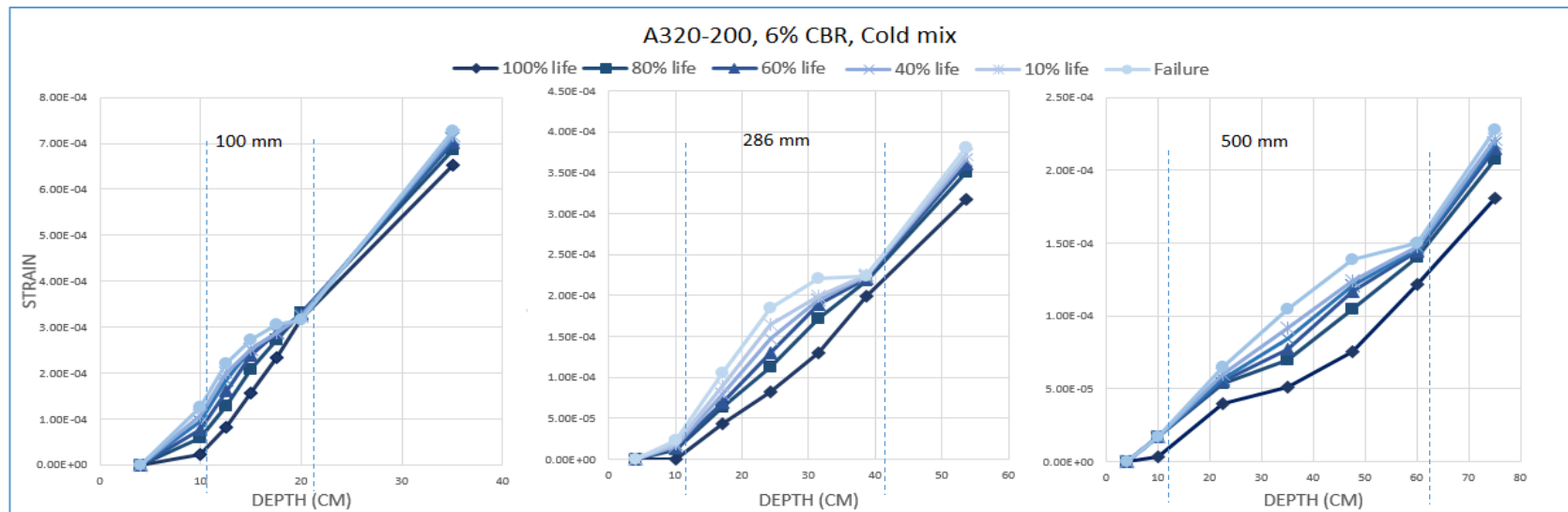


Figure 71 Strain evolution with different aircraft for HMA



In Table 35 the difference between the life calculated with the fatigue equation and the life calculated with the deterioration analysis is presented ($\sum l_i/N_1 \cdot 100$). Results show how CRBM deteriorates more than HMA, having more life reduction than HMA.

Table 35 Life difference with deterioration analysis compared with fatigue equation

ACN	Aircraft	Subgrade	100 mm base		286 mm base		500 mm base	
			Life %		Life %		Life %	
			CRBM	HMA	CRBM	HMA	CRBM	HMA
46.8	A320-300	6	103%	101%	67%	87%	43%	76%
42.4		10	97%	102%	66%	85%	44%	76%
40.5		15	80%	97%	67%	85%	46%	76%
75.3	A380-800	6			84%	95%		
63.7		10			75%	90%		
58.4		15			73%	88%		
72.3	B777-300	6			84%	95%		
58.6		10			75%	90%		
52.5		15			73%	87%		

It can be appreciated how the scenarios with the same aircraft and same base thickness present comparable results. However, when the aircraft (or ACN) and the thickness change, the life difference changes.

In Figure 74 the life reduction is presented versus the ACN. These results show how the greater the ACN, the lower the life reduction. This could be due to the fact that higher ACN gives lower life; therefore, as the deterioration method works with percentages, for lower ACN the interval should be smaller and work with more iterations.

In Figure 75 the same analysis has been carried out using the different thicknesses. In this case, low thicknesses give lower lives; therefore, less difference between methods.

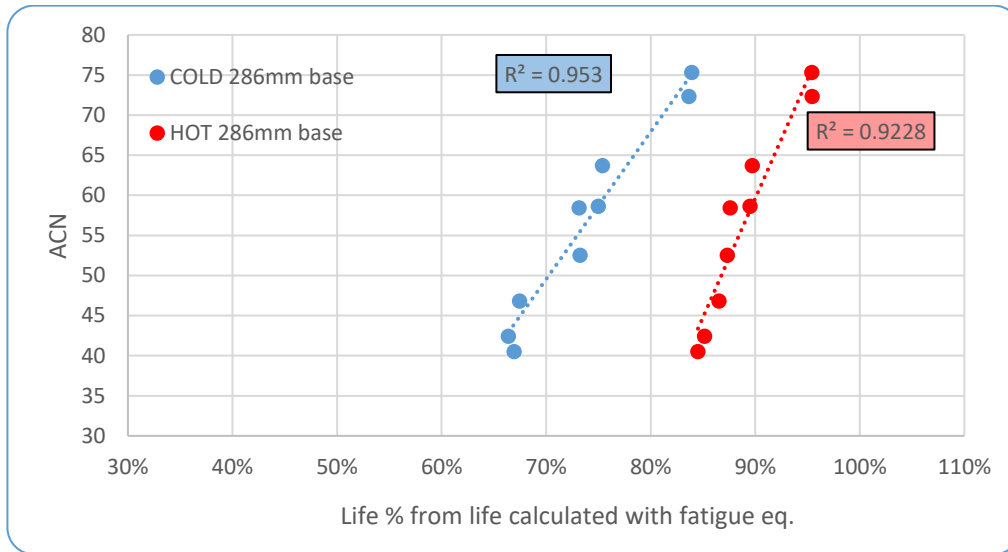


Figure 74 Life reduction for different aircraft

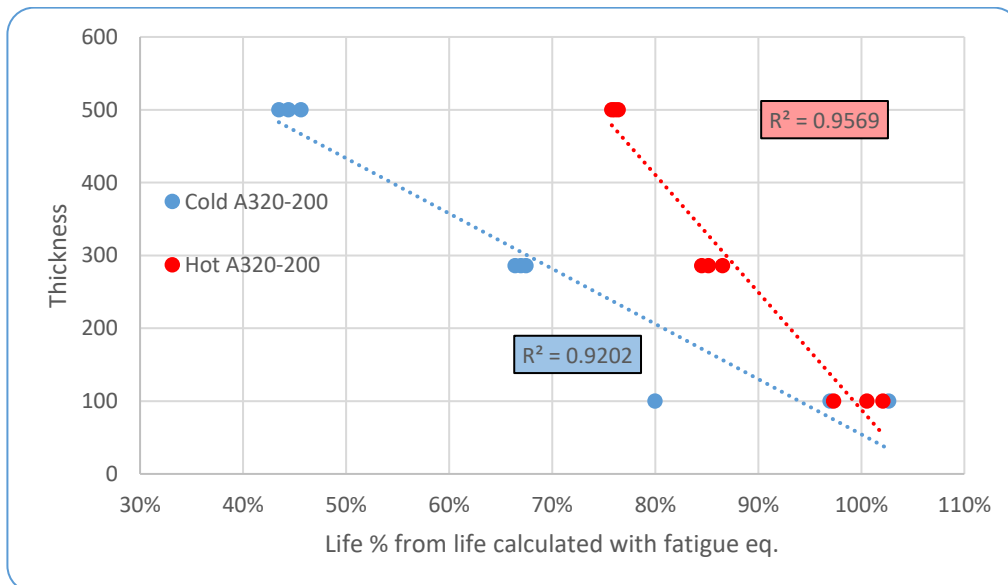


Figure 75 Life reduction for different base thicknesses

It should be noted that the life calculated with the deterioration method is lower than the one calculated with the fatigue equation for both CRBM and HMA.

This analysis highlights the relationship between life reduction and initial life calculated. The greater the initial life calculated is (high thicknesses and low ACN) the greater the difference with the life calculated with the deterioration analysis. In Figure 76 this relationship is presented.

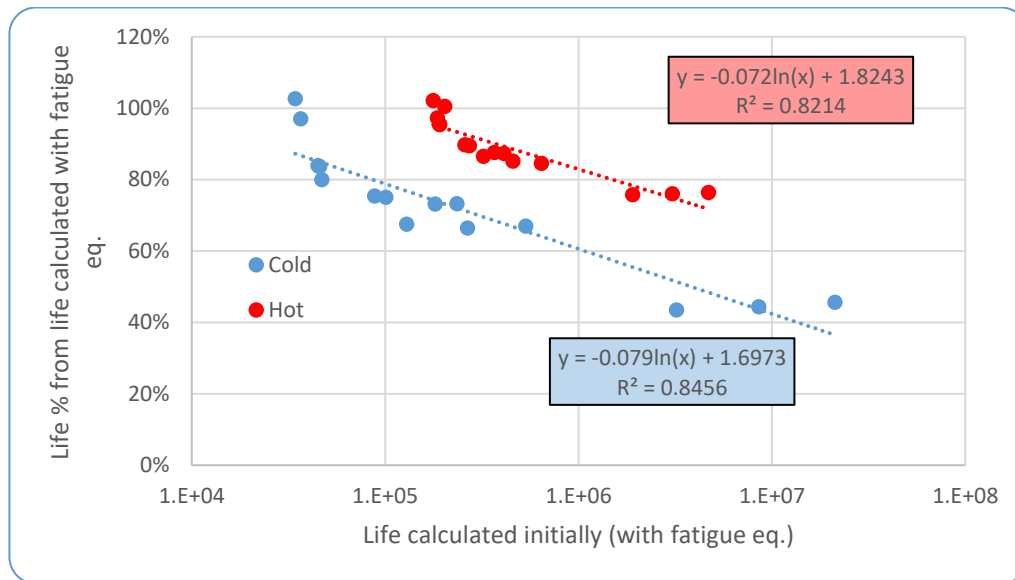


Figure 76 Relationship between life initially calculated and reduction with deterioration analysis

To analyse how this affects the pavement design, the equation presented in Figure 76 is used to recalculate the life of each case scenario of section 7.2. using laboratory stiffnesses and the fatigue equation for CRBM and HMA respectively. Two analysis were carried out. In the first analysis, only fatigue and deterioration is taken into account. This shows how pavement thicknesses should change when using CRBM if only fatigue is considered. In Figure 77 the results of this analysis are presented. It can be noticed how, on average, CRBM thicknesses should be increased by 2% with respect to HMA; however, the maximum difference between points is 14.4%; therefore, if only fatigue is taken into account, the HMA thickness should be increased by 2% when using CRBM.

Nevertheless, in this Thesis the pavement design has been done taking into account the subgrade permanent deformation also. In the second analysis, this is taken into account and the results are presented in Figure 78. Here, the average difference between analyses is 9.0%, having a peak difference of 22.7%.

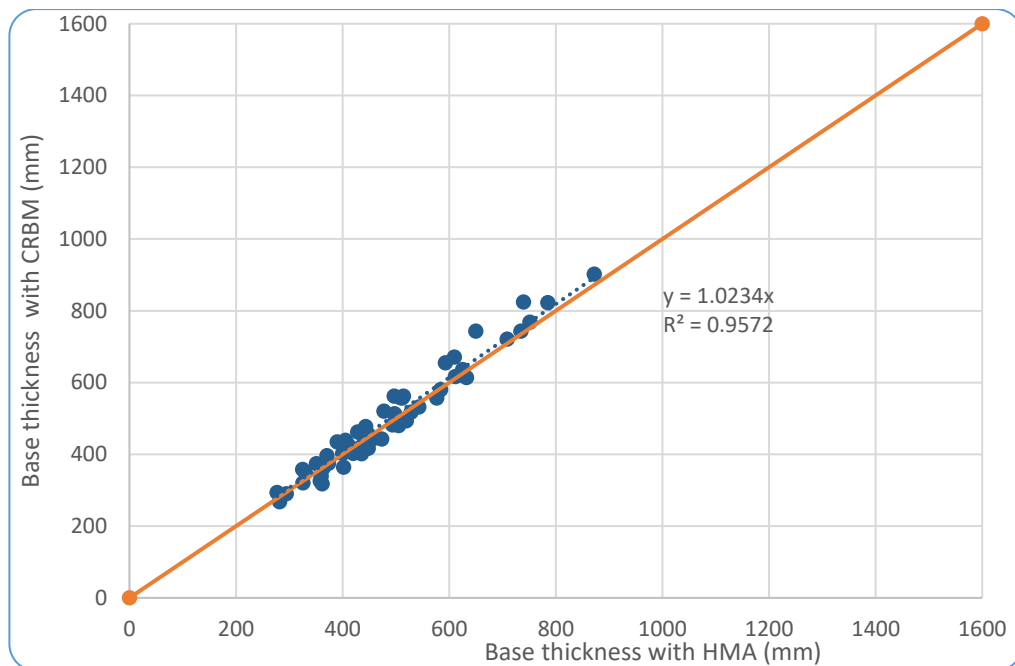


Figure 77 Comparison between HMA and CRBM pavement thicknesses using laboratory stiffnesses considering only fatigue and deterioration analysis

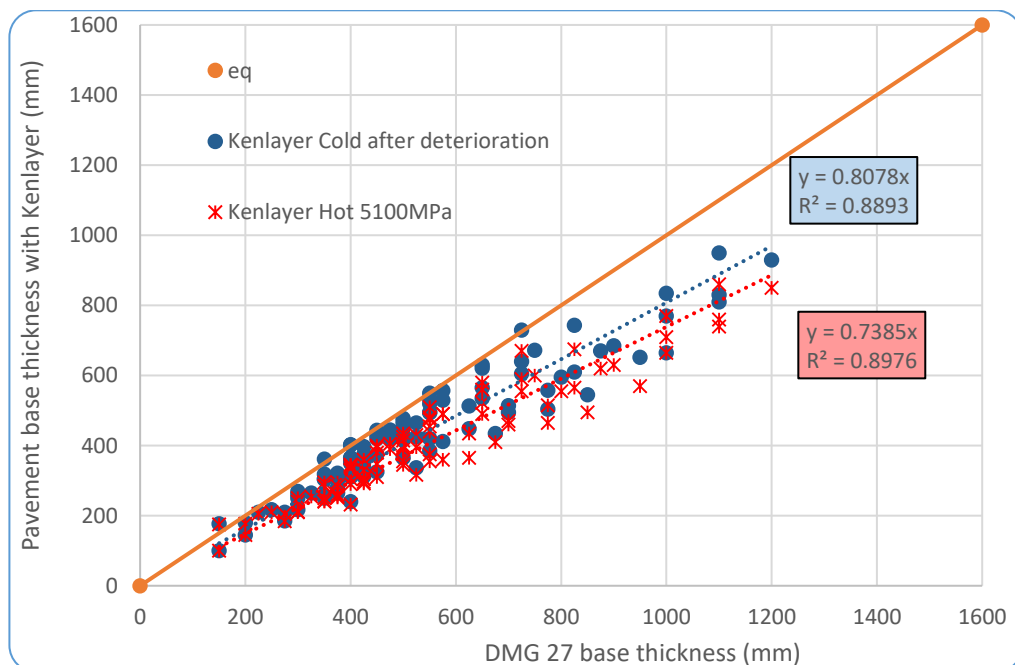


Figure 78 Comparison between DMG 27 derived base pavement thicknesses and analytical approach using laboratory stiffnesses considering CRBM deterioration analysis

Chapter 8

DISCUSSION AND RECOMENDATIONS

As stated in Chapter 5, the CRBM studied in this investigation has the potential to be used in airfield pavements, having good performance in terms of resistance to permanent deformation, fatigue, temperature susceptibility and durability.

In this section, Kenlayer was selected as the pavement design software to analyse airfield pavement structures with CRBM.

The first step was to establish the variables needed to carry out this analysis. With this purpose, 96 case scenarios were analysed with HMA, with different aircraft, traffic and subgrade conditions. The results obtained were compared to those of a well-established empirical design method, namely DMG 27. The results showed that Kenlayer gives thicker pavement structures (8% on average) than DMG 27. However, having a R^2 of 0.92, it was concluded that the software and the inputs selected were appropriate for this investigation.

The next step was to analyse the same case studies substituting the base HMA with the CRBM investigated in this research. The results showed that an average increase of 7.8% of the base layer was needed when using CRBM. However, further analysis was carried out to understand CRBM behaviour.

The deterioration analysis showed how fatigue data can be used to obtain more accurate pavement design life, modelling pavements in a more realistic way, changing strains throughout the pavement life.

With the results from the deterioration analysis a new pavement life was obtained and it could be concluded that DMG 27, Chart 7, can be used for designing airfield pavements using CRBM increasing the base thickness by 9%, with a minimum DLC layer of 150 mm.

In Table 36 an example of how to establish the base thickness when using CRBM is presented.

Table 36 *Proposed design thickness for CRBM*

Base thickness with Chart 7	Final base thickness using CRBM	CRBM	DLC
200	220	70	150
300	330	180	150
400	440	290	150
500	550	400	150
600	660	510	150
700	770	620	150
800	880	730	150

This analysis was also carried out only taking into account the deterioration analysis and fatigue of the asphalt layers only. The conclusion of this analysis was that the HMA thickness should be increased by 2% when using CRBM.

In several investigations where a pavement design life is calculated from laboratory tests, a shift factor is used to adjust the results, as it is well known that the allowable number of repetitions for pavements is much greater than those obtained in the laboratory (Ebels, 2008; Jitareekul, 2009; Ojum, 2015; Oke, 2010; Oliveira, 2006). This shift factor accounts for in-service effects such as lateral wheel load distribution, rest periods and crack propagation, establishing a relationship between laboratory and field performance (Oke, 2010). Unfortunately, there is no information about shift factors for CRBM (Ojum, 2015); therefore, the results presented here are a conservative approach as they are not multiplied by any factor and the use of the average variation is justified.

This research was carried out comparing one CRBM to a HDM50; therefore, further investigation is needed to open the model to other HMA and CRBM. Consequently, the design guidance presented here is a first step towards an airfield pavement design guide and further study is needed to optimise it.

For better understanding of the modelling process, in Figure 79 a flow chart is presented explaining limitations and assumptions.

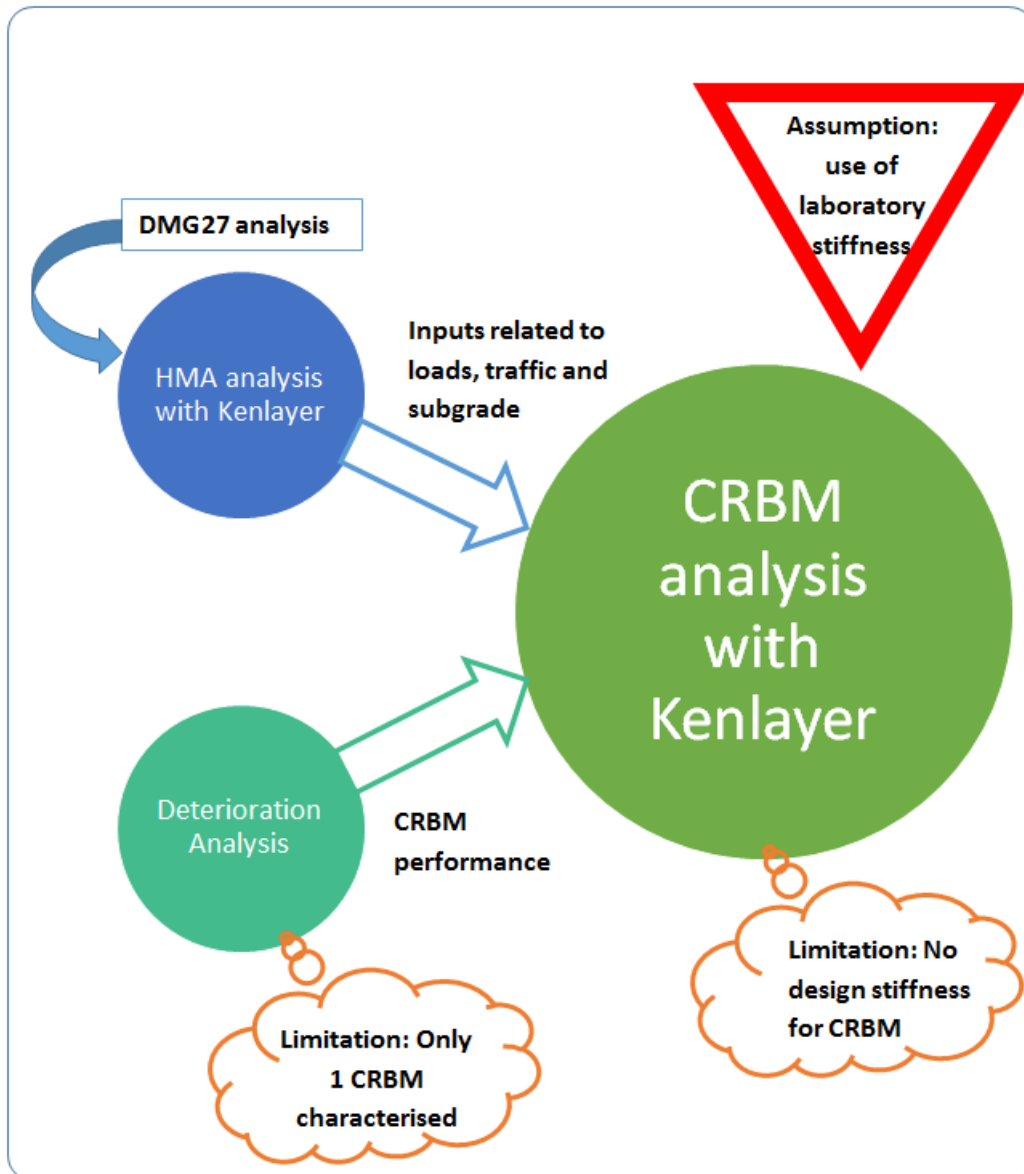


Figure 79 Modelling approach

SECTION 5. CONCLUSIONS AND FUTURE WORK

Chapter 9

CONCLUSIONS

The work presented in this Thesis was carried out as part of the Marie Curie SUP&R ITN project (Sustainable Pavement and Railways Initial Training Network) funded under the European Union's Seventh Framework Programme for research. The SUP&R ITN project aimed to help form a new generation of multidisciplinary researchers capable of conceiving, planning and executing sustainable road and railway infrastructures. The project was divided into three areas: pavement technologies, railway technologies and sustainability assessment. The project presented here is part of the pavement technologies group and the research focused on pavement design with cold recycled materials.

The aim of this specific research project was to develop a design methodology to use CRBMs in pavements. In order to achieve this aim, 3 objectives were established as listed below:

- O1 Review of field experience on performance of cold-mix asphalt, in-situ and off-site recycled, and pavement design methods.
- O2 Design an appropriate laboratory test programme in order to measure effects of key variables on performance and establish practical limitations on the use of cold recycled materials.
- O3 Develop a pavement design methodology. Propose appropriate specifications and design guidance for airfield authorities and practitioners.

After a thorough literature review the following main conclusions were developed:

- Cold recycled Bound Materials (CRBM) use is increasing in road construction and rehabilitation; however, its use in airfield is less common.

- There is no specific airfield pavement design guidance to use CRBM; therefore, an investigation focusing on airfield pavements is justified.
- CRBM can be used in an airfield base layer.
- An analytical design software is needed to analyse airfield pavements with CRBM and Kenlayer was selected for this purpose.
- Inputs for carrying out pavement analysis were identified: load, traffic, subgrade condition and material properties (stiffness, fatigue and Poisson's ratio).
- The use of RAP increases the level of sustainability, reducing raw material consumption and avoiding material disposal.
- When tar is present in old asphalt layers that need rehabilitation, hot recycling or recycling into unbound materials is prohibited; therefore, cold recycling becomes an economically attractive option.
- The optimum RAP grading is established for its use in UK pavements by Transport Research Laboratory report TRL611, which is primarily developed on experience within low to medium traffic volume roads.
- Foamed bitumen was chosen instead of bitumen emulsion as the cold technology for this study. It is more economic, commonly used in the UK, needs less curing time and it present less challenges with moisture content and early-stage performance.
- Cement improves the CRBM mix performance when used in relatively low dosages compared to that used in hydraulically bound mixtures.

- The performance of CRBM using RAP as the coarse aggregate is different to that using uncoated aggregates, and some published information utilises relatively high cement additions. Both of the above point are important when considering the likely performance of CRBM.
- Two curing procedures were selected: 3 days at 40°C and 28 days at 20°C. These are based on industry and research. It is acknowledged that they are laboratory curing procedures and may not reflect conditions on site.
- Characterisation tests were identified as mixture volumetrics, binder penetration grade and softening point, ITSM, RLAT, ITST and ITFT in stress and strain control mode.
- Fatigue is a research priority as it is the main failure mechanism in flexible pavements.
- Using dissipated energy concepts, the failure mechanism and crack propagation can be analysed.
- Wheel Tracking tests can also be useful for analysing crack propagation.

After this literature review, objective O1 was achieved and the research could be focused on objective O2. With this aim, a laboratory programme was established. The main objective was to characterise CRBM and analyse if current design practice could be optimised. At the moment, when an airfield pavement is designed with CRBM, its properties are generally equated to those of a hot mix; therefore, to understand if this practise is accurate, CRBM properties were compared to those of a HMA commonly used in airfield base course. Conclusions drawn after this laboratory programme are summarised as:

- CRBM manufactured with RAP, fly ash, cement and foamed bitumen (with optimised mix design) has acceptable properties for use in airfield pavements in terms of resistance to permanent deformation, fatigue, temperature susceptibility and durability.
- The measured results for CRBM differ from conventional hot mixes; therefore, actual design guides should be optimised and calibrated to include performance of CRBM. The main difference is found in the fatigue behaviour.
- In previous investigations, it was established that curing specimens wrapped for 28 days at 20°C simulated CRBM field performance just before trafficking; and for 3 days at 40°C approximates 6 months of curing on field. However, the results obtained in this investigation showed very similar results between both curing procedures; therefore, an accelerated curing procedure could be developed for this type of mix using 40°C over 3 days. However, more research is needed in this regard to establish an appropriate curing procedure.

Once the principal characteristics of CRBM were established, and after carrying out a further literature review on fatigue, it was decided to focus on fatigue performance and the failure mechanism of CRBM. This decision was made in order to understand how CRBM behaves under cyclic loading and to be able to properly model CRBM layers within a pavement.

For this purpose, two testing methodologies were established: ITFT in strain control mode and Wheel track test (WTT). The conclusions are summarised below:

- CRBM develops dispersed micro-cracking that lowers the mixture stiffness without producing macro cracks for over large numbers of cycles.

- In HMA, the appearance of macro cracks is related to a stiffness reduction of 50%, and this is the established failure criterion.
- For CRBM the appearance of macro cracks occurs at 70% stiffness reduction.
- WTT showed dispersed macro cracking in the CRBM, agreeing with the results from ITFT in strain control.
- A new failure criterion is established as 70% stiffness reduction for this mixture and a fatigue law could be calculated.

Once objective O2 was achieved, the research was focused on pavement modelling to achieve objective O3. The first step was to establish the proper inputs for the analysis. For this purpose, 96 theoretical scenarios were analysed with HMA, with different aircraft, traffic and subgrade conditions. The same case studies were then analysed replacing the HMA base with the CRBM investigated in this research. The main conclusions of this study can be summarised as:

- Kenlayer and the inputs selected were appropriate for this investigation.
- A relationship between CRBM and HMA base thicknesses has been found. However, further study is needed to validate and optimise CRBM layer thickness as laboratory fatigue results are not directly equivalent to field behaviour.
- The deterioration analysis highlighted the different behaviour of CRBM and how fatigue data could be used to analyse pavement life in a more realistic way.

- If only asphalt layer fatigue behaviour is taken into account when designing a pavement, the relative HMA thickness (note this is a HDM50) should be increased 2% when substituting in CRBM. However, it should be considered that the thickness increase varies between 0 and 14.4%.
- DMG 27, Chart 7, can be used for designing airfield pavements using CRBM increasing the base thickness by 9%, with a minimum DLC layer of 150 mm. The same consideration should be done here as in the previous point: these thickness increase varies between 0 and 22.7%.

It must be considered that this analysis was carried out comparing one CRBM and one HMA, namely HDM50. Therefore, further research is needed to analyse different HMA and CRBM mixes, analysing the variability of the model. However, the analysis was carried out without taking into account any shift factor to relate fatigue behaviour in the laboratory and in the field; thus, the use of an average thickness variation is justified.

Chapter 10

FUTURE WORK

This section discusses the areas of research that could improve airfield pavement design with CRBM. Four main areas of investigation could be enhanced: mix design, curing, testing and fatigue data analysis.

1. Mix design:

Carrying out further research with different RAPs and mix designs is important as it will help to analyse the variability of the mixture properties and how they affect the design model. With this information, the variability of the design model could be established, helping authorities and practitioners to understand how the CRBM should be selected.

This investigation focused on CRBM with foamed bitumen and cement; however, there is need of further research to understand how other types of fillers, such as hydrated lime, affect the mix behaviour. It is also known that a high amount of cement can improve the mixture strength, however it also makes it more brittle; therefore, other types of filler could avoid this drawback.

2. Curing procedure:

An investigation should be carried out analysing curing procedures in the laboratory and establish a relationship with the results obtained on site. This requires an extensive curing period on site since coring of CRBM is prone to damage the samples if done within 6-12 months of in-service life. Also, analysing if the accelerated curing procedure is accurate could save time in the laboratory and quicken the investigation of these kinds of materials.

3. Testing:

Further investigation into the suitability of tests is also needed. Nowadays the tests developed for HMA are the ones used for CRBM. As has been proved in this investigation, the behaviour of CRBM differs from HMA; therefore, the adequacy of the tests should be investigated.

During this research, there were some problems with the WTT in terms of confinement and instrumentation; hence, investigation to improve this device is needed, accounting for crack initiation and propagation measurement.

Finally, large scale testing is needed. This will help to analyse how micro cracking affects the bearing capacity of the CRBM layer and to properly develop failure criteria. Also, a shift factor to adjust fatigue life obtained in the laboratory and real behaviour could be established using this large-scale testing.

With further investigation, the design guidance presented here could be optimised, reducing layer thicknesses and giving confidence in the mix, extending its use and improving the sustainability of airfield pavements.

4. Fatigue data analysis

Fatigue tests are carried out in strain or stress control. However, this is not a realistic scenario, as strains and stresses vary during the pavement life.

There is a need for research in this area, developing a deterioration analysis based on laboratory fatigue data, that analyses the pavement life while changing the strain.

Reducing the deterioration intervals used in this research will be helpful to obtain more accurate results; however, this will only be achievable if first a software that reduces computational effort is developed.

Further research is also needed to establish a shift factor that relates fatigue behaviour in the laboratory and on site.

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APPENDIX A

LABORATORY TRIALS OF COLD RECYCLED FOAMED BITUMEN ASPHALT: RAF WADDINGTON

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Abstract

The Defence Infrastructure Organisation requested a pavement evaluation on RAF Waddington and the results indicated that runway rehabilitation and reprofiling was needed in order to meet the physical design requirements set out in the Manual of Aerodrome Design & Safeguarding. The presence of tar in a layer of the old pavement promoted the option of cold recycling this material into the new structure. This paper presents the results from a laboratory investigation into the suitability of cold recycled foamed bitumen asphalt to be used in the structural layers of an airfield pavement.

Laboratory mixture designs with foamed bitumen, incorporating asphalt planings from RAF Waddington runway, were produced in URS Infrastructure and Environment Ltd. laboratory. Specimens were used to assess mix performance and in order to add confidence to the design. The last objective of the research was to demonstrate that asphalt planings from RAF Waddington could be recycled into foamed asphalt for incorporation in the runway rehabilitation works. The optimum binder content was determined from Indirect Tensile Stiffness Modulus tests and Indirect Tensile Strength tests, concluding that the optimum binder content was 3.3% by mass. As a common practice in the UK, up to 1.5% by mass of cement was added to the mixture to improve early life performance. To assess the foamed bitumen samples' performance with time, specimens were prepared and cured for 28, 180 and 360 days at different temperatures. Post curing, the specimens were tested for a range of performance criteria including fatigue, stiffness and durability.

The study found that asphalt sampled from the runway at RAF Waddington can be recycled into foamed asphalt, meeting the requirements of Defence Infrastructure Organisation Specification 050.

Introduction

A pavement evaluation on RAF Waddington was undertaken by URS in June 2012 which highlighted the need for Runway Rehabilitation and Reprofiling. High levels of Polycyclic Aromatic Hydrocarbons (PAH) were identified in this evaluation, which highlighted the presence of tar. This presented some limitations on recycling and expensive cost of disposal, for this reason, and because Cold Recycled Bound Materials (CRBM) were successfully used in other schemes, the potential for incorporation of foamed asphalt in the runway renewal scheme was explored.

Use of foam mix asphalt in the runway would be a UK first; therefore, a laboratory recycling trial was carried out by URS in order to add confidence and demonstrate that site won materials could be recycled into foamed asphalt to meet the requirements of the specifications.

This paper presents results of the recycling study that assesses the performance of laboratory manufactured CRBM incorporating reclaimed asphalt taken from the runway at Waddington.

The pavement evaluation report (URS, June 2012) identified that the construction of the pavement was typically asphalt between 315 and 410 mm in thickness overlying 210 mm thickness of pavement quality concrete (PQC) or dry lean concrete (DLC) layer. In some areas of the runway, the lower layers of the asphalt were found to contain high levels of PAH, the depth of these vary in thickness between 225 mm and 280 mm.

At time of commencement of this study it was not clear whether non-tar material would be incorporated in the runway refurbishment, therefore two material designs were assessed: one with Reclaimed Asphalt Planings (RAP) containing tar (herein “tar bound (Bottom RAP) mixture”) and the other for material free of tar (herein “non-tar bound (Top RAP) mixture”).

A three stage laboratory study was undertaken:

- Stage 1: Confirm recyclability of the materials
- Stage 2: Mixture testing to determine optimum binder content
- Stage 3: Mixture testing to assess CRBM mixture performance

This laboratory program aimed to provide mixture performance results with a view to developing CRBM runway design and material specifications in the future. The study comprised a phased approach, with the three laboratory study stages outlined in this paper.

Literature Review

The UK has adopted the concept of sustainable development and two key measurements of the level of sustainability achieved are the volumes of new resources consumed and the waste sent for disposal. The aim should be to minimise the waste generated and maximise the quantities of materials reused or recycled. Using recycled materials not only avoids waste generation but also the equivalent use of primary materials (CIRIA 1999).

Recycling asphalt is a pavement rehabilitation technique which involves reusing materials from the layers constituting a pavement that has already been in service, materials that have lost some of their original properties because of use or ageing, such as cohesion, texture and composition, but have the potential for being reused and forming new layers.

This material recovered from aged asphalt pavements is known as RAP and is one of the most important elements to bear in mind during the recycling process as it has great influence on the end product characteristics.

When the use of RAP is the selected option for pavement rehabilitation, the next step is to identify the recycling agent to be used. These agents have the purpose of altering the properties of old or aged binders and can be classified as softening agents or rejuvenating agents. The difference between these two types of agents are that softening agents only lower the binder viscosity while rejuvenating agents restore the physical and chemical properties of the aged binder. Foamed bitumen is used as rejuvenator.

Foamed bitumen is produced by injecting air and water droplets under a high pressure (5 bars) into hot (160-180°C) liquid bitumen, resulting in bitumen taking the form of foam as schematically illustrated in Figure 1. The volume of bitumen will increase while viscosity considerably reduces (Jitareekul 2009).

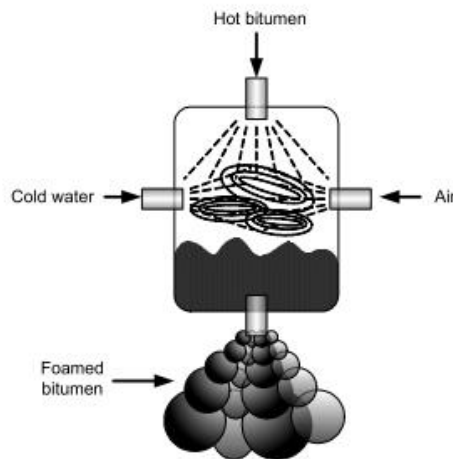


Figure 1 *Foamed bitumen production*

Normally between 3% and 5% of foam bitumen is added, but when the recycled material contains a high proportion of bitumen, this can be reduced to 2% or 3% (Lanre 2010), (Jitareekul 2009), (Botasso et al. 2008).

Foamed bitumen has been used with success in several sites, having similar strength and stiffness that conventional hot mixes after a period of curing, and giving the pavement a good structural performance (Sunarjono 2008).

The design guide and specifications used in the UK for road maintenance is the TRL Report TRL611. For Airfields there is the Specification 050 "Recycled bound materials for airfields".

Stage 1: Recyclability and Waste Acceptance Criteria

Asphalt layers of each of the cores taken from the runway during the pavement evaluation were tested with PAK-Marker, sprayed in a thin layer along the length of the core sample. After allowing the spray to dry the colour is assessed. The spray remains white in colour if sample has no polycyclic aromatic compounds, whilst a yellow colour is produced if high quantities of PAHs are present. It was found that

bituminous samples from the top (approximately 200mm) of the pavement in general did not show a positive result for PAH when tested with the PAK-marker. However, in some areas of the runway, the lower layers of the asphalt showed positive results indicative of the presence of tar. The layers containing tar were sent to a laboratory for Waste Acceptance Criteria (WAC) testing to confirm the levels of PAH and Benzo(a)pyrene. The results from this analysis are summarised in Table 1.

Table 1. PAH results from WAC testing (from the lower asphalt layers)

Material	PAH (mg/kg)	Benzo(a)pyrene (mg/kg)	Implications
Limitations	>25	NA	Not recycled into hot mix asphalt
	>1000	>100	Not recycled into unbound applications. Classified as Hazardous waste if disposed
	NA	>50	Classified as Hazardous Waste and denoted as bituminous mixtures containing coal tar
Tar bound (Bottom RAP) mixture	64000	3000	Cannot be recycled into hot mix or unbound material. Classified as Hazardous waste if disposed.

The limitations for recycling bituminous materials are summarised:

- Bituminous materials with PAH over 25mg/kg shall not be recycled into hot mix asphalt (County Surveyors Society Guidance Note 2008).
- Bituminous materials with PAH over 1000 mg/kg and benzo(a)pyrene levels above 100 mg/kg are not permitted to be recycled into unbound applications (County Surveyors Society Guidance Note 2008).
- The acceptable threshold values of Total PAH and Benzo(a)pyrene are 1000 mg/kg and 100 mg/kg respectively. Above these limits material is classed as hazardous waste if disposed of (The Stationery Office 2005).
- The Environment Agency considers that if benzo[a]pyrene is present at a concentration of 50 mg/kg or more in the asphalt arisings (excluding any other waste material), the waste should be classed as hazardous and denoted by the EWC code 17 03 01 (bituminous mixtures containing coal tar) (ADEPT 2013).

This confirms that if disposed of, it is likely that the lower layers of the original runway pavement would be classified as Hazardous Waste, leading to a high disposal cost. In addition, this material cannot be recycled into hot mix asphalt or unbound material. Recycling into cold recycled bound material is permitted and is likely to offer a cost effective and viable engineering solution.

Stage 2: Mixture Testing to Determine Optimum Binder Content

To determine optimum binder content of the foamed mixture a sample preparation and mixing technology was developed.

Sample Preparation

Ten large 450mm diameter cores were taken from RAF Waddington runway and were separated into tar and non-tar containing layers. Then the samples were processed through a jaw crusher and graded (results are presented in Table 2).

Table 2. Initial particle size distribution

Sieve size (mm)	% Passing		Specification (min.-max.)
	Non-tar bound (Top RAP) mixture	Tar bound (Bottom RAP) mixture	
20	100	100	100
14	100	100	85-100
10	88	94	68-95
4	6	53	38-69
2	0.3	36	26-53
0.5	0.2	16	13-33
0.25	0.1	5.2	9-25
0.063	0	0.6	5-20

The RAP was then split into 0/4 mm and 4/20 mm size fractions. Each fraction was graded to BS EN 933-1 and a blend of the size fractions was calculated which targeted the specified grading curve. The bottom end of the grading did not meet the specifications. To solve this, the Non-tar bound (Top RAP) mixture was mixed with Crushed Rock Fines (CRF) and Fly Ash (FA) and for the Tar bound (Bottom RAP) mixture only FA was needed. The mix design and final grading achieved with the addition of CRF and FA to meet the specification are presented in Table 3 and Table 4.

Table 3. Mix design to determine optimum binder content

Constituent	Supplier	Source/Product	% constituent by dry mass	
			Non-tar bound (Top RAP) mixture	Tar bound (Bottom RAP) mixture
Bitumen	Nynas	Nyfoam 100	Varied 3 - 3.6%	Varied 3 - 3.6%
Cement	Cemex	CEM I (52.5kN), Rugby	1%	1%
CRF	Lafarge Tarmac	Mountsorrel Granite	30%	0%
FA	Cottam power Station	Conditioned FA	5.8%	4.8%
RAP	Derived from site cores	RAF Waddington	52%	93%
Water	N.A	N.A	4.1%	4.1%

Table 4. Final particle size distribution

Sieve size (mm)	% Passing		Specification (min.-max.)
	Non-tar bound (Top RAP) mixture	Tar bound (Bottom RAP) mixture	
20	100	100	100
14	91.2	95.9	85-100
10	75.7	86.6	68-95
4	51.3	54.1	38-69
2	32.1	30.2	26-53
0.5	16.1	14.7	13-33
0.25	12.0	11.4	9-25
0.063	8.6	8.4	5-20

Wirtgen classify the bitumen in the RAP as being “active” or “inactive”, as described in the Wirtgen Manual (Wirtgen GmbH 2012). To investigate the state of the bitumen in the RAP, it was tested following the Wirtgen Manual methodology. As a rule, “active” RAP contains sticky, shiny bitumen whilst “inactive” RAP is dull black rock. For this test, RAP was heated to 70 °C and a 100 mm Marshal plug manufactured. Samples were then soaked in water for 24 hours and Indirect Tensile Strength (ITS) test was carried out. Samples with ITS values higher than 100 kPa were considered “active”. Results are presented in Table 5.

Table 5. RAP classification

RAP layer	ITS results	Classification
Non-Tar bound (Top RAP) mixture	468 kPa	Active
Tar bound (bottom RAP) mixture	Sample does not hold together	Inactive

This results show that the newest bitumen is still active and it has to be taken into account when studying the optimum binder content. It also has implications in the grading, as the lack of fines in the Non-Tar bound (Top RAP) mixture observed and presented in Table 2 is likely to be due to the “active” bitumen holding the fine particles.

Laboratory Mixing Methodology

The mix design and testing was carried out by URS laboratory in Nottingham and comprised the following stages:

1. Foaming the bitumen: A Wirtgen WLB10 was used as the foam generator. The foamed bitumen expansion ratio, half-life and injection mass were optimised and tested prior to the commencement of mixing.

Nyfoam 100 bitumen supplied by Nynas was used to produce the foam, with an air pressure of 5 bar, a water pressure of 4 bar and bitumen temperature of 170°C, as suggested by Nynas based on past experience. An investigation into the effect of water content on half-life and expansion ratio was carried out to determine the optimum water content required to achieve a half-life greater than six seconds and an expansion ratio greater than 8 (Wirtgen GmbH 2012). Figure 2 provides a summary of the results from this investigation and shows that the optimum water content in the bitumen is between 1 and 1.6% water.

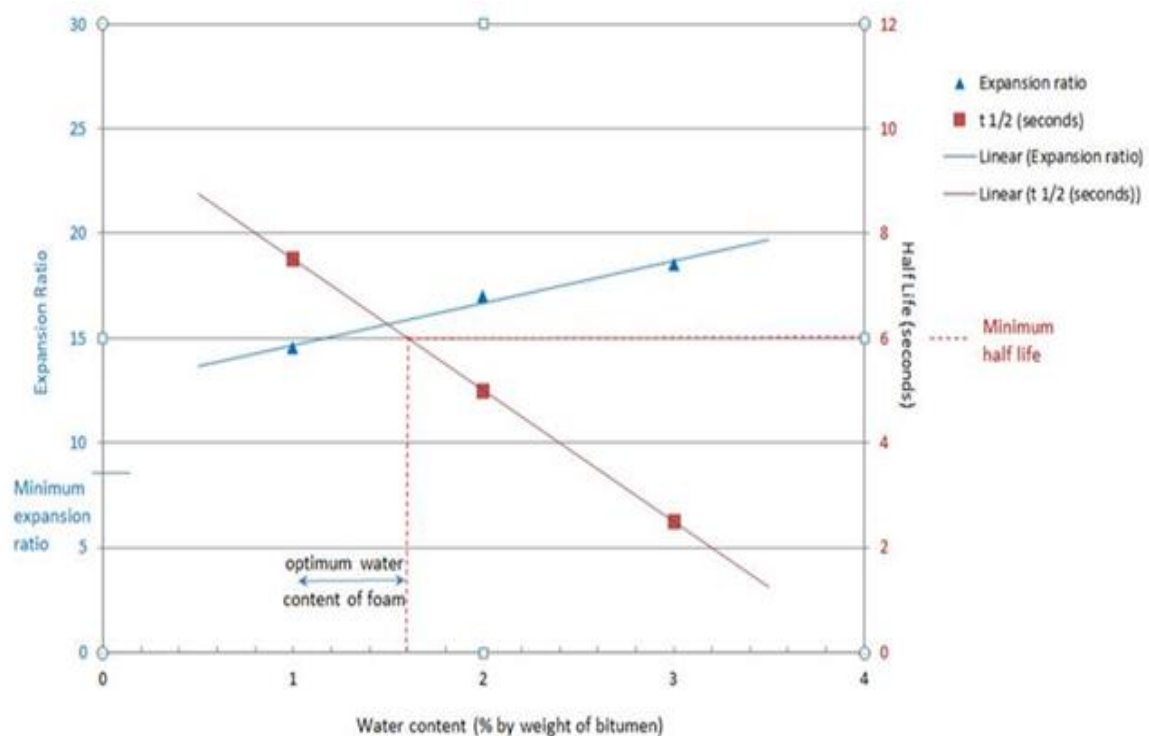


Figure 2. Influence of water content on bitumen foaming properties

2. Aggregate constituents were weighed in the correct proportions to achieve the target grading curve.
3. The aggregate constituents were mixed for two minutes in a Wirtgen WLM30 mixer. Water was added until aggregates reached their target moisture content which is known as the “fluff point”(Wirtgen GmbH 2012), where there are no dry fines in the mix and the mix holds together when squeezed by hand. The mass of added water was recorded.
4. Foamed bitumen was injected into the mixer and mixed for two minutes.
5. Any visible balls of bitumen and fines were broken down by hand and mixed for a further minute.
6. Test specimens were compacted within two hours of mixing.
7. Compacted specimens were double wrapped in cling film and sealed in a bag in their modulus for curing at 40°C for 28 days.
8. After curing, Indirect Tensile Stiffness Modulus (ITSM) to BS EN 12697-26:2004 Annex C and Indirect Tensile Strength (ITS) to BS EN 12697-23:2003 were determined to establish the optimum binder content.

Testing to Determine Optimum Binder Content

Foamed asphalt mixtures were produced as described above at three different bitumen additions: 3.0, 3.3 and 3.6% bitumen. 1% cement content was set for the initial mixtures to improve curing and total strength of the specimens. Results of the laboratory work are presented in Table 6. The average ITSM is calculated with 6 samples and the average ITS with 3 samples.

Table 6. Test results for samples cured at 40°C for 28 days

Bitumen content	Tar bound (Bottom RAP) mixture			Non-tar bound (Top RAP) mixture			Specification
	3.0%	3.3%	3.6%	3.0%	3.3%	3.6%	Min 3.0%
Average 28 day ITSM (MPa)	920	3650	3405	3327	3528	3010	-
Average ITS (kPa)	112	322	324	159	321	160	>250
Water sensitivity ratio	1.5	1.0	1.0	0.9	1.0	1.1	>0.8
Bulk Density (kg/m ³)	2123	2139	2111	2126	2146	2164	-
Maximum density (kg/m ³)	2346			2341			-

Results suggest that 3.3% added bitumen content is the optimum for both the non-tar bound (Top RAP) mixture and the tar-bound (Bottom RAP) mixture. Although having “active” bitumen in the non-tar bound (Top RAP) mixture, the addition of CRF reduces the overall bitumen content in the aggregate, this could explain why both materials have the same optimum binder content.

Asphalt material stiffness relates to its load spreading ability, one of the key parameters used to assess pavement structural condition. In a structural layer, high stiffness indicates good load-spreading ability.

The ITSM values rise from 3.0% bitumen to 3.3% bitumen and then lower for the 3.6% bitumen. This could be due to having insufficient bitumen to bind the mixture in the first case and having too much bitumen in the second case what gives lower stiffness values to the mixture. The distribution of the bitumen or mixing efficiency could also be a possible contributing factor to variation on results.

The ITS test is an indirect measure of the tensile strength and reflects the flexibility and flexural characteristics of the mixture. It is an economic method for investigation the effectiveness of the bitumen (Wirtgen GmbH 2012). This method was added to the investigation since the ITSM results can be distorted with the addition of cement, making the mixture stiff enough to meet the stiffness specification but may not be a well graded, well mixed cohesive mixture.

ITS values were found to be below the required value for mixtures at 3.0% and 3.6% bitumen (for the non-tar bound (Top RAP) mixture only). This is likely to be due to the bitumen content and potentially poor distribution of bitumen, which was evident based on visual assessment. In order to improve the non-tar mixture, the proportion of CRF in the non-tar mixture was reduced during stage 3.

Stage 3: Mixture Testing to Assess CRBM Mixture Performance

Test specimens were produce at 3.3% added bitumen content and 1.5% cement content. The cement content was increased from 1.0% used in Stage 2, to 1.5% in order to replicate common practice for foamed asphalt mixtures in the UK. In addition, the constituent proportions for the non-tar bound (top RAP) mixture were adjusted in order to reduce the proportion of CRF contained in the mixture and maximise the recycled content, improving ITS results. The mixture proportions are summarised in Table 7.

Table 7. Constituent proportions for stage 3

Ingredient	Constituent proportion (by dry mass)	
	Tar bound (Bottom RAP) mixture	Non-tar bound (Top RAP) mixture
Bitumen	3.3%	3.3%
Cement	1.5%	1.5%
RAP	88.0%	71.6%
Crushed rock fines	0%	17.4%
Fly ash	7.2%	6.2%
Total water	4.5%	4.5%

119 samples were prepared as described in Stage 2, and cured at different temperatures (20°C and 40°C) over varied times (28, 180 and 360 days).

The testing methodology comprised:

- Determination of ITSM to BS EN 12697-26:2004 Annex C, to assess the load spreading ability and durability after wet conditioning immersed at 20°C for 7 days (immediately following the curing period).
- Determination of the ITS to BS EN 12697-23:2003 before and after wet conditioning immerse in water at 20°C for 24 hours (immediately following the curing period).
- Determination of Indirect Tensile Fatigue (ITF)(British Board of Agreement n.d.). Fatigue performance of a material is one of the key parameters used to assess a pavement's expected life. The fatigue test is undertaken at several stress conditions, with the number of loading cycles to failure recorded. This enables a predicted 'fatigue life' to be calculated and plotted.

Laboratory test results are summarised in Tables 8 and 9.

Table 8. Laboratory mixtures results summary for samples cured at 40°C

Material property	Tar bound (bottom RAP) mixture	Non-tar bound (top RAP) mixture			Specification Requirement (at 28 days only)
	28 days	72 h	14 days	28 days	
ITSM (MPa)	4812 (4370-5390)	-	3483 (3070-4220)	5090 (4400-5940)	3,500 (min) 10,000 (max)
Moisture sensitivity (retained stiffness)	0.97	-	-	0.95	0.8 minimum
ITS (kPa)	364 (340-408)	274 (259-288)	-	138 (132-145)	>225 at 28 days
ITS _{wet} (kPa)	300 (265-347)	-	-	193 (155-231)	Information only
Bulk density (kg/m ³)	2098 (2085-2124)	-	2104 (2086-2131)	2104 (2086-2131)	Information only

Note: Values shown are mean (with minimum and maximum values in brackets).

Table 9. Laboratory mixture results summary for samples cured at 20°C

Material property	Tar bound (bottom RAP) mixture			Non-tar bound (top RAP) mixture		
	28 days	180 days	360 days	28 days	180 days	360 days
ITSM (MPa)	2915 (2200-3420)	4515 (3820-5590)	4763 (4400-5190)	3028 (2810-3580)	4146 (3980-4410)	4908 (4450-5580)
Moisture sensitivity (retained stiffness)	1.01	1.08	0.87	0.91	0.94	0.83
ITS (kPa)	285 (242-339)	384 (318-449)	467 (409-511)	116 (109-125)	444 (394-497)	261 (225-314)
ITS _{wet} (kPa)	239 (205-261)	351 (339-362)	453 (407-537)	156 (126-184)	227 (220-238)	209 (195-237)
TSR	84%	91%	97%	134%	51%	80%
ITFT linear regression line	3.27 - 0.30 log (cycles to failure) R ² = 0.9	2.67 - 0.13 log (cycles to failure) R ² = 0.65	2.77-0.15 log (cycles to failure) R ² = 0.98	2.84 - 0.15 log (cycles to failure) R ² = 0.9	2.69 - 0.15 log (cycles to failure) R ² = 0.69	2.82 - 0.20 log (cycles to failure) R ² = 0.76
Bulk density (kg/m ³)	2084 (2063-2109)	2092 (2076-2147)	2106 (2080-2129)	2095 (2068-2131)	2080 (2061-21.3)	2106 (2088-2121)

Note: Values shown are mean (with minimum and maximum values in brackets).

Comparing ITSM and ITS results from both tables it can be said that samples cured at 40°C for 28 days showed similar results to those of samples cured at 20°C for 360 days. Curing samples in the laboratory is an area of ongoing research, which is developing as the relative long term performance of these mixtures is being furthered.

The ITSM and moisture sensitivity results are within specification limits and suggest that the samples are not moisture susceptible.

Some ITS values from Non-tar bound (top RAP) mixture are below the specifications requirements. A possible explanation could be due to poor bitumen distribution in the mixture, associated with the presence of CRF.

The fatigue test is undertaken at several stress conditions, with the number of loading cycles to failure recorded. This enables a predicted 'fatigue life' to be calculated and plotted. Fatigue performance of the non-tar bound (Top RAP) mixture is not very consistent as reflected by the low values of R². This can be due to the poor mixture of the bitumen addressed before.

In order to quantify the differences in fatigue characteristics between the mixtures, the number of cycles to failure can be calculated at particular strain levels with the linear regression line. For asphalt pavements, analyses confirmed by measurements have shown that the tensile strain at the base of the bound layers, under a moving wheel load, is of the order of 100 microstrain (Cooper & Pell 1974).

In Table 10 the calculated cycles to failure are presented, and in

Figure 3 the fatigue lines for samples cured at 20°C for 360 days and two standard asphalt materials for comparison purposes are plotted.

Table 10. Calculated fatigue for 100 microstrain

Material	Fatigue Life (Number of Cycles to Failure) at Tensile Strain = 100 $\mu\epsilon$
Tar bound (Bottom RAP) mixture after 28 days curing at 40°C	17,113
Tar bound (Bottom RAP) mixture after 180 days cured at 20°C	142,510
Tar bound (Bottom RAP) mixture after 360 days curing at 20°C	135,935
Non-tar bound (Top RAP) mixture after 28 days curing at 40°C	13,335
Non-tar bound (Top RAP) mixture after 180 days cured at 20°C	39,810
Non-tar bound (Top RAP) mixture after 360 days curing at 20°C	10,826
20 mm DBM 100. Reference material (Read 1996)	71,724
20 mm HDM 50. Reference material (Read 1996)	26,389

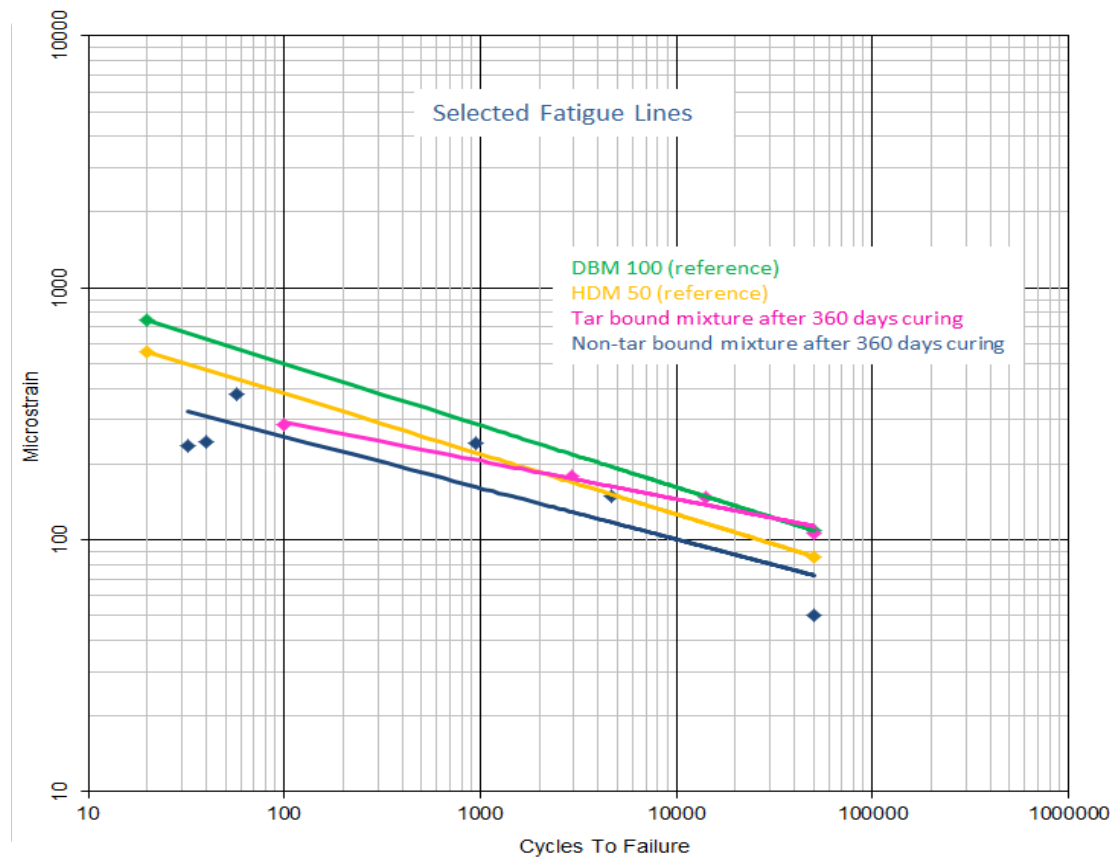


Figure 3. Fatigue lines comparison

The fatigue lines show a comparable performance of the Tar bound (Bottom RAP) mixture with the DBM 100 for a strain around 100. This means that, under the same loading conditions, both materials will have similar performance. In the case of the Non-tar bound (Top RAP) mixture, specimens show lower resistance to fatigue than the tar bound (Bottom RAP) mixture. As already said, this can be due to the addition of CRF, as it is not helping the mixture, being the bitumen not well distributed and affecting the performance of the mixture in terms of fatigue. This also affects the dispersion of the results.

Conclusions

The results show that tar bound (Bottom RAP) mixture samples achieved the specified requirements for ITSM and ITS. Some variation was found for the ITS tests with the non-tar bound (Top RAP) mixture falling below the specified limits. This is considered to be due to the distribution of the bitumen which could be associated with the incorporation of CRF. Also, results suggest that the incorporation of CRF into foam mix asphalt does not improve the performance in terms of ITS or fatigue, when compared with the tar-bound CRBM mixture, which did not contain CRF. Hence, it is suggested to avoid adding CRF in future works.

Therefore, this test program demonstrates that RAP from RAF Waddington can potentially be recycled into the foamed asphalt which meets specifications for the runway refurbishment works.

The study also found that samples cured at 40°C for 28 days achieved similar ITSM results to those cured for 360 days at 20°C. With further study, an accelerated curing procedure could be developed.

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Comparison between laboratory results for cold recycled bound materials and DBM 50 used in airfield pavements

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ABSTRACT

Cold recycled materials are widely used in UK road construction, but their use in airfield pavements is less common. The UK has adopted the concept of sustainable development and cold recycling of asphalt into foamed or emulsion bitumen bound materials minimizes the waste generation and reduces the consumption of finite resources. In addition, this technique has the added benefit of readily incorporating recycled asphalt which contains tar, making cold recycling an attractive option for the rehabilitation of airfield pavements. The challenge is that there is no specific pavement design guidance for these materials and the current practice is to conservatively equate these materials to an assumed performance of a hot asphalt mixture. The objective of this paper is to verify if this practice is accurate when analyzing these materials' performance, especially through laboratory testing. The results presented in this paper show that the pavement design could be optimized and that a design guide based on cold recycled materials is needed to gain confidence and extend the use of cold recycled bitumen bound materials in airfields. This conclusion forms the basis for the SUP&R ITN project ESR1: "Pavement design for cold recycled materials".

Keywords: Cold Asphalt, Foam, Mechanical Properties, Reclaimed asphalt pavement (RAP) Recycling

1. INTRODUCTION

The construction industry is playing a key role in improving the efficient use of materials, as part of sustainable development; by addressing what, where and how it builds; and by planning for maintenance, future development and disposal [1]. In the context of pavements, the main aims for the efficient use of materials are to reduce demand on primary resources and divert materials from going to landfill as waste. Recycling asphalt is a pavement rehabilitation technique which involves recycling materials from the layers constituting a pavement that has already been in-service. These materials have often lost some of their original properties due to mechanical deterioration and/or ageing; such as cohesion, texture and composition; however the materials still have the potential for being recycled and forming new layers [2].

The potential benefits of the wider use of recycled materials within pavement designs include:

- Reduced demands on the environment for finite resources (e.g. primary aggregate abstraction and bitumen).
- Reduced demand on the environment (reduction in material for landfill).
- Potential for comparative benefits in pavement layer performance, facilitating reductions on the overall thickness of pavement design.

Material recovered from aged asphalt pavements is known as RAP (Reclaimed Asphalt Pavement) and it has the potential to be recycled into various applications dependent on its characteristics. RAP can be used in all pavement applications [3] but it is normally used in unbound mixtures for sub-base [4], hydraulically bound mixtures for sub-base and base [4] and bitumen bound layers [5]. The objective should be to increase the RAP content in asphalt mixtures without sacrificing durability. The main benefit of using RAP into asphalt is the reduction in demand on bitumen, which is a finite resource. In the other hand, UK pavements constructed prior to 1980 or surfaced in the late 1980's may contain tar. Tar is a potentially hazardous material, in terms of waste disposal, and it is classified as "Hazardous Waste" by European Directives which were implemented in the UK via the Hazardous Waste Regulations 2005 (HWR). Pavements built during those years are now commonly requiring reconstruction and the problem is that when RAP contains tar, there are some limitations on use based on health and/or environmental considerations [6]:

- Bituminous materials with polycyclic aromatic hydrocarbons (PAH) over 25 mg/kg should not be recycled into hot mix asphalt.
- Bituminous materials with PAH over 1000 mg/kg and Benzo(a)pyrene levels above 100 mg/kg are not permitted to be recycled into unbound applications.
- The acceptable threshold values of total PAH and Benzo(a)pyrenes are 1000 mg/kg and 100 mg/kg respectively.

Above these limits the material is classified as Hazardous Waste, if disposed.

This means that tar layers cannot be recycled into hot mix asphalt or unbound applications and their disposal is relatively expensive. Therefore, cold recycling is a safe and sustainable option for asphalt plannings containing tar and has become increasingly interesting, as it is a method which not only minimizes waste and use of raw material, but also saves energy because it is not necessary to heat the mixture [6].

Once the possibility of cold recycling a pavement is considered, the next step is to identify the recycling agent to be used. These agents extract maximum value from the old or aged binders in RAP. The most common agents used are foamed bitumen and bitumen emulsion [7].

This research will focus on cold recycled bound materials (CRBM) using foamed bitumen as it is the most commonly used cold technique in UK road construction, although their use in airfield pavements is less common. The problem is that there is no specific pavement design guidance for these materials and the current practice is to conservatively equate these materials to an assumed performance of a hot asphalt mixture.

CRBM will be used in the base layer of the pavement, so to ascertain the suitability of this practice CRBM performance will be compared to a hot asphalt base, namely Dense Bituminous Macadam 50 (DBM50).

2. LITERATURE REVIEW

2.1 Foamed Bitumen

Foamed bitumen is produced by injecting air and water droplets under high pressure (5 bars) into hot (typically 160-180°C) liquid bitumen, resulting in bitumen taking the form of foam as schematically illustrated in Figure 1. The volume of bitumen will increase while viscosity considerably reduces [7].

Normally between 3% and 5% of foam bitumen is added to the mixture, but when the recycled material contains a high proportion of bitumen, this can be reduced to 2% or 3% [8], [7], [2]. Foamed bitumen has been used with success in roads and airfields, having similar strength and stiffness to conventional hot mixes after a period of curing, and giving the pavement good structural performance [9].

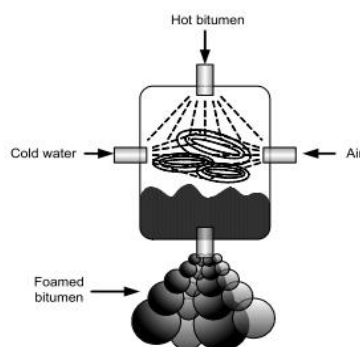


Figure 1: Foamed bitumen production [7]

2.2 Curing of CRBM

It is accepted that hot mix asphalt reaches its mature state after a very short time. This means that its characteristics can be measured almost immediately after paving. But this is not the case for CRBM mixes [10]. CRBM mixes are evolutive materials, as the initial cohesion between the binder and the aggregates is relatively low and it grows gradually. These materials only reach their mature state after a period of time [10]. This period of time is called the curing phase and opening to traffic and long term performance depends on it [11] as this material only develops full strength after a large percentage of its moisture content is lost [12].

The objective of curing specimens in the laboratory is to simulate pavement conditions as experienced in the field. There are several studies on curing procedures and some guidance notes [5][13], but the problem is that no standard curing procedure has been established for CRBM and it is difficult to determine how realistic these curing conditions in the laboratory are.

It is known that the mechanical and performance properties of CRBM are intimately related to the curing condition [14], which is why several studies related to the curing process have been carried out to establish the most appropriate practice. Some of the conclusions of these studies have highlighted the importance of curing temperature and time, and how moisture loss is not the only mechanism involved in curing. It has been proved that curing specimens fully wrapped at 20°C for 28 days is an appropriately conservative practice to understand CRBM actual performance on site, where a surface course is typically applied a few days after construction of the base layer (CRBM mix) with trafficking commencing after a further few days [11]. This curing procedure will therefore be used in this study.

2.3. Asphalt pavements in airfields

Asphalt is widely used in airfield pavements, but in each section of an airfield (runway, taxiway, aprons, parking areas, etc.) different considerations apply. Commonly, asphalt is used in runways and taxiways in the surface, binder and base courses [15].

Defence infrastructure organisation (DIO) specifications for CRBM are available for the lower layers of taxiways and runways [16], but these materials use is limited due to unknown long term performance and risk of failure with high rehabilitation cost and traffic delay. In this case, CRBM mixes would be substituting an Asphalt Concrete (AC) base [17]. When using CRBMs, as there is no airfield design guide available, the current practice is to conservatively equate them to AC and this is why in this paper a CRBM mix has been compared to this type of material, to assess if this practice is accurate or can be improved.

There are a several widely used design guides for airfield pavement design, some examples being:

- DMG27 [17] – a UK design guide where separate design charts are plotted for different type of materials. These charts are based on resistance to permanent deformation, water sensitivity, durability, stiffness, resistance to fatigue and tensile strength. The inputs in this case are: traffic, type of subgrade and type of material.
- DIO[16] – lays down specification requirements for airfield pavement works using recycled bound materials, but no design guidance is included in terms of layer thickness.
- BAA[18] – design guide based also in graphs where the inputs are traffic, subgrade category and failure criteria. This guide is based on the California bearing ratio method with some analytical equations but no CRBM are included.
- FAARfield – airfield design software from the Federal Aviation Administration in the USA where the inputs are: traffic, design life, layer thickness, type of material (standard hot mixes) and stiffnesses. This software is based on failure by fatigue models, making iterative calculations to meet the design life selected.

The problem with these guides is that only the inputs can be changed, not the parameters used for the main calculations such as fatigue or permanent deformation. This is why a laboratory plan has been devised to obtain the characteristics mentioned for both mixtures (CRBM and HDM 50) and the results compared to see if a design for a hot mix can be applied for a CRBM mix.

3. LABORATORY PLAN

To undertake this investigation a CRBM mixture (foamed asphalt) was characterized completely in the laboratory while hot mix data were obtained from a previous study carried out in the same laboratory [19]. The materials and testing are described as follows.

3.1. Materials

For the CRBM mixture manufacture, the following materials were used:

- 0-10mm and 10-20mm RAP
- Fly Ash and Cement
- Bitumen penetration grade 100/150
- Water

The hot mix AC base was a HDM 50 [19]:

- Virgin aggregate (granite): 31.5mm, 20mm, 10mm and fines
- Filler (limestone)
- Bitumen penetration grade 40/60

3.2. Raw material characterization

The first step was to characterize the materials used in the investigation. With this objective the following tests were undertaken:

- RAP and hot mix aggregates combined mixture grading [20]

The gradations used are presented in Table 1 and Table 2.

Table 1: RAP grading

Sieve size (mm)	% Passing	Specification	
		(min.	- max.)
31.5	100	86	100
20	100	65	100
14	100	53	90
10	88	47	77
4	6	30	55
2	0.3	22	41
0.5	0.2	10	27
0.25	0.1	8	25
0.063	0	6	20

Table 2: Hot mix aggregates grading

Sieve size (mm)	% Passing	Specification	
		(min.	- max.)
40	100	100	100
31.5	100	90	100
20	88	71	95
6.3	55.1	44	66
2	29.4	20	40
0.25	12.5	6	20
0.063	7.1	7	11

- Bitumens characterization: penetration grade [21] and softening point [22]. Bitumen results are shown in Table 3.

Table 3: Bitumens characteristics

Binder	Penetration grade (tenths of mm)	Softening point (°C)	Purpose
40/60	47	72.1	HDM50
100/150	107	44.2	CRBM mix

3.3. Manufacture of CRBM mix specimens

3.3.1 Mix design

The mix design was selected to mirror material being manufactured at the airfield from which the RAP was obtained. At this airport, rehabilitation and reprofiling was needed and a study on CRBM mix design with foamed bitumen had been carried out. During this study an optimum mix design was developed and this was selected for this investigation, as it is a real case and it will be useful to compare laboratory results and results on site in the future.

In Table 4 and Table 5 the mix design of the CRBM used is presented. As can be appreciated in Table 1, the RAP contains less fines than the specification demands, so fly ash was added to correct the gradation. The final mix design grading is presented in Table 6 and both mix gradings are compared in Figure 2.

Table 4: Aggregates for CRBM mix

Aggregates	Proportion by mass (%)
0-10mm RAP planings	48.1
10-20 mm RAP planings	43.2
Fly ash	6.9
Cement	1.8

Table 5: CRBM mix design

Ingredient	Proportion by mass (%)
Aggregates	90.5
Foamed bitumen	3
Total water content	6.5

Table 6: CRBM mix final grading

Sieve size (mm)	% Passing	Specification	
		(min.)	- max.)
31.5	100	86	100
20	97.4	65	100
14	85	53	90
10	72	47	77
6.3	56	38	64
4	41.8	30	55
2	28.7	22	41
0.5	14.6	10	27
0.25	11	8	25
0.063	8	6	20

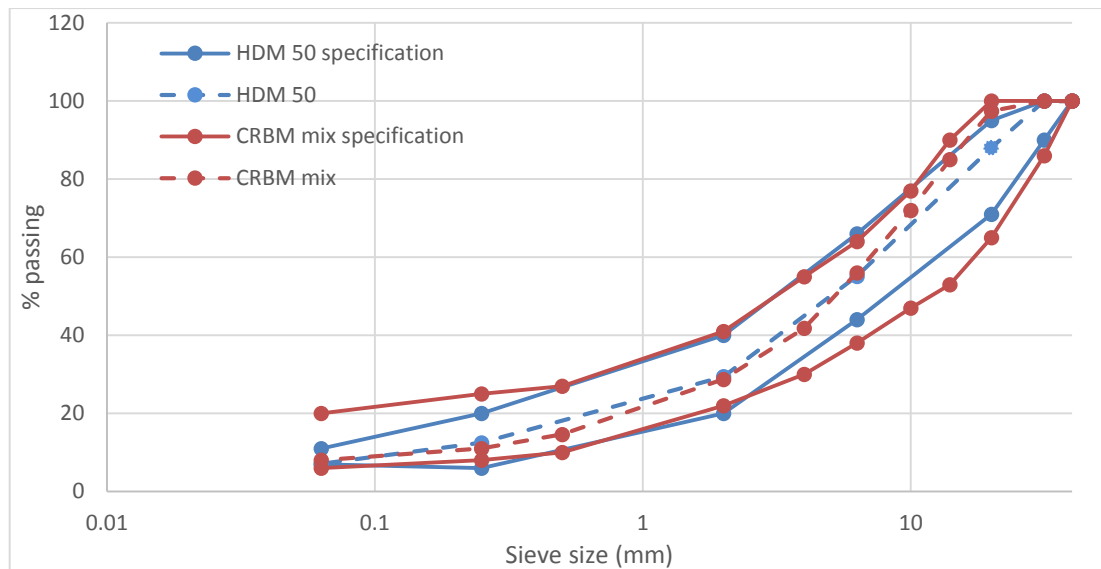


Figure 2: CRBM mix vs hot mix aggregates grading

3.3.2. Foaming

Foamed bitumen was produced in the laboratory using a mobile foaming plant type Wirtgen WLB 10 as shown in Figure 3. The foaming conditions are presented in Table 7. The half-life of the bitumen was measured using a Wirtgen measurement instrument (dipstick), and the results show a half-life greater than the recommended value of 6 seconds [23].

Before starting the Wirtgen WLB 10, all the components needed for manufacturing the samples had to be batched to comply with the mix design specified in Table 4 and Table 5. The bitumen needed to be heated at the set temperature (170°C) between 3 and 5 hours before using the equipment.

Once the materials were ready and the parameters set in the equipment, foam mix batches could be manufactured using a mixer coupled to the foaming plant, to avoid losing material. The aggregates had to be mixed with the water for 1 minute before adding the foam, and then another 3 minutes with the foam.

Table 7. Foaming conditions

Water Pressure	4 bar
Air pressure	5 bar
Bitumen type	100/150
Bitumen temperature	170°C
Water addition	1%



Figure 3. Wirtgen WLB 10

3.3.3. Compaction of foam mix specimens

Specimens were compacted using a Coopers Gyratory Compactor in accordance with BS EN 12697-31 [24], at a constant pressure of 600kPa, a speed of 30rpm and a gyration angle of 1.25°. The calculated amount of mixture was placed in the mould and compacted to a target void content of 5%. This resulted in cylinders with a diameter of 100mm and 60mm high. Specimens were de-moulded after 24h and placed for curing.

3.3.4. Specimen curing

For curing, test specimens were double wrapped in cling-film plastic using two separate sheets, as shown in Figure 4. Once wrapped, the specimens were stored in air at 20°C for 28 days. This curing procedure is a conservative approximation to simulate actual performance of CRBM on site [11], [16].

After this period all specimens were unwrapped, characterized and tested as described in section 3.5. In Figure 5 the specimens after curing are shown.



Figure 4: Double wrapped specimens



Figure 5: Cured specimens

3.4. Hot mix specimen manufacture

The process for hot mix sample manufacturing comprised [19]:

- Perform Particle Size Distributions on the virgin aggregates.
- Perform Penetration Tests on the bitumen.
- Perform Softening Point Tests on the bitumen.
- Create an Aggregate Blend to fit within the specification envelope, as shown in Figure 2.
- Use prior knowledge of the aggregates and bitumen to estimate the Binder Content (4-2%).
- Perform Maximum Density Tests on trial mixes to confirm that the targeted volumetrics can be achieved.
- Create a Mixture Composition based on the above.
- Manufacture slabs using a Laboratory Roller Compactor.
- Core 100mm and 200mm diameter specimens using a diamond tipped asphalt coring rig.

3.4.1. Mix design

The mix design selected to achieve the desired air void content is presented in Table 8.

Table 8: HDM 50 mix design

Constituents	Proportion by mass (%)
31.5 mm agg.	10.5
20 mm agg.	19.2
10 mm agg.	19.2
Crushed rock fines	43.1
Filler	3.8
Bitumen	4.2

3.5. Testing methodology

After the materials had been characterised, and the specimens manufactured and cured (CRBM), the following tests were carried out.

- Bulk density BS EN 12697-6:2012, procedure C [25], max density to BS 12697-5:2009, method A [26] and air voids to BS EN 12697-8:2003 [27].
- Determination of indirect tensile stiffness modulus (ITSM) to BS EN 12697-26:2004 Annex C, to assess the load spreading ability and durability after wet conditioning, immersed at 20°C for 24 hours (immediately following the curing period).
- Resistance to permanent deformation under dynamic loading was assessed by repeated load axial test without confinement (RLAT) to BS DD 226.
- Determination of the indirect tensile strength (ITS) to BS EN 12697-23:2003 before and after wet conditioning.
- Determination of indirect tensile fatigue (ITF) [28]. Fatigue performance of a material is one of the key parameters used to assess a pavement's expected life. The fatigue test was undertaken under several stress conditions, with the number of loading cycles to failure recorded. This enabled a 'fatigue life' to be calculated and plotted.

4. LABORATORY RESULTS: COMPARISON BETWEEN MATERIALS

CRBM test results obtained in the laboratory are presented below and compared to DMB 50 test results obtained by E. Draper [19].

4.1. Mixture volumetrics

Table 9: Mixture volumetrics

Mixture	Bulk density (Kg/m ³)	Max density (Kg/m ³)	Air voids (%)
CRBM	2146	2515	15
HDM50	2442	2551	4.3

4.2. Stiffness

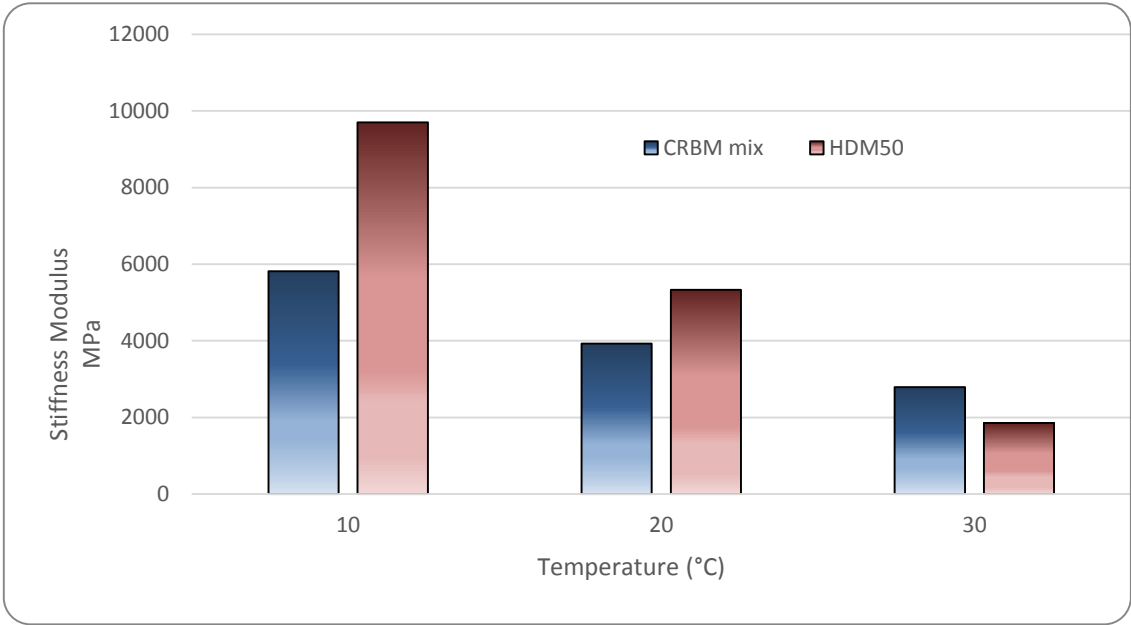


Figure 6: Average stiffness values

4.3. Strength

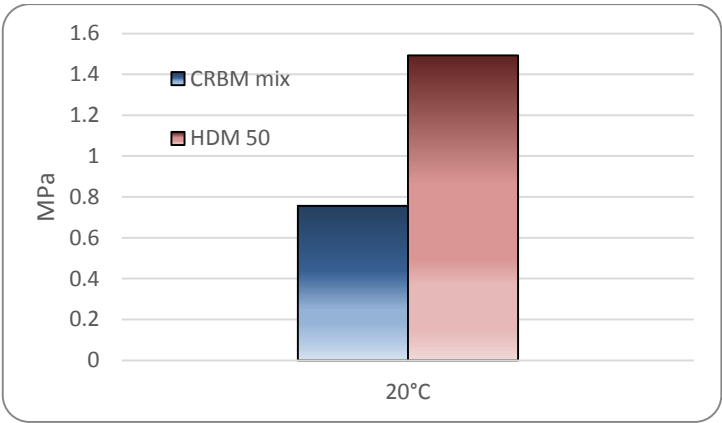


Figure 7: Average strength values

4.4. Durability

Table 10: Moisture sensitivity

Mixture	Retained stiffness (%)	Retained strength (%)
CRBM	120	92
HDM50	92	NA

4.5. Resistance to permanent deformation

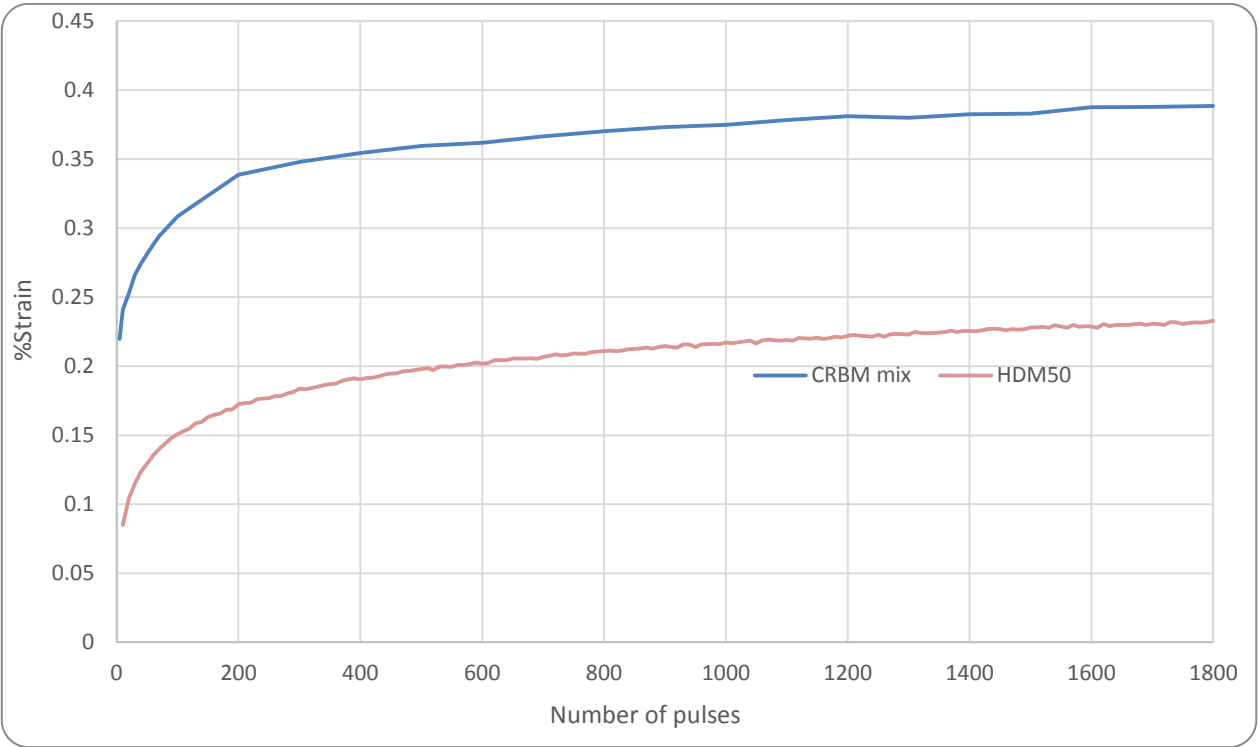


Figure 8: Permanent deformation

4.6. Fatigue

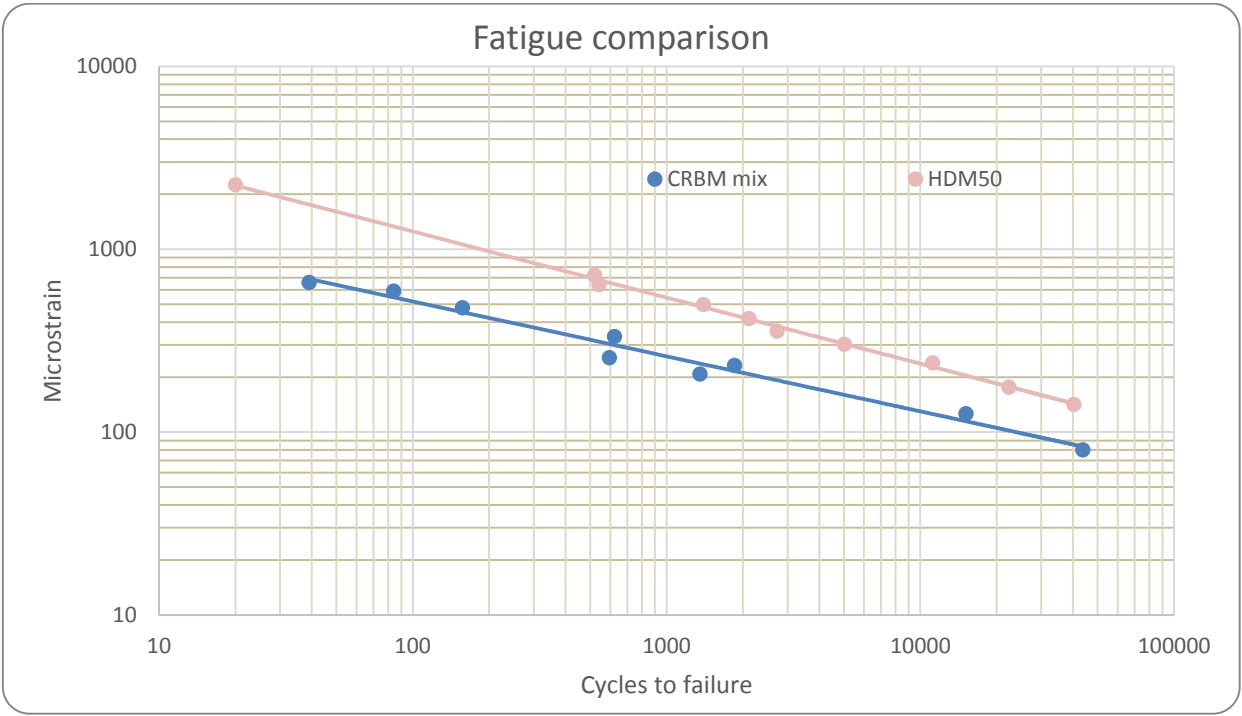


Figure 9: Fatigue characteristics

4.7. Results discussion

It can be appreciated that there are differences in all the results obtained between hot mix asphalt and the CRBM. As expected the performance of the CRBM mix is not as good as the hot mix in some aspects, despite it being within specification limits [16].

Particular mention should be made of the air void content of the CRBM shown in Table 9. The value obtained in the laboratory (15%) is very high compared with the target (5%) but similar to other foam mix studies [29]. Inspection of a broken specimen in Figure 10, reveals that these voids are not readily apparent. It may therefore be concluded that they are in the form of 'micro-voids' within the binder-filler-fines mortar.

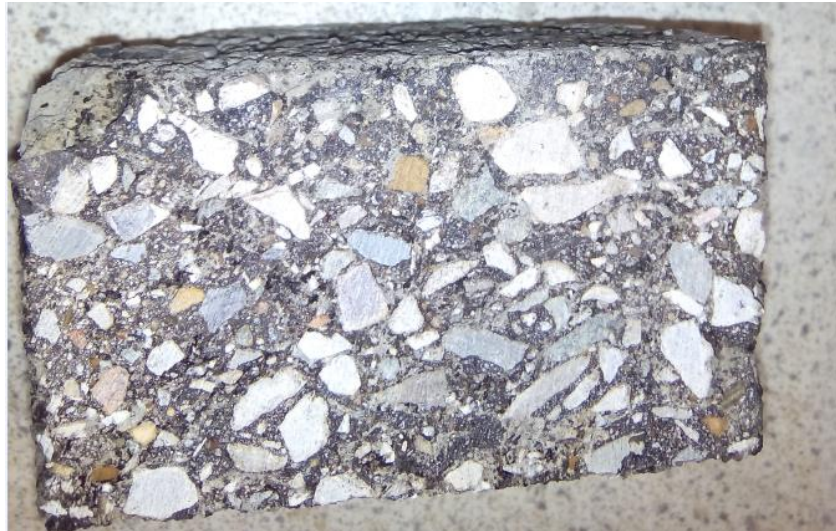


Figure 10: CRBM broken specimen

Figure 6 shows that the CRBM mixture was less temperature susceptible than HDM 50, behaving more consistently in terms of stiffness. This is likely to be due to the action of cement within the mixture. Regarding the stiffness value used for design (at 20°C), it can be noticed that the difference between the two mixtures is moderate (4000MPa vs 5300MPa) if it is taken into account that the HDM50 evaluated has an average stiffness value, being possible to find HDM50 with lower stiffness and still be within specifications.

Strength and stiffness results after wet conditioning were obtained to assess mixture durability, see Table 10. It could be considered surprising that CRBM stiffness after 7 days soaking is greater than before, obtaining a retained stiffness greater than 100%, but this is likely to be due to the content of cement, which was still active and used the moisture to gain strength, making the specimens stiffer than before. However, these results show that both mixtures have good durability properties.

In terms of permanent deformation, in Figure 8, both mixtures have much better rut resistance than the UK highways requirements (below 2% strain [5]), and high rut resistance in very heavily stressed sites is an important characteristic, therefore important for airfield pavements.

Finally, the fatigue results in Figure 9 show a difference amounting to a factor of about four at 100 microstrain (typical highway strain levels), rising to about eight at 400 microstrain (more typical of airfields).

5. CONCLUSIONS

With the results obtained from the laboratory investigation it is concluded that CRBM mixes have acceptable properties for use in airfield pavements. Resistance to permanent deformation, fatigue, temperature susceptibility and durability results show that these materials give reasonable performance but differ from conventional hot mixes. This means that, for the design approach with CRBM, actual design guides should be optimised and calibrated to include performance of CRBM.

As the current practice is to assume material equivalency with hot mix and it is not possible to include material characteristics other than stiffness and Poisson's ratio in these guides, the next step of the author's investigation will be to analyse some of the guides mentioned (DMG 27, FAArfield and BAA) against mechanistic/empirical software where fatigue and permanent deformation laws can be changed. Once the software is chosen, the first step will be to use hot mix performance to understand differences between methods and reproduce the design guides mentioned with the selected software. Then, pavements structures containing CRBM will be analysed with this software obtaining a design guidance. Finally, CRBM will be studied in terms of failure mechanisms, taking into account the curing process and the crack development in the mixture. The final objective will be to develop a new design methodology and guidance for using CRBM in airfield pavements.

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ANALYSIS OF STIFFNESS AND FATIGUE RESISTANCE OF COLD RECYCLED ASPHALT MIXTURES MANUFACTURED WITH FOAMED BITUMEN FOR THEIR APPLICATION TO AIRFIELD PAVEMENT DESIGN/ ANALISIS DEL MÓDULO DE RIGIDEZ Y LA RESISTENCIA A FATIGA DE MEZCLAS ASFÁLTICAS RECICLADAS EN FRÍO FABRICADAS CON BETÚN ESPUMADO PARA SU USO EN EL DISEÑO DE FIRMES PARA AEROPUERTOS

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ABSTRACT: Cold recycled bound materials (CRBMs) provide an economic and environmental advantage for pavements since they decrease energy and raw material consumption. However, design methods for airfield pavements do not include key CRBM properties. In this paper an empirical-mechanistic method is used to study airfield pavement design with CRBM in order to develop design guidance. The aim of the paper is to obtain the inputs related to material properties needed for use in this method. For this purpose, CRBM containing reclaimed asphalt, with fly ash, cement and foamed bitumen as stabilising agents, was characterised. The methodology included indirect tensile stiffness modulus (ITSM) and indirect tensile fatigue tests (ITFT) in strain control mode. The inputs needed for a pavement design analysis with CRBM were then obtained. The results showed the importance of further study on CRBM fatigue to understand the behaviour of these mixes under cyclic loading.

KEYWORDS: mechanical properties, modulus of elasticity, fatigue, characterisation, fly ash.

RESUMEN: El uso de mezclas asfálticas recicladas en frío (MARF) proporciona ventajas tanto económicas como medioambientales al disminuir el consumo de energía y materias primas. Sin embargo, los métodos de diseño para firmes de aeropuertos no incluyen las propiedades de MARFs. En este artículo un método empírico-mecanístico se emplea para estudiar el diseño de firmes de aeropuertos con MARF. El objetivo es obtener los inputs relacionados con las propiedades de MARF necesarios para llevar a cabo el diseño del pavimento. Con este propósito, MARF con asfalto reciclado, ceniza volante, cemento y betún espumado ha sido caracterizado. La metodología incluye ensayo de tracción indirecta para la obtención del módulo de rigidez y ensayo de fatiga con tracción indirecta en modo de deformación controlada. Los inputs necesarios han sido obtenidos y los resultados muestran la importancia de un estudio adicional del comportamiento a fatiga de MARF para entender su comportamiento bajo cargas cíclicas.

PALABRAS CLAVE: propiedades mecánicas, módulo elástico, fatiga, caracterización, ceniza volante.

1. INTRODUCTION

Material that is recovered from aged asphalt pavements is known as RAP (Reclaimed Asphalt Pavement) (1; 2). The requirement for using RAP in new asphalt is becoming increasingly urgent (3) because of the economic and environmental benefits (3), reducing demand on finite resources, generation of waste materials and embodied energy (4).

Cold recycling of asphalt is a proven technique that reduces energy consumption (5,6,7). This reduction is largely achieved by avoiding aggregate drying (8) and mixing the material at ambient temperature instead of 170 °C-185 °C, as required for hot mix asphalt (HMA) (9). The use of bituminous stabilising agents produces a flexible layer with superior fatigue performance to those with purely cementitious binders (10). This study looks at foamed asphalt with cement, material which is classified in the UK as a cold recycled bound material (CRBM) (8; 11).

Foamed bitumen is produced by injecting air and water droplets under high pressure (e.g. 5 bar) into hot (160-180 °C) liquid bitumen, resulting in the formation of foam (12). The volume of bitumen increases while viscosity considerably reduces (13). Typically foam bitumen is added to the mixture at between 3% and 5% by weight of aggregate; however, when the bitumen content of the recycled material is high, this can be reduced to 2-3% (2; 13; 14).

Early life CRBM mechanical properties change over time (15). This phenomenon, during which the cohesion between the binder and the aggregates increases as the mixture loses water, is known as curing (16; 17). No standard curing procedure has been established for CRBM; however, from previous research, it has been demonstrated that curing specimens fully wrapped at 20°C for 28 days is an appropriately conservative practice (16; 18); therefore, this curing procedure was chosen for this study.

Despite the increasingly common use of CRBMs in roads (11; 19), the specifications for the use of these materials in airfields are underdeveloped (8) and there is no guidance to ensure that pavement design with these materials is trustworthy (20–22). Design guides for airfield pavements such as FAARfield (23), BAA (24) or Design and Maintenance Guide 27 (DMG27) (25) do not readily allow the introduction of new material properties (20), making it difficult for authorities and practitioners to use these materials on airfield pavements. A new design approach is therefore required (22). In Figure 1 a harmonised approach for analytical pavement design of pavements using CRBM is proposed (22).

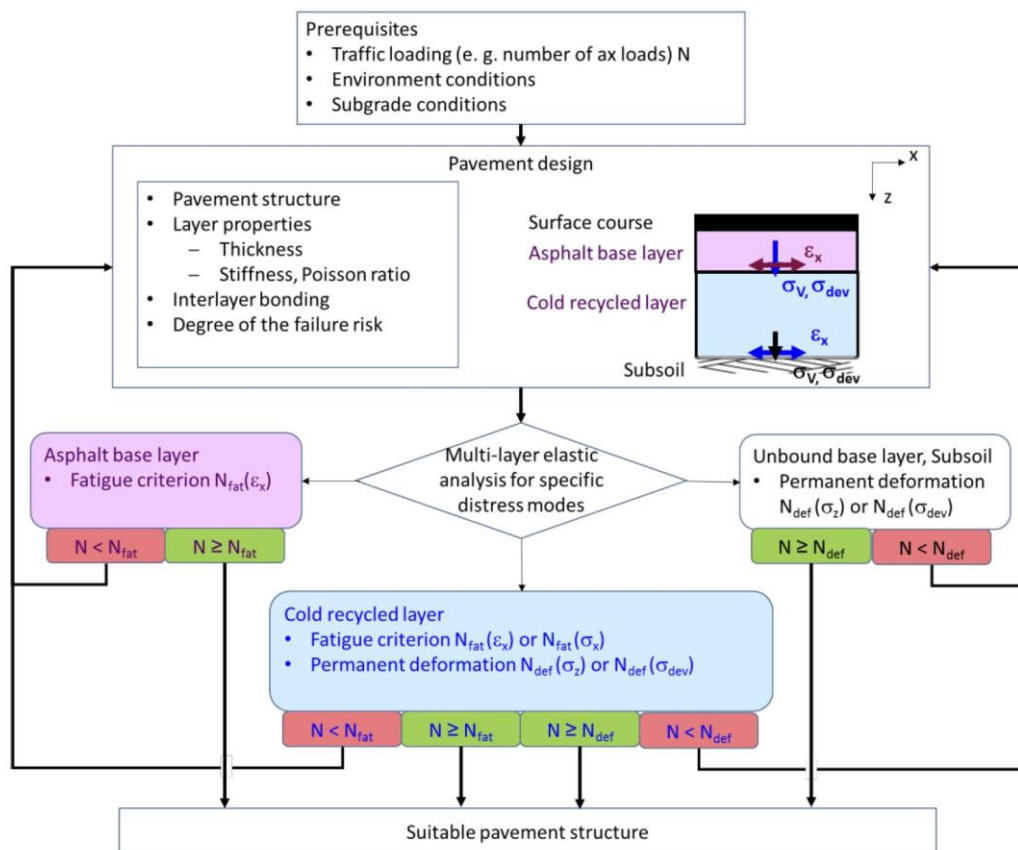


Figure 1 Analytical design principle for pavements with cold recycled layer (22)

In this investigation Kenlayer, an empirical-mechanistic software package, was selected to undertake the multilayer-elastic analysis. This software allows analysis which can incorporate CRBM behaviour (26). It was decided to use a mechanistic design method because of the lack of the necessary performance data to undertake an empirical analysis (21; 22). Furthermore a mechanistic method provides a theoretically sound approach as it relates the stresses, strains and deflections within a pavement structure with the loads and material properties.

The structure of airfield pavements comprises surface, binder and base courses laid on a foundation as shown in Figure 2, for new build and full rehabilitation designs (24; 25). The surface and binder courses have typical thicknesses of 40 and 60 mm respectively; the base course thickness is designed as a function of traffic, subgrade conditions and desired design life (26).

According to the BAA approach the base thickness calculated is then divided into 1/3rd asphalt and 2/3rd dry lean concrete (24), while DMG27 requires a minimum of 120 mm of asphalt base to avoid reflective cracking for high traffic scenarios (25).

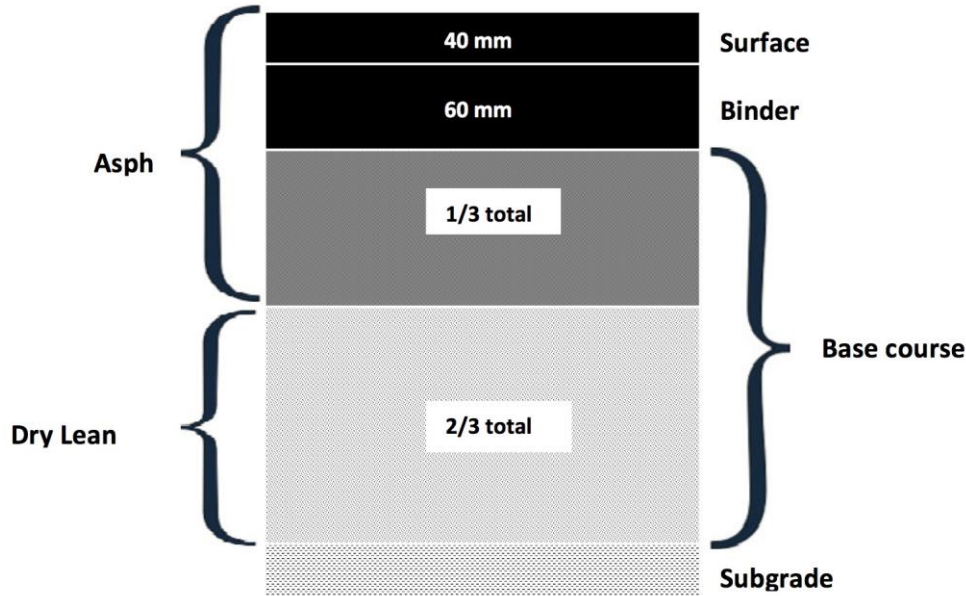


Figure 2 Airfield flexible pavement structure

To carry out a pavement analysis with Kenlayer (or any other multi-layer linear elastic program), material mechanical properties need to be defined, such as stiffness, Poisson's ratio and failure criteria; these parameters therefore have to be determined for CRBM.

The material stiffness can be obtained from conventional indirect tensile stiffness modulus (ITSM) tests (27). Poisson's ratio has a relatively small effect on the pavement response (26); thus a typical value of 0.3 has been adopted here for CRBM (28). The failure criteria can be adjusted by modifying cracking and permanent deformation algorithms (26). Fatigue cracking is a common distress that affects pavement service life (29).

The failure criterion for permanent deformation is expressed by equation 1:

$$N_d = f_1 (\varepsilon_c)^{-f_2} \quad [1]$$

where N_d is the allowable number of load repetitions to limit permanent deformation, ε_c is the compressive strain at the top of the subgrade, and f_1 and f_2 are coefficients determined from road tests or field performance (26). The compressive strain at the top of the subgrade is used as a failure criterion as the permanent deformation is considered to be caused by subgrade weakness rather than by the overlying layers (26). Taking this into account, for this research f_1 and f_2 were selected as 1.365×10^{-9} and 4.477 respectively, these values being taken from the Asphalt Institute analytical design procedure (26; 30).

The failure criterion for fatigue cracking is expressed by equation 2:

$$N_f = f_3 (\varepsilon_t)^{-f_4} (E_1)^{-f_5} \quad [2]$$

where N_f is the allowable number of load repetitions to prevent fatigue cracking, ε_t is the tensile strain at the bottom of the asphalt layer, E_1 is the elastic modulus of the asphalt layer and f_3 , f_4 and f_5 are coefficients determined from laboratory fatigue tests, with f_3 modified to correlate with field performance observations (26).

A key aim of this paper was to evaluate at a laboratory level the parameters needed to perform a pavement analysis with Kenlayer incorporating CRBM with foamed bitumen layers, namely stiffness and fatigue coefficients, f_3 , f_4 and f_5 .

2. MATERIALS AND METHODS

To obtain the inputs needed for Kenlayer mentioned in the introduction, a laboratory program was established. The materials used were specified fully in previous work (20) and are summarised in the next section.

2.1. Materials

For CRBM mixture manufacture, RAP, fly ash, cement and foamed bitumen were used with the mix design shown in Table 1. Fly ash was added as the RAP contained less fines than the specification demands. More information about the mix design optimization can be found in the authors' previous works (20; 31).

Table 1 CRBM mix design

Ingredient	Proportion by mass (%)
0-10mm RAP	43.5
10-20 mm RAP	39.1
Fly ash	6.3
Cement	1.6
Foamed bitumen	3
Total water content	6.5

The binder contents in the RAP and in the final mixture were calculated in accordance with BS 598-102 (32). Binder was recovered in accordance with BS EN 12697-3 (33) for characterisation. The recovered bitumen and the bitumen used for foaming were characterised in terms of softening point in accordance with BS EN 1427-2007 (34) and penetration grade in accordance with BS EN 1426-2007 (35). The results are shown in Table 2.

Table 2 Bitumen characterisation

Bitumen	Binder content (%)	Penetration (25°C,1/10 mm)	Softening point (°C)
100/150	NA	107	44.2
RAP 0-10 mm	7.2	30	58.6
RAP 10-20 mm	4.4	32	55.0
Recovered from mixture	7.5	46	52.4

A Wirtgen WLB 10 mobile foaming plant, with the settings established in Table 3, and a gyratory compactor were used for specimen manufacture. Specimens were double wrapped in cling film and cured for 28 days at 20°C.

Table 3 Foaming conditions

Water Pressure	4 bar
Air pressure	5 bar
Bitumen type	100/150
Bitumen temperature	170°C
Water addition	1%

2.2. Methodology

The testing methodology comprised:

- Determination of indirect tensile stiffness modulus (ITSM) to BS EN 12697-26:2004 Annex C (36)
- Indirect Tensile Fatigue Tests (ITFT) in strain control mode to BS EN 12697-24:2012 Annex E (37)

2.2.1. ITSM

Asphalt material stiffness relates to its load spreading ability and temperature susceptibility, parameters used to assess pavement structural condition. In a structural asphalt layer, high stiffness indicates good load-spreading ability.

In determining ITSM the rise-time, which is the time taken for the applied load to increase from the initial contact load to its maximum value, was selected as 124 ms. 10 conditioning pulses were applied to set the load needed to obtain a peak horizontal deformation of 5µm. To calculate the stiffness modulus 5 pulses were applied across two perpendicular diameters (36).

As stated before, the stiffness value is a material property required to undertake analysis with Kenlayer and it was measured on 37 specimens at 10, 20 and 30 °C.

2.2.2. ITFT in strain control mode

This test was developed at the University of Nottingham (38) and monitors the stiffness variation for a specified repeated strain value and the number of cycles until failure occurs. These parameters were needed to determine the fatigue coefficients in equation 2. This is a relatively simple test and suitable for cylindrical specimens; therefore, the manufacture of test specimens was straight-forward, saving materials and using the same compaction method as in the author's previous research, gyratory compaction. This also avoided the need to cut specimens from a slab, a process that can affect CRBM behaviour.

The strains selected for the ITFT were between 150 and 300 µε (39) and the loading frequency was 2 Hz. The test was performed at 20 °C on 14 specimens and the failure criterion used was the conventional target of 50% reduction of stiffness value (40).

3. ANALYSIS OF RESULTS AND DISCUSSION

3.1. Stiffness

The stiffness results presented in Figure 3 show low temperature susceptibility of CRBM mixes when compared to HMA (20). This is likely to be due to the action of cement within the mixture. Regarding the stiffness value used for design (at 20°C), 3500 MPa is comparable to the values of HMA assumed in airfield base layers, for example DBM50 for which stiffness values range between 2400 MPa and 5000 MPa (41). It would therefore appear that CRBM with foamed bitumen has appropriate stiffness modulus for airfield pavement design.

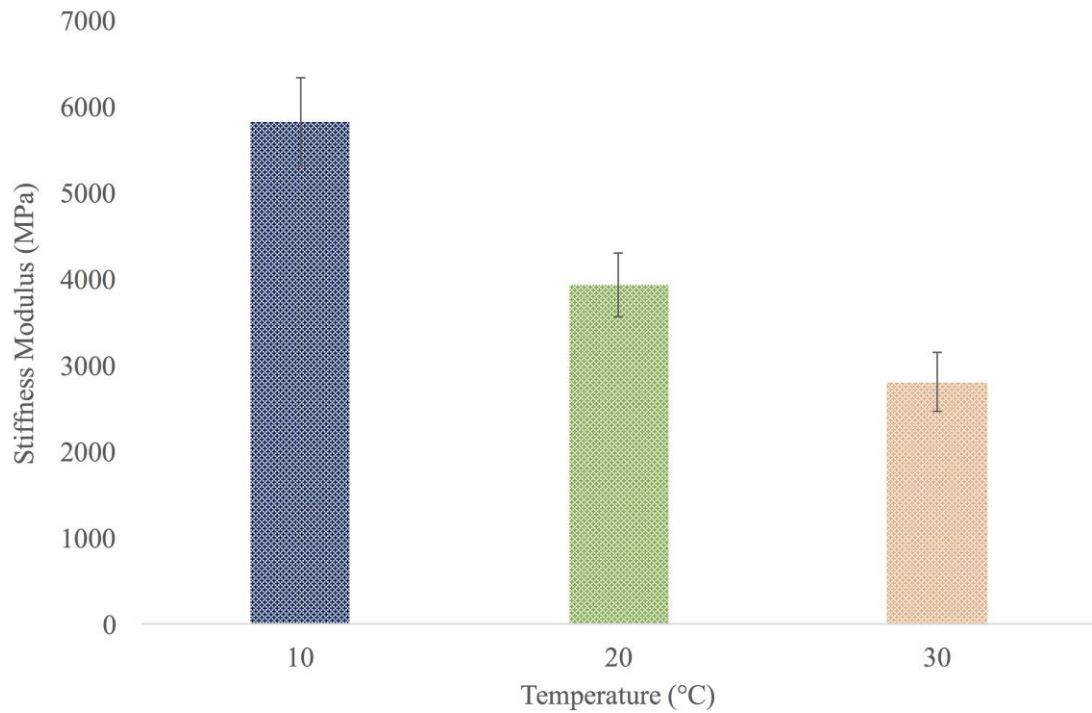


Figure 3 CRBM stiffness modulus

3.2. ITFT in strain control mode

Table 4 shows the results from the ITFT in strain control mode for CRBM.

To obtain the coefficients, the difference between the N_f obtained in the laboratory and the N_f calculated using equation 2 was minimised by optimising the values required for f_3 , f_4 and f_5 . The fatigue curves obtained from laboratory tests and from calculations are presented in Figure 4.

Table 4 Experimental data from strain control fatigue tests for CRBM

N_f	ϵ_t	E_1 (MPa)
184279	0.00015	3426
202363	0.00015	3081
165563	0.00018	2219
78554	0.00018	2330
123526	0.0002	2343
64123	0.0002	1660
88723	0.0002	2895
65933	0.00022	1996
90173	0.00025	1711
36413	0.00025	2179
82223	0.00027	1390
59043	0.00027	1343
15183	0.0003	1432
21673	0.0003	1646

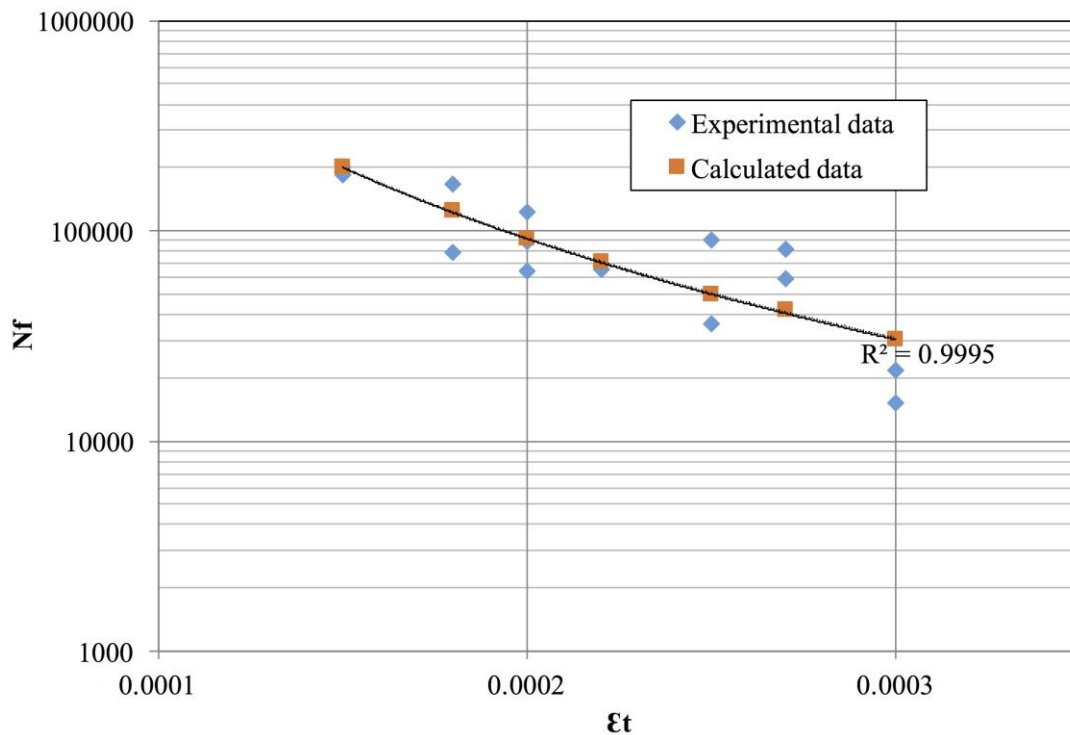


Figure 4 CRBM fatigue law calculation

Maggiore's data (38), shown in Table 5, were also used for analysing the adequacy of this fatigue test method, and the values of f_3 , f_4 and f_5 compared to those for HMA calculated by the Asphalt Institute and Shell (26; 30). HMA results and CRBM fatigue coefficients are presented in Table 6.

Table 5 Experimental data from strain control fatigue tests for HMA

N_f	ϵ_t	E_1 (MPa)
149243	0.000125	10900
126500	0.000135	10329
88923	0.000145	11081
42613	0.000155	10231
49383	0.000165	10220
22393	0.000175	10582
30963	0.000185	9828
18683	0.0002	9928
17773	0.00022	9245

Table 6 Fatigue coefficients

	f_3	f_4	f_5
Calculated HMA	0,074	4.842	3.109
Shell factors	0.0685	5.671	2.363
The Asphalt institute factors	0.0796	3.291	0.854
Calculated CRMB	$7.61 \cdot 10^{-6}$	2.826	0.110

It is noted that the values calculated using Maggiore's HMA data and the values proposed by Shell are comparable. Thus it seems likely that the ITFT in strain control mode is a suitable test for fatigue coefficient calculation.

Previous researchers report various values for these coefficients, with the typical range of values for f_4 being between 3 and 6 (42); however, f_3 varies by several orders of magnitude, and these points relate to values obtained for HMA.

With the new coefficients obtained in Table 6, the fatigue law for CRBM can be expressed as in equation 3.

$$N_f = 7.61 \cdot 10^{-6} (\epsilon_t)^{-2.826} (E_1)^{-0.11} \quad [3]$$

The fatigue curves from Maggiore's data and the CRBM mix are compared in Figure 5. This comparison highlights the difference in behaviour between HMA and CRBM. It should be noted that at the same strain, CRBM has a greater life. However, the fact that CRBM stiffness is lower than that of HMA has to be taken into account; thus, when HMA reaches 50% of its initial stiffness it is deemed to have failed but the stiffness is still greater than the initial stiffness of CRBM. This highlights the necessity for further study on failure criteria.

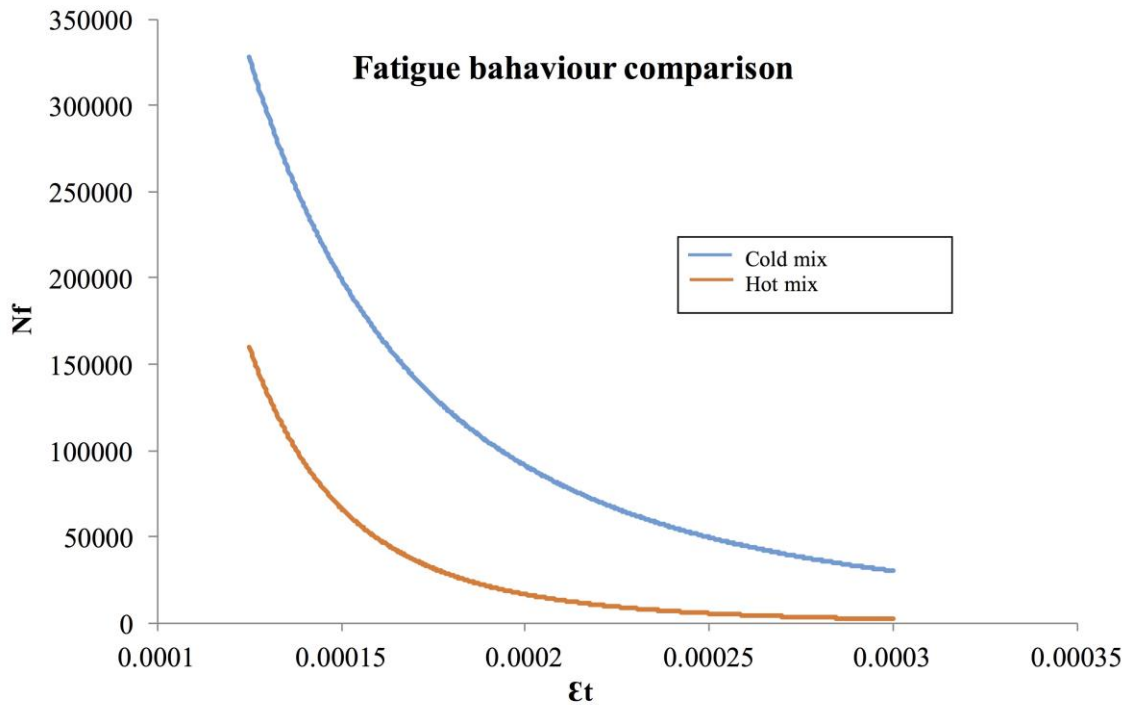


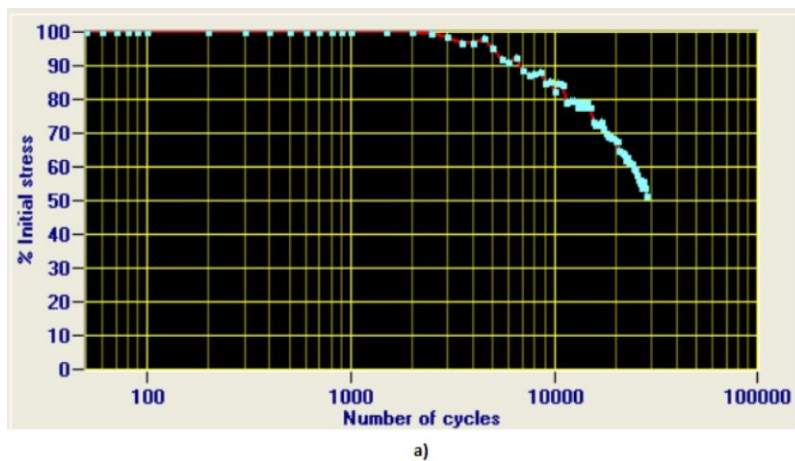
Figure 5 Fatigue laws comparison

The fundamental material input variables for CRBM assessed with the Kenlayer model have been identified as stiffness and fatigue with permanent deformation being dependent on the subgrade. These variables are summarised in Table 7.

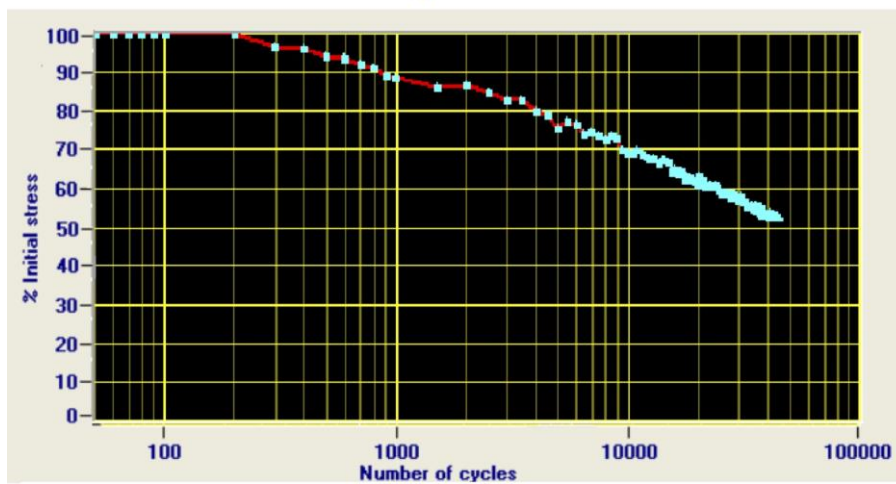
Table 7 Kenlayer inputs

Input		Value
Poisson's Ratio		0.3
Stiffness Modulus	20°C	3500 MPa
	f_3	$7.61 \cdot 10^{-6}$
Fatigue factors	f_4	2.826
	f_5	0.110

It is also interesting to study the material behaviour in terms of stress evolution during testing to analyse if the modes of failure of the two materials are comparable. In Figure 6 it can be appreciated that the stress evolution is different for HMA and CRBM mixes. HMA has a near-constant stress at the beginning and then it falls relatively rapidly. For CRBM the stress starts reducing from the beginning, but at a moderate slope. This highlights the importance of studying the mode of failure for CRBM, since it does not appear to be comparable with HMA.



a)



b)

Figure 6 a) HMA fatigue behaviour (38) b) CRBM mix fatigue behaviour

4. CONCLUSIONS

In this paper, the fundamental CRBM input variables for undertaking a pavement design analysis with Kenlayer have been identified as stiffness and fatigue life.

Laboratory determination of these inputs showed significant difference in the performance of CRBM versus HMA. In terms of stiffness, calculated values for CRBM are within specifications; therefore this material is identified as potentially appropriate for airfield pavement design. Fatigue coefficients have been established for CRBM; however, the failure criterion used in this research was the conventional target of 50% reduction of stiffness value, as generally used for HMA, and it remains to be investigated whether this failure criterion is also valid for CRBM.

Fatigue is a determining factor for understanding CRBM behaviour under cyclic loading. For this reason, further investigation is needed in order to develop fuller understanding of how CRBM performs, and how pavement design should best be progressed.

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ANALYSIS OF THE FAILURE MECHANISM OF COLD RECYCLED BOUND MATERIALS EVALUATING FATIGUE PERFORMANCE BASED ON DISSIPATED ENERGY, DISPLACEMENT RATIO AND ENERGY STIFFNESS RATIO.

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ABSTRACT

It is well known that fatigue is one of the main failure mechanisms in flexible pavements but this is not so well established in cold recycled bound materials (CRBM) as in hot mix asphalt. Using CRBM provides an environmental and economic advantage for pavements since they decrease energy and raw material consumption. Having information about how these mixes fail is essential to develop a trustworthy design methodology. This paper presents a laboratory fatigue study using indirect tensile fatigue tests in strain control mode. The material investigated contained reclaimed asphalt pavement (RAP), fly ash, and foamed bitumen and cement as stabilising agents. The data obtained in the laboratory has been analysed using four different approaches: Dissipated energy ratio (DER), Ratio of dissipated energy change (RDEC), displacement ratio approach and energy stiffness ratio. With these approaches crack initiation and propagation have been studied. The aim is to understand CRBM behaviour under cyclic loading and analyse the differences compared to hot mixes, investigating if the current practice of equating both materials' failure criteria is accurate. The results presented in this paper show that the CRBM used behaves differently, and that the accepted failure criterion of 50% stiffness reduction for hot mix asphalt, is evidently not appropriate for this CRBM. Nonetheless, further understanding of the fatigue test, the failure criterion and how micro cracking affects the bearing capacity of CRBM layers is required.

Keywords: *cold recycled materials, fatigue, failure mechanism, dissipated energy.*

1 INTRODUCTION

2 Using cold recycled bound material (CRBM) is a proven technique for pavement construction.
3 It is widely recognised to potentially reduce energy and raw material consumption relative to
4 conventional materials used in pavement construction (1,2). Reclaimed asphalt pavement
5 (RAP) (3,4) is a major component of CRBM, and it can be reused into most pavement
6 applications (5); however, it is most commonly used in unbound mixtures for sub-base,
7 hydraulically bound mixtures for sub-base and bitumen bound layers (6,7). The authors'
8 research focuses on recycling RAP into CRBM, using foamed bitumen and cement, for use in
9 airfield pavement base course layers.

10 In terms of the mechanical performance of materials, fatigue is one of the main failure
11 mechanisms in flexible pavements (8,9); thus, its study is considered a research priority (9). In
12 particular, understanding fatigue behaviour is fundamental to develop a refined design
13 methodology beyond that developed for conventional hot asphalt mixtures (8). Fatigue
14 behaviour of hot mix asphalt (HMA) has been studied extensively in the past; however, for
15 CRBM this is not so well established (8).

16 The failure criterion commonly used for fatigue is the phenomenological approach with a
17 conventional target of 50% reduction of stiffness value (10); however, this failure criterion has
18 been established based on the testing of HMA (10). In this paper, a fatigue study of CRBM
19 using dissipated energy, displacement ratio and energy stiffness ratio approaches is presented.
20 The aim is to understand CRBM behaviour under cyclic loading and observe the failure
21 mechanism to establish an appropriate failure criterion.

23 Dissipated energy fatigue approach

24 There are three generic approaches to obtain fatigue performance of materials. The
25 phenomenological approach is based on experience and has no theoretical basis. The fracture
26 mechanics approach studies only the crack propagation, while the dissipated energy approach
27 can be used for analysing crack propagation and failure; therefore this last approach was
28 selected for this study (11).

29 Moreover, the dissipated energy method reduces the gap between research in the laboratory
30 and field behaviour of the material, creating a link between research and reality (11). When
31 applying a load to a material energy is being input into that material (12), and this energy can
32 be represented as the area under the stress-strain curve (12,13). During the loading-unloading
33 process the energy is either recoverable or lost. If the loading curve and the unloading curve
34 are the same, all the energy has been recovered (typical of elastic materials)(2); however, if the
35 curves are different, a loss of energy has occurred (typical of visco-elastic and frictional
36 materials) (9). This loss means that part of the energy is dissipated in the form of mechanical
37 work, heat generation or damage (9,12). This phenomenon is called "Hysteresis" (9,12) and
38 the energy lost in each load cycle is the dissipated energy (9). The dissipated energy in each
39 cycle is the area within the hysteresis loop (Figure 1) (9,12,14).

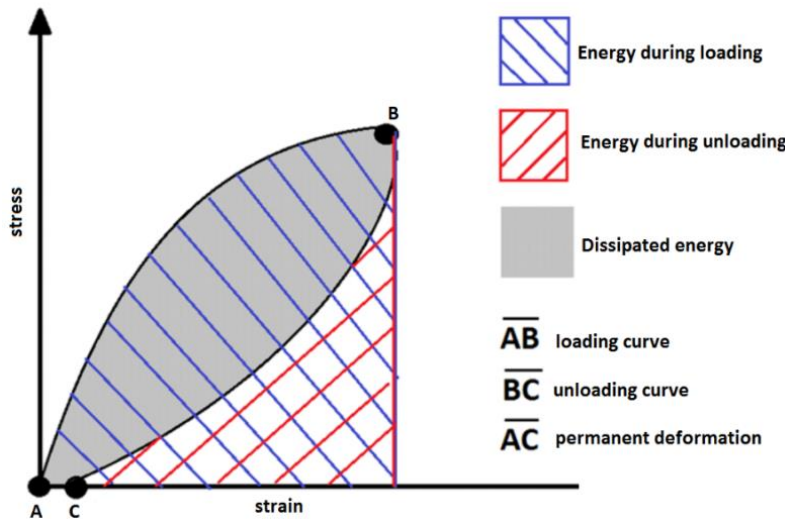


FIGURE 1 Hysteresis loop (9)

The Indirect Tensile Fatigue Test (ITFT) in strain control mode was selected to study the fatigue behaviour of CRBM under laboratory conditions. The ITFT in strain control mode was developed at the University of Nottingham (11). It is a relatively simple test and suitable for cylindrical specimens; therefore, the stress distribution is relatively uniform during testing within the range of displacement measurement, and specimen manufacture can be undertaken using the gyratory compactor which realistically simulates site compaction (15). Use of this method also avoids the requirement to cut specimens from a slab, which removes consideration of any micro cracking/coring induced damage to the CRBM specimens, prior to them being tested.

In the literature, several dissipated energy based methods have been developed to analyse the data obtained from laboratory tests (11). The initial dissipated energy (IDE) approach was found not to be appropriate for fatigue tests for the whole loading range (11,12); also it does not account for the effect of healing (12). The cumulative dissipated energy (CDE) approach is mix and temperature dependent (11,12); therefore, it is not a good parameter to describe the fatigue phenomenon. The dissipated energy ratio approach (DER) (16) suggests a fatigue life based on the difference between the energy needed for the creation of micro cracks and the energy needed for the growth of these cracks (11). The ratio of dissipated energy change (RDEC) is an improvement on the DER approach. It is able to eliminate other forms of dissipated energy due to mechanical work or heat generation; so it can be considered as a good parameter to describe the fatigue process (9,11,12,17). It is valid for the whole range of loading, different mixture types, loading modes and loading conditions (12).

The displacement ratio relates the dissipated energy with the material deformation, allowing the identification of micro-crack initiation and propagation (13). The energy stiffness ratio at failure represents material behaviour at which damage accumulation has produced an inability of the mix to resist further damage, and it is independent of the loading mode (18).

In this research it was decided to focus on DER, RDEC, displacement ratio and energy stiffness ratio approaches for analysing crack propagation and failure.

DER approach

With this approach and equation 1 crack propagation can be studied (9,12,16).

$$DER_n = \frac{n \cdot \omega_0}{\omega_n} \quad (1)$$

Key: DER_n is the energy ratio in cycle n , n is the cycle number, ω_0 is the dissipated energy in the first cycle and ω_n is the dissipated energy in cycle n (16).

With the ITFT in strain control mode and representing the DER against the number of cycles, the curve shown in Figure 2 can be obtained. This curve has an initial linear part where the dissipated energy ratio is constant versus the number of cycles. When permanent deformation occurs (cracking) the line develops a rapid increase. N_i represents the initiation of micro crack development in a bituminous material. When this curve arrives at the inflexion point, the macro cracks are starting to develop (N_M) until complete failure (N_f) (11,13). It can be seen that the appearance of macro cracks in a HMA sample is related to the failure criterion of 50% stiffness reduction (N_{50} in Figure 2) (11,19).

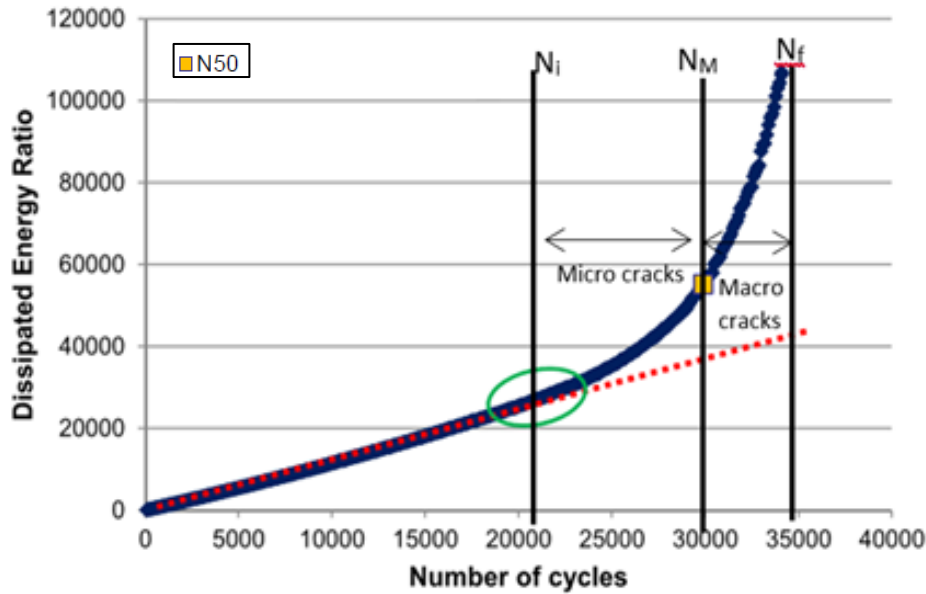


FIGURE 2 Evolution of DER in strain control fatigue (11)

RDEC approach

The RDEC is calculated with equation 2 where RDEC is the ratio of the dissipated energy change per load cycle, ω_a is the dissipated energy produced in load cycle a , ω_b is the dissipated energy produced in load cycle b and a and b are load cycles (12,14).

$$RDEC = \frac{\omega_a - \omega_b}{\omega_a(b - a)} \quad (2)$$

The RDEC presents 3 stages when represented against number of cycles (Figure 3). After an initial period, Stage I, the RDEC presents a plateau period, Stage II, where the percentage of input energy being turned into damage is constant. Finally, in Stage III, there is a dramatic increase in RDEC, which signifies fatigue failure (12,14) and is typically related to the visual appearance of macro cracking (9,11). Once the macro crack appears there is often rapid irreversible damage occurring with every loading cycle (9). In this approach, the 50% stiffness reduction occurs at the end of stage II.

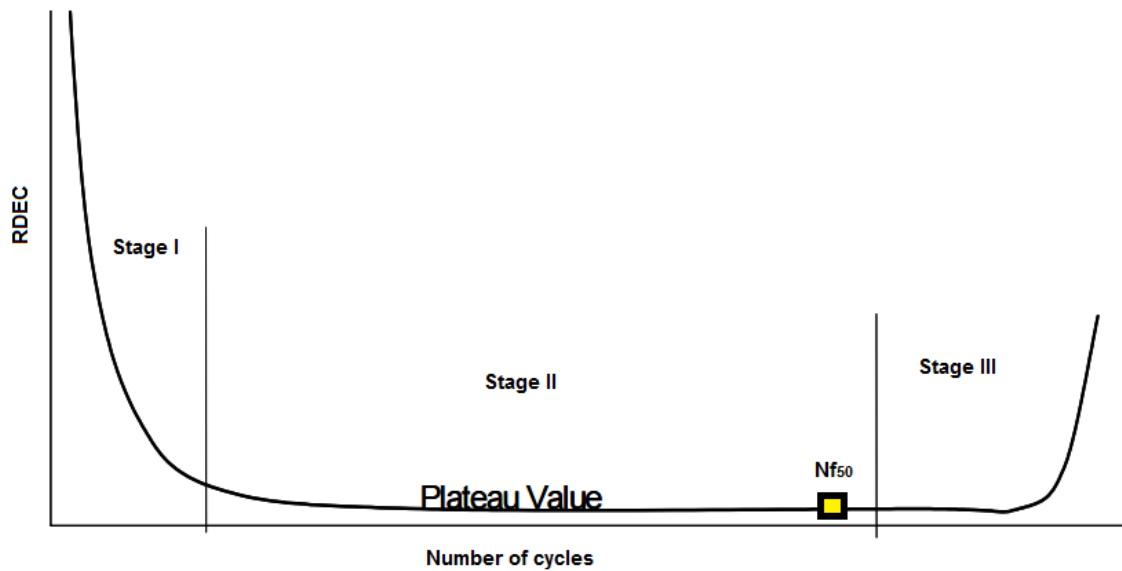


FIGURE 3 RDEC evolution (14,20)

Displacement ratio approach

The displacement ratio represents the relationship between horizontal and vertical displacements, representing the changes produced in the geometry of the material in each load cycle (13). It is calculated with equation 3, where δ_n is the displacement ratio in cycle n , δ_{hn} is the horizontal displacement in cycle n and δ_{vn} is the vertical displacement in cycle n . With this method the phases of crack propagation can be studied, as in the example shown in Figure 4. Here, the first stage is where plastic deformations are taking place, so no damage is induced to the material. Then crack propagation begins, starting with micro cracks (N_i) and then macro cracks (N_M) until complete failure (N_f).

$$\delta_n = \frac{\delta_{hn}}{\delta_{vn}} \quad (3)$$

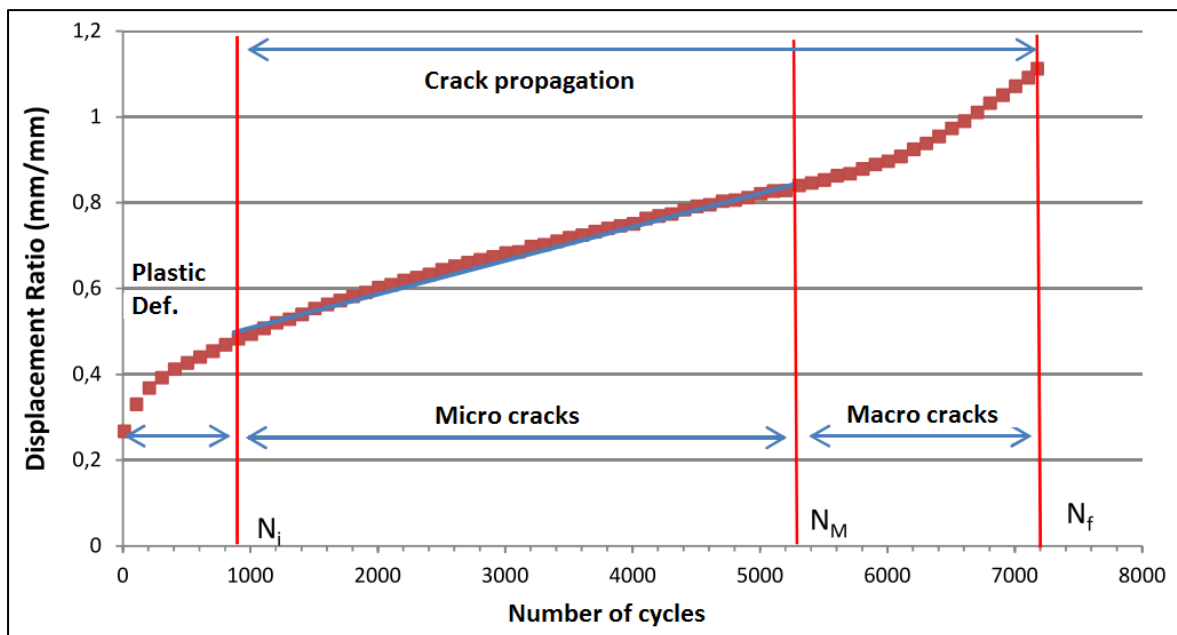


FIGURE 4 Crack propagation study with displacement ratio approach (9)

Energy stiffness ratio

For analysing the accuracy of the failure criterion used for HMA, the energy stiffness ratio concept was defined by Abojaradeh (18) and it is described by equation 4. N_i is the cycle number, S_i is the stiffness at cycle i and S_0 is the initial stiffness taken at cycle 50 (18).

$$\text{Energy stiffness ratio} = \frac{N_i S_i}{S_0} \quad (4)$$

With this approach, failure is defined as the number of cycles at the peak value of the curve, when the energy stiffness ratio is plotted against the number of cycles (Figure 5) (18). For benchmarking research, using this approach (18) confirms that the failure with this method for HMA can reliably be defined at 50% stiffness reduction.

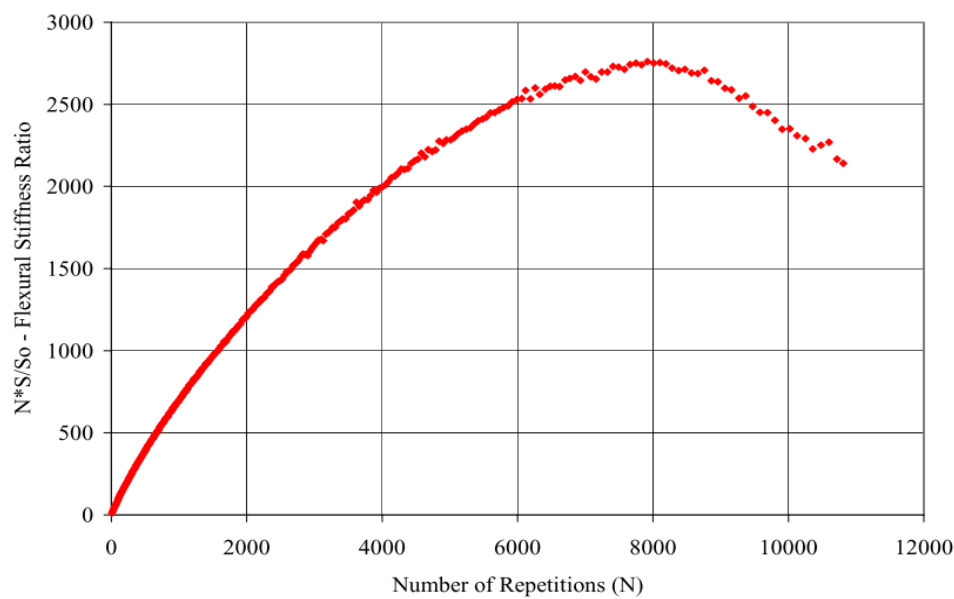


FIGURE 5 Energy stiffness ratio (18)

MATERIALS AND METHODOLOGY

To generate the data sets for CRBM, a laboratory test program was established. The materials used are specified in detail in (21) with a summary provided in the next section.

Materials

The CRBM mixture comprises RAP, fly ash, cement and foamed bitumen, with the mixture volumetric proportions given in Table 1.

TABLE 1 CRBM mix design

Ingredient	Proportion by mass (%)
0-10mm RAP	43.5
10-20 mm RAP	39.1
Fly ash	6.3
Cement	1.6
Foamed bitumen	3
Total water content	6.5

A mobile Wirtgen foaming plant type WLB 10, with the settings established in Table 2, and a gyratory compactor were used for the manufacturing of the test specimens. Post compaction, the samples of CRBM were cured double wrapped in cling film (poly vinyl chloride (PVC)) for 28 days at 20°C. The PVC wrapping is designed to ensure the CRBM does not dry out during curing. The duration and temperature selected for the curing procedure were based upon extensive research in the UK which reports that the regime is suitable to simulate CRBM in-service performance (22,23). This has been calibrated for a range of tracking scenarios on UK roads where a minimum HMA surface course was installed over the CRBM mix, with trafficking commencing after a further few days (6,22,23).

TABLE 2 Foaming conditions

Water Pressure	4 bar
Air pressure	5 bar
Bitumen type	100/150 pen grade
Bitumen temperature	170°C
Water addition	1%

Methodology

The testing methodology for the assessment of the CRBM comprised ITFT in strain control mode (11). With this test the stiffness variation for a specific strain value, between 150 and 300 μ strain, and the dissipated energy are measured and the number of cycles until failure recorded (11). The test was initially carried out with the standard failure criterion of 50% stiffness reduction at 20°C with a loading frequency of 2 Hz. However, after analysing the data obtained, it was decided to carry out more tests until complete failure of the specimens. This was undertaken to further understand the micro and macro crack propagation in CRBM. In total, 26 specimens were tested and are reported as part of this study.

With the results obtained in the laboratory the failure mechanism was studied applying the DER, RDEC and displacement ratio approaches. Finally the energy stiffness ratio was applied to analyse the failure criterion.

RESULTS AND DISCUSSION

With a traditional bending fatigue test the stress and strain are applied with a continuous sinusoidal waveform (24); however, in the ITFT a haversine waveform is applied, thus the dissipated energy looks like half of a typical hysteresis loop (11). In Figure 6 the type of hysteresis loop obtained for the ITFT in strain control mode with CRBM is presented.

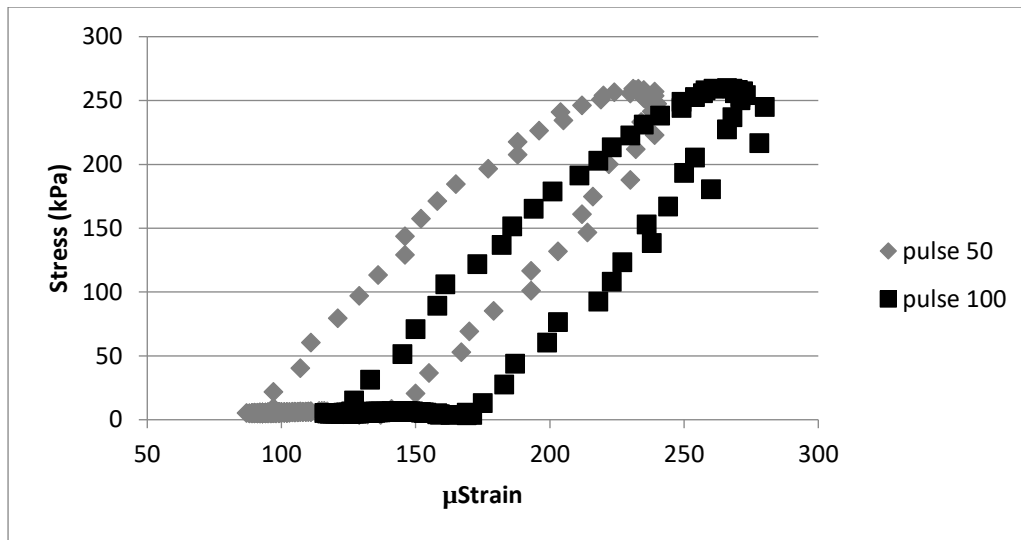


FIGURE 6 Hysteresis loop with haversine waveform obtained with ITFT in strain control mode

In the ITFT in strain control mode, the specimen does not reach complete fracture along the vertical diameter at failure; it is characterised by localised deformations at the edge of the loading strips with visible cracks observed along the vertical plane of the specimen, as shown in Figure 7.



FIGURE 7 Failure in strain control mode

In Figure 8 an example of the different approaches studied in this paper with the ITFT results from CRBM is presented. It can be appreciated that, in this example, the micro cracking starts around 45000 cycles according to the DER approach; however, according to the RDEC and displacement ratio approaches the micro cracks start before 10000 cycles. This discrepancy is related to the type of micro cracks (9). Ni in the DER represents the widening of hairline cracks and the creation of a crack network (11); therefore if the cracks are dispersed in the specimen the DER approach will not detect them as early as in the displacement ratio approach (9). With this information, it is noted that for this CRBM there is a long period (between 10000 and 45000 cycles in this example) where dispersed micro cracks are being developed.

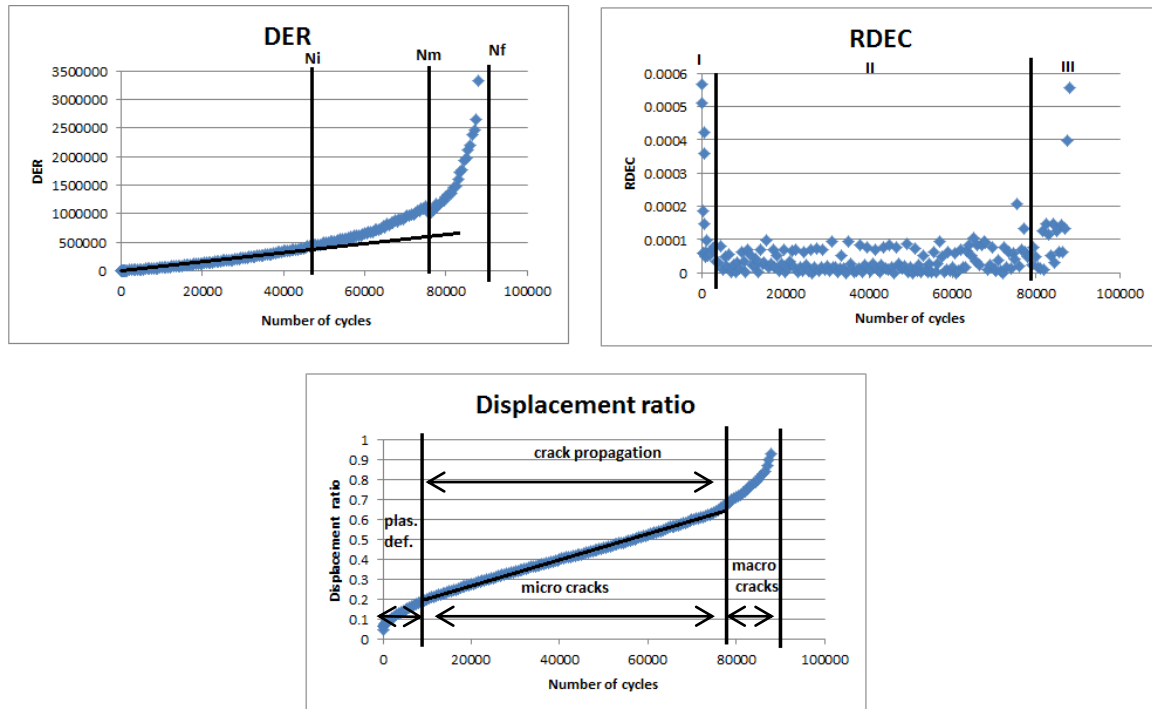


FIGURE 8 Dissipated energy and displacement ratio results for 10% stiffness failure criterion with 300μstrain

In Figure 9 the stiffness and DER evolution are plotted. It can be seen that the failure criterion of 50% reduction of stiffness occurs around 40000 cycles while macro cracks develop around 75000 cycles. This failure mechanism differs from HMA, where the 50% stiffness reduction is related to the appearance of macro cracks (11,19) as stated in the introduction (see Figure 2).

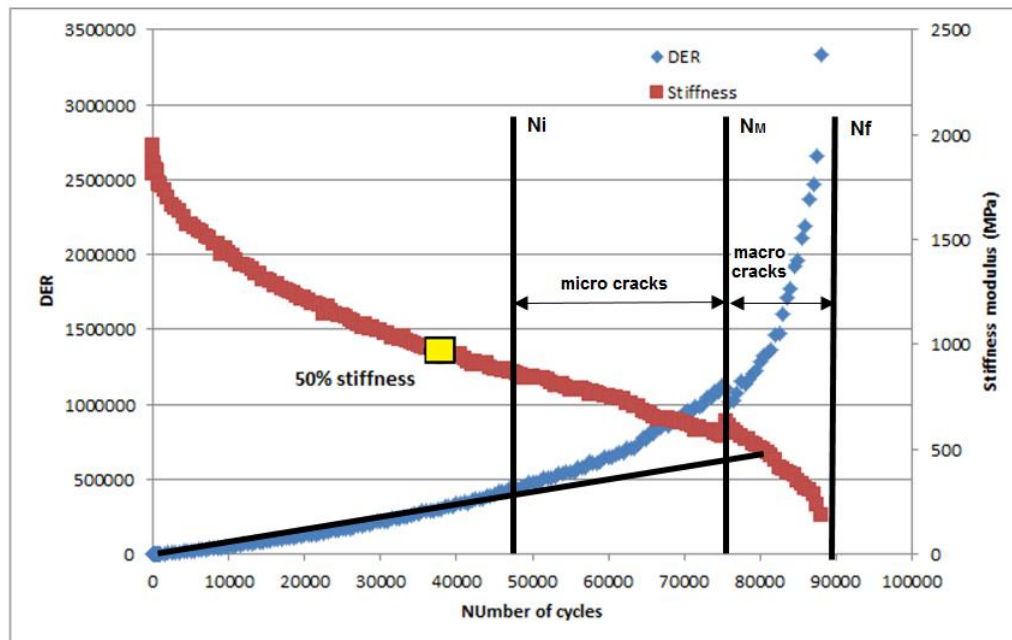


FIGURE 9 DER and stiffness evolution with 300μstrain

Applying the energy stiffness ratio concept, as shown in Figure 10, it is noted that the failure related to the appearance of macro cracks occurs after the 50% stiffness reduction. This phenomenon was observed for all the strain levels tested.

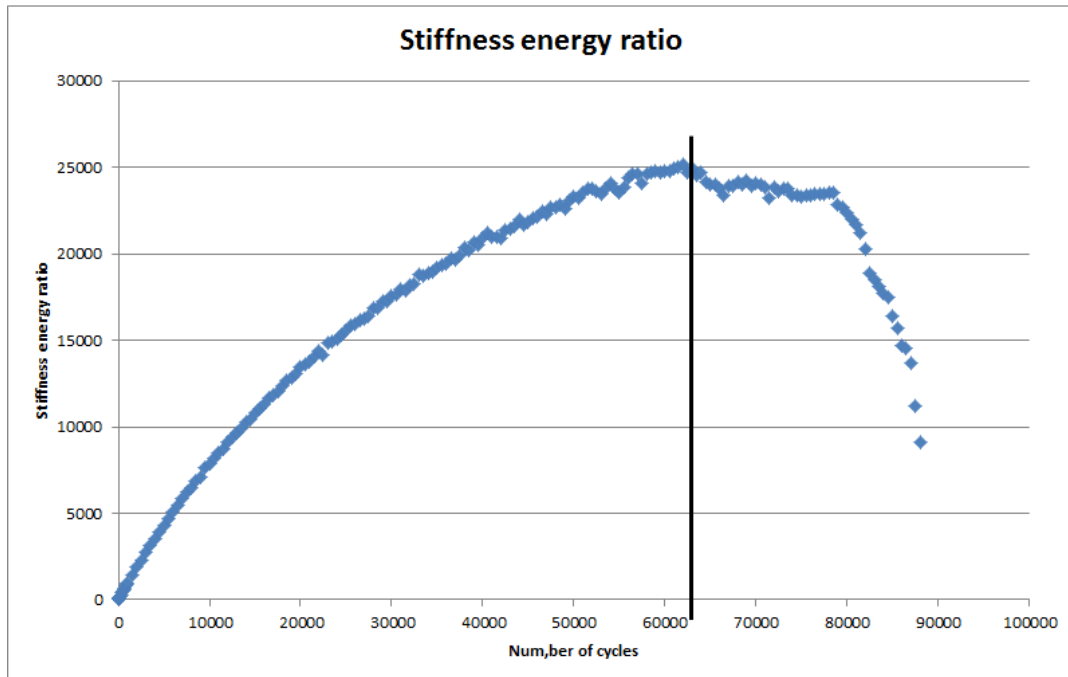


FIGURE 10 Energy stiffness ratio

In Table 3 a summary of the results obtained with the four approaches and the tests carried out until complete failure of the specimens are presented.

TABLE 3 Results summary

	DER			RDEC		Displacement ratio			Energy stiffness ratio peak value (failure)
	N _i	N _M	N _f	Stage II	Stage III	N _i	N _M	N _f	
Main value	37.3	22.4	13.1	69.9	18.4	62.5	20.7	12.9	27.7
Maximum value	47.5	30.5	22.0	80.0	30.5	76.0	30.5	22.0	40.0
Minimum value	24.0	14.5	10.0	58.0	10.5	40.4	15.0	10.0	18.5
Standard deviation	6.6	5.5	4.1	7.5	6.3	11.0	5.1	4.2	7.0

It can be noticed that the appearance of macro cracks occurs between 20.7% and 27.7%, depending on the method used. This highlights the different performance of CRBM in comparison with HMA. However, deeper study is still needed in order to establish a failure criterion for CRBM, analysing different mix designs, temperatures, tests and field performance.

CONCLUSIONS

In this paper, a study on CRBM using data from the ITFT in strain control fatigue is presented. Three failure mechanism approaches were used to analyse the data obtained: DER, RDEC and displacement ratio. These approaches give the same crack propagation analysis, taking into account that the DER approach does not detect dispersed micro cracks as early as the RDEC and displacement ratio.

For understanding the suitability of the 50% stiffness reduction failure criterion, the energy stiffness ratio concept was used to analyse the laboratory data. In this case specimen failure was found to occur at 27.7% initial stiffness.

With these results it can be confirmed that this CRBM fails in a different way from HMA; therefore, a different failure criterion should be established.

Studies of crack propagation show that, unlike conventional HMA, micro cracking still predominates at the 50% stiffness, and macro cracking only tends to occur at around 20-27% of the initial stiffness.

The study shows a clear fundamental difference in failure mechanism between this CRBM and HMA. The implications of these findings are that basing fatigue models for CRBM on established practice for HMA is likely to be overly conservative.

This means that there is a gap in understanding of how CRBM fails; therefore, further investigation into the suitability of tests and failure criteria is needed. Also, CRBM should be studied at full scale to confirm how micro cracking affects the bearing capacity of the CRBM layer and to properly develop a design methodology for this material.

ACKNOWLEDGEMENTS

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Towards airfield pavement design using cold recycled bound materials

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ABSTRACT: Using cold recycled bound material (CRBM) is a sustainable technique that saves energy and raw material consumption. This technique is widely used in UK roads but its application in airfields is limited as there is no specific pavement design guidance for using CRBM. As part of the author's research in the SUP&R ITN project this paper proposes appropriate design guidance for using CRBM in airfield pavements. With this aim an analytical approach using a multilayer elastic model (Kenlayer) was used to analyse different airfield pavements with conventional asphalt materials for specific loading and subgrade conditions. First, pavement designs obtained from Kenlayer were compared to those from an existing design guide, namely DMG 27 from Defence Estates, UK, to prove the suitability of the approach and inputs for airfield design purposes. After this, the software was used to analyse pavement structures where CRBM was substituted for asphalt concrete in the base layer. The material properties were assigned based laboratory testing and established design parameters. The results show that different asphalt concrete and CRBM layer thicknesses were needed for the same design conditions, due to the different material properties. This allowed an appropriate factor to be derived to increase the thickness given by DMG27 (or any conventional airfield pavement design method) to account for a CRBM base.

1 INTRODUCTION

The requirement to use recycled asphalt plantings (RAP) in new asphalt is becoming increasingly important (Copeland 2011). The main benefits of using RAP include reducing demand on finite resources, reducing the generation of waste materials and reducing embodied energy (CIRIA 1999). Cold recycling of asphalt is a proven technique that reduces energy consumption (Kazmierowski et al. 1992, Thenoux et al. 2007). This study looks at foamed asphalt with cement and this type of material is classified in the UK as a cold recycled bound material (CRBM) (Merrill et al. 2004, Defence Estates 2009).

Despite the increasingly common use of CRBMs in roads (Merrill et al. 2004, West & Willis 2014), specifications for the use of these materials in airfields are underdeveloped (Defence Estates 2009). The results presented by Lacalle Jiménez et al. (2016) demonstrate, as expected, that the mixture behaviour differs from a CRBM to a hot mix asphalt (HMA), so design guidance is needed to ensure that a pavement

design incorporating these materials is trustworthy (Cardno 2012, Lacalle Jiménez et al. 2016). Nevertheless CRBM have acceptable properties for use in airfield pavements, as well as being within specifications (Lacalle Jiménez et al. 2016).

As established in the authors' previous works, design guides for airfield pavements such as FAArfield (Muniraj 2003), BAA (BAA plc Group Technical Services, 1993) or Design and Maintenance Guide 27 (DMG27) (Defence Estates 2011) do not readily allow the introduction of materials with non-standard properties (Lacalle Jiménez et al. 2016); therefore, a new design approach is needed.

In this paper, airfield pavements are modelled using Kenlayer incorporating the relevant material property, namely stiffness modulus. The results for conventional asphalt are calibrated against the DMG27 standard. The objective is to establish design guidance for using CRBM in airfield pavements. With this aim, an understanding of how airfield pavements are designed is first needed.

1.1 Airfield pavement design

Airfield pavements are classified as (Defence Estates 2011):

- Rigid: A rigid pavement comprises either wholly or partly concrete construction which can be plain, reinforced or prestressed and which distributes the aircraft loading to the subgrade by means of its high flexural stiffness.

- Flexible: A flexible pavement is composed of bound or unbound granular materials. Cement-bound granular bases beneath bituminous surfacings make pavements quite rigid in their early years. However, this type of construction is treated as a flexible pavement for design and evaluation purposes.

This research deals with the inclusion of CRBM as base course under asphalt surfacing; therefore, it is classified as a flexible pavement.

Defence Infrastructure Organisation (DIO) material specifications for CRBM are available for the lower layers of taxiways and runways (Defence Estates 2009); nonetheless the use of these materials as a replacement to hot mix asphalt is limited due to lack of design guidance, unknown long term performance and risk of failure with high rehabilitation costs and traffic delays.

Traditional design models can be classified into three groups: empirical, analytical or mechanistic and analytical-empirical or mechanistic-empirical.

Empirical methods are based on the observed performance of a pavement, monitoring it under traffic loading and different environmental conditions to establish the appropriate construction thickness. These methods are based on design charts, where the pavement thickness can be obtained for a given traffic level, material type and subgrade strength. This means that these methods are restricted to the conditions for which they were originally developed (Hakim, 2011). One example of this method is the DMG27 (Defence Estates 2011).

Analytical pavement design uses mathematical models (multi-layer elastic system, plate on elastic foundation or finite element) and fundamental engineering properties of materials to calculate layer thickness. The stresses and strains calculated with the mathematical models are compared with allowable values for the materials used, to control fatigue cracking and pavement deformation (or other deterioration mechanisms), and thereby to determine the necessary thickness of the pavement layer to achieve a given design life. These methods allow non-standard materials and construction types (Hakim, 2011).

The analytical-empirical (or mechanistic-empirical) approach has two steps. The first is based on analytical methods, calculating strains and stresses from mathematical models, and the second step is to compare these results with permissible stresses or strains. In all models, the analytical tool used is a version of the theory of elasticity and the damage caused by the stresses and strains is determined from empirical relations (Ullidtz 2002). Pavement design methods such as BAA and AASHTO 2008 are analytical-empirical.

In this paper, the BAA analytical design approach is used incorporating Kenlayer structural model to calculate stresses and strains within the pavement layers.

2 INPUTS FOR AIRFIELD PAVEMENT DESIGN

The first step was to demonstrate that the proposed analytical approach using Kenlayer model was appropriate for carrying out the required airfield pavement design and to establish the inputs needed. For this purpose a range of conventional flexible pavements were analysed for different traffic levels and foundation conditions and the results were compared with those from a conventional well established design guide. DMG27(Defence Estates 2011) was selected as the reference because it is the most commonly used guide for airfield pavement design in the UK, where the research was carried out.

In order to ensure a sufficiently representative sample of aircraft load scenarios, 8 different types of aircraft were chosen, of varying gear type, 4 different types of subgrade and 3 traffic levels. With these inputs the total number of cases to be analysed was 96.

2.1 DMG27 design

For performing pavement design with DMG 27, the first step is to establish the Aircraft Classification Number (ACN) for each subgrade condition. The ACN of an aircraft expresses the relative loading severity on a pavement supported by a subgrade of particular strength (Defence Estates 2011). In Table 1 the calculated ACN's are presented for the different aircraft and types of subgrade.

These cases have been studied at 3 traffic levels:

- Low: 10,000 coverages per design life
- Medium: 100,000 coverages per design life
- High: 250,000 coverages per design life

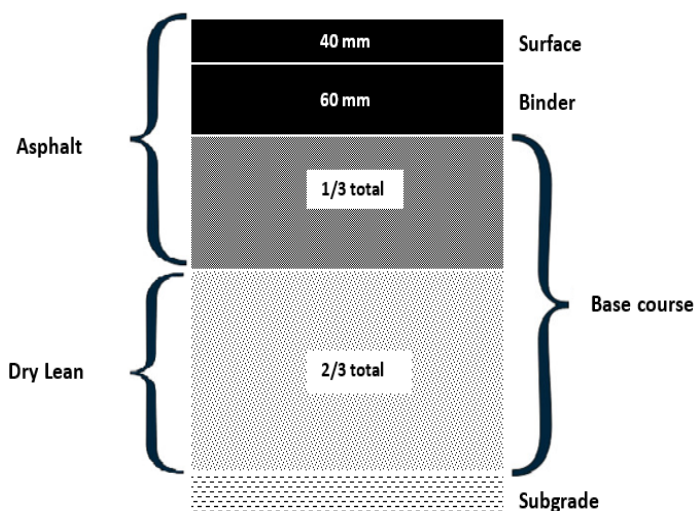
Table 1. Summary of Aircraft Classification Number for a Range of Subgrade Strengths

Aircraft	Gear type	Subgrades			
		15% CBR	10% CBR	6% CBR	3% CBR
Bae 146-100	Dual	16.3	18.0	20.2	23.9
A320-200	Dual	40.5	42.4	46.8	52.6
A380-800	Dual Tandem	58.4	63.7	75.3	105.5
A318	Dual	32.7	33.8	37.4	43.1
A340-300	Dual Tandem	62.3	65.6	74.2	89.2
A340-500	Dual Tandem	75.3	82.2	97.8	129.8
B767-300	Dual Tandem	39.5	43.3	51.1	69.9
B777-300	Tridem	52.5	58.6	72.3	100.1

Note: CBR denotes subgrade California Bearing Ratio

Coverages represent the number of times a particular point in the pavement is expected to receive a maximum stress as a result of a given number of aircraft passes. The relationship between passes and coverages depends on several factors including the type of aircraft and its given by the Pass-to –Coverage Ratio (Defence Estates 2011).

Chart 7 of DMG27 for flexible pavement design using conventional hot mix asphalt concrete base is used. A typical pavement structure is shown in Figure 1. The surface and binder course have standard thicknesses of 40 and 60 mm respectively; the base course thickness varies depending on traffic and subgrade conditions.

**Figure 1.** Airfield flexible pavement structure

The base thickness derived from chart 7 is commonly divided into 1/3 of asphalt concrete and 2/3 of Dry Lean concrete (DLC) for economic reasons, with a minimum of 120 mm of asphalt base to avoid reflective cracking (Defence Estates 2011).

2.2 Proposed analytical method design

The first step for analysing the stresses and strains in a pavement structure with Kenlayer is to define the type and properties of material that is going to be modelled. Most analytical models used for the design of flexible pavements are based on linear elastic theories which assume that each layer is homogenous, linear elastic with a constant modulus of elasticity, isotropic and that the surface layer is free from shearing and normal stresses (Ojum 2015). For these reasons the type of material selected in the software was assumed to be ‘linear’.

The load inputs are: gear type, contact tyre radius, tyre pressure and wheel coordinates. The above parameters together with the critical locations of pavement response of subgrade strains for each aircraft are presented in Table 2. To determine the design pavement structure, the strain at the top of the subgrade was used to calculate the pavement life based on the equation used by BAA (1993).

The stiffness moduli underpinning BAA (1993) were used for conventional asphalt (1380MPa) and for DLC (1207MPa). These are design stiffnesses that account for the pavement design life; therefore, their values are lower than the obtained in the laboratory. These stiffnesses have been calibrated based on experience and account for the long term durability. The analytical pavement design thickness results using Kenlayer and BAA stiffnesses/performance model are plotted in Table 2 against the DMG27 derived thicknesses. The results (Figure 2) show on average slightly more conservative thicknesses calculated using the analytical approach. Nevertheless, the match is considered acceptably good and it can be concluded that the proposed analytical approach is calibrated against a well-established empirical design method, namely DMG27. Therefore, the input parameters used are appropriate for airfield pavement analysis.

Table 2. Load input parameters and locations of pavement subgrade response

Aircraft Used	Gear Type	AUM (kg) MTOW	Load on One Main Gear Leg (%)	Tyre Press- ure (MPa)	Tyre Pressure (kPa)	No. of wheels in each main gear	Contact Tyre Radius (cm)	Wheel Coordinates (m)												Critical Locations to Check (m)											
								1		2		3		4		5		6		1		2		3		4		5		6	
								X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
Bae 146-100	Dual	37308	46	0.965	965	2	16.66	0	0	0	0.66									0	0	0	0.33								
A320-200	Dual	77400	46.6	1.44	1440	2	19.78	0	0	0	0.927									0	0	0	0.4635								
Airbus A380-800	Dual Tandem	562000	19	1.5	1500	4	23.57	0	0	0	1.35	1.8	0	1.8	1.35					0	0	0	0.675	0.9	0	0.9	0.675				
A318	Dual	68400	44.5	1.02	1020	2	21.59	0	0	0	0.927									0	0	0	0.4635								
A340-300	Dual Tandem	275900	37.5	1.42	1420	4	23.85	0	0	0	1.397	1.981	0	1.981	1.397					0	0	0	0.6985	0.9905	0	0.9905	0.6985				
Airbus A340-500	Dual Tandem	369200	36.5	1.61	1610	4	25.56	0	0	0	1.4	1.98	0	1.98	1.4					0	0	0	0.7	0.99	0	0.99	0.7				
B767-300	Dual Tandem	159665	46.1	1.34	1340	4	20.71	0	0	0	1.143	1.422	0	1.422	1.143					0	0	0	0.5715	0.711	0	0.711	0.5715				
B777-300	Tridem	300280	47.4	1.48	1480	6	22.37	0	0	0	1.397	1.45	0	1.448	1.397	2.896	0	2.896	1.397	0	0	0	0.6985	0.724	0	0.724	0.6985	1.45	0	1.45	0.6985

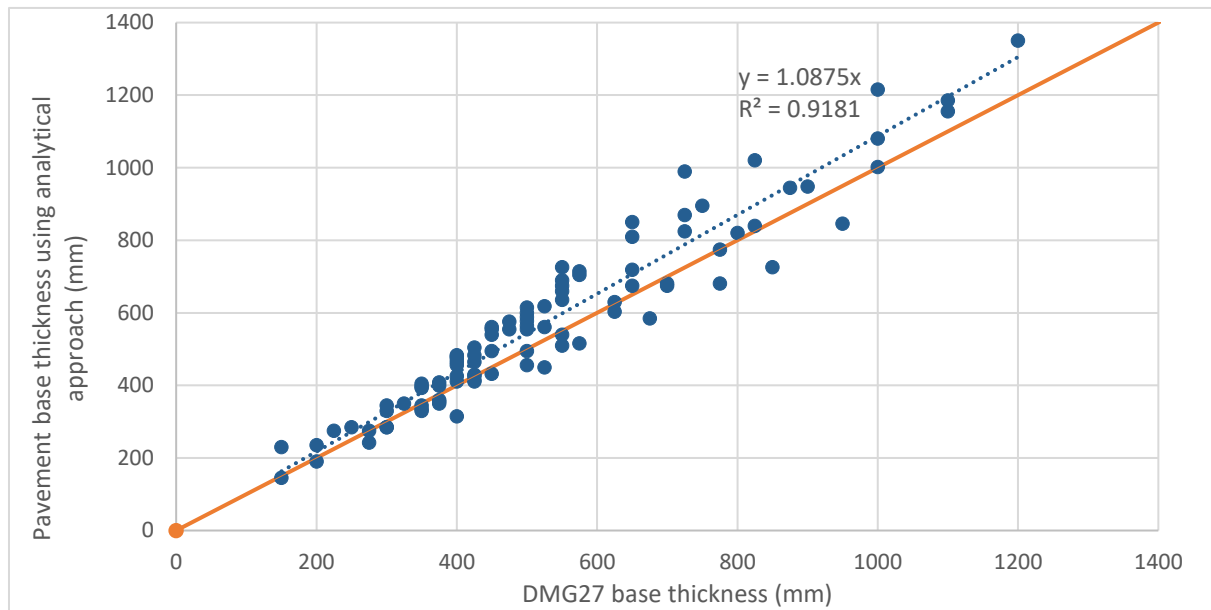


Figure 2. Comparison between DMG27 derived base pavement thickness using asphalt concrete and thicknesses calculated using the proposed analytical approach

3 PAVEMENT DESIGN WITH CRBM

Once the inputs have been established, the analysis with CRBM characteristics can be carried out. However, while the BAA asphalt stiffness has been used as the design stiffness, based on field experience, no such value is known for CRBM. Thus, two different analyses were carried out. Reference is made to laboratory data for a 50 penetration grade asphalt concrete (HDM50 (Defence Estates 2005)), commonly used in airfields, and for a particular CRBM incorporating 82.6% of Reclaimed Asphalt Pavement (RAP), 1.6% cement, 6.3% fly ash and 3% foamed bitumen and with a total water content of 6.5% (Lacalle Jiménez et al. 2016) (stiffness modulus obtained to BS EN 12697-26:2004 Annex C (British Standard, 2012)).

For design purposes, the 15th percentile of the laboratory stiffness values is selected as a conservative approach (Table 3) and a thickness of 150mm for DLC is fixed to analyse the effect of the asphalt in the base.

Table 3. Statistical results for stiffness moduli at 20°C (MPa)

	CRBM	HDM50
Average	3925	5330
Min	3353	5020
Max	4854	5890
Standard deviation	368	213
15th percentile	3567	5117
Stiffness used in Kenlayer	3500	5100

In the first analysis it was assumed that the CRBM deteriorates in the same way as HMA; therefore, the design stiffness of 947MPa for CRBM is calculated from the laboratory stiffness of 3500MPa using the HMA laboratory to design stiffness ratio of 0.27 (see Table 4). The design stiffnesses for CRBM and HMA are used in the analytical design approach and pavement base thicknesses are compared with the DMG27 derived thicknesses in Figure 3.

Table 4. Design stiffnesses

	HDM50	CRBM
Lab. Stiffness (MPa)	5100	3500
Design stiffness (MPa)	1380	947
Ratio	0.27	0.27

In this case it can be appreciated how the average difference between analyses is 5.6% (Figure 4), having a maximum difference between points of 10.5%. With these results it was concluded that DMG27, Chart 7, can be used for designing airfield pavements using CRBM increasing the base thickness a 10.5%, as a conservative approach.

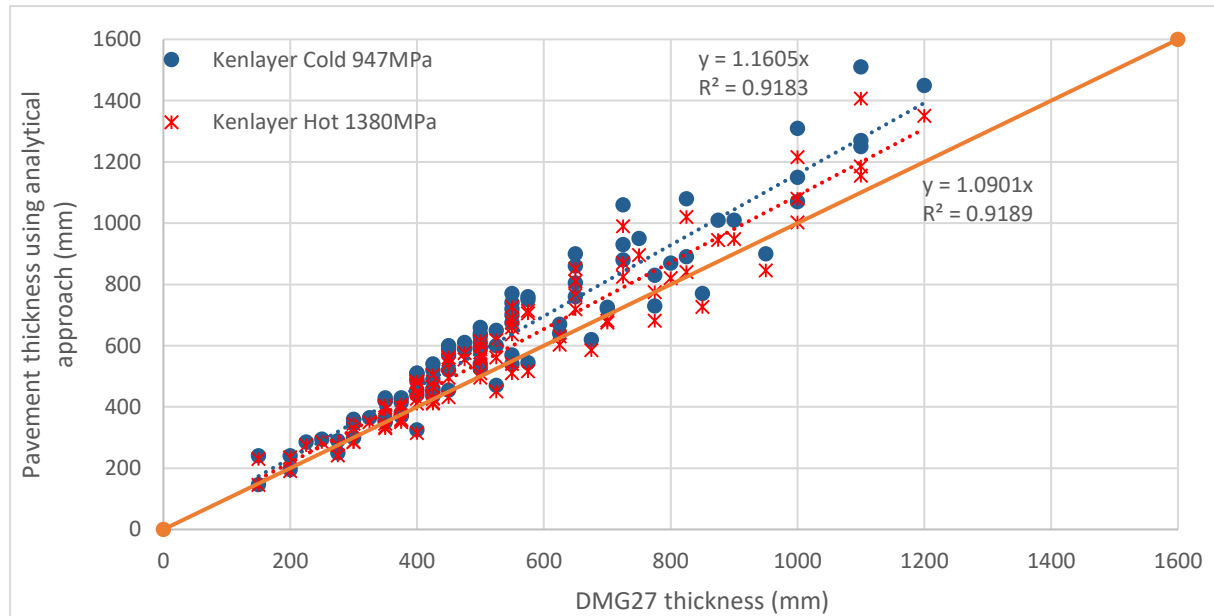


Figure 3. Comparison between DMG27 derived base pavement thicknesses and analytical approach using hot and cold asphalt design stiffnesses

It can be appreciated that the base thickness using CRBM is 7% thicker in average than with HMA. Analysing point by point, the maximum difference is found to be 8.5%.

The second analysis was carried out using the laboratory stiffnesses of HDM50 and CRBM in the analytical approach (Figure 4).

Here the comparison against DMG27 is made for calibrating purposes, not to use Kenlayer for pavement design; therefore, the variability ($R^2=0.89$) is judged as appropriate.

In Table 5 an example of how to establish the base thickness when using CRBM is presented.

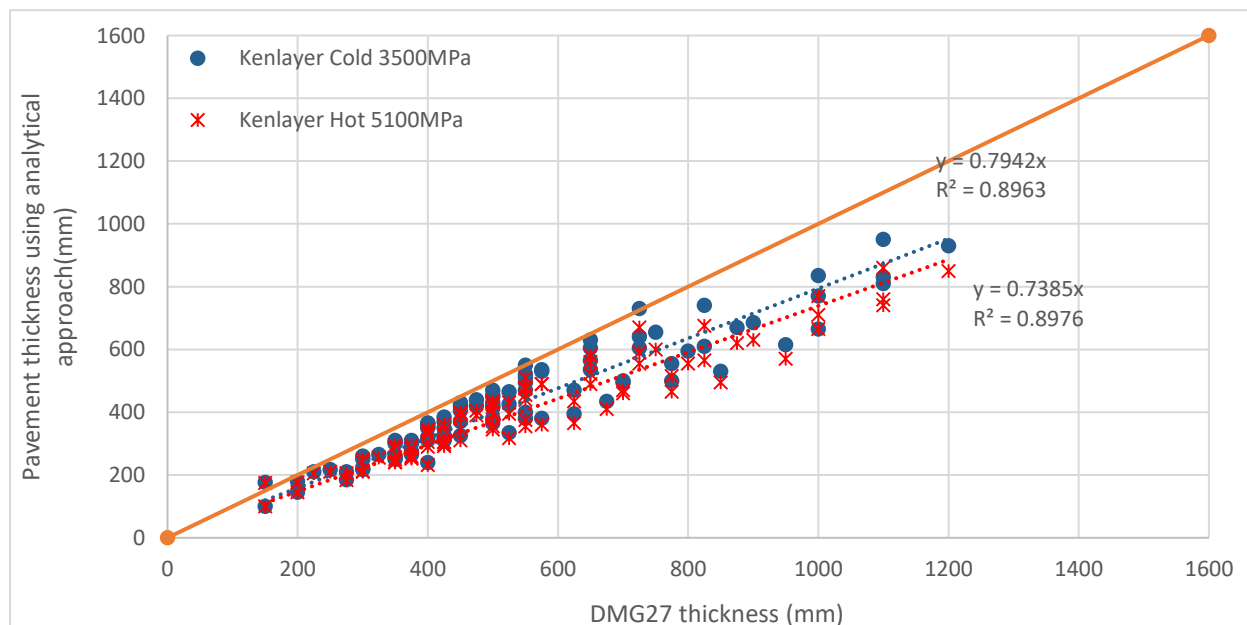


Figure 4. Comparison between DMG27 derived base pavement thicknesses and analytical approach using laboratory stiffnesses

Table 5. Proposed design thickness for CRBM

Base thickness with Chart 7	Increase of 10.5%	Final base thickness using CRBM
200	221	230
300	332	340
400	442	450
500	553	560
600	663	670
700	774	780
800	884	890

In addition to this analysis, in a previous study (Lacalle Jiménez & Tuck 2015) it was concluded that this material has the potential to be used in airfield pavements, having good moisture sensitivity and long term characteristics (after 360 days). However, the study has been carried out analysing CRBM as a HMA in terms of fatigue and load distribution. Therefore, further study is needed to understand if the assumptions made are appropriate and analyse how CRBM behave under cyclic loading in a realistic scenario. Also long term behaviour should be studied in more depth to ensure good pavement design.

4 SUMMARY AND CONCLUSIONS

In this paper, an airfield pavement thickness design approach using Cold Recycled Bound Material (CRBM) is proposed. An analytical approach incorporating Kenlayer multi-layered model and BAA design stiffness and performance model is calibrated against a well-established design method, DMG27, for a wide range of pavement and traffic scenarios.

As BAA is based on design materials properties, based mostly in field experience, and there is no available information in this regard for CRBM, two analyses were carried out with Kenlayer. The first was assuming that CRBM deteriorate in the same way as HMA, and therefore, the CRBM design stiffness is calculated based on laboratory stiffness with the same HMA laboratory to design stiffness ratio. The second study was carried out using laboratory stiffnesses for both materials in the analytical method.

This CRBM mixture has been characterised in previous studies giving good performance; however, some assumptions were made for this study; therefore, future work is needed to establish a trustworthy design guidance. This future works should focus on failure criteria and long term performance of CRBM.

The conclusions can be summarised as:

- Using CRBM in airfield base course is viable according to mechanical properties and long term performance.
- Design guidance is needed due to the different behaviour compared to HMA.
- Analytical analysis suggests to increase the asphalt base thickness, obtained from Chart 7 of DMG27 by 10.5% compared with HMA when using CRBM.

5 ACKNOWLEDGMENTS

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APPENDIX B

Example of pavement calculation for medium traffic, with a critical plane ACN of 90 with Dual gear and a subgrade of 10%. The pavement must have 625 Bound Base Material (BBM); therefore the whole structure comprises 40 mm surface course (HMA), 60 mm binder course (HMA), 208 base course (HMA) and 417mm base (DLC).

