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FACTORS AFFECTING STRENGTH, STIFFNESS AND FATIGUE BEHAVIOUR OF COLD RECYCLED CEMENT-TREATED MIXTURES

William Fedrigo

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To my dearest family

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Emancipate yourselves from mental slavery
None but ourselves can free our minds
Robert Nesta Marley

ABSTRACT

FEDRIGO, W. Factors affecting strength, stiffness and fatigue behaviour of cold recycled cement-treated mixtures. 2019. Doctoral Thesis – Post-graduate Programme of Civil Engineering: Construction and Infrastructure, UFRGS, Porto Alegre.

Full-depth reclamation with Portland cement (FDR-PC) is a technique that rehabilitates and enhances the performance of a distressed pavement by introducing a cold recycled cement-treated material (CRCTM) layer. It has been used in Brazil for many years, especially due to technical, environmental and economic advantages. However, the lack of standards has been inhibiting a larger diffusion of FDR-PC and resulting in the adoption of different design criteria, sometimes leading to low efficiency. This thesis, which consists of four papers, focuses on factors affecting strength, stiffness and fatigue behaviour of CRCTM. The first paper provides a comprehensive literature review on FDR-PC. The second paper evaluates the effects of reclaimed asphalt pavement (RAP) residual binder on the mechanical behaviour of CRCTM. The study comprised the laboratory production of RAP samples with different binder types, contents and ageing conditions, as well as the production and testing of CRCTM (laboratory-produced RAP, graded crushed stone and cement). Results showed that the indirect tensile strength of CRCTM is mainly affected by the RAP binder type, while their resilient modulus is significantly affected by RAP binder type, content and ageing. The third paper evaluates the flexural static and cyclic behaviour of cement-treated mixtures of RAP and lateritic soil (LS). Results showed that higher cement contents lead to stronger and stiffer mixtures. Besides, flexural strength, strain at break and resilient modulus of such mixtures increase with RAP percentage. The study also resulted in fatigue relationships for such mixtures. Mechanistic analyses showed that the fatigue life of base layers made of such mixtures increases with the RAP percentage and with the thickness of the layers (surface and base); the effect of cement content depends on the thickness of the layers. Finally, the fourth paper focuses at (a) analysing the effects of compaction method, test setup and displacement rate on the flexural behaviour of lightly cement stabilised materials (LCSM), (b) characterising the fatigue behaviour of South African LCSM, and (c) comparing them with Brazilian materials. Results showed similarities when using vibratory and compression compactions. Moreover, the South African test setup results in lower strength and stiffness, while the displacement rate does not have a significant effect. Besides, the strain at break is not affected by testing characteristics. The study also resulted in laboratory fatigue relationships for South African LCSM, which lead to fatigue lives shorter than the South African transfer functions. Furthermore, strain-based relationships provide a better fatigue life prediction than stress-based relationships. The study also identified similarities between some South African LCSM and Brazilian CRCTM. Following are the most meaningful findings of this thesis: a) RAP asphalt binder affects the mechanical behaviour of CRCTM and hence the mix and structural design; b) it is possible to recycle pavements with LS base layers overlaid by thick asphalt layers without compromising mechanical and fatigue behaviour of the recycled layer; c) laboratory fatigue relationships of South African LCSM and cement-treated mixtures of RAP and LS reported for the first time; and d) strain at break is preferred for pavement design since it is not affected by testing characteristics.

Keywords: *pavement; full-depth reclamation with cement; cold recycling; mechanical behaviour, fatigue*.

RESUMO

FEDRIGO, W. Fatores que afetam a resistência, a rigidez e a fadiga de misturas recicladas a frio com cimento. 2019. Tese (Doutorado em Engenharia) – Programa de Pós-Graduação em Engenharia Civil: Construção e Infraestrutura, UFRGS, Porto Alegre.

A reciclagem profunda com cimento Portland é uma técnica usada para recuperação de pavimentos que vem sendo utilizada no Brasil há vários anos, por vantagens técnicas, econômicas e ambientais. Contudo, uma maior utilização é limitada pela escassez de normas, resultando na adoção de diferentes critérios, que, por vezes, levam a uma baixa eficiência. Essa pesquisa teve o objetivo de estudar fatores que afetam o comportamento mecânico e de fadiga de misturas recicladas com cimento. A pesquisa foi dividida em quatro partes, apresentadas no formato de artigos. O primeiro artigo apresenta uma revisão bibliográfica sobre a técnica. No segundo artigo, são avaliados os efeitos do ligante asfáltico do fresado no comportamento mecânico de misturas recicladas com cimento. Para tal, amostras de fresado com diferentes tipos, teores e condições de envelhecimento de ligante asfáltico foram produzidas. Então, misturas recicladas (fresado de laboratório, brita graduada e cimento) foram preparadas e ensaiadas. Os resultados mostraram que a resistência à tração de tais misturas é afetada pelo tipo do ligante do fresado, já seu módulo de resiliência é afetado pelo tipo, teor e envelhecimento do ligante do fresado. O terceiro artigo apresenta o comportamento na flexão estático e cíclico de misturas de fresado, solo laterítico e cimento. Verificou-se que maiores teores de cimento levam a misturas mais resistentes e rígidas. Além disso, a resistência à tração na flexão, a deformação na ruptura e o módulo de resiliência das misturas aumenta com a porcentagem de fresado. Modelos de fadiga foram obtidos e usados em análises mecanísticas, mostrando que a vida de fadiga de bases de tais misturas aumenta com a porcentagem de fresado e com as espessuras do revestimento e da base; o efeito do cimento depende da espessura das camadas. O terceiro artigo foca em (a) analisar o efeito do método de compactação, da configuração de ensaio e da taxa de carregamento no comportamento na flexão de materiais cimentados, (b) caracterizar o comportamento de fadiga de materiais cimentados sul-africanos e (c) comparar materiais sul-africanos e brasileiros. Verificaram-se resultados semelhantes usando-se compactação vibratória ou por compressão. A configuração de ensaio sul-africana resulta em menor resistência e rigidez, já a taxa de carregamento não tem efeito significativo. Além disso, a deformação na ruptura não é afetada pelas características do ensaio. Foram obtidos modelos de fadiga de laboratório para materiais sul-africanos; as vidas de fadiga obtidas são inferiores às dos modelos de campo sul-africanos. Ainda, os modelos baseados na deformação levam a uma melhor previsão da vida de fadiga do que os baseados na tensão. Por fim, foram observadas semelhanças entre materiais sul-africanos e brasileiros. As principais contribuições dessa tese foram: a) o ligante asfáltico do fresado influencia o comportamento da mistura reciclada com cimento e, assim, sua dosagem e o dimensionamento do pavimento; b) pavimentos com base de solo laterítico e revestimentos espessos podem ser reciclados com cimento sem prejuízo no comportamento mecânico ou de fadiga; c) foram obtidos modelos de fadiga de laboratório até então não existentes; e d) a deformação na ruptura parece ser a melhor propriedade para dimensionamento, já que as características do ensaio não a afetam.

Palavras-chave: *pavimento; reciclagem profunda com cimento; reciclagem a frio, comportamento mecânico; fadiga*.

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LIST OF ABBREVIATIONS, ACRONYMS AND SYMBOLS

4PBT: Four-Point Bending Test

AASHTO: American Association of State Highway and Transportation Officials

ABNT: Associação Brasileira de Normas Técnicas (Brazilian Association of Technical Standards)

AM: AASHTO Modified effort

ANOVA: Analysis of Variance

APT: Accelerated Pavement Testing

ASTM: American Society for Testing and Materials

Austroads: Association of Australian and New Zealand Road Transport and Traffic Authorities

BI: Brazilian Intermediate effort

CAPES: Coordenação de Aperfeiçoamento de Pessoal de Nível Superior (Brazilian Coordination for the Improvement of Higher Education Personnel)

CBR: California Bearing Ratio

CDM: Compressive Dynamic/complex Modulus

CEM: Compressive Elastic Modulus

CIMBÉTON: Centre d'information sur le ciment et ses applications

CNPq: Conselho Nacional de Desenvolvimento Científico e Tecnológico (Brazilian National Council for Scientific and Technological Development)

CRCTM: Cold Recycled Cement-Treated Mixture

CSIR: Council for Scientific and Industrial Research (South Africa)

CSRA: Committee of State Road Authorities (South Africa)

CTCS: Cement-Treated Crushed Stone

D: Diameter of specimen

DEINFRA-SC: Departamento Estadual de Infraestrutura de Santa Catarina (Santa Catarina State Department of Infrastructure)

DER-PR: Departamento de Estradas de Rodagem do Estado do Paraná (Highway State Department of Paraná)

DER-SP: Departamento de Estradas de Rodagem do Estado de São Paulo (Highway State Department of São Paulo)

DNIT: Departamento Nacional de Infraestrutura de Transportes (Brazilian National Department of Transport Infrastructure)

DTEM: Direct Tensile Elastic Modulus

DTS: Direct Tensile Strength

FDR-PC: Full-Depth Reclamation with Portland Cement

FEM: Flexural Elastic Modulus

FHWA: Federal Highway Administration

FRM: Flexural Resilient Modulus

FS: Flexural Strength

FSM: Flexural Static Modulus

FTS: Flexural Tensile Strength

$FT\varepsilon_b$: Flexural Tensile strain at break

FWD: Falling Weight Deflectometer

GC: Gyratory Compactor

GCS: Graded Crushed Stone

GM: Grading Modulus

h: Height of specimen

H: Horizontal displacement of specimen @ ITS test

HRB: Highway Research Board

IECA: Instituto Español del Cemento y sus Aplicaciones

ICS: Initial Consumption of Stabiliser

ITMr: Indirect Tensile Resilient Modulus

ITRM: Indirect Tensile Resilient Modulus

ITS: Indirect Tensile Strength

JCI: Japan Concrete Institute

L: Length between supporting rollers @ flexural test

LAGEOTEC: Laboratório de Geotecnologia (Geotechnology Laboratory) – UFRGS

LAPAV: Laboratório de Pavimentação (Pavements Laboratory) – UFRGS

LCSM: Lightly Cement Stabilised Materials

LEME: Laboratório de Ensaios e Modelos Estruturais (Structural Models and Testing Laboratory) – UFRGS

LG: Lateritic Gravel

LL: Liquid Limit

LS: Lateritic Soil

LVDT: Linear Variable Differential Transducer

LWD: Light Weight Deflectometer

MBV: Methylene Blue Value

MDD: Maximum Dry Density

MDUW: Maximum Dry Unit Weight

MeDiNa: Método de Dimensionamento Nacional (Brazilian National Design Method)

MEPDG: Mechanistic-Empirical Pavement Design Guide

M_{FWD}: modulus back-calculated using FWD data

M_i: Initial flexural resilient modulus

M_{LWD}: modulus back-calculated using LWD data

MOR: Modulus Of Rupture

M_T: Total mass of specimen

N: Fatigue life, Load cycles, Number of cycles, Number of load repetitions

NCHRP: National Cooperative Highway Research Program

NDEM: Elastic modulus (non-destructive methods)

OMC: Optimum Moisture Content

P: Peak load @ ITS test

P_C: Percentage by mass of cement

PCA: Portland Cement Association

P_{GCS}: Percentage by mass of GCS

P_i: Applied load @ flexural test

PI: Plasticity Index

PPGCI: Programa de Pós-Graduação em Engenharia Civil: Construção e Infraestrutura (Post-graduate Programme of Civil Engineering: Construction and Infrastructure) – UFRGS

P_{RAP}: Percentage by mass of RAP

R^2: Coefficient of determination

RAP: Reclaimed Asphalt Pavement

RILEM: International Union of Laboratories and Experts in Construction Materials, Systems and Structures

RL: Reliability Level

RTFOT: Rolling Thin Film Oven Test

SAMDM: South African Mechanistic Design Method

SANRAL: South African National Roads Agency SOC Limited

SBS: Styrene-Butadiene-Styrene

SD: Standard Deviation

SEE: Standard Error of Estimate

SEM: Scanning Electron Microscopy

SL: Stress Level

SP: Standard Proctor effort

TF: Transfer Function

TRH: Technical Recommendations for Highways (South Africa)

TRM: Triaxial Resilient Modulus

UCS: Unconfined Compressive Strength

UFRGS: Universidade Federal do Rio Grande do Sul (Federal University of Rio Grande do Sul)

UP: University of Pretoria

VC: Vibratory Compactor

V_m: Voids in the mixture

V_T: Total volume of specimen

W: Moisture content

w: Width of specimen

WSDOT: Washington State Department of Transportation

δ_i: Mid-span deflection/displacement @ flexural test

ε_b: Flexural tensile strain at break

ε_i: Initial flexural tensile strain

ν: Poisson's ratio

ρ_b: Wet density

ρ_C: Particle density of cement

ρ_d: Dry density

ρ_{GCS}: Particle density of GCS

ρ_m: Maximum density

ρ_{RAP}: Particle density of RAP

σ_i: Flexural tensile stress

1 INTRODUCTION

The economic development experienced by Brazil in the decade of 2005-2014 significantly increased highway usage. The number of commercial vehicles increased over the years, as well as their total gross weight and axle loads. As a consequence, pavement design results in increasingly thicker asphalt layers. Twenty years ago, a 200 mm thick granular base layer overlaid by a 100 mm thick asphalt layer was considered a thick pavement. However, in recent times, high volume traffic pavements are designed with at least 200 mm thick asphalt layers to avoid premature fatigue failure.

Although thick asphalt layers increase the pavement fatigue resistance, they increase shear failure risk as well, which may lead to rutting and top-down cracking. Increasing asphalt layers thickness also yield in higher construction costs. Moreover, this is not considered a sustainable practice, due not only to the high consumption of natural resources and energy but also to the high emission volumes.

Pavement rehabilitation shows a similar scenario. For instance, typical techniques like mill and overlay sometimes show low efficiency and have environmental impacts, mainly because of the generation of reclaimed asphalt pavement (RAP). However, pavements must be designed, constructed and rehabilitated to last. Public opinion and governmental agencies increasingly demand durable pavements that can keep their function of providing safe and comfortable traffic.

Countries with advanced pavement technology (e.g. Western European countries, the USA, Australia and South Africa) frequently use treated layers in pavement construction and rehabilitation. Almost all French pavements have some treated material layer; sometimes even the subgrade. South Africa, which is a country that has a similar economy and wheater as Brazil, commonly uses stabilised pavement bases or subbases. That is, countries are choosing semi-rigid pavements instead of typical flexible pavements.

The usage of cement-treated materials is limited in Brazil, but it is increasing because full-depth reclamation with Portland cement (FDR-PC) is becoming a popular pavement rehabilitation technique. The technique gained importance on the 1980s due to the development of high technology equipment, the knowledge on cement stabilised materials and the increasing environmental concern. Brazil has been using FDR-PC since the 1990s, but the lack of national standards and procedures sometimes inhibits its usage. For instance, there is no structural design procedure, resulting in the adoption of different design criteria, which sometimes leads to low efficiency.

Cold recycled cement-treated mixtures (CRCTM) obtained using FDR-PC differ from conventional cement-treated materials because of the presence of RAP. RAP consists aggregates coated by asphalt binder and agglomerations of fines and asphalt binder. The amount of RAP in the mixture depends on the old asphalt layer thickness and will significantly affect the properties of the recycled layer. Although literature states that the increase of RAP in the mixture decreases its strength and stiffness, the effects of the RAP residual asphalt binder on the behaviour of cold recycled mixtures remain not well known, affecting mix and structural designs. For instance, the RAP effect may change depending on its asphalt binder type, content and ageing.

Fatigue is generally the main design criteria for the long-term performance of cold recycled cement-treated mixtures, but just a few researchers reported studies on this subject. There were only studies on the fatigue behaviour of cold recycled mixtures made of RAP, graded crushed stone base and cement. Nonetheless, other types of materials are commonly used in base layers and may end up being a constituent of cold recycled mixtures. For instance, lateritic soils are often used as pavement materials in some regions of tropical countries (e.g. Brazil), due to their excellent behaviour after compaction and to the lack of high-quality granular materials.

The four-point bending test is often used to characterise the behaviour and to obtain design parameters of cement stabilised materials (with or without RAP). The test generates stress states similar to those in stabilised layers; development of tensile and compressive stresses at the bottom and top of the beam, respectively. Researchers have extensively investigated the flexural behaviour of cement stabilised materials. However, there is no universal test method, leading to different results of mechanical and fatigue behaviour, and hence affecting pavement design.

The abovementioned facts state that there are still doubts concerning mechanical and fatigue behaviour of cold recycled cement-treated mixtures. Therefore, it is necessary to study the behaviour of these materials aiming at filling these knowledge gaps; this may result in procedures that are simple and at the same time qualified in a scientific point of view, which would increase the correct usage of FDR-PC technique.

The research reported in this thesis is a product of a comprehensive joint university/industry research project aiming at developing standards and procedures for FDR-PC, which focus mainly on the definition/adaptation of a rational structural design method for pavements with cold recycled cement-treated layers. A group of researchers of the Pavements Laboratory (Laboratório de Pavimentação – LAPAV), Federal University of Rio Grande do Sul (Universidade Federal do the Rio Grande do Sul – UFRGS), was responsible for the research, in partnership with two other laboratories of the same university, the Geotechnology Laboratory (Laboratório de Geotecnologia – LAGEOTEC) and the Structural Models and Testing Laboratory (Laboratório de Ensaios e Modelos Estruturais – LEME). The last paper presented in this thesis (Chapter 5) is the result of a research developed in the University of Pretoria (UP), South Africa, as part of a cooperation agreement between UFRGS and UP.

1.1 OBJECTIVES

1.1.1 General objective

The general objective of this research was to study strength, stiffness and fatigue behaviour of cold recycled cement-treated mixtures.

1.1.2 Specific objectives

The following were the specific objectives of this research:

a) To evaluate the effects of asphalt binder present in reclaimed asphalt pavement (type, content and ageing) on the mechanical behaviour of cement-treated mixtures of reclaimed asphalt pavement and graded crushed stone.

b) To determine flexural tensile strength, flexural tensile strain at break and flexural modulus (static and cyclic) of cement-treated mixtures of reclaimed asphalt pavement and lateritic soil.

c) To obtain laboratory fatigue relationships of cement-treated mixtures of reclaimed asphalt pavement and lateritic soil.

d) To evaluate the effects of flexural static test characteristics (compaction method, test setup and displacement/loading rate) on the flexural behaviour of lightly cement stabilised materials, following Brazilian and South African procedures.

e) To obtain laboratory fatigue relationships of South African lightly cement stabilised materials.

f) To compare the flexural behaviour of Brazilian cold recycled cement-treated mixtures and South African lightly cement stabilised materials.

1.2 THESIS STRUCTURE

The present doctoral thesis consists of six chapters. This section summarises the content of each chapter.

Chapter 1 presents the introduction, which explains the relevance and objectives of the research.

Chapters 2–5 consist of four papers. The thesis author is the first and corresponding author of all papers. Although these studies have had collaborators, the first author was the only involved in all parts, including testing, analysis and writing (draft, review and editing).

Chapter 2 presents a literature review article on full-depth reclamation of pavements with Portland cement, which produces cold recycled cement-treated mixtures. It focuses on the following topics: history, construction steps, mix design, structural design, and behaviour, especially the mechanical one.

Chapter 3 presents a published article in which are evaluated the effects of asphalt binder present in reclaimed asphalt pavement (type, content and ageing) on the mechanical behaviour of cold recycled cement-treated mixtures (reclaimed asphalt pavement, graded crushed stone and cement).

Chapter 4 presents a published article in which are described the effects of cement content and reclaimed asphalt pavement percentage on the flexural static and cyclic behaviour of cold recycled cement-treated mixtures (reclaimed asphalt pavement, lateritic soil and cement). This chapter also presents laboratory fatigue relationships for such mixtures for the first time.

Chapter 5 presents a published article in which are reported the effects of flexural static test characteristics (compaction method, test setup and testing displacement/loading rate) on the flexural behaviour of lightly cement stabilised materials (LCSM). This chapter also compares Brazilian and South African test methods and materials. Furthermore, it presents laboratory fatigue relationships for South African LCSM for the first time, as well as a comparison of such relationships with the traditional South African transfer functions for cement stabilised materials.

The last chapter (Chapter 6) summarises the conclusions of each paper. It also presents the most meaningful contributions to the knowledge and recommendations for further research.

Appendix A1 presents the comments made by the examiners and the responses by the author.

2 FULL-DEPTH RECLAMATION OF PAVEMENTS WITH PORTLAND CEMENT: A LITERATURE REVIEW

Abstract:

This paper presents a literature review on full-depth reclamation of pavements with Portland cement (FDR-PC). The paper consists of the following topics: history, construction steps, advantages and disadvantages, mix design, structural design, and behaviour in laboratory and field. All sections make comparisons between international and Brazilian experiences. The paper highlights that FDR-PC is used worldwide with several benefits. While mix design methods are well established internationally, structural design methods still need to be adapted in most countries. The paper presents ranges of strength and stiffness based on previous research. Analysis of previous research also identified an adequate test for field modulus prediction and general effects of some characteristics on the behaviour of FDR-PC materials. Furthermore, it identified knowledge gaps for future research, which could promote FDR-PC application.

Keywords: pavement cold recycling; full-depth reclamation with cement; reclaimed asphalt pavement; mix design; structural design; mechanical behaviour

1 Introduction

Industry development increases the efforts on the expansion/enhancement of road transportation systems, which includes rehabilitating existing pavements. Recently, alternative pavement rehabilitation technologies started gaining importance because of their environmental-friend aspects. Among others, cold recycling is an example of such technologies (Xiao et al., 2018).

Full-depth reclamation/recycling with Portland cement (FDR-PC) is a cold technique that can treat most distresses of old pavements. FDR-PC consists of in situ pulverising the existing asphalt layer and blending it with a predetermined amount of underlying base material while stabilising these materials with cement to, after compaction, produce a new base layer (Federal Highway Administration, FHWA, 1997; Portland Cement Association, PCA, 2005a).

Due to the presence of reclaimed asphalt pavement (RAP), materials produced using FDR-PC (i.e. cold recycled cement-treated mixtures) may have different behaviour as that of conventional cement stabilised materials. Therefore, standards and methods (e.g. mix and structural design) need to be adapted or even developed to cover such materials.

In this regard, many researchers have studied the complex behaviour of FDR-PC materials, using laboratory and field testing. However, despite FDR-PC technical, environmental and economic benefits, the road industry still considers it a non-traditional rehabilitation technique and further effort are necessary to make it a standard choice.

Xiao et al. (2018) reported a comprehensive literature review on cold recycling technology of asphalt pavements. However, the authors mainly focused on recycling using asphalt stabilisers. To the date, there is no literature review specifically on FDR-PC, hence promoting the application of the technique or serving as a guide for future research.

Taking this into account, this paper aimed at collecting findings and data from previous research and identifying gaps for further research. The paper consists of six sections. Firstly, it focuses on the history of FDR-PC, briefly describes its construction steps and shows its advantages and disadvantages. Then, the paper presents mix and structural design methods for FDR-PC. All these sections present comparisons between international and Brazilian experiences. At last, the paper shows a comprehensive review of the laboratory and field behaviour of materials produced using FDR-PC. The last section analyses and compares over 1800 testing data collected from 55 studies.

2 History

This section presents a brief history of the FDR-PC technique, divided into international and Brazilian experiences.

2.1 International experience

Jasienski & Rens (2001) reported that the usage of pavement recycling with cement addition started on the fifties; the USA and France were the pioneers.

In Malaysia, the first project using FDR-PC occurred in 1985 through the recycling of 15 km of pavement in Pahang state. Since then, FDR-PC has become widely accepted in the country, being one of the main solutions for pavement rehabilitation (Sufian et al., 2009a).

In 1989, Belgium introduced the technique by recycling a pavement area of 6000 square meters in the city of Vaux-sur-Sûre. Between 1989 and 2001, around 300,000 square meters of pavements were recovered in the country using the technique (Jasienski & Rens, 2001).

FDR-PC was introduced in South Africa in 1991 when 23 km of a federal highway was recycled. Currently, there are several recycling machines in that country, and thousands of km of pavements were recycled, especially using cement as stabilising binder (Collings, 2001).

According to Vorobieff & Wilmot (2001), the first recycling machines arrived in Australia in 1992. Portland cement is the most used binder in recycling works in that country, due to its ability to stabilise most pavement materials, low price and availability.

The technique became widespread in Spain after the country's first experience in 1992. Studies estimate that more than 2.5 million square meters of pavements are recycled annually (Instituto Español del Cemento y sus Aplicaciones, IECA, 2013).

Table 1 presents a compilation of locations where FDR-PC was employed internationally. It also provides brief information on the reported works. The table only considers cases that provided info on construction year and used technique, and it does not include test sections.

Table 1

2.2 Brazilian experience

Brazil has been using FDR-PC for about three decades. According to Paiva & Oliveira (2009), the technique has already recovered millions of square meters of pavements in the country.

For instance, the state of São Paulo frequently uses the technique, which has rehabilitated thousands of kilometres of pavements in that state. Authors mention several highways in which the technique was successfully used (SP-294, SP-272, SP-300, SP-304, SP-264, SP-141, SP-079, and SP-563) (Oliveira, 2003; Paiva & Oliveira, 2010; Paiva et al., 2013). Besides, there are reports of FDR-PC usage in other Brazilian regions, especially in the state of Minas Gerais (highways BR-040, BR-459 and BR-135) (Gusmão, 2008; Oliveira, 2003; Paiva & Oliveira, 2009).

Table 2 provides a brief description of some FDR-PC works in Brazil. The table only presents cases that provided info on construction year and used technique, not including test sections.

Table 2

3 Construction

The four main steps of FDR-PC are cement spreading, pulverisation, compaction, and application of a surface layer (FHWA, 1997). Since most documents on FDR-PC are construction specifications, presenting a detailed description of the process is not one of the main objectives of this paper. However, this section briefly describes each of the mentioned steps based on the USA experience, but construction is similar in any part of the world.

Cement is generally spread in a controlled manner by spreader trucks specifically designed for this matter (PCA, 2005a). However, it is possible to perform this operation manually as well. If it is necessary to correct the grain size distribution, it is also possible to spread additional aggregates (FHWA, 1997).

Pulverisation usually occurs to a depth of 100 mm to 300 mm, but modern reclaimers can pulverise to depths of 450 mm. The same reclaimer hooked to a water truck also mixes the existing materials with cement/virgin aggregates while injecting the proper amount of water into the mixing chamber (FHWA, 1997; PCA, 2005a).

Compaction generally happens using heavy smooth-wheeled vibrating rollers and padfoot vibrating rollers operating in high-amplitude vibration mode (Xiao et al., 2018). After compaction, using a proper curing method is necessary to achieve the required strength and inhibit shrinkage cracks.

Once the recycled layer is stable, it is possible to apply a surface layer of any kind, completing the process and providing a new pavement structure (PCA, 2005a). During the process, other typical road construction operations also occur (e.g. grading).

The existing Brazilian standards on FDR-PC are also construction specifications developed by the National Department of Transport Infrastructure (DNIT), by the Santa Catarina State Department of Infrastructure (DEINFRA-SC), and by the Highway State Departments of Paraná (DER-PR) and São Paulo (DER-SP). These specifications are as follow:

- DER-PR ES-P 33/05 – Pavements: In situ pavement recycling with cement addition (DER-PR, 2005);
- DER-SP ET-DE-P00/035 – In situ asphalt pavement recycling with cement and crushed stone (DER-SP, 2006);
- DNIT 167/2013-ES – Pavements: Full-depth reclamation with Portland cement – Construction specification (DNIT, 2013);
- DEINFRA-SC ES-P 09/16 – Pavements: Full-depth reclamation (DEINFRA-SC, 2016).

As mentioned, these standards only present best-practices for FDR-PC construction; they do not provide any information on the mix or structural design. Table 3 presents a comparison between them. The table shows that the standards suggest the same curing method and mainly use the strength and the degree of compaction to assure quality control; those developed by DER-PR and DER-SP also suggest ranges for field moisture content control.

Table 3

Furthermore, Table 3 shows how different can be the standards of a country, which becomes evident when comparing the most recently published standards (DNIT and DEINFRA-SC) with those published a decade earlier (DER-PR and DER-SP). The main difference concerns the required grain size distribution. Figure 1 shows that there is no agreement between the standards, excepting for those developed by DER-PR and DER-SP.

Figure 1

4 Advantages and disadvantages

4.1 Advantages

This section presents a summary of technical, economic and environmental advantages of FDR-PC based on the literature (IECA, 2013; Minguela, 2011; PCA, 2005a; PCA, 2005b; Wirtgen, 2012).

4.1.1 Technical

- Allows rehabilitating a distressed pavement or upgrading a weak pavement structure, due to the generation of a stabilised layer which will be homogeneous, stable and thicker, providing better mechanical characteristics.
- Reduces the compressive stress at the subgrade and the tensile stress at the asphalt wearing course, due to the inclusion of a stabilised layer.
- Provides a moisture- and frost-resistant stabilised layer.
- Recycles and improves existing materials which most of the times do not show adequate technical characteristics anymore.
- Provides a homogenisation of the pavement strength and geometry.
- Allows rehabilitation under traffic since traffic is usually allowed on one side of the road while construction occurs on the other.
- Keeps road elevation, avoiding problems with curb/gutter and overhead clearances.
- Generates minimal disturbance due to construction traffic because of the fast construction cycle and low material volume transported in or out.
- Allows performing improvements in road geometry simultaneously with its pavement rehabilitation.
- Portland cement acceptance and availability. The material is well-known by the construction industry.

4.1.2 Economic

- Reduces costs with new material, as well as with its production and transportation since it reuses existing pavement materials.
- Allows a quick return of local traffic and avoids detours, which may reduce user costs.
- One of the least-cost alternatives for rehabilitating a distressed pavement, especially in comparison to thick structural overlay or removal and replacement. It is usually 25–50% cheaper than the latter.
- Portland cement is usually cheaper than asphalt cement.
- Most of the mentioned technical advantages result in economic benefits.

4.1.3 Environmental

- Conserves natural resources by recycling existing materials and avoids the disposal of materials in landfills, especially in comparison to the construction of a new structure (Table 4).
- Reduces energy consumption since it is a cold recycling technique (performed at ambient temperature), especially in comparison to the construction of a new structure (Table 4).
- Decreases the carbon dioxide emission and the impact in the adjacent area (erosion, dust, etc) due to the reduced transport needed.
- Causes minimal environmental impacts since it is an in situ technique, avoiding plant installation problems (vegetation removal, drainage change, etc).

Table 4

Complementing the advantages reported above, Table 5 presents a comparison between FDR-PC and other rehabilitation techniques (structural overlay and removal and replacement).

Table 5

4.2 Disadvantages

Some disadvantages and limitations of FDR-PC are as follows:

- The recycling of a non-uniform structure, regarding both materials and thicknesses, may result in a heterogeneous recycled mixture (Minguela, 2011).
- FDR-PC is a single lane strategy, requiring adequate precautions to avoid longitudinal cracks (Minguela, 2011).
- Cement stabilised materials inherently shrink and, as a consequence, some shrinkage cracks may reflect through the asphalt wearing course. Adequate curing methods and crack minimisation strategies can help inhibiting such problems (PCA, 2005a; Wirtgen, 2012).
- FDR-PC may generate a brittle material, which may result in the fracture of the layer, reflecting cracks in the pavement surface. Thicker layers with less strength (low cement content) are preferred to avoid such brittleness (PCA, 2005a; Wirtgen, 2012).
- Although agency costs are generally lower than for other rehabilitation techniques (including recycling with foamed asphalt or asphalt emulsion), FDR-PC might become more expensive when considering both agency and user costs (Braham, 2016).

5 Mix design

This section presents different mix design methods for FDR-PC employed internationally and in Brazil. It also presents comparisons and analyses regarding such methods.

5.1 International experience

Table 6 summarises the main characteristics of some internationally known mix design methods. The table presents an Australasian (Austroads), an American (PCA) and three European methods (IECA, Wirtgen and the French method). Austroads method is a guide for in situ stabilisation in general (not specifically FDR-PC) including virgin and recycled materials.

Table 6

Since the test is simple and cheap, 7-day unconfined compressive strength is the main design parameter in most of the methods (excepting the French one, which suggests 28- or 360-day direct tensile strength). There is no consensus in the level of strength to be achieved, but there is an agreement that the compaction should follow AASHTO (American Association of State Highway and Transportation Officials) modified effort (only PCA does not suggest this effort).

Furthermore, in addition to strength tests, all methods suggest tests to characterise other important properties (e.g. durability).

Figure 2 presents the grain size distribution envelopes suggested by some of these methods. The figure shows that the envelopes suggested by Wirtgen and Austroads are similar, although the first is wider than the latter. PCA uses only three sieves for grain size distribution control, but it limits the RAP content by 50% unless approved by the engineer and included in an adequate mixture design.

Figure 2

In general, PCA and Wirtgen methods seem the most detailed, since both covers all considered characteristics except one (typical cement content and maximum RAP content, respectively). On the other hand, the French method is the one that most diverges from the others, especially because of the suggested tests (e.g. methylene blue value and direct tensile strength). Designing a mix using the latter also depends on traffic and qualities of recycling equipment and recycled mixture.

5.2 Brazilian experience

There is no consensus on a mix design method for FDR-PC in Brazil. Although the standards mentioned in Section 3 present technical characteristics that the recycled layer should follow (Table 3), there is no info on the mix design. Some of them (DER-PR, 2005; DNIT, 2013) only mention the importance of an adequate mix design, covering possible material and thickness variations along the length of the road.

In the absence of such a method, it is common to use standards developed for soil-cement mix design in FDR-PC works (Oliveira, 2003). As a consequence, some authors started focusing on this subject (Fedrigo et al., 2017a; Gusmão, 2008). For instance, Fedrigo et al. (2017a) adapted the Austroads method to Brazilian pavement recycled materials. Their work included different cement and RAP contents and four base materials of existing pavements (graded crushed stone, soil-cement, cement-treated crushed stone and lateritic soil), resulting in a non-official draft standard.

6 Structural design

There are few studies on the structural design of FDR-PC layers in the literature, and the common practice is using empirical methods. In this regard, authors have determined layer structural coefficients of such materials based on the AASHTO guide for design of pavement structures (1993), obtaining similar values to those of conventional cement-treated materials (Taha et al., 2002; Mallick et al., 2002a; Puppala et al., 2011).

Several authors reported studies on rational design parameters (e.g. resilient modulus) or even on the fatigue behaviour of FDR-PC materials (Luvizão, 2014; Minguela, 2011). However, those aiming at developing/adapting mechanistic-empirical structural design methods for FDR-PC started being reported only in the last few years (Amarh et al., 2017; Castañeda López et al., 2018; Jones et al., 2015; Wu et al., 2015). The following sections present details on some mechanistic-empirical pavement design methods including cement stabilised materials.

6.1 International experience

Table 7 summarises the main characteristics of some internationally known structural design methods. The table presents methods from the USA (AASHTO Mechanistic-Empirical Pavement Design Guide, MEPDG), South Africa (South African Mechanistic Design Method, SAMDM), Australia and New Zealand (Austroads Pavement Design Guide), and France (French design method). It also presents a new Brazilian method (discussed in Section 6.2). The characteristics shown in Table 7 are those concerning cement stabilised layers, but the documents cited in the table provide the complete pavement design procedures (i.e. materials, traffic, climate, etc).

Table 7

The methods follow similar theoretical bases and are usually available in packages containing software and manuals. None of them includes recycled materials, only conventional cement stabilised materials. The French method is the exception, but it states that the behaviour of FDR-PC materials is similar to that of conventional cement stabilised materials, and so is its structural design, which is not exactly true.

Table 7 shows that fatigue is the main structural design criterion, even though the methods from South Africa and the USA also evaluate a second failure mode related to the compressive stress at the top of the layer. Another agreement is using flexural tests to characterise the material properties for design; once again, the only exception is the French method. Testing the materials is always preferred, but using estimated/typical property values is also possible.

Although theoretically similar, each method has its particularity. For instance, South African and Australian methods incorporate the evaluation of the stabilised layer in a post-cracked condition, considering properties and failure modes of a granular layer. On the other hand, the methods from the USA and France present models for predicting the durability of the layer. Moreover, the French method presents structural design catalogues containing recycled layers, developed using its mechanistic-empirical approach (CIMBÉTON, 2013). Spain also uses the same practice (IECA, 2013; Minguela, 2011).

6.2 Brazilian experience

Brazilian researchers have been working on a mechanistic-empirical method for decades, releasing its first version in 2007 (Franco, 2007). The current version, known as MeDiNa (National Design Method), comes with a software package (Franco & Motta, 2018). After a testing phase, it will become the method of the Brazilian National Department of Transport Infrastructure (DNIT). Table 7 presents some of its characteristics.

The Brazilian method also considers fatigue as the main failure mode for conventional cement stabilised materials. As with most international methods, it also does not take into account recycled materials. Furthermore, although it states that testing is preferable, the method presents data on properties of Brazilian cement stabilised materials (Balbo, 1993; Ceratti, 1991). There are also typical properties of roller-compacted concrete (Trichês, 1993).

However, the method has deficiencies even regarding conventional cement stabilised materials. It uses fatigue models based only on laboratory results without any field calibration (no transfer function or even laboratory to field shift factor), thus showing the importance of advancing in the knowledge of such materials, including FDR-PC.

7 Behaviour

This section presents characteristics of the laboratory and field behaviour of mixtures produced using FDR-PC.

7.1 Laboratory

Based on previous studies, Tables 8 and 9 present ranges of strength of FDR-PC materials. Similarly, Tables 10 and 11 present ranges of elastic modulus and cyclic modulus, respectively. These tables consider only mixtures containing RAP and specific software allowed obtaining values presented in graphs by some researchers.

Table 8

Table 9

Table 10

Table 11

Figure 3 shows box and whisker plots for all values of strength found in the literature (Tables 8 and 9). Figure 3 production included approximately 1200 values from 43 studies. Average values of unconfined compressive strength (UCS), indirect tensile strength (ITS), flexural tensile strength (FTS) and direct tensile strength (DTS) are 4.0 MPa, 0.5 MPa, 0.9 MPa and 0.6 MPa, respectively. The values of ITS and DTS are similar. Ratios of tensile to compressive strength varies from 10% to 25%. Furthermore, at least 75% of the individual values of UCS, ITS, FTS and DTS are lower than 5.2 MPa, 0.65 MPa, 1.2 MPa and 0.7 MPa, respectively. In addition, flexural tensile strain at break values varies from 100 με to 1200 με for FDR-PC (Castañeda López et al., 2018; D'avila et al., 2017).

Figure 3

Figure 4 shows box and whisker plots for the values of elastic modulus (Figure 4a) and cyclic modulus (Figure 4b) found in the literature (Tables 10 and 11). The figures also present field modulus values (discussed in Section 7.2). Figure 4 production included approximately 700 values from 42 studies. The average values of elastic modulus under compressive (CEM), flexural (FEM) and direct tensile (DTEM) conditions are similar (5500–5900 MPa). However, the average elastic modulus obtained using non-destructive methods (e.g. ultrasonic pulse velocity test), NDEM, is higher (16,500 MPa). Besides, more than 75% of the individual values of CEM, FEM, DTEM and NDEM are lower than 9600 MPa, 7200 MPa, 5900 MPa and 20,600 MPa, respectively.

Figure 4

Figure 4b shows similarities between the average resilient modulus under triaxial (TRM) and flexural (FRM) conditions (3600 MPa and 3400 MPa, respectively), which agrees with statements by Fedrigo et al. (2018b). Moreover, compressive dynamic/complex modulus (CDM) average value (5900 MPa) is similar to the average elastic modulus measured using static loading conditions (Figure 4a). However, the indirect tensile resilient modulus (ITRM) average value (9900 MPa) is higher than all modulus values measured using static or cyclic loading conditions. Furthermore, 75% of the individual values of TRM, ITRM, FRM and CDM are lower than 1900 MPa, 13,900 MPa, 4400 MPa and 7200 MPa, respectively.

The graphs in Figures 3 and 4 disregard all specific testing characteristics (e.g. cement and RAP contents, curing time, testing temperature, and so on). However, based on the analysed laboratory works, Table 12 shows the effect of some of these characteristics on the mechanical and durability properties of mixtures produced using FDR-PC. The following sections discuss some of these effects in detail.

Table 12

7.1.1 Reclaimed asphalt pavement effect

In general, increasing RAP contents reduce strength and stiffness of FDR-PC materials (Castañeda López et al., 2018; Dalla Rosa & Muller, 2016; D'avila et al., 2017; Dellabianca, 2004; El Euch Khay et al., 2014; Fedrigo et al., 2017b; Fedrigo et al., 2018b; Ghanizadeh et al., 2018; Grilli et al., 2013; Guthrie et al., 2007; Ji et al., 2016; Kleinert et al., 2019; Kolias, 1996a; Kolias, 1996b; Suebsuk et al., 2019; Sufian et al., 2009b; Taha et al., 2002; Yuan et al., 2011). However, some studies on FDR-PC mixtures containing lateritic soils report the opposite trend (Kleinert et al., 2017; Schreinert, 2017; Schreinert et al., 2018).

Based on the mentioned works, some reasons causing this reduction trend are: 1) RAP has agglomerations formed by fines and asphalt binder, which present air voids and are weaker than natural aggregates, resulting in higher deformations under loading; 2) residual asphalt binder in RAP reduces the surface area that could be coated by cement, inhibiting the generation of bonding points between aggregates and cement paste; and 3) this residual asphalt binder also affects the shape of the aggregate, which becomes rounded, reducing interlocking. Studies using scanning electron microscopy (SEM) confirm some of these facts (Ji et al., 2016).

RAP addition also turns the cement stabilised mixture time- and temperature-dependent. However, this viscoelastic behaviour is not as strong as for asphalt mixtures or even cold recycled mixtures with asphalt binders (Adresi et al., 2019; Amarh et al., 2017; Godenzoni et al., 2018; Graziani et al., 2019; Grilli et al., 2013; Kolias, 1996b; Wu et al., 2015). A possible explanation is that cement, which is not temperature sensitive, governs the behaviour of FDR-PC mixtures, inhibiting the thermal sensitivity of the residual asphalt binder in RAP (Isola et al., 2013).

RAP is generally a heterogeneous material, presenting, for instance, different residual asphalt binder contents. However, Yuan et al. (2011) state that RAP asphalt binder content does not have a strong effect on the strength of FDR-PC mixtures, even for asphalt binder contents as high as 8%.

Some authors state that high RAP contents reduce the fatigue life of FDR-PC mixtures (Jiang et al., 2018; Paiva et al., 2017) and that their behaviour is intermediate between those of conventional cement stabilised mixtures and asphalt mixtures (Kolias et al., 2001). However, studies also report that RAP content effect on the fatigue behaviour of such mixtures is more complex, depending on the cement content and on the thickness of the layers (Castañeda López et al., 2018).

Because of the mentioned effects, some authors argue that it is necessary adding virgin aggregates to FDR-PC mixtures. It increases the aggregate surface area to be coated by cement and the interlocking, resulting in higher strength and stiffness (Ji et al., 2016; Jiang et al., 2018; Ma et al., 2015).

7.1.2 Portland cement effect

Increasing cement contents increase the strength and stiffness of FDR-PC mixtures; the higher the cement content, the higher the hydration reactions, generating bonding points between aggregates (Castañeda López et al., 2018; Dalla Rosa & Muller, 2016; D'avila et al., 2017; Fedrigo et al., 2017b; Fedrigo et al., 2018b; Ghanizadeh et al., 2018; Guthrie et al., 2007; Ji et al., 2016; Kleinert et al., 2017; Kleinert et al., 2019; Paiva & Oliveira, 2013; Puppala et al., 2011; Schreinert et al., 2018; Taha et al., 2002; Trichês et al., 2013; Yuan et al., 2011). Cement increase also reduces porosity, which results in less moisture sensitivity and higher durability (Fedrigo et al., 2017b; Guthrie et al., 2007; Kleinert, 2016; Yuan et al., 2011).

Although high cement contents increase mechanical and durability properties, its usage is often limited to a minimum, due to shrinkage problems (Fedrigo et al., 2017b). FDR-PC works make use of cement in the form of powder or slurry. The latter leads to slightly weaker mixtures; hence, in this case, the cement content must be increased to achieve the desired strength (Dixon et al., 2012).

The type of Portland cement also affects the strength of FDR-PC mixtures (Paiva & Oliveira, 2009). Paiva & Oliveira (2009) recommend using cement types with fewer clinker and gypsum, which reduces the heat of hydration and setting time, and, consequently, shrinkage effects. In this regard, Portland composite cement types are the most indicated (Paiva & Oliveira, 2009; World Road Association, WRA, 2003).

7.1.3 Compaction effect

The compaction effort causes similar effects as the cement content; that is, when increased, it reduces the mixture porosity and increases its strength and stiffness. (Ely, 2014; Fedrigo et al., 2017b; Fedrigo et al., 2018a). Aranha (2013) states that this effect is stronger on compressive behaviour than on tensile behaviour.

Therefore, using a higher compaction effort and achieving an adequate degree of compaction may counterbalance using lower cement contents, reducing shrinkage effects and costs (Fedrigo et al., 2017b; Paiva & Oliveira, 2010). It is also necessary to avoid compaction delays since it can reduce the strength of the FDR-PC material (Oliveira, 2003).

7.1.4 Curing effect

Like any cementitious material, FDR-PC mixtures become stronger and stiffer while curing (Fedrigo et al., 2018b; Ghanizadeh et al., 2018; Gusmão, 2008; Ji et al., 2016; Kleinert et al., 2017; Kolias, 1996b; Taha et al., 2002; Trichês et al., 2013; Wilson & Guthrie, 2011). Besides, their moisture resistance also increases with age (Fedrigo et al., 2017b; Kleinert, 2016).

In the field, traffic release usually happens a few hours after recycling (with adequate surface protection applied). However, some recommend longer curing times before traffic release. Laboratory studies show that shrinkage effects are stronger in the early ages (El Euch Khay et al., 2014; Fedrigo et al., 2017b; Kleinert et al., 2019), which can result in cracks, compromising the quality of the layer. Thus, proper curing is necessary to avoid such problems; this can be achieved by continuous water spraying or applying a sealing compound or membrane (PCA, 2005a).

7.1.5 Existing base material effect

Since FDR-PC often incorporates the existing base layer, the base material affects the behaviour of the recycled layer. Generally, an FDR-PC mixture containing a coarser base material (e.g. graded crushed stone or gravel) will present higher strength and stiffness (Castañeda López et al., 2018; Fedrigo et al., 2017b; Fedrigo et al., 2018b; Katsakou & Kolias, 2007; Kleinert, 2016; Kleinert et al., 2017; Kleinert et al., 2019; Schreinert et al., 2018). However, some authors verified similar behaviour between FDR-PC mixtures with gravel or sand base materials (Ghanizadeh et al., 2018), while others reported that higher fines content led to higher strength (Yuan et al., 2011).

FDR-PC mixtures with higher fines content also tend to be highly sensitive to water and shrinkage effects, then presenting less durability (Fedrigo et al., 2017b; Kleinert, 2016; Kleinert et al., 2019). This behaviour was observed for conventional cement stabilised materials as well (Chakrabarti & Kodikara, 2003; Chakrabarti & Kodikara, 2005; Lee et al., 2004).

7.2 Field

In comparison to laboratory studies, field studies are still limited, but they show that FDR-PC not only rehabilitates a distressed pavement but also enhances its performance; the deflection values reduce and become more homogeneous along the length of the road (Lewis et al., 2006; Oliveira et al., 2005; Silva & Miranda Jr, 2000; Trichês & Santos, 2011). Besides, FDR-PC pavements generally show lower deflections than pavements recycled with other cold technologies (Jones et al., 2015; Wu et al., 2015).

After recycling, deflections keep reducing since the FDR-PC layer stiffness keeps increasing due to its curing (Amarh et al., 2017; Aranha, 2013; Godenzoni et al., 2018; Isola et al., 2013; Tataranni et al., 2018; Trichês & Santos, 2013; Wilson & Guthrie, 2011; Wu et al., 2015). However, under traffic loading, deflections tend to increase, which is a result of FRD-PC layer stiffness decreasing due to the breakdown of cement bonds and resultant microcracking. Once microcracks evolve to macrocracks, the layer fails due to fatigue (Jones et al., 2015; Wu et al., 2015).

FDR-PC layers achieve higher initial (post-construction) and residual (post-traffic) moduli than layers recycled without any or with asphalt stabilisers (e.g. asphalt emulsion or foamed asphalt) (Jones et al., 2015; Wu et al., 2015). They also show a higher rate of modulus decrease, which is more pronounced in the wheelpaths (Jones et al., 2015; Wilson & Guthrie, 2011; Wu et al., 2015). However, FDR-PC pavements can outperform pavements recycled with asphalt stabilisers, attaining a longer life (Amarh et al., 2017).

Water-related and shrinkage effects may accelerate the deterioration of FDR-PC layers. Adequate mix design can reduce the first, while techniques such as induced micro-cracking, construction joints or even using geosynthetics have been very successful in minimising shrinkage cracks (CIMBÉTON, 2013; IECA, 2013; PCA, 2005a; Rens, 2003; Trichês & Santos, 2011; Wilson & Guthrie, 2011).

The level of other defects (e.g. irregularity and permanent deformation) in FDR-PC pavements are generally low, especially in comparison with pavements recycled without or with different stabilisers (Jones et al., 2015; Lewis et al., 2006; Wu et al., 2015). Laboratory studies also confirm that FDR-PC materials are resistant against permanent deformation (Mallick et al., 2002b; Romeo et al., 2019). However, cement stabilised materials may still undergo permanent deformation due to crushing at the top of the layer (compressive fatigue-erosion), especially when overlaid by thin surface layers (De Beer, 1985; De Beer, 1990; Litwinowicz & De Beer, 2013; National Cooperative Highway Research Program, NCHRP, 2014). Proper structural design can avoid such a problem.

As in the laboratory, field studies show that FDR-PC layers have little sensitivity to temperature, especially in comparison to asphalt mixtures. This fact corroborates with the hypothesis that the addition of cement not only improves the strength but also reduces the temperature sensitivity of the residual asphalt binder in RAP (Amarh et al., 2017; Godenzoni et al., 2018; Wu et al., 2015).

Table 13 shows the ranges of back-calculated modulus of FDR-PC layers, obtained by some authors using field data (falling weight deflectometer, FWD, or light weight deflectometer, LWD). Table 13 data comes from test sections of in service pavements or laboratory test tracks subjected to accelerated pavement testing (APT). Specific software allowed obtaining values presented in graphs by some researchers. Figure 4 (Section 7.1) shows the average value and the standard deviation (SD) of all back-calculated moduli obtained by the authors while comparing them with the moduli obtained using laboratory tests.

Table 13

Figure 4 shows that indirect tensile cyclic tests and non-destructive methods overestimate the modulus of FDR-PC layers. Furthermore, 75% of the modulus obtained using other laboratory tests fall within the range of average field modulus plus or minus SD; the average field and laboratory values are also close. However, while most tests can still overestimate field modulus (sometimes by more than twice), the flexural test with cyclic loading is the only that leads to a similar range of modulus as back-calculation. The latter supports statements by Fedrigo et al. (2018b) and the preference for flexural tests of most structural design methods.

8 Conclusions and recommendations

The following are conclusions based on the data analysed in this paper:

- There are reports of FDR-PC usage all over the world. However, despite the well-known advantages of the technique, the lack of standards or even the divergences between them might be limiting FDR-PC further application in some countries (e.g. Brazil).
- Internationally, there are several mix design methods for FDR-PC materials. The analysed methods are similar, excepting for the French one. Brazilian documents still do not provide such a method.
- While most countries use structural design methods developed for conventional cement stabilised materials, the French one also considers FDR-PC mixtures (up to 20% RAP). All analysed methods consider fatigue as the primary failure mode of such materials and suggest using flexural tests (excepting the French one). The Brazilian method is based only on laboratory testing.
- The majority of reported research on FDR-PC are laboratory studies on the mechanical behaviour of such materials; only a few are field studies or focus on durability properties. The paper presents ranges of strength and stiffness based on several studies. According to these data, cyclic load flexural tests lead to a better prediction of field

modulus, while indirect tensile tests (one of the most used worldwide) overestimate field modulus values. Furthermore, the paper tabulates the effects of some characteristics on the behaviour of FDR-PC materials, emphasising that there are doubts regarding some effects while others are still unknown.

The following are recommendations for further research:

- To evaluate economic and environmental issues of FDR-PC using life-cycle cost analysis and life cycle assessment, respectively. Such studies (only one study reported in the literature) could show the advantages of FDR-PC, helping to make the technique a standard choice.
- To perform more field studies to help in the development/calibration of mechanistic-empirical structural design methods. Even though it is harder than in the laboratory, varying cement and RAP contents in test sections could help to understand their effect on the behaviour of FDR-PC materials. Accelerated pavement testing (APT) could be useful to this matter (only one study reported in the literature).
- To further study the fatigue behaviour of FDR-PC materials, since there are still doubts, especially regarding RAP effect.
- To further evaluate the viscoelastic behaviour of FDR-PC materials. Although such materials have little sensitivity to temperature and frequency, this might change for different ranges of cement and RAP.
- To further investigate the durability of FDR-PC, since there are only a few studies on this matter and several effects remain unknown (e.g. RAP content and existing base material).
- To determine the flexural resilient modulus of different FDR-PC materials, since such test properly predicts field modulus. Although most studies focus on the mechanical behaviour of FDR-PC, only a few made use of such tests.
- To study the effect of the asphalt binder present in RAP on the behaviour of FDR-PC materials, since much remains unknown.

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Year	Location	Characteristics	Source
1985	Pahang, Malaysia	15-km-long section	Sufian et al. (2009a)
1989	Pahang, Malaysia	55-km-long section	Sufian et al. (2009a)
1990	Belgium	Area of 10,000 m^2	Jasienski & Rens (2001)
1991	National Route N2, South Africa	23-km-long section	Collings (2001)
1992	Znojmo, Czech Republic	300-mm-thick layer	Stehlik et al. (2001)
1993	Belgium	Area of 50,000 m^2	Jasienski & Rens (2001)
1994	Valladolid, Spain	-	Minguela (2011)
1995	Acedera, Spain	220-mm-thick layer	Minguela (2011)
1995	Amarillo, Texas, USA	260-mm-thick layer	FHWA (1997)
1995	Ruta de la Plata, Spain	36-km-long section	Segarra (2001)
1996	Mariembourg, Belgium	Area of 16,300 m^2	Jasienski & Rens (2001)
1997	Ávila, Spain	4.5- km-long section	Minguela (2011)
1998	Saint-Ghislain, Belgium	Area of 33,600 m^2	Jasienski & Rens (2001)
1999	Spain	Area of 1,220,000 m^2	Minguela (2011)
2000	Spain	Area of 1,547,000 m^2	Minguela (2011)
2000	KwaZulu-Natal, South Africa	60-km-long section	Paige-Green & Ware (2006)
2001	Belgium	Area of 7000 m^2	Jasienski & Rens (2001)
2002–04	Malaysia	90-km-long section	Sufian et al. (2009a)
2004	Long County, Georgia, USA	1.8-km-long section	Lewis et al. (2006)
2007	Laramie County, Wyoming, USA	First use in the region	PCA (2012)
2008	Powhatan and Goochland, Virginia, USA	-	Amarh et al. (2017)
2009	Richland County, Montana, USA	-	PCA (2012)
2009	Wyoming, USA	230-mm-thick layer	Wilson & Guthrie (2011)
2012	Hennepin, Minnesota, USA	-	Ghasemi et al. (2018)

Table 1. Brief compilation of the international experience using FDR-PC

Year	Location	Characteristics	Source
1998	Highway BR-381, São Paulo–Belo Horizonte	Volume of 30,000 m^3	Silva & Miranda Jr (2000)
2000	Highway SP-352, São Paulo	-	Paiva et al. (2013)
2004	Highway SP-351, Bebedouro–Palmares Paulista, São Paulo	22-km-long section	Oliveira et al. (2005)
2007	Highway SC-150 (BR-282), Santa Catarina	30-km-long section	Trichês & Santos (2013)
2011	Highway BR-381, São Paulo–Belo Horizonte	-	Aranha (2013); Bessa et al. (2016)
2012	Highway GO-222, Anápolis–Nerópolis, Goiás	8-km-long section	Santos et al. (2017)
2013	Highway SC-463, Santa Catarina	23-km-long section	Luvizão (2014)
2016	Highway SC-453, Tangará–Luzerna, Santa Catarina	35-km-long section	Fedrigo et al. (2018a)

Table 2. Brief compilation of the Brazilian experience using FDR-PC

Characteristic	Brazilian standard			
	DER-PR (2005)	DER-SP (2006)	DNIT (2013)	DEINFRA-SC (2016)
Reclaimer minimum milling depth (mm)	-	120	300	300
Maximum cement content (%)	-	-	-	3
Maximum RAP content (%)	-	-	50	50
Compaction effort	Brazilian intermediate[1]	Brazilian intermediate	AASHTO modified	AASHTO modified
Minimum degree of compaction (%)	100	100	98	100 (top) and 98 (bottom)
Field moisture content (%)	OMC [-1; +1]	OMC [-2; +1]	-	-
7-day UCS (MPa)	3.5–8.0	-	2.1–2.5	> 2.1
7-day ITS (MPa)	-	-	0.25–0.35	> 0.25
Curing method	Asphalt prime-coat	Asphalt prime-coat	Asphalt prime-coat	Asphalt prime-coat
Traffic release	After 7 days of curing	After surface treatment and adequate strength	After surface treatment; 3–7 days to verify possible deficiencies	After surface treatment

RAP: Reclaimed Asphalt Pavement; AASHTO: American Association of State Highway and Transportation Officials; OMC: Optimum Moisture Content; UCS: Unconfined Compressive Strength; ITS: Indirect Tensile Strength; [1]Approximately half the effort of AASHTO modified

Table 3. Comparison between Brazilian standards

Characteristic	FDR-PC base	New base
Number of trucks	12	180
New materials (t)	300	4500
Material landfilled (m^3)	0	2100
Fuel consumed (L)	1900	11,400

Based on 1.6 km of 7.3-m-wide 2-lane road, 150-mm base

Table 4. Comparison of energy use and materials for FDR-PC base and new base (PCA, 2005b)

Benefit	Rehabilitation technique		
	FDR-PC	**Structural overlay**	**Removal and replacement**
New structure	X	X	X
Fast construction	X	X	
Minimal traffic disruption	X		
Minimal haulage	X		
Conserve resources	X		
Maintain existing elevation	X		X
Low cost	X		

Table 5. Benefits of FDR-PC in comparison to other rehabilitation techniques (PCA, 2012)

Characteristic	PCA (2005a)	IECA (2013)	Wirtgen (2012)	Austroads (2002)	French method (CIMBÉTON, 2013; Abdo et al., 2013)
Country	USA	Spain	Germany	Australia and New Zealand	France
Existing material characterisation	Sieve analysis	Sieve analysis and Atterberg limits	Sieve analysis and Atterberg limits	Sieve analysis and Atterberg limits	Sieve analysis and MBV
Grain size distribution envelope	Figure 2	-	Figure 2	Figure 2	European Standard EN 13285
Maximum RAP content (%)	50	-	-	-	Usually between 5 and 20
Compaction effort	Standard Proctor	AASHTO modified	AASHTO modified	Standard Proctor or AASHTO modified	AASHTO modified
Main design parameter	UCS	UCS	UCS	UCS	Direct tensile strength and stiffness (or estimated from ITS)
Specimen dimensions (mm)	101.6 (diameter) × 116.4 (height)	-	150 (diameter) × 127 (height)	-	-
Typical cement content (%)	-	≥ 4	2–4	1–5.5	4–6
Minimum number of cement contents	3	-	3	-	-
Minimal number of specimens	2	3	2	-	-
Curing type	Moisture room	Moisture room	Moisture room (or accelerated in oven)	Moisture room (or accelerated in oven)	Moisture room
Curing time (days)	7	7	7	28	360 (or estimated from 28-day results)
Strength (MPa)	2.1–2.8	> 2.5 MPa (depending on traffic and subgrade)	< 4: Lightly cemented; 4–10: Cemented	< 1: Modified; 1–4: Lightly bound; > 4: Heavily bound	≥ 0.5
Additional tests	Moisture sensitivity (tube suction test)	Density sensibility and workability	ITS	Capillary rise, swell, drying shrinkage and erodibility	Fatigue (two-point bending test)

RAP: reclaimed asphalt pavement; PCA: Portland Cement Association; IECA: Instituto Español del Cemento y sus Aplicaciones; CIMBÉTON: Centre d'information sur le ciment et ses applications; MBV: methylene blue value; AASHTO: American Association of State Highway and Transportation Officials; UCS: unconfined compressive strength; ITS: indirect tensile strength

Table 6. International FDR-PC mix design methods

Characteristic	**SAMDM (Theyse et al., 1996; SANRAL, 2014)**	**Austroads Pavement Design Guide (Austroads, 2017)**	**MEPDG (AASHTO, 2015; NCHRP, 2014)**	**French method (CIMBÉTON; 2009; CIMBÉTON, 2013; Abdo et al., 2013)**	**MeDiNa (Franco, 2007; Franco & Motta, 2018)**
Country	South Africa	Australia and New Zealand	USA	France	Brazil
Material	Conventional cement stabilised	Conventional cement stabilised	Conventional cement stabilised (typical values for recycling)	Cement stabilised or recycled	Conventional cement stabilised
Main failure mode	Fatigue (bottom) and crushing (top)	Fatigue (bottom)	Tensile fatigue (bottom) and compressive fatigue-erosion (top)	Fatigue (bottom)	Fatigue (bottom)
Transfer functions/models development	Flexural test and APT	Flexural test and APT	Flexural test, cyclic impact erosion test and field data (FWD and traffic)	Two-point bending test	Flexural test and indirect tensile test
Fatigue transfer function/model type	Strain-based	Strain-based	Stress-based	Stress-based	Stress-based
Main tests required	7-day UCS and FTS ($FT\varepsilon_b$)	28-day UCS and 90-day FTS	28-day UCS and 28-day FTS	360-day direct tensile strength	FTS and ITS
Typical UCS (MPa)	1.125–2.250	-	1.8–5.5	-	-
Typical tensile strength (MPa)	-	1.0–1.5 (flexural)	0.25–0.75 (flexural)	0.35–0.70 (at 10^6 cycles)[1]	0.78–2.27 (flexural/indirect)
Typical modulus (MPa)	1500–2000 (APT)	3000–5000 (flexural)	750–1100 (flexural)	13,000–2,0000 (direct tensile)[1]	6000–16,000 (flexural/indirect)
Typical $FT\varepsilon_b$	125–145 (flexural)	-	-	-	-
Observations	Two-phase analysis: effective fatigue and equivalent granular	FTS can be estimated using UCS; Post-cracked phase may be considered	Three levels of inputs (1: testing; 2: estimating; 3: typical values); Durability and shrinkage models	Properties can be estimated using 28-day results and ITS tests; Frost-thaw evaluation	Fatigue models based only on laboratory results

SAMDM: South African Mechanistic Design Method; SANRAL: South African National Roads Agency SOC Limited; MEPDG: Mechanistic-Empirical Pavement Design Guide; AASHTO: American Association of State Highway and Transportation Officials; NCHRP: National Cooperative Highway Research Program; CIMBÉTON: Centre d'information sur le ciment et ses applications; APT: accelerated pavement testing; FWD: falling weight deflectometer; UCS: unconfined compressive strength; FTS: flexural tensile strength; $FT\varepsilon_b$: flexural tensile strain at break; ITS: indirect tensile strength; [1]Depends on qualities of recycling equipment and recycled mixture

Table 7. International and Brazilian structural design methods

Source	Country	Existing base material	Cement content (%)	RAP content (%)	Compact. method	Curing time (days)	Test temp. (°C)	UCS (MPa)	ITS (MPa)	FTS (MPa)	DTS (MPa)
Adresi et al. (2019)	Iran	Granular	3–7	40–80	AM	7	25–50	1.20-5.80	0.20-0.95	-	-
Castañeda López (2016); Castañeda López et al. (2018)	Brazil	GCS	2–6	20–70	AM	28	24 ± 3	-	-	0.21–1.53	-
Chakravarthi et al. (2019)	India	Granular	2–6	25–100	AM	7	-	0.40–3.40	0.06–0.74	-	-
Dalla Rosa & Muller (2016)	Brazil	Granular	1–7	60–100	AM	7	-	-	-	0.06–1.66	-
Dalla Rosa et al. (2015)	Brazil	Granular	3.4	35	AM	28	-	-	0.45	-	-
D'avila (2015); D'avila et al. (2017)	Brazil	CTCS	2–6	20–70	AM	28	24 ± 3	-	-	0.44–1.34	-
Dellabianca (2004)	Brazil	LG	2	25	BI	3–28	-	0.70–1.30	-	-	-
El Euch Khay et al. (2014)	Tunisia	Granular	6	25–100	AM	7–28	-	5.85–14.7	1.04–1.55	1.08–2.38	-
Ely (2014)	Brazil	GCS	4	70	BI	3–14	-	1.29–2.08	0.19–0.28	-	-
Ely (2014)	Brazil	GCS	4	70	AM	3–14	-	2.03–2.72	0.21–0.45	-	-
Fedrigo (2015); Fedrigo et al. (2018b)	Brazil	GCS	2–4	20–50	AM	3–14	-	1.61–6.08	0.34–1.0	-	-
Fedrigo (2015); Fedrigo et al. (2018b)	Brazil	GCS	4–6	20–50	BI	3–14	-	1.61–5.78	0.29–1.11	-	-
Ghanizadeh et al. (2018)	Iran	Clayey gravel	3–6	20–60	AM	7–28	-	1.60–4.10	-	-	-
Ghanizadeh et al. (2018)	Iran	Clayey sand	3–6	20–60	AM	7–28	-	1.30–4.80	-	-	-
Gonzalo-Orden et al. (2019)	Spain	Granular	3	35	AM	90	-	3.73	0.40	0.60	-
Grilli et al. (2013)	Italy	Granular	3	50–80	GC	7	25	4.20–6.20	0.21–0.31	-	-
Gusmão (2008)	Brazil	GCS	3–5	40–60	BI	1–28	-	0.49–3.00	0.04–0.42	-	-
Guthrie et al. (2007)	USA	Granular	0.5–2	25–100	AM	7	-	0.76–4.55	-	-	-
Isola et al. (2013)	Italy	Granular	3.5–4	30–70	AM	7	-	3	1.20–1.40	-	-
Ji et al. (2016)[1]	China	CTCS	3–4	30–100	VC	7–90	-	2.50–7.70	0.25–0.77	-	-
Jiang et al. (2018)[1]	China	CTCS	3–4	30–100	VC	90	-	-	0.27–0.78	-	-

RAP: reclaimed asphalt pavement; GCS: graded crushed stone; CTCS: cement-treated crushed stone; LG: lateritic gravel; AM: AASHTO modified; BI: Brazilian intermediate; GC: gyratory compactor; VC: vibratory compactor; UCS: unconfined compressive strength; ITS: indirect tensile strength; FTS: flexural tensile strength; DTS: direct tensile strength; [1]Only mixtures without virgin aggregates

Table 8. Ranges of strength reported for FDR-PC materials (part 1)

Source	Country	Existing base material	Cement content (%)	RAP content (%)	Compact. method	Curing time (days)	Test temp. (°C)	UCS (MPa)	ITS (MPa)	FTS (MPa)	DTS (MPa)
Katsakou & Kolias (2007); Kolias et al. (2001)	Greece	Granular	3–5	25–100	VC	1–60	0–35	1.0–15.5	0.15–1.50	0.10–3.45	0.1–1.7
Kleinert (2016)	Brazil	CTCS	1–7	7–92	AM	3–14	-	1.00–5.57	0.17–1.18	-	-
Kleinert (2016); Kleinert et al. (2019)	Brazil	Soil-cement	1–7	7–92	AM	3–14	-	1.89–6.49	0.21–1.22	-	-
Kolias (1996a); Kolias (1996b)	Greece	Granular	5	25–100	VC	7–360	20	5.03–11.7	0.43–1.43	1.67–2.16	-
Ma et al. (2015)	China	-	1–3	100	VC	7	-	-	0.16–0.50	-	-
Melese et a.l (2019)	Canada	Granular	2–6	20–60	SP	7–56	-	1.50–4.20	0.43–0.48	-	-
Minguela (2011)	Spain	Granular	2.5–4.5	33	AM	7–548	-	1.31–5.17	0.19–0.50	0.45–0.69	-
Mohammadinia et al. (2014)	Australia	-	2–4	100	AM	1–28	-	2.80–5.40	-	-	-
Oliveira & Paiva (2019)	Brazil	LG	3	35	AM	3–28	-	2.14–3.48	0.29–0.55	-	-
Oliveira (2003)	Brazil	Soil-cement	3–5	30	SP	7	-	0.97–2.82	-	-	-
Oliveira (2003)	Brazil	LG	3–5	85	SP	7	-	1.33–2.17	-	-	-
Paiva & Oliveira (2009)	Brazil	LG	4	34	BI	3-28	-	1.60–4.99	-	-	-
Paiva & Oliveira (2010)	Brazil	Soil-cement	3	77	AM	7	-	0.72–2.28	0.10–0.45	-	-
Schreinert (2017)	Brazil	Lateritic soil	1–7	7–92	AM	3–28	24 ± 3	-	0.10–0.98	0.28–1.43	-
Suddeepong et al. (2018)	Thailand	GCS	3–7	20–80	AM	7–28	-	1.80–14.5	-	-	-
Suebsuk et al. (2019)	Thailand	LG	1–5	30–100	AM	7–28	-	0.10–4.90	-	-	-
Sufian et al. (2009b)	Malaysia	GCS	3	25–100	AM	1–28	-	0.40–6.03	0.14–0.55	-	-
Taha et al. (2002)	Oman	Granular	3–7	70–100	AM	3–28	-	0.80–3.60	-	-	-
Trichês et al. (2013)	Brazil	Granular	3–4.5	35	AM	3–28	25	0.97–4.84	0.21–0.71	0.24–0.82	-
Trichês et al. (2013)	Brazil	GCS	2–4	30	BI	3–28	25	0.54–2.65	0.22–0.84	-	-
Trichês et al. (2013)	Brazil	Granular	2–4	25	AM	7–28	25	1.24–2.86	0.27–0.70	-	-
Wilson & Guthrie (2011)	USA	Silty sand	4	50–70	AM	7–90	-	1.52–6.96	-	-	-
Yuan et al. (2011)	USA	Granular	2–6	50–100	SP	7	-	0.70–7.00	0.12–0.72	-	-

RAP: reclaimed asphalt pavement; GCS: graded crushed stone; CTCS: cement-treated crushed stone; LG: lateritic gravel; AM: AASHTO modified; BI: Brazilian intermediate; SP: standard Proctor; VC: vibratory compactor; UCS: unconfined compressive strength; ITS: indirect tensile strength; FTS: flexural tensile strength; DTS: direct tensile strength

Table 9. Ranges of strength reported for FDR-PC materials (part 2)

Source	Country	Existing base material	Cement content (%)	RAP content (%)	Compact. method	Curing time (days)	Test temp. (°C)	CEM (MPa)	FEM (MPa)	DTEM (MPa)	NDEM (MPa)
Castañeda López (2016); Castañeda López et al. (2018)	Brazil	GCS	2–6	20–70	AM	28	24 ± 3	-	1422–13,255	-	-
Chakravarthi et al. (2019)	India	Granular	2–6	25–100	AM	7	-	20–380	-	-	-
D'avila (2015); D'avila et al. (2017)	Brazil	CTCS	2–6	20–70	AM	28	24 ± 3	-	1800–7600	-	-
El Euch Khay et al. (2014)	Tunisia	Granular	6	25–100	AM	28	-	-	-	-	4687–15,430
Ghanizadeh et al. (2018)	Iran	Clayey gravel	3–6	20–60	AM	28	-	9.3–38.5	-	-	-
Ghanizadeh et al. (2018)	Iran	Clayey sand	3–6	20–60	AM	28	-	18.4–38	-	-	-
Grilli et al. (2013)	Italy	Granular	3	50–80	GC	28	20	-	-	-	8100–10,000
Ji et al. (2016)[1]	China	CTCS	3–4	30–100	VC	7–90	-	476–1521	-	-	-
Katsakou & Kolias (2007); Kolias et al. (2001)	Greece	Granular	3–5	25–100	VC	1–60	20	1200–18,500	3500–20,000	500–20,000	-
Kolias (1996a); Kolias (1996b)	Greece	Granular	5	25–100	VC	28–720	0.5–30	5000–24,100	-	-	15,100–31,200
Melese et al. (2019)	Canada	Granular	2–6	20–60	SP	28	-	2400–11,500	-	-	-
Minguela (2011)	Spain	Granular	2.5–4.5	33	AM	7–481	-	690–3502	-	-	-
Yuan et al. (2011)	USA	Granular	2–6	50–100	SP	7	-	-	-	-	5200–14,000

RAP: reclaimed asphalt pavement; GCS: graded crushed stone; CTCS: cement-treated crushed stone; AM: AASHTO modified; SP: standard Proctor; GC: gyratory compactor; VC: vibratory compactor; CEM: compressive elastic modulus; FEM: flexural elastic modulus; DTEM: direct tensile elastic modulus; NDEM: elastic modulus (non-destructive methods); [1]Only mixtures without virgin aggregates

Table 10. Ranges of elastic modulus reported for FDR-PC materials

Source	Country	Existing base material	Cement content (%)	RAP content (%)	Compact. method	Curing time (days)	Test temp. (°C)	Load freq. (Hz)	TRM (MPa)	ITRM (MPa)	FRM (MPa)	CDM (MPa)
Castañeda López (2016); Castañeda López et al. (2018)	Brazil	GCS	2–6	20–70	AM	28	24 ± 3	5	-	-	2913–7725	-
Dalla Rosa et al. (2015)	Brazil	Granular	3.4	35	AM	28	-	1	-	16,000	-	-
Dellabianca (2004)	Brazil	LG	2	25	BI	7	-	-	450–800	-	-	-
Ely (2014)	Brazil	GCS	4	70	BI	3–14	24 ± 3	1	-	3663–7892	-	-
Ely (2014)	Brazil	GCS	4	70	AM	3–14	24 ± 3	1	-	4865–8420	-	-
Fedrigo (2015); Fedrigo et al. (2018b)	Brazil	GCS	2–4	20–50	AM	3–14	24 ± 3	1	-	10,873–25,719	-	-
Fedrigo (2015); Fedrigo et al. (2018b)	Brazil	GCS	4–6	20–50	BI	3–14	24 ± 3	1	-	10,390–24,842	-	-
Fedrigo et al. (2018b)	Brazil	GCS	2	50	AM	3–14	24 ± 3	1	88–1412	-	-	-
Godenzoni et al. (2018)	Italy	Granular	3	33	Not specified	2520	0–50	0.1–20	-	-	-	3752–9390
Graziani et al. (2019)	Italy	Granular	3	33	Not specified	2520	0–50	0.1–20	-	-	-	3300–7500
Grilli et al. (2013)	Italy	Granular	3	50–80	GC	28	10–30	0.1–20	-	-	-	3300–9600
Kleinert (2016)	Brazil	CTCS	1–7	7–92	AM	3–14	24 ± 3	1	-	484–20,031	-	-
Kleinert (2016); Kleinert et al. (2019)	Brazil	Soil-cement	1–7	7–92	AM	3–14	24 ± 3	1	-	2199–19,357	-	-
Kolias (1996b)	Greece	Granular	5	25–100	VC	720	4–30	1–16	-	-	-	7100–23,600
Louw et al. (2019)	USA	Granular	5	50	AM	1440	-	1	12,000–17,500	-	-	-
Mallick et al. (2002a); Mallick et al. (2002b)	USA	Granular	5	67	GC	90	-	-	-	10,469	-	-
Mohammadinia et al. (2014)	Australia	-	2–4	100	AM	1–7	-	1	200–3200	-	-	-
Oliveira & Paiva (2019)	Brazil	LG	3	35	AM	28	25	1	-	8076	-	-
Puppala et al. (2011)	USA	-	2–4	100	SP	7	-	1	200–515	-	-	-
Romeo et al. (2019)	Italy	Granular	3	Not specified	AM	90	-	1	650–750	-	-	-
Schreinert (2017)	Brazil	Lateritic soil	1–7	7–92	AM	3–28	24 ± 3	1	-	860–16,927	-	-
Sufian et al. (2009b)	Malaysia	GCS	3	25–100	AM	1–28	25	-	-	1267–15,500	-	-
Trichês et al. (2013)	Brazil	GCS	2–4	30	BI	28	25	1	602–2615	-	-	-
Trichês et al. (2013)	Brazil	Granular	2–4	25	AM	7–28	25	1	2669–5518	-	-	-

RAP: reclaimed asphalt pavement; GCS: graded crushed stone; CTCS: cement-treated crushed stone; LG: lateritic gravel; AM: AASHTO modified; BI: Brazilian intermediate; SP: standard Proctor; GC: gyratory compactor; VC: vibratory compactor; TRM: triaxial resilient modulus; ITRM: indirect tensile resilient modulus; FRM: flexural resilient modulus; CDM: compressive dynamic/complex modulus

Table 11. Ranges of cyclic modulus reported for FDR-PC materials

Behaviour	Property	Increase of						Source
		Cement content	RAP content	Compact. effort	Curing time	Temp.	Load freq.	
Mechanical	Unconfined compressive strength	I	D	I	I	D	-	Works in Table 8
	Indirect tensile strength	I	D[1]	I	I	D	-	Works in Table 8
	Direct tensile strength	I	D	U	I	U	-	Works in Table 8
	Flexural tensile strength	I	D[1]	U	I	U	-	Works in Table 8
	Flexural tensile strain at break	I[1]	I	U	U	U	-	Works in Table 8
	Compressive elastic modulus	I	D	U	I	D	-	Works in Table 9
	Direct tensile elastic modulus	I	D	U	I	U	-	Works in Table 9
	Flexural elastic modulus	I	D	U	I	D	-	Works in Table 9
	Elastic modulus (non-destructive tests)	I	D	U	I	D	-	Works in Table 9
	Triaxial resilient modulus	I	D	U	I	U	U	Works in Table 10
	Indirect tensile resilient modulus	I	D[1]	I	I	U	U	Works in Table 10
	Flexural resilient modulus	I	D	U	U	U	U	Works in Table 10
	Compressive dynamic/complex modulus	U	D	U	U	D	I	Works in Table 10
	Phase angle	U	I	U	U	I	D	Kolias (1996b); Grilli et al. (2013); Graziani et al. (2019)
	Fatigue life	I[2]	D[2]	U	U	U	U	Kolias et al. (2001); Paiva et al. (2017); Jiang et al. (2018)
Durability	Moisture sensitivity (tube suction test)	D	D	U	U	U	-	Guthrie et al. (2007); Yuan et al. (2011)
	Shrinkage	I	N	N	I	U	-	El Euch Khay et al. (2014); Fedrigo et al. (2017b)
	Capillary rise	D	D	D	U	U	-	Kleinert (2016); Fedrigo et al. (2017b); Oliveira & Paiva (2019)
	Absorption	D	D	D	U	U	-	Kleinert (2016); Fedrigo et al. (2017b); Oliveira & Paiva (2019)
	Erodibility	D	D	D	U	U	-	Kleinert (2016); Fedrigo et al. (2017b)
	Swell	N	N	N	N	U	-	Kleinert (2016); Fedrigo et al. (2017b); Oliveira & Paiva (2019)
	Mass loss (wet-dry cycles test)	D	I	U	U	U	-	Suddeepong et al. (2018)

I: increase; D: decrease; N: no effect; U: still unknown; RAP: reclaimed asphalt pavement; [1]Authors reported the opposite trend for mixtures with lateritic soils (Schreinert, 2017); [2]Authors stated that cement and RAP effects are more complicated (Castañeda López et al. 2018)

Table 12. Effects of some characteristics on the properties of FDR-PC materials

Source	Country	Existing base material	Cement content (%)	RAP content (%)	Curing time (days)	M_{FWD} (MPa)	M_{LWD} (MPa)
Amarh et al. (2017)	USA	Not specified	5	Not specified	21–780	2600–7200	-
Godenzoni et al. (2018)	Italy	Granular	3	33	40–2520	1072–5650	-
Isola et al. (2013)[1]	Italy	Granular	4	70	1	-	360–400
Jones et al. (2015); Wu et al. (2015)	USA	Granular	5	50	28–550	3000–22,000	-
Trichês & Santos (2011)	Brazil	Granular	3	40	35	732	-
Wilson & Guthrie (2011)	USA	Silty sand	4	50–70	3–90	-	621–12,765

RAP: reclaimed asphalt pavement; M_{FWD}: modulus back-calculated using falling weight deflectometer data; M_{LWD}: modulus back-calculated using light weight deflectometer data; [1]M_{FWD} values are not presented since the authors combined surface and recycled base layers into a single layer

Table 13. Ranges of field modulus reported for FDR-PC layers

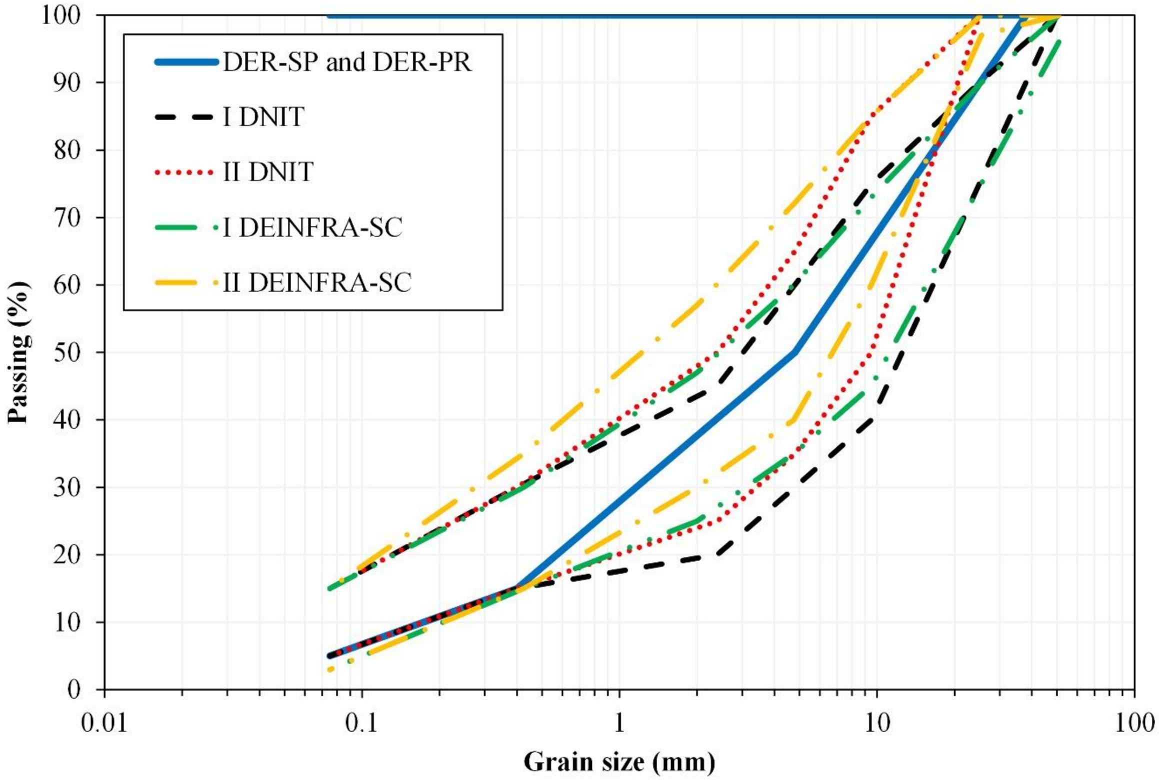

Figure 1. Grain size distribution envelopes suggested by Brazilian standards

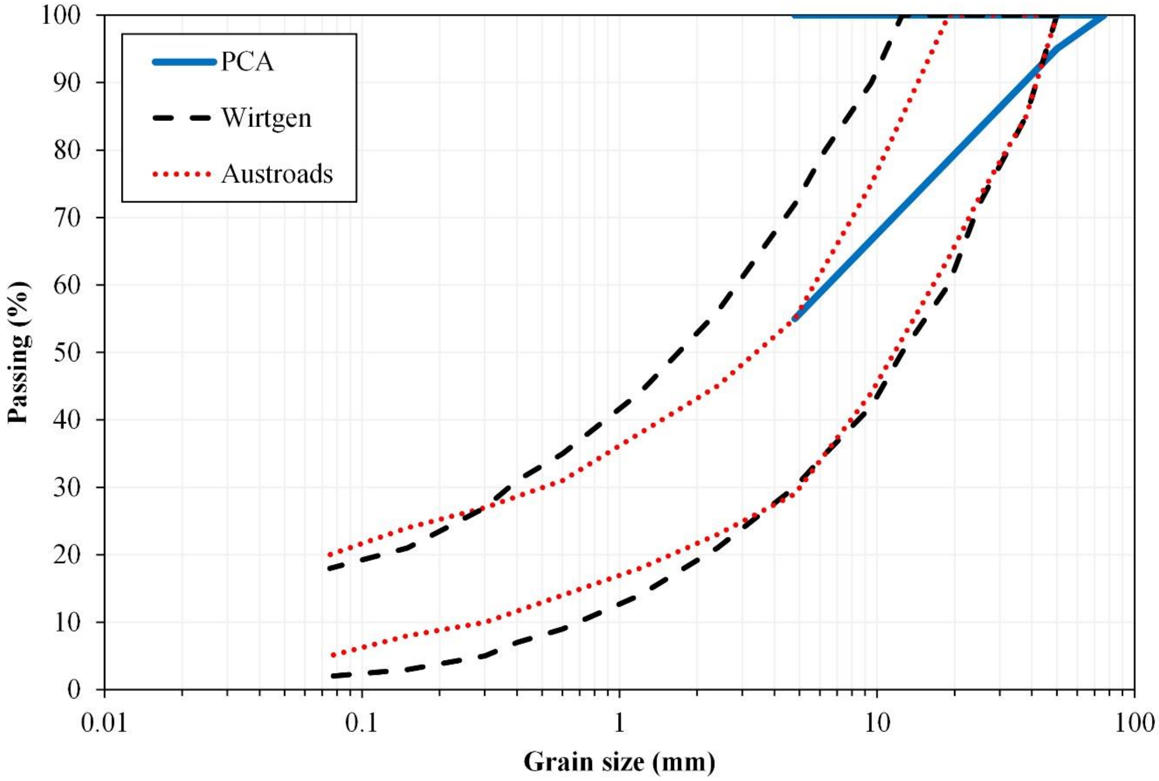

Figure 2. Grain size distribution envelopes suggested by mix design methods

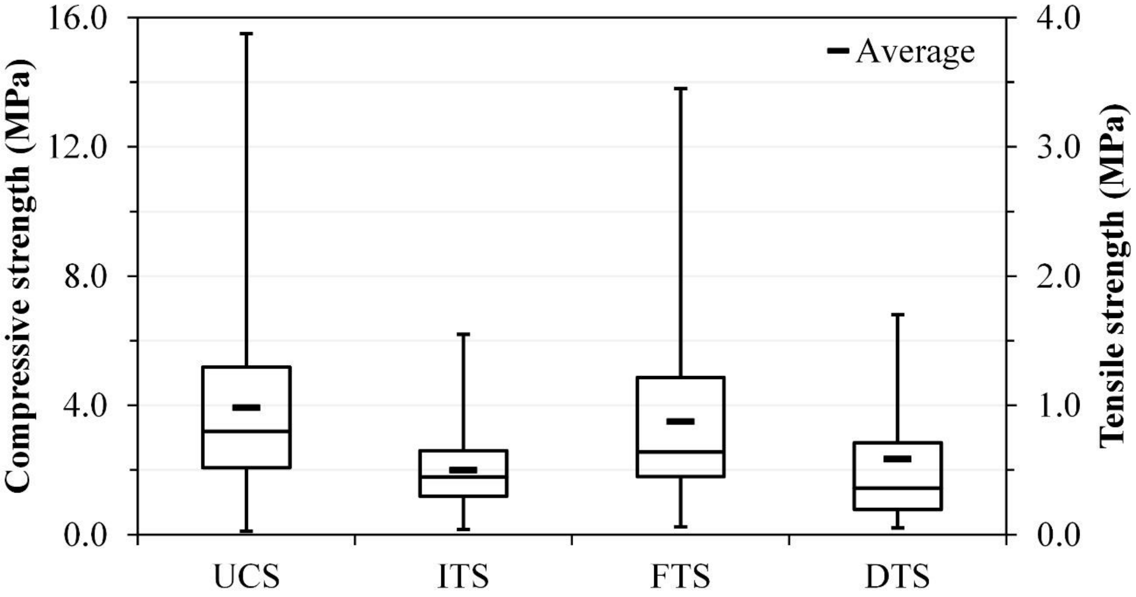

Figure 3. Box and whisker plots for the strength values reported in previous research

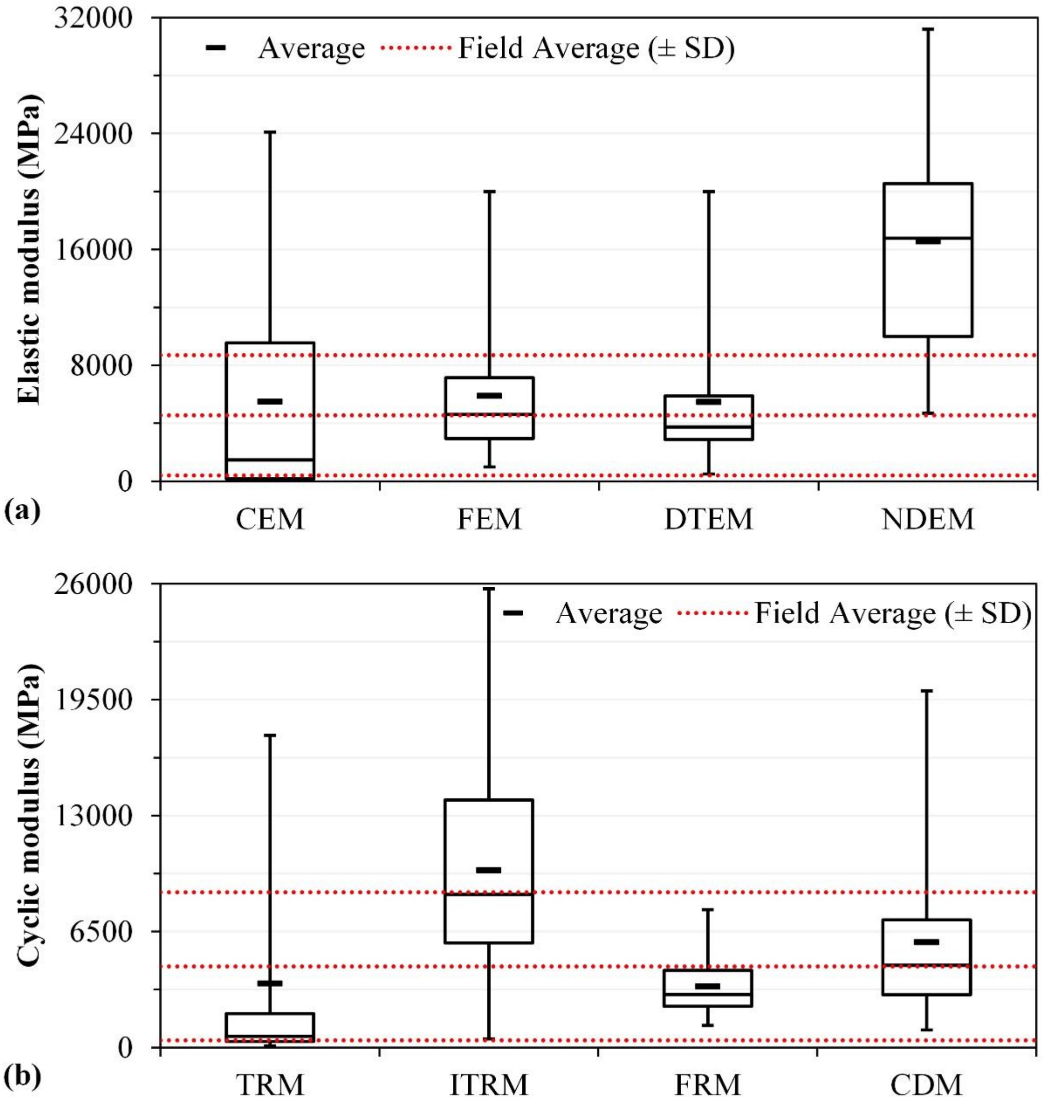

Figure 4. Box and whisker plots for the modulus values reported in previous research: (a) elastic; and (b) cyclic

3 EFFECTS OF RAP RESIDUAL ASPHALT BINDER TYPE, CONTENT AND AGEING ON THE MECHANICAL BEHAVIOUR OF COLD RECYCLED CEMENT-TREATED MIXTURES

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Abstract:

This paper evaluates the effects of reclaimed asphalt pavement (RAP) residual binder characteristics on the mechanical behaviour of cold recycled cement-treated mixtures. RAP samples with different binder types (conventional, polymer and rubber), contents (4.5% and 5.5%) and ageing conditions were produced in laboratory. Cold recycled mixtures were prepared using the produced RAP, graded crushed stone and cement. The strength of cold recycled cement-treated mixtures is mainly affected by the binder type, while their stiffness is significantly affected not only by the binder type but by the binder content and ageing condition as well.

Keywords: pavement recycling; full-depth reclamation with cement; reclaimed asphalt pavement; asphalt binder; mechanical behaviour

1 Introduction

Asphalt pavements of many countries are approaching the end of their design lives. In most of these countries, using waste materials and recycling techniques are progressively becoming common, especially due to their environmental-friendly and cost-effective characteristics (Arulrajah et al., 2015; Paige-Green & Ware, 2006). Full-depth reclamation with portland cement (FDR-PC) is one of the main used recycling techniques. It consists of the in situ milling of the existing damaged layers (i.e. asphalt wearing course and base layer) in conjunction with cement treatment. The resulting mixture is compacted to form a new layer that will increase the bearing capacity of the pavement structure. A new surface layer is then applied, completing the FDR-PC process (Katsakou & Kolias, 2007; Portland Cement Association, PCA, 2005).

The mixtures produced using FDR-PC differ from regular cement-treated materials because of the presence of reclaimed asphalt pavement (RAP). This material consists of a mixture of aggregates coated with asphalt binder and agglomerations of fines bonded together with asphalt binder. The quantity of RAP in the mixture depends on the asphalt layer thickness and will significantly affect the properties of the recycled layer (Kolias et al., 2001).

All around the world, several studies on FDR-PC and cement-treated materials with RAP were reported (Amarh et al., 2017; Bessa et al., 2016b; Dalla Rosa et al., 2015; Dixon et al., 2012; Fedrigo et al., 2019a; Godenzoni et al., 2018; Gonzalo-Orden et al., 2019; Graziani et al., 2019; Jiang et al., 2018; Jones et al., 2015; Liebenberg & Visser, 2003; Mallick et al., 2002a; Mallick et al., 2002b; Mohammadinia et al., 2015; Paiva et al., 2017; Puppala et al., 2011; Santos et al., 2017; Tataranni et al., 2017; Wilson & Guthrie, 2011; Wu et al., 2015). Many studies indicate that the increase of RAP in the mixture decreases its strength and stiffness (Adresi et al., 2017; El Euch Khay et al., 2014; Ghanizadeh et al., 2017; Grilli et al., 2013a; Grilli et al., 2013b; Romeo et al., 2019; Suddeepong et al., 2018; Taha et al., 2002). Some authors state that this reduction happens due to the asphalt binder coating the aggregate particles, which reduces the available surface area that can be coated with cement, inhibiting the generation of cementitious bonds between the cement and the aggregates and allowing greater deformation or slip (Guthrie et al., 2007; Ji et al., 2016; Ma et al., 2015). Kolias (1996a; 1996b) also mentions that the agglomerations of fines bonded together with the asphalt binder present in RAP show low modulus and strength, especially in comparison to crushed stone particles.

Suebsuk et al. (2017) even developed a model in which the strength of this kind of recycled mixture decreases as the asphalt binder content in RAP increases. Yuan et al. (2011) reported an opposite conclusion. They studied the effect of different RAP sources on strength and stiffness of cement-treated mixtures and verified that the asphalt binder content in RAP does not seem to show a considerable impact. Fedrigo et al. (2016) performed a study to evaluate the effect of RAP asphalt binder type on the strength of cement-treated mixtures. The authors reported that mixtures with RAP containing asphalt rubber showed the higher strength values, followed by those containing conventional and polymer asphalt binders, respectively. However, in both the last-mentioned studies the used RAP had different aggregate types and grain size distributions, affecting the results. Therefore, the effects of the RAP residual asphalt binder on the behaviour of cold recycled cement-treated mixtures remain not well known, affecting mix and structure designs.

In this regard, Chapter 7 of RILEM State-of-the-Art Report "Advances in Interlaboratory Testing and Evaluation of Bituminous Materials" (de la Roche et al., 2013) states that laboratory manufactured RAP is important for assessing the recyclability of asphalt mixtures. A recommendation for laboratory production of RAP, including an ageing procedure for the asphalt mixture, was then proposed. Other authors also produced RAP materials in laboratory to evaluate the effect of the RAP residual binder (mainly its age) on stiffness, fatigue and permanent deformation of cold recycled mixtures with asphalt emulsion (Ojum & Thom, 2017). Authors also produced artificial RAP substrates in laboratory to evaluate the bond strength between RAP and bitumen emulsion composites (Cardone et al., 2018).

The main objective of this research was to evaluate the effects of RAP residual asphalt binder type, content and ageing on the mechanical behaviour of cold recycled cement-treated mixtures.

2 Experimental programme

Comparing the behaviour of cold recycled cement-treated mixtures containing RAP from different sources (with different asphalt binder type and content, aggregate type, grain size distribution, age and so on) would not lead to reliable results. Therefore, to achieve the research objective, RAP with three different commercially available asphalt binders (conventional, polymer and rubber) and two contents (4.5% and 5.5%) as well as with two different ageing conditions (namely, unaged and aged) were produced in laboratory. These asphalt binder types and contents are commonly used in Brazilian asphalt materials. The same aggregates and grain size distributions were used for all RAP to avoid any undesirable influence. These RAP were then mixed with aggregates and cement to simulate the full-depth reclamation of an actual asphalt pavement. The effects of the studied variables on compactability and mechanical behaviour (indirect tensile strength and resilient modulus) of cold recycled mixtures were evaluated.

2.1 RAP preparation

Twelve asphalt mixtures were prepared according to Table 1, characterizing a full factorial design. The physical properties of the asphalt binders are presented in Table 2 (Barros et al., 2018; Godoi et al., 2017). The properties of the basalt aggregates followed the required limits used in Brazil; its grain size distribution was within the Brazilian National Department of Transport Infrastructure (DNIT) “C envelope”, commonly used for asphalt concrete production in Brazil (DNIT, 2006). More details of the used materials were reported by Barros et al. (2018) and Godoi et al. (2017). The mixtures were prepared using a high shear mixer and compacted using a gyratory compactor (100 gyrations) at appropriate temperatures (Barros et al., 2018; Godoi et al., 2017). Two specimens (100 mm in diameter and 150 mm in height) were prepared for each mixture.

Table 1

Table 2

Ageing was performed by putting the loose mixes in an oven at 85 °C for 8 days before compaction, following the procedures suggested by Elwardany et al. (2017). Similar methods were used by de la Roche et al. (2013) and Preti et al. (2019). Binder extraction was not feasible due to the extremely long time necessary to produce RAP in the necessary amount with that purpose. To verify the efficiency of the ageing method, the different asphalt binders were aged independently. The procedures and equipment were the same as those used for the loose mixtures. The thickness of the asphalt binder was kept at 5 mm. Results presented in Table 2 prove the soundness of the ageing method. Besides, the ageing effect must have been even stronger for the asphalt binder in loose mixtures, since the thickness of asphalt binder film that coats aggregates is thinner than 5 mm (Al-Khateeb, 2018; Elseifi et al., 2008; Heitzman, 2005).

It is highlighted that de la Roche et al. (2013) tested asphalt binders extracted from laboratory produced RAP and proved the adequacy of the used ageing method.

The specimens were left to cool and then were crushed using a small size jaw crusher to obtain twelve laboratory produced RAP (Table 1). Figure 1 shows the general appearance of the RAP obtained, which resembles that of an actual field RAP. Figure 2 shows the grain size distributions of the obtained RAP. The obtained grain size distributions were very similar for the different asphalt types and contents. Therefore, the grain size distributions were divided into aged RAP and unaged RAP and error bars were added. The grain size distributions of the laboratory produced RAP were similar to those of field RAP reported elsewhere (Castañeda López et al., 2018; Fedrigo et al., 2018; Fedrigo et al., 2017; Fedrigo et al., 2019b). However, field RAP has a higher quantity of fines, possibly due to its ageing and because of higher crushing energy produced by recycling or milling machines.

Figure 1

Figure 2

2.2 Cold recycled cement-treated mixtures

2.2.1 Materials

Twelve recycled mixtures were prepared using the twelve produced RAP (50%), a graded crushed stone (GCS) (50%) and cement (2%, based on the dry mass of RAP + GCS). The cement and RAP contents follow the maximum limits reported in the most recent Brazilian protocol for FDR-PC (Santa Catarina State Department of Infrastructure, DEINFRA-SC, 2016), which are 3% and 50%, respectively. The GCS was collected from the base layer of a pavement structure in Southern Brazil during its recycling. Figure 2 shows the grain size distributions of GCS and mixtures of RAP + GCS (considering aged and unaged RAP). The same figure also presents an envelope for cold recycled cement-treated mixtures suggested by Wirtgen (2012). Portland cement with ground-granulated blast-furnace slag addition (Brazilian type CP II E 32) following Brazilian standards (Brazilian Association of Technical Standards, ABNT, 1991; ABNT, 2018) was used to treat the mixtures of RAP + GCS. The particle density of GCS and cement were 2.93 g/cm^3 and 2.96 g/cm^3, respectively. Particle densities of the twelve RAP were similar, showing an average value of 2.55 g/cm^3 (coefficient of variation = 1%).

2.2.2 Compaction

The mixtures maximum dry density and optimum moisture content were not determined due to the high quantity of RAP that would have to be produced to perform compaction tests. Moisture content was determined based on work by Isola et al. (2013). According to those authors' experience, a water-cement ratio of 1.7 was adopted for all mixtures. Despite lower than the typically used for cement-treated materials, the adopted moisture content proved to be adequate. This could be related to the lack of fines (Figure 2), which led to a small specific surface. To produce the mixtures, the solids (RAP, GCS, and cement) were mixed and then water was added while continuing mixing. The amount of cement was computed as a percentage of the mass of RAP and GCS (1 kg per specimen) and the amount of water as a percentage of the mass of solids.

Marshall procedure was used to compact the mixtures into cylindrical moulds (102 mm in diameter and approximately 64 mm in height). 75 blows were applied on both sides of the specimens using a mechanical hammer (4.54 kg mass and 457 mm drop height). It is highlighted that the effort produced using Marshall procedure (approximately 2900 kNm/m^3) is similar to the Modified Proctor (2700 kNm/m^3), which is commonly used for cement-treated materials (Tebaldi et al., 2014). Besides, the production of specimens of cold recycled mixtures using the Marshall procedure was reported elsewhere (Bessa et al., 2016a; Dal Ben & Jenkins, 2014; Fu et al., 2010; Gandi et al., 2019, Kim & Lee, 2006; Tebaldi et al., 2014; Wirtgen, 2012). No expulsion of water or fines was noticed during compaction, possibly due to the small amount of both materials. Specimens were demoulded right after compaction since they were stable. The mass of the specimens was measured (M_T) and the volume (V_T) was derived from their dimensions. Then, they were sealed in hermetic recipients to avoid moisture loss and cured at room temperature (~25 °C) for 7 days. For each mixture, 4 specimens were prepared, totalling 48 specimens. Figure 3 (a) shows the appearance of a specimen prior to testing.

Figure 3

The compactability of the mixtures was evaluated by means of dry density (ρ_d) and voids in the mixture (V_m) (Graziani et al., 2016; Grilli et al., 2018; Grilli et al., 2012; Grilli et al., 2016; Raschia et al., 2019a; Raschia et al., 2019b), according to Equations (1), (2), (3) and (4).

$$\rho_b = \frac{M_T}{V_T} \tag{1}$$

$$\rho_m = \frac{100}{\frac{P_{GCS}}{\rho_{GCS}} + \frac{P_{RAP}}{\rho_{RAP}} + \frac{P_C}{\rho_C}} \tag{2}$$

$$\rho_d = \frac{\rho_b}{W+1} \tag{3}$$

$$V_m = \frac{\rho_m - \rho_d}{\rho_m} 100 \tag{4}$$

where ρ_b is the wet density (g/cm^3); M_T and V_T are respectively the total mass (g) and volume (cm^3) of the specimens; ρ_m is the maximum density (g/cm^3); P_{GCS}, P_{RAP} and P_C are respectively the percentage by mass of GCS, RAP and cement (expressed as percent by mass of solid materials); ρ_{GCS}, ρ_{RAP} and ρ_C are respectively the particle density of GCS, RAP and cement (which were measured and are presented in Section 2.2.1); and W is the moisture content.

2.2.3 Mechanical tests

The specimens were tested at a temperature of 24 ± 3 °C and relative humidity of 55 ± 15%. For each mixture, two specimens were used to determine the indirect tensile strength (ITS) to be used in indirect tensile resilient modulus (ITMr) tests. The other two specimens were subjected to ITMr tests and then to ITS tests. The procedures used for ITS tests followed ASTM D6931 (American Society for Testing and Materials Standard, ASTM, 2012). Tests were carried using a constant displacement rate of 50 mm/min. Figure 3 (b) shows the appearance of a specimen after ITS test. Test procedures for ITMr were mainly based on ASTM D7369 (ASTM, 2011). Haversine-shaped cyclic load pulses were applied at a frequency of 1 Hz. The stress level was kept at 30% of the ITS. The horizontal displacements were measured by a linear variable differential transducer (LVDT). Specimens were subjected to 75 load cycles (first 50 cycles were pre-conditioning). Brazilian standards recommend these number of cycles and stress levels (DNIT, 2010; DNIT, 2018). ITS and ITMr, in MPa, were calculated using Equation (5) and Equation (6), respectively.

$$\mathrm{ITS} = \frac{2P}{\pi D h} \tag{4}$$

$$\mathrm{ITMr} = P\frac{(\nu + 0.27)}{Hh} \tag{5}$$

where P is the peak load (N); ν is the Poisson's ratio (assumed as 0.17 based on work by Kleinert et al. (2017)); H is the horizontal displacement of the specimen after application of load (mm); D is the diameter of the specimen (mm); and h is the height of the specimen (mm).

3 Results

3.1 Compactability

Figure 4 shows the dry density (ρ_d) as a function of asphalt binder type and content for unaged (Figure 4 (a)) and aged (Figure 4 (b)) conditions. The results are the average of four specimens and the error bars represent the standard deviation. Similar dry densities were observed for the different mixtures. Figure 5 shows that similar voids in the mixture (V_m) were also achieved since the ρ_d is the only variable that changes when calculating V_m.

Figure 4

Figure 5

An Analysis of Variance (ANOVA) was performed for the compactability results. Due to the abovementioned reason, analysing ρd or V_m led to the same results (Table 3). The effect of the asphalt binder content on ρ_d and V_m was significant since the obtained p-value was less than the considered significance level of 0.05. Besides, a comparison of means (Fisher Pairwise Comparison) was performed, showing significant differences.

Table 3

3.2 Indirect tensile strength

Figure 6 shows the indirect tensile strength (ITS) as a function of asphalt binder type and content for unaged (Figure 6 (a)) and aged (Figure 6 (b)) conditions. The results are the average of four specimens. Error bars showing the standard deviation are also presented. Similar values of ITS were observed for specimens that were or not subjected to ITMr tests before ITS tests, indicating that no damage was caused during ITMr tests. Furthermore, a statistical analysis (one-way ANOVA) performed for all mixtures proved that ITMr test does not affect the ITS values (p-values > 0.05).

Figure 6

ITS values ranged from 0.18 to 0.27 MPa. Mixtures containing RAP with conventional asphalt binder showed the highest ITS. ITS of mixtures containing RAP with polymer asphalt binder were higher than those of mixtures containing RAP with rubber asphalt binder for a content of 4.5%. The opposite trend was observed for a content of 5.5%. The observed trend diverges from that presented by Fedrigo et al. (2016) for cement-treated mixtures with RAP. The increase of the asphalt binder content in RAP decreased the ITS of mixtures containing RAP with conventional and polymer asphalt binders, possible due to an increase in flexibility. However, the opposite trend was observed for mixtures containing RAP with rubber asphalt binder.

Trichês and Morilha Junior (2005a; 2005b) studied the effect of different asphalt binders on the indirect tensile strength of asphalt mixtures. Differently from the results presented in this study, the authors reported that mixtures containing polymer asphalt binder were stronger than those made with conventional and rubber asphalt binders, respectively. The authors also reported that mixtures with modified asphalt binders (polymer and rubber) showed higher ageing resistance. However, the impact of the ageing procedure used in this study caused only slight increases in the ITS of the mixtures.

ANOVA was performed for ITS and the results are presented in Table 4. Only the asphalt binder type and the interaction between asphalt binder type and content showed significant effects (p-values < 0.05). It is highlighted that the effect of the aforementioned interaction may also be related to the significant effect of the asphalt binder content on the compactability, since the latter affects the ITS. The fact that the interaction between asphalt binder type and content is significant in some way disagrees with Yuan et al. (2011). The authors stated that the asphalt binder content in RAP does not seem to show a considerable impact on strength and stiffness of cold recycled cement-treated mixtures. Although there was significant interaction, a comparison of means was performed for the asphalt binder type and there was no significant difference between polymer and rubber asphalt binders.

Table 4

Figure 7 shows the effect of the interaction between asphalt binder type and content on the mean of ITS. A multiple comparison of means was also performed, and three groups were defined (Table 5). Means that do not share a group letter are significantly different. It should be highlighted that ANOVA results are consistent with the comments written above based on Figure 6.

Figure 7

Table 5

3.3 Indirect tensile resilient modulus

Figure 8 shows the indirect tensile resilient modulus (ITMr) as a function of asphalt binder type and content for unaged (Figure 8 (a)) and aged (Figure 8 (b)) conditions. The results are the average of six values (two specimens were tested, and three modulus values were determined per specimen). The standard deviation is also presented (error bars).

Figure 8

ITMr values varied from 2,151 MPa to 6,699 MPa. As observed for ITS, the increase of the asphalt binder content in RAP led to ITMr reductions, increasing the mixtures flexibility. Generally, mixtures containing RAP with conventional asphalt binder showed the highest modulus values, followed by those containing RAP with polymer and rubber asphalt binders, respectively. For asphalt mixtures, Trichês and Morilha Junior (2005a; 2005b) reported that mixtures containing polymer asphalt binder were stiffer than those made with conventional and rubber asphalt binders, respectively. Xiao and Amirkhanian (2008; 2009) also state that the stiffness of asphalt mixtures decreases with the rubber content. Therefore, the presence of rubber in asphalt mixtures and cold recycled cement-treated mixtures seems to lead to similar effects on the stiffness.

Modulus increased with ageing for all mixtures, showing that the asphalt binders became stiffer. However, Ojum and Thom (2017) reported the opposite trend when evaluating the effect of the RAP residual binder age on the stiffness of cold recycled mixtures with asphalt emulsion. The literature states that the modification of asphalt binders with polymers and rubber increases their ageing resistance (Lu & Isacsson, 1998; Ruan et al., 2003; Tarefder & Yousefi, 2016; Trichês & Morilha Junior, 2005a; Trichês & Morilha Junior, 2005b; Xiao & Amirkhanian, 2008; Xiao & Amirkhanian, 2009). In this study, this was observed for cold recycled cement-treated mixtures containing RAP with polymer asphalt binder, since the ageing procedure slightly increased their ITMr. However, the ITMr of mixtures containing RAP with rubber asphalt binder increased approximately 100% with ageing.

ANOVA was performed for the ITMr results (Table 6). All the studied variables and the interaction between asphalt binder type and ageing showed significant effects (p-value < 0.05). The significant effect of the asphalt binder content disagrees with the statements made by Yuan et al. (2011). Since the effect of the asphalt binder content on the compactability was also significant, the latter may have affected the ITMr. The main effects were analysed even though there is a significant interaction. A comparison of means showed significant differences for all means of the main effects.

Table 6

The effect of the interaction between asphalt binder type and ageing on the mean of ITMr is presented in Figure 9. A multiple comparison of means was performed resulting in four groups (Table 7). Means that do not share a group letter are significantly different. The ANOVA results for stiffness are consistent with the comments written above based on Figure 8.

Figure 9

Table 7

4 Conclusions

Based on the results presented and analysed, it can be concluded that the characteristics of the asphalt binder in RAP (type, content, and ageing) affect the compactability and the mechanical behaviour of cold recycled cement-treated mixtures, hence affecting mix and structure designs. The following conclusions can be pointed out:

- The asphalt binder content has significant effect on the compactability of cold recycled cement-treated mixtures. Mixtures containing RAP with higher asphalt binder content (5.5%) generally showed higher dry densities and lower voids. Therefore, significant effects of the asphalt binder content on the mechanical behaviour may also be related to compactability of the mixture.
- The asphalt binder type and the interaction between asphalt binder type and content have significant effects on the strength of cold recycled cement-treated mixtures.

Mixtures containing RAP with conventional asphalt binder showed the highest strengths. The strength of mixtures containing RAP with polymer asphalt binder was higher than those of mixtures containing RAP with rubber asphalt binder for an asphalt binder content of 4.5%. The opposite trend was observed for a content of 5.5%.

- The asphalt binder content, type, and ageing, as well as the interaction between asphalt binder type and ageing, have significant effects on the resilient modulus of cold recycled cement-treated mixtures. The increase in asphalt binder content in RAP led to stiffness reductions, increasing the flexibility of the mixtures. Mixtures containing RAP with conventional asphalt binder showed the highest stiffness values, followed by those containing RAP with polymer and rubber asphalt binders, respectively. The ageing procedure increased the stiffness of all mixtures, due to hardening of the asphalt binders. The weakest and strongest effects due to ageing occurred for mixtures containing RAP with polymer and rubber asphalt binders, respectively.

It is acknowledged that laboratory production of RAP is important for assessing the recyclability of asphalt pavements. In this regard, future research could focus on constructing test sections with asphalt mixtures produced with the same aggregates and grain size distribution, but different asphalt binder types and contents. These sections could then be subjected to traffic and climatic effects and milled at different ages to produce RAP materials that would be even more representative of field RAP.

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Asphalt binder type	Asphalt binder content (%)	Asphalt mixture ageing
Conventional	4.5	Yes
		No
	5.5	Yes
		No
Polymer (SBS, styrene-butadiene-styrene)	4.5	Yes
		No
	5.5	Yes
		No
Rubber	4.5	Yes
		No
	5.5	Yes
		No

Table 1. Characteristics of asphalt mixtures and RAP

Property	Conventional	Polymer	Rubber
Brazilian classification	CAP 50/70	AMP 60/85	AB-8
Penetration @ 25 °C, 100 g, 5 s (0.1 mm)	61	59	64
Brookfield viscosity @ 135 °C (cP)	327.5	1,220	-
Brookfield viscosity @ 150 °C (cP)	165	603	-
Brookfield viscosity @ 177 °C (cP)	61.5	222.5	1575
Density @ 25°C (kg/m³)	1009.1	1003.0	1023.7
Softening point (°C)	48	67	55
Penetration index	-1.3	-	-
Elastic recovery @ 25°C, 200 mm (%)	-	94	78
After Rolling Thin Film Oven Test (RTFOT)			
Penetration @ 25 °C, 100 g, 5 s (0.1 mm)	38	45	51
Softening point (°C)	50	73	60
Elastic recovery @ 25°C, 200 mm (%)	-	92	86
After oven ageing (8 days @ 80 °C)			
Penetration @ 25 °C, 100 g, 5 s (0.1 mm)	39	37	40
Softening point (°C)	49	63	59
Elastic recovery @ 25°C, 200 mm (%)	-	88	82

Table 2. Physical properties of asphalt binders

Factor	***p*-value**
Ageing	0.568
Content	0.026
Type	0.109
Ageing*Content	0.116
Ageing*Type	0.236
Content*Type	0.321
Ageing*Content*Type	0.340

Table 3. ANOVA results for ρ_d and V_m

Factor	***p*-value**
Ageing	0.086
Content	0.753
Type	0.000
Ageing*Content	0.597
Ageing*Type	0.587
Content*Type	0.028
Ageing*Content*Type	0.822

Table 4. ANOVA results for ITS

Content*Type	N	Mean of ITS (MPa)	Grouping
4.5%*Conventional	8	0.264	A
5.5%*Conventional	8	0.254	A and B
5.5%*Rubber	8	0.230	B and C
4.5%*Polymer	8	0.228	B and C
4.5%*Rubber	8	0.198	C
5.5%*Polymer	8	0.197	C

Table 5. Multiple comparison of means for the effect of interaction between asphalt binder type and content on the ITS

Factor	***p*-value**
Ageing	0.000
Content	0.000
Type	0.000
Ageing*Content	0.823
Ageing*Type	0.000
Content*Type	0.054
Ageing*Content*Type	0.830

Table 6. ANOVA results for ITMr

Ageing*Type	N	Mean of ITMr (MPa)	Grouping
Yes*Conventional	12	6580	A
Yes*Polymer	12	5761	B
Yes*Rubber	12	5263	C
No*Polymer	12	5198	C
No*Conventional	12	5082	C
No*Rubber	12	2555	D

Table 7. Multiple comparison of means for the effect of interaction between asphalt binder type and ageing on the ITMr

Figure 1. General appearance of prepared RAP

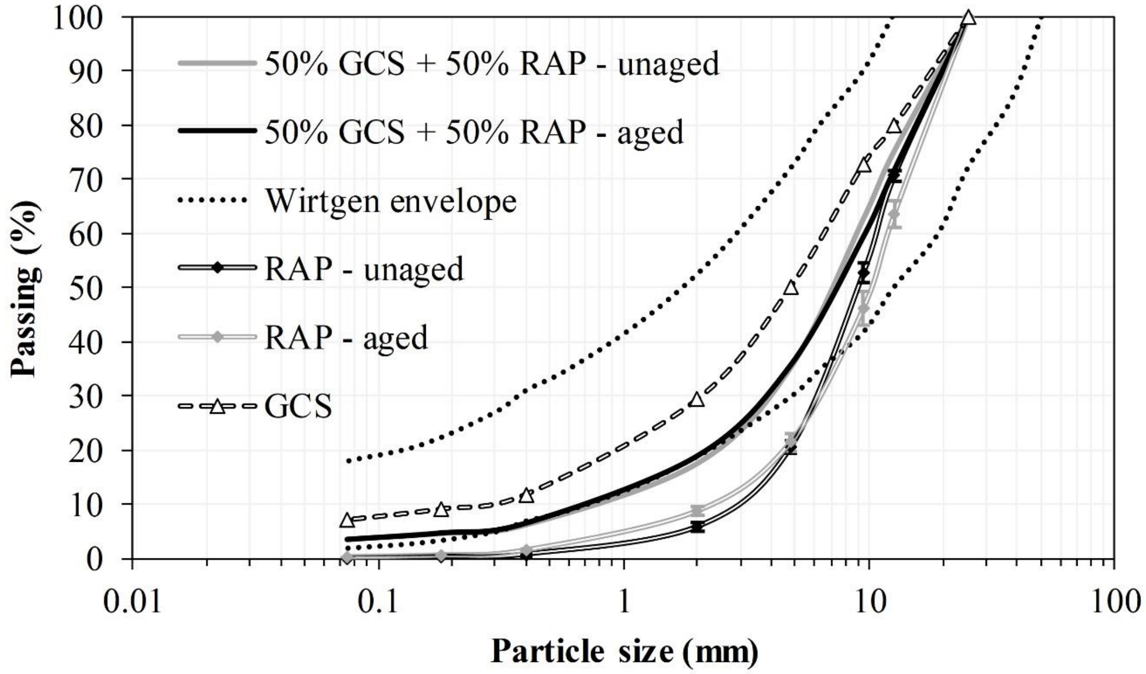


Figure 2. Grain size distributions of RAP, GCS and mixtures of RAP and GCS

Figure 3. Specimen of cold recycled cement-treated mixture: (a) before and (b) after ITS test

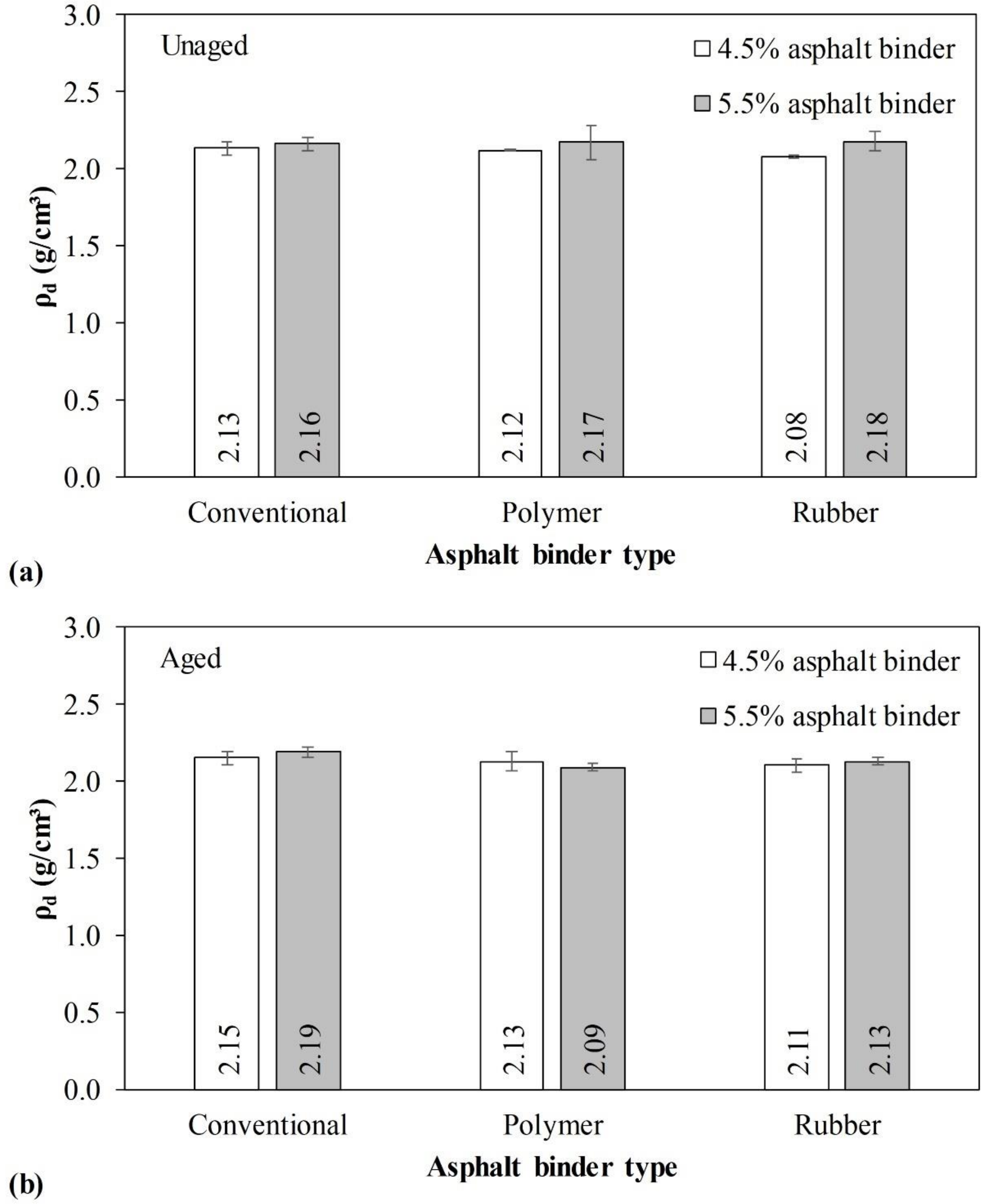
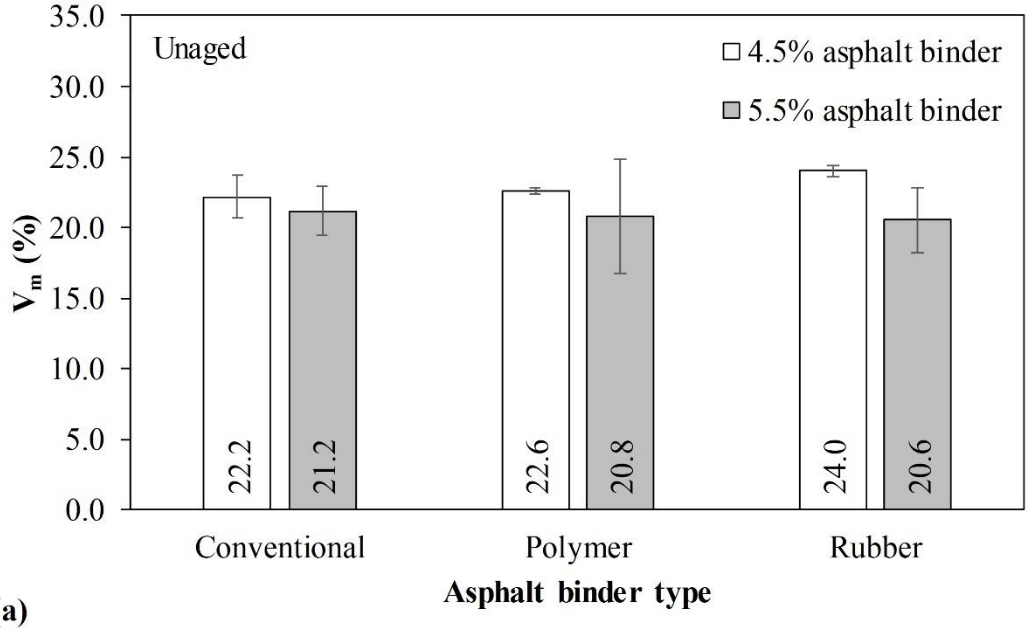
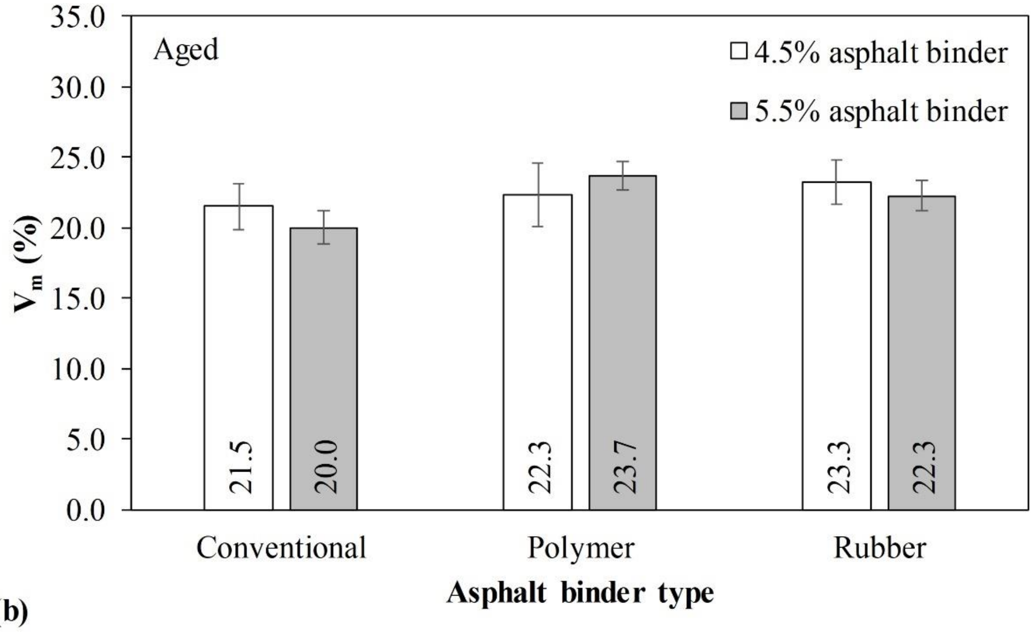


Figure 4. ρ_d of cold recycled cement-treated mixtures as a function of asphalt binder type and content for (a) unaged and (b) aged conditions



(a)



(b)

Figure 5. V_m of cold recycled cement-treated mixtures as a function of asphalt binder type and content for (a) unaged and (b) aged conditions

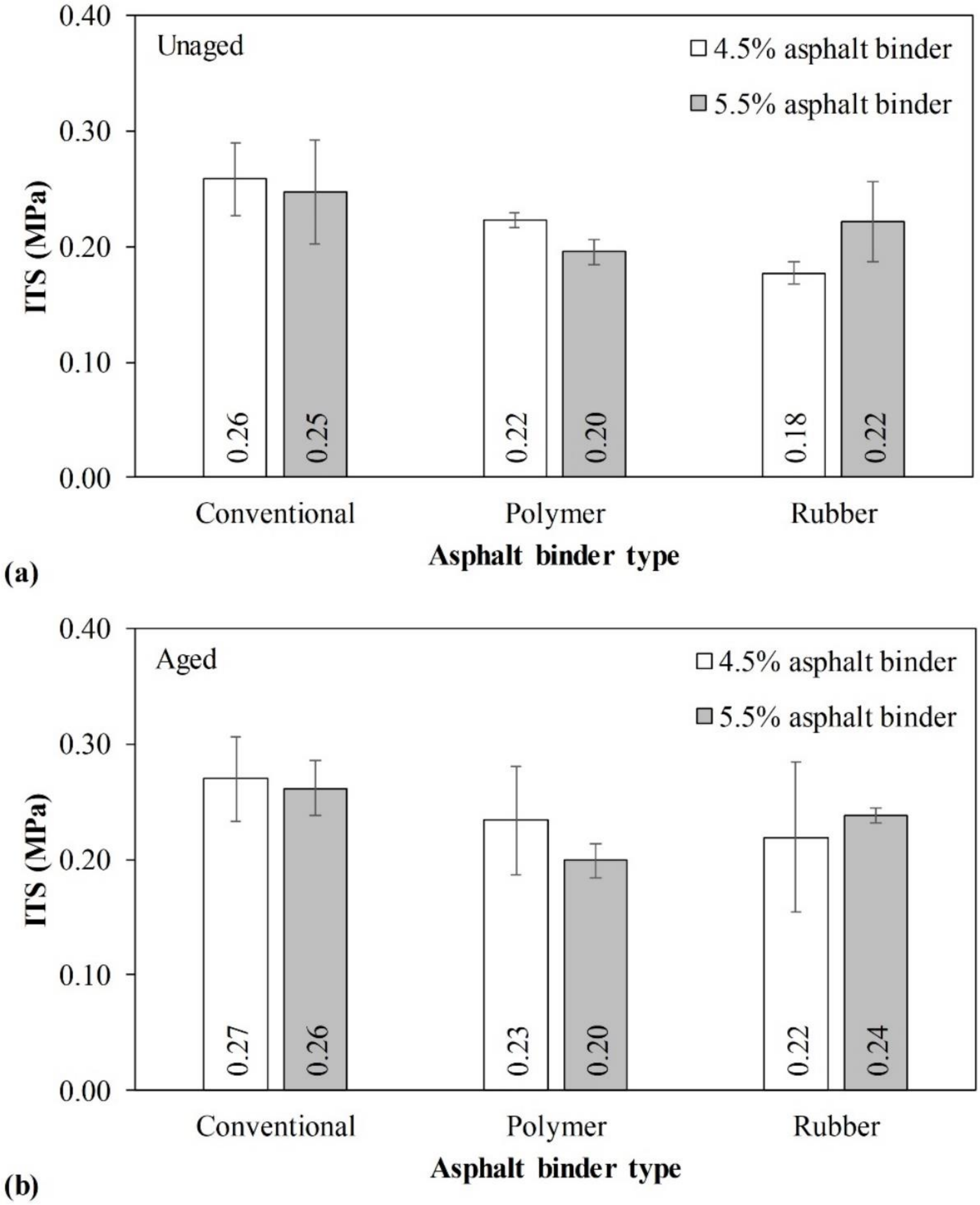


Figure 6. ITS of cold recycled cement-treated mixtures as a function of asphalt binder type and content for (a) unaged and (b) aged conditions

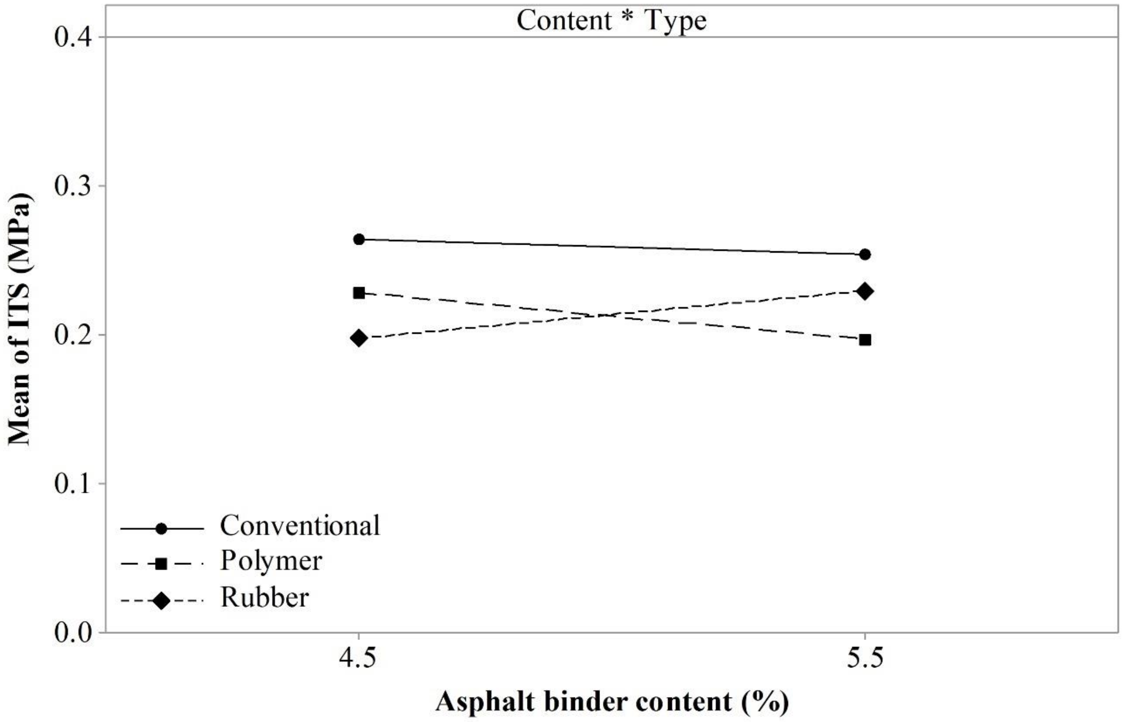


Figure 7. Effect of the interaction between asphalt binder type and content on the ITS mean

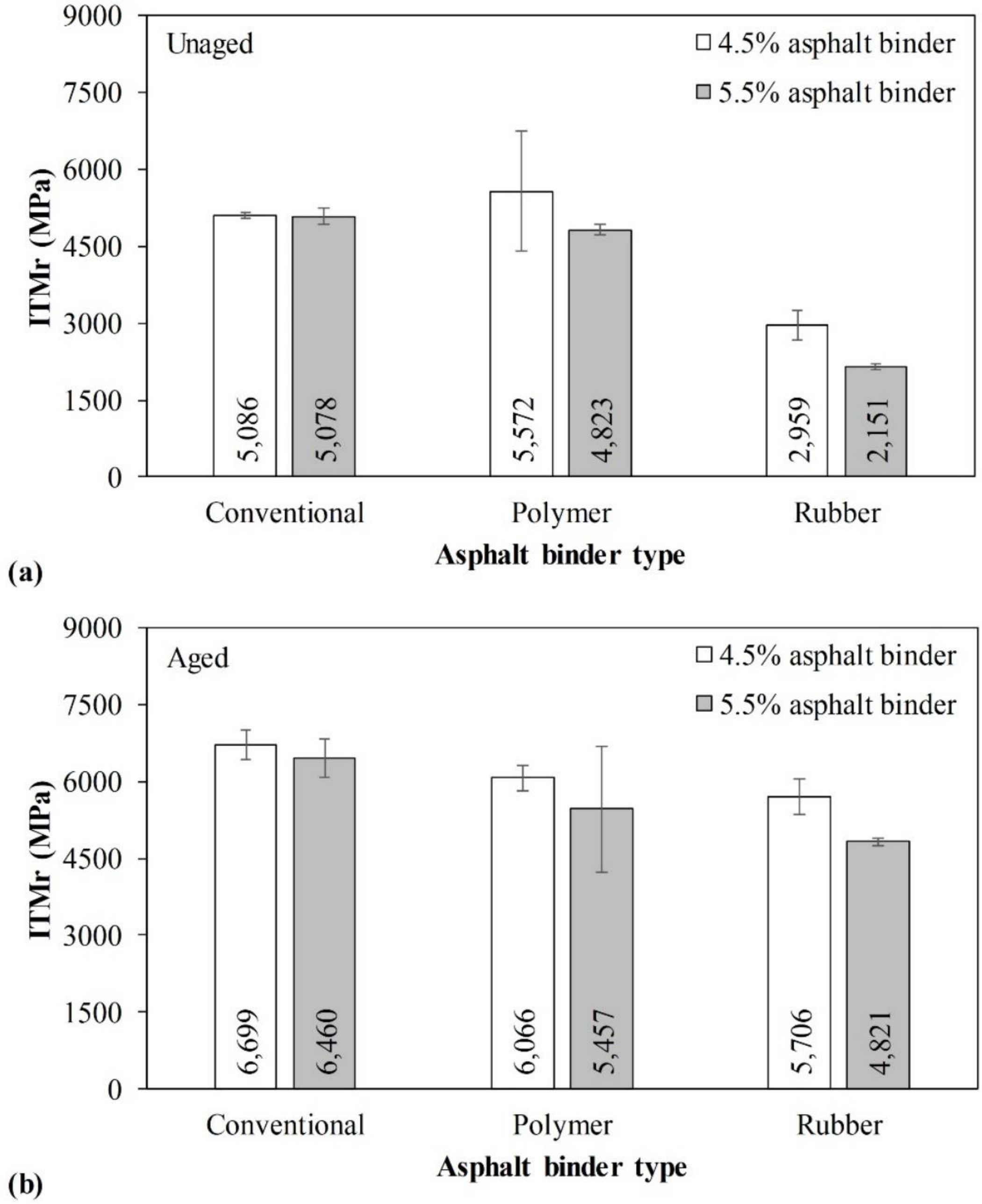


Figure 8. ITMr of cold recycled cement-treated mixtures as a function of asphalt binder type and content for (a) unaged and (b) aged conditions

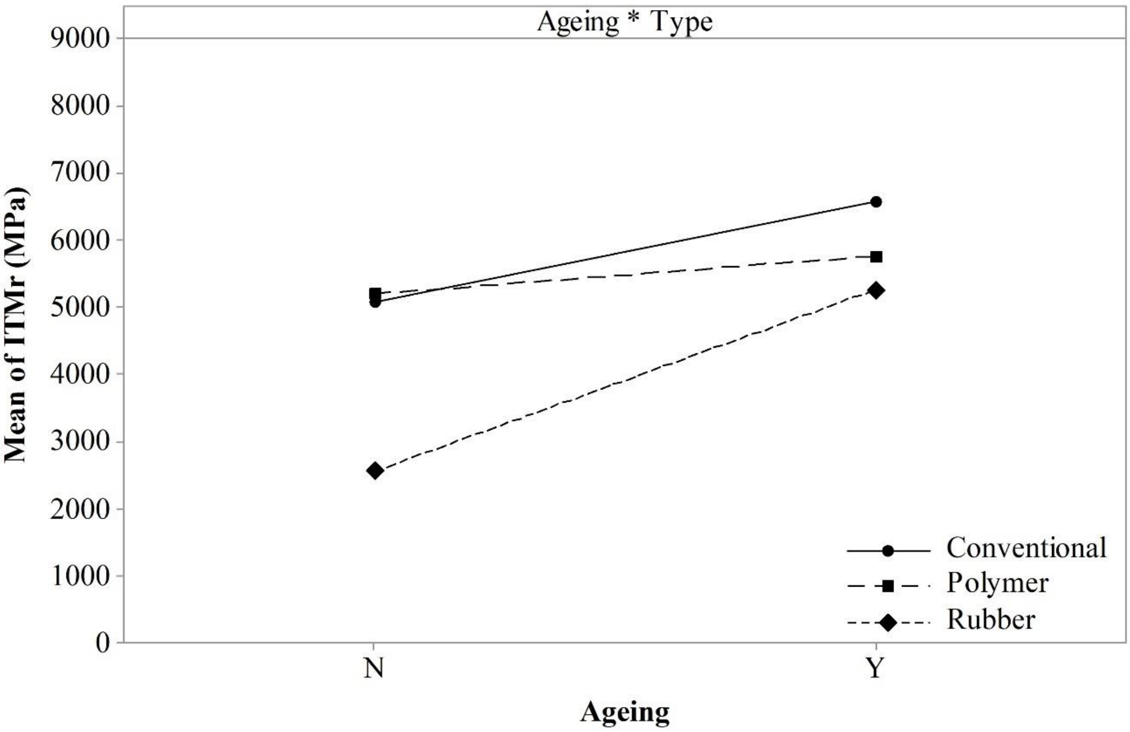


Figure 9. Effect of interaction between asphalt binder type and ageing on the ITMr mean

4 FLEXURAL STRENGTH, STIFFNESS AND FATIGUE OF CEMENT-TREATED MIXTURES OF RECLAIMED ASPHALT PAVEMENT AND LATERITIC SOIL

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Abstract:

This paper evaluates the flexural static and cyclic behaviour of cement-treated mixtures of reclaimed asphalt pavement (RAP) and lateritic soil (LS). Flexural strength and resilient modulus increased with RAP percentage. Strain at break increased with higher RAP and lower cement amounts, increasing mixtures flexibility. Higher cement contents led to stronger and stiffer mixtures. Strain-based fatigue relationships were obtained. Mechanistic analyses showed that the fatigue life of cement-treated recycled base layers of RAP and LS increases with RAP percentage and with the thicknesses of asphalt wearing course and recycled base layer. The effect of cement content depends on the thickness of the layers.

Keywords: pavement recycling; full-depth reclamation with cement; fatigue; flexural behaviour; reclaimed asphalt pavement; lateritic soil

1 Introduction

Asphalt roads of many developed and developing countries are approaching the end of their design lives and require rehabilitation. Most of these countries are also facing lack of natural resources and traditional materials used in civil engineering activities are becoming increasingly scarce. Therefore, more environmental-friendly and cost-effective methods must be considered (Arulrajah et al., 2015; Paige-Green & Ware, 2006). In this scenario, the use of recycled and waste materials in pavement structures became accepted worldwide (Behak & Núñez, 2008) as well as the recycling of damaged pavements.

Full-depth reclamation (or recycling) with portland cement (FDR-PC) is a technique that consists of in situ milling and mixing with cement the existing upper layers of a damaged pavement structure (mainly the asphalt wearing course and the base layer). The resulting mixture is then compacted to form a new layer that will increase the bearing capacity of the upgraded pavement (Katsakou & Kolias, 2007; Portland Cement Association [PCA], 2005). These recycled layers are considered as cement-treated materials and the design requirements in terms of material properties and layer thickness are mainly based on this assumption (Kolias et al., 2001). That is, although these recycled layers initially show little distress, they inherently exhibit fatigue deterioration under cyclic loading and may rapidly deteriorate once distress initiates. Therefore, fatigue failure usually is the main design criterion for the long-term performance of pavements with a cement-treated recycled layer (Jitsangiam et al., 2016; Sounthararajah et al., 2018, Sounthararajah et al., 2019).

FDR-PC generates a mixture that differs from regular cement-treated materials, due to the presence of reclaimed asphalt pavement (RAP). RAP consists of a mixture of aggregates coated with asphalt binder and agglomerations of fines bonded together with asphalt binder. The amount of this material in the mixture depends on the asphalt layer thickness and will significantly alter the properties of the recycled layer (Kolias et al., 2001). Studies on FDR-PC or cement-treated materials with RAP were reported worldwide. Authors investigated the mechanical behaviour (Dalla Rosa et al., 2015; Dixon et al., 2012; Grilli et al., 2013a; Mallick et al., 2002a; Mallick et al., 2002b; Mohammadinia et al., 2014; Puppala et al., 2011), durability properties (Fedrigo et al., 2017a; Guthrie et al, 2007; Suddeepong et al., 2018; Yuan et al., 2011) and the field performance of this kind of material (Amarh et al., 2017; Bessa et al., 2015;

Godenzoni et al., 2018; Jones et al., 2015; Santos et al., 2017; Tataranni et al., 2017; Wilson & Guthrie, 2011; Wu et al., 2015).

Some authors reported that the presence of RAP reduces the strength and stiffness of the cement-treated mixture (Adresi et al., 2017; El Euch Khay et al., 2014; Fedrigo et al., 2018; Ghanizadeh et al., 2018; Grilli et al., 2013b; Isola et al., 2013; Ji et al., 2016; Kolias, 1996a; Kolias 1996b; Ma et al., 2015; Taha et al., 2002). However, the effect of RAP on the fatigue behaviour of the mixture is not well known. Therefore, fatigue studies on FDR-PC materials are necessary to allow mechanistic-empirical structural design, but only a few were reported. Some authors stated that the fatigue performance of cement-treated aggregates decreases with increasing RAP percentage (Jiang et al., 2018; Kolias et al., 2001). On the other hand, Castañeda López et al. (2018) concluded that the effect of RAP on the fatigue of these mixtures is more complicated. The authors reported that layers with high RAP percentages can show greater fatigue life than those with low RAP percentages. There are also studies on the fatigue behaviour of recycled pavement materials simultaneously stabilised with cementitious and asphalt binders, such as emulsion or foamed asphalt (Bocci et al., 2010; Bocci et al, 2011; Bocci et al., 2012, Pitawala et al., 2019).

Moreover, lateritic soils (LS) are often used as pavement materials in some regions (especially in tropical countries, due to the lack of high-quality granular materials) and may end up being a constituent of FDR-PC layers (Paiva & Oliveira, 2017; Paiva et al., 2017). However, no fatigue studies on cement-treated mixtures of RAP and LS were reported. Past researches on this kind of mixtures were restricted to testing and modelling unconfined compressive and indirect tensile strengths (Fedrigo et al., 2017b; Paiva & Oliveira, 2017; Suebsuk et al., 2017), which is not enough for design purposes. There are also studies evaluating the resilient modulus of such mixtures using the indirect tensile test (Kleinert et al., 2017), but this test overestimates the stiffness of cement-treated recycled mixtures, not being recommended for design (Fedrigo et al., 2018).

The reported research aimed at evaluating flexural strength, strain at break and flexural modulus (static and resilient) and obtaining fatigue relationships of cement-treated mixtures of RAP and LS, as well as verifying the effects of cement content and RAP percentage on the mentioned properties.

2 Experimental programme

2.1 Materials and specimens

Laboratory tests were performed on mixtures made of cement, LS and RAP (20%, 50% and 70%, in ratio to LS). LS was a clayey lateritic soil (LG'), according to the MCT (Miniature, Compacted, Tropical) methodology (Nogami & Villibor, 1981). LS liquid limit and plasticity index were, respectively, 44% and 12%. The used RAP had 5.3% of asphalt binder in its composition (6.0% in the fine fraction and 4.6% in the coarse fraction). The grain size distribution of the materials and mixtures are presented in Figure 1. Two contents (2% and 4%, based on the dry mass of LS+RAP mixtures) of portland cement with ground-granulated blast-furnace slag addition (Brazilian type CP II E 32) were used.

Figure 1

For comparison purposes, the amounts of cement and RAP adopted in this study were the same as those used by Castañeda López et al. (2018) for cement-treated mixtures of RAP and crushed aggregates (graded crushed stone). The levels of cement content were established based on what is typically used in Brazil for FDR-PC works, while the levels of RAP percentage were defined to simulate the recycling of pavements with both thin and thick asphalt wearing courses.

Modified Proctor compaction tests were undertaken to determine the optimum moisture content (OMC) and the maximum dry unit weight (MDUW) of each mixture to enable the preparation of the specimens. The tests were carried out following Method D described in AASHTO T180-01 (American Association of State Highway and Transportation Officials, 2010). The mixtures were compacted in 5 layers in a mould with approximately 0.0021 m^3. Blows were applied using a hammer of 4.54 kg mass and 457 mm drop height to produce an effort of 2700 kNm/m^3. Figure 2 shows the dry unit weight as a function of moisture content and Table 1 shows the values of OMC and MDUW for each mixture. OMC decreased with RAP percentage and increased with cement content, which is related to fines content in the mixture. The higher the fines content, the higher the specific surface and consequently the amount of water to reach the optimum condition. MDUW increased with RAP percentage since RAP addition leads to well-graded grain size distributions (Figure 1).

Figure 2

Table 1

Prismatic beams (100 mm x 100 mm x 400 mm) were prepared from the mixtures of LS+RAP, with the specified amount of cement and water at OMC (based on the dry mass of solids). The samples of RAP and LS were air-dried until mass constancy. After the samples of RAP, LS, cement and water had been weighted, the solids were mixed, and water was added while continuing mixing. The mixtures were statically compacted in three equal layers to achieve the MDUW using a hydraulic press. To improve the bond between layers, the surface of the compacted layers was carefully scarified. The specimens were cured at room temperature for 28 days in sealed plastic bags to maintain the mixtures moisture content. For each mixture, three specimens were prepared for static tests (18 specimens) and nine were prepared for fatigue (cyclic) tests (54 specimens). The acceptance criteria were: (a) dry unit weight of at least 95% of the MDUW; (b) moulding moisture content deviating by less than 1% of the OMC, and; (c) dimensions of the beams deviating by less than 3%.

2.2 Testing procedures

A 250 kN capacity load testing machine was used for static tests and a pneumatic testing machine capable of applying haversine load pulses was used for fatigue tests. Static and fatigue tests were conducted in a controlled stress mode. Following the methods employed by Castañeda López et al. (2018), testing temperature and relative humidity were maintained at 24±3 °C and 55±15%, respectively. Four-point bending test configuration was used. The mid-span deflection was measured using two linear variable differential transducers (LVDTs), mounted using a yoke arrangement based on JCI SF-4 (Japan Concrete Institute [JCI], 1984). Authors stated that tensile strain values derived from the mid-span deflection are consistent with tensile strains directly measured using strain gauges (Litwinowicz & Brandon, 1994) and with strain data recorded by distributed fibre optic sensing techniques (Sounthararajah et al., 2017). The experimental setting was the same for both static and fatigue tests, as presented in Figure 3. This experimental setting was also used by Castañeda López et al. (2018) and Fedrigo et al. (2018) for the evaluation of fatigue behaviour and resilient modulus of cement-treated mixtures of RAP and crushed aggregates.

Figure 3

2.2.1 Static tests

Flexural strength tests were performed in accordance with the experience of the National Cooperative Highway Research Program (Mandal et al., 2016; Mandal et al., 2018; NCHRP, 2014). A monotonic increase of stress was applied at a rate of 0.69 MPa/min. Eq. (1) was used to calculate the flexural stress. The tensile strain was calculated using Eq. (2). Strain at break (ε_b) corresponds to 95% of the ultimate load to avoid excessive variations and over-optimistic design since it is reported that flexural tensile strains are affected by local micro-cracking (Austroads, 2013; Litwinowicz & Brandon, 1994). Flexural static moduli were determined from the stress-strain relationships presented in Section 3 (secant modulus corresponding to 40% of flexural strength).

$$\sigma_i = \frac{P_i * L}{w * h^2} \tag{1}$$

$$\varepsilon_i = \frac{108 * h * \delta_i * 10^6}{23 * L^2} \tag{2}$$

Where σ_i (MPa) is the flexural stress corresponding to force P_i (N); ε_i (microstrain) is the flexural tensile strain corresponding to LVDTs average displacement δ_i (mm); L is the length between supporting rollers (300 mm), and; w and h are the average width and height of the specimen (mm), respectively.

2.2.2 Fatigue tests

Flexural fatigue tests were based on Austroads experience (Austroads, 2008; Austroads, 2012; Austroads, 2013; González et al., 2013). Specimens were subjected to 5 Hz haversine-shaped cyclic loading without resting periods (Figure 4). The magnitude of the load pulses (stress level, SL) varied within a range of 10% to 40% of the peak load, accordingly to each mixture (Table 2, Section 4). The tests were terminated either after the rupture of the specimen or after one million load cycles. The flexural resilient modulus was calculated based on the ratio of a given flexural stress and the corresponding resilient tensile strain. Initial resilient modulus and initial tensile strain were defined as the average values between the 50th and 100th load pulses.

Figure 4

3 Static tests results

Figure 5 and Figure 6 show the effects of cement content and RAP percentage on flexural strength and strain at break, respectively. The standard deviation is also presented. The average values of strain at break decreased with cement content and increased with RAP percentage; for instance, 70% RAP mixtures were more flexible than 20% RAP mixtures. A possible explanation is that RAP has agglomerations of fine grains bonded together with asphalt binder (mastic), forming a kind of "aggregate" that presents low stiffness (Kolias, 1996a). However, it is emphasised that the standard deviation values were high for the 2% cement mixtures, suggesting similar results for different RAP percentages.

Flexural strength increased with cement content and RAP percentage. Strength increasing with RAP percentage diverges from the literature, which shows that cement-treated recycled pavement materials tend to lose strength with RAP addition. This could be an effect of the increasing in LS percentage, which increased the finer fraction (clay) of the mixtures and consequently their specific surface; the higher the specific surface, the higher the cement content required to bond the particles together. Besides, Figure 1 shows that increasing RAP percentages led to well-graded grain size distributions, increasing strength.

Figure 5

Figure 6

The flexural strength values are comparable to those reported by Castañeda López et al. (2018), whereas the values of strain at break are higher. The mentioned authors studied the flexural properties of cement-treated mixtures of RAP and aggregates using the same equipment and considering the same factors (cement content and RAP percentage) and levels as those used in this study. However, they used a different curing method (humidity chamber at temperature of 21 ^{o}C and relative humidity of 90%).

The normalised flexural stress (ratio of the stress applied during the test to the peak stress) was plotted against the resulting strain and the average graphs are presented in Figure 7. Figure 7 also shows that the mixtures flexibility generally increases with RAP percentage and decreases with cement content. Flexural static modulus results are presented and discussed in Section 4.1.

Figure 7

4 Fatigue tests results

Table 2 summarises the fatigue tests results. The number of beams tested for each mixture is presented, as well as the ranges of stress level (% of the peak stress), cycles to failure, initial resilient modulus and initial tensile strain. Some specimens failed quite prematurely, and the test data could not be collected.

Table 2

4.1 Flexural resilient and static moduli

Figure 8 presents the flexural initial resilient and static moduli as a function of cement content and RAP percentage. For each mixture, the resilient modulus values are the average of all tested specimens. The mixtures moduli increased with cement content, a trend also observed in other studies on cement-treated recycled pavement materials (Castañeda López et al., 2018; Fedrigo et al., 2018; Grilli et al., 2013b). The resilient modulus also increased with RAP percentage, which could be related to the same reason that was highlighted for flexural strength (Section 3). However, the static modulus was similar for different RAP percentages and, consequently, the influence of RAP percentage did not show a well-defined trend. In general, the moduli obtained in this study are lower than those reported by Castañeda López et al. (2018), which used the same experimental program to evaluate the flexural behaviour of cement-treated mixtures of RAP and crushed aggregates. These lower moduli may be related to the high values of strain at break showed by specimens of cement-treated RAP+LS, which allow high deformation levels before failure. Moreover, most of the values of modulus obtained in this study are within the range of flexural resilient modulus reported by Biswal et al. (2018a; 2018b) for cement-treated lateritic soils without RAP addition.

Figure 8

Figure 9 shows the relationship between flexural initial resilient modulus and flexural static modulus; a regression model and the correspondent coefficient of determination (R^2) are also presented. Although some mixtures showed similar modulus under static and cyclic loading, the obtained model has very low accuracy. Relationships between moduli (static and resilient) and flexural strength are presented in Figure 10. One of the regression models presented in Figure 10 allows estimating the flexural resilient modulus using flexural strength results for similar mixtures with high accuracy, which could be useful for design purposes since static tests are more common and cheaper to execute than cyclic tests. However, a poor relationship is observed between the flexural static modulus and flexural strength, though the two properties were obtained using the same test condition.

Figure 9

Figure 10

4.2 Failure characteristics

The fatigue progressive failure was recorded for all specimens. Some observed damage evolution patterns are presented in Figure 11 in terms of normalised resilient modulus (ratio of the resilient modulus at any load cycle to the initial resilient modulus) reduction. The graphs presented in Figure 11 correspond to specimens of a mixture with 4% of cement and 70% of RAP, but similar trends were observed for other mixtures. No visible cracks were observed in the specimens until the imminence of failure. The typical pattern of conventional cement-treated materials showing three damage phases (González et al., 2013; Jia et al., 2018; Mandal et al., 2016; Mandal et al., 2018) was only observed for mixtures subjected to the lower stress levels used in this study (10-15% of the peak load). For the others, the stiffness reduced according to a nearly constant rate from the beginning until end of the test. Fedrigo et al. (2019) observed similar trends for lightly cement stabilised materials.

Figure 11

Specimens subjected to the higher stress levels failed with higher normalised resilient modulus than those subjected to the lower stress levels, which failed following the usual half initial modulus condition. Failures before the attainment of half initial modulus were reported in similar studies (González et al., 2013; Gnanendran & Paul, 2016). Moreover, this kind of effect of the stress level on the modulus degradation is supported by Nguyen et al. (2018), which simulated flexural fatigue tests of cement-treated materials using discrete element modelling. On the other hand, Jia et al. (2018) stated that the stiffness at break is not dependent on the stress level.

It is highlighted that the stress levels applied in this study are lower than those often used for fatigue testing of cement-treated materials, which are reported to be within 60-100% (Arulrajah et al., 2015; Disfani et al., 2014; González et al., 2013; Jia et al., 2018; Jitsangiam et al., 2016; Kolias et al., 2001; Mandal et al., 2016; Mandal et al., 2018; Nascimento & Albuquerque, 2018; Sounthararajah et al., 2018). However, the studied mixtures were highly sensitive to stress levels as high as 60%, failing after a few cycles. In this regard, authors stated that the micro-cracking in flexural tests starts at about 35% of the peak load (Otte, 1972; Paige-Green, 2014) and that a slight increase in the applied stress level may lead to an abrupt failure of specimens (Jitsangiam et al., 2016).

It is also acknowledged that cement-treated materials are always susceptible to shrinkage and that fatigue cracking often initiates near the shrinkage cracking. The fact that the specimens were not cured in high humidity and temperature-controlled environment may have caused micro-cracking due to drying shrinkage, consequently reducing the strength and stiffness of some specimens (and the allowable stress level to be applied). High drying shrinkage strains were measured for the studied mixtures (> 0.2%) and reported for cement-treated lateritic soils without RAP addition (Biswal et al., 2018c). Even though, it is highlighted that, in the field, shrinkage cracking usually starts at the top of the layer (there is less variation of moisture content at the bottom of the layer), while fatigue cracking initiates at the bottom of the layer.

Moreover, it may be possible that some mixtures showed low fatigue resistance due to inadequate cement content, which was not enough to produce bound materials with fatigue properties (González et al., 2013). This can be especially true for mixtures with high percentages of LS, due to the higher specific surface. In this case, the main design criterion for a pavement layer made of the studied mixtures could be the crushing at its top instead of the fatigue at its bottom (De Beer, 1990; Litwinowicz & De Beer, 2013; Theyse et al., 1996).

4.3 Laboratory fatigue life relationships

Several authors expressed the fatigue life of cement-treated materials as a function of the strain ratio (González et al., 2013; Otte, 1978; Theyse et al., 1996). Authors also reported that strain-based models provide a better prediction of the fatigue life than stress-based models (Fedrigo et al, 2019; Pretorius, 1970). Stress- and strain-based fatigue relationships were obtained for the studied mixtures, but only strain-based relationships are presented since they showed higher accuracy (R^2), following the trends reported by previous studies. The strain-based fatigue relationships, in which the fatigue life (N) is a function of the ratio of the initial tensile strain (ε_i) to the strain at break (ε_b), and the corresponding coefficients of determination (R^2) are presented in Figure 12.

Figure 12

The effects of cement content and RAP percentage on the fatigue life parameters did not show a well-defined trend. The studied mixtures showed to be highly sensitive to strain; that is, a slight increase in the initial strain may greatly reduce the mixtures fatigue life. The values of the fatigue parameters are close to those recommended by Austroads (Austroads, 2008; Austroads, 2012; Austroads, 2013) for designing conventional cement-treated layers (without RAP). These values are also similar to those obtained by Castañeda López et al. (2018) for cement-treated mixtures of RAP and aggregates.

4.4 Mechanistic pavement analysis

The Everstress layered elastic analysis software was used to compute the tensile strains at the bottom of cement-treated recycled base layers made of the studied mixtures. The obtained tensile strains were then used as input (ε_i) in the fatigue relationships shown in Figure 12. A typical four-layered structure was used (Table 3). Two 20 kN-wheel loads and contact pressure of 560 kPa were considered. Figure 13 shows the fatigue life of the mixtures as a function of the thicknesses of asphalt wearing course and recycled base layer.

Table 3

Figure 13

Generally, the fatigue life of the recycled base layers increased with recycled base thickness, with asphalt wearing course thickness and with RAP percentage. The fatigue life of mixtures with 20% of RAP increased with the cement content. The effect of cement content on the fatigue life of mixtures with 50% and 70% of RAP seems to depend on the thickness of the layers. For instance, mixtures with 50% of RAP overlaid by a 50 mm thick asphalt layer showed higher fatigue lives with lower cement content (2%) for base layers thinner than 300 mm, while for thicker base layers the fatigue life was higher with 4% of cement. Similar conclusions were drawn by Castañeda et al. (2018) for cement-treated layers of RAP and aggregates.

The analyses showed that a 400 mm thick base layer with 4% of cement and 70% of RAP, overlaid by 100 mm of asphalt wearing course performs well, carrying about 100,000 cycles, and making clear that pavements with thick asphalt layers might be in situ recycled with cement. Sounthararajah et al. (2018) reported that 400 mm thick base layers of aggregates treated with 4% of cement can carry more than a million cycles. However, pavement removal and reconstruction would probably generate higher costs and more environmental issues than in situ recycling.

5 Conclusions

The following are conclusions based on the data analysed in this paper:

- The flexural strength of cement-treated mixtures of RAP and LS increases with cement content and RAP percentage. Strength values (0.28-0.96 MPa) are comparable to those of cement-treated mixtures of RAP and aggregates reported elsewhere.
- The strain at break of cement-treated mixtures of RAP and LS decreases with cement content and increases with RAP percentage. Strain at break values (273-1,059 microstrain) are generally higher than those of cement-treated aggregates with or without RAP reported elsewhere.
- The flexural modulus of cement-treated mixtures of RAP and LS increases with cement content independently of the loading condition (static and cyclic) and the resilient modulus increases with RAP percentage. Stiffness values (983-4,163 MPa) are generally lower than those of cement-treated aggregates with or without RAP reported elsewhere.
- A model that allows estimating the flexural resilient modulus using flexural strength results with high accuracy was obtained. This is important for practical use since static tests are more common and cheaper to execute than cyclic tests.
- Generally, strength, stiffness and flexibility of cement-treated mixtures of RAP and LS increase with RAP percentage. Therefore, flexible pavements with LS base layers and thick asphalt wearing courses could be in situ recycled without compromising mechanical behaviour.

- The fatigue progressive failure of cement-treated mixtures of RAP and LS seems to be affected by the stress level. Specimens subjected to high stress levels tend to fail with high stiffnesses, while those subjected to low stress levels fail following the usual half initial modulus condition. The low fatigue resistance of some mixtures could be explained by insufficient cement contents or by micro-cracking due to drying shrinkage. Adequate mix design and field curing by maintaining the moisture condition could improve the recycled pavement performance.
- Fatigue relationships for cement-treated mixtures of RAP and LS were reported for the first time. The exponent values of the fatigue relationships (6.65-21) are comparable to those of cement-treated aggregates with or without RAP reported elsewhere.The fatigue life of this kind of mixture is highly dependent on the tensile strain, emphasising the importance of using mechanistic-empirical methods for FDR-PC design.
- Mechanistic analyses indicate that the fatigue life of cement-treated recycled base layers of RAP and LS increases with the thicknesses of asphalt wearing course and recycled base layer. The effect of cement content depends on the thickness of the layers. Mixtures with a high RAP percentage generally perform better than those with a low percentage, confirming that pavements with thick asphalt layers could be in situ recycled.

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Cement content (%)	RAP percentage (%)	OMC (%)	MDUW (kN/m³)
2	20	14.2	17.1
2	50	8.3	19.3
2	70	7.1	20.7
4	20	14.2	17.4
4	50	10.9	19.6
4	70	10.1	20.1

Table 1. Compaction parameters of the studied cement-treated mixtures of RAP+LS

Cement (%) + RAP (%)	Lowest (L) and highest (H) values of								Number of beams that		
	stress level (%)		cycles to failure		initial resilient modulus (MPa)		initial strain (microstrain)		were tested	failed prematurely	did not fail
	L	H	L	H	L	H	L	H			
2 + 20	15	25	99	1,000,000	862	1508	29	77	9	2	1
2 + 50	10	40	99	1,000,000	1111	2010	25	99	9	2	1
2 + 70	10	25	99	325,299	1714	3382	34	83	9	0	0
4 + 20	10	20	99	1,000,000	1777	2538	25	65	9	1	1
4 + 50	10	20	199	1,000,000	3083	4143	23	55	9	1	1
4 + 70	13	25	299	407,299	3620	5054	29	43	9	1	0

Table 2. Summary of fatigue tests results

Material	Thickness (mm)	Resilient Modulus (MPa)	Poisson's ratio	Unit weight (kN/m^3)
Asphalt wearing course	50; 100	4500	0.30	23.5
Recycled base layer	180; 200; 250; 300; 350; 400	Initial resilient modulus*	0.17**	MDUW*
Granular subbase	150	$RM = 200(\theta/\sigma_{atm})^{0.6}$	0.40	20.0
Subgrade	-	100	0.35	18.0

*Values for each cement-treated mixture of RAP and LS (Figure 8 and Table 1); **Kleinert et al. (2017)

Table 3. Materials and thicknesses used in the mechanistic pavement analyses

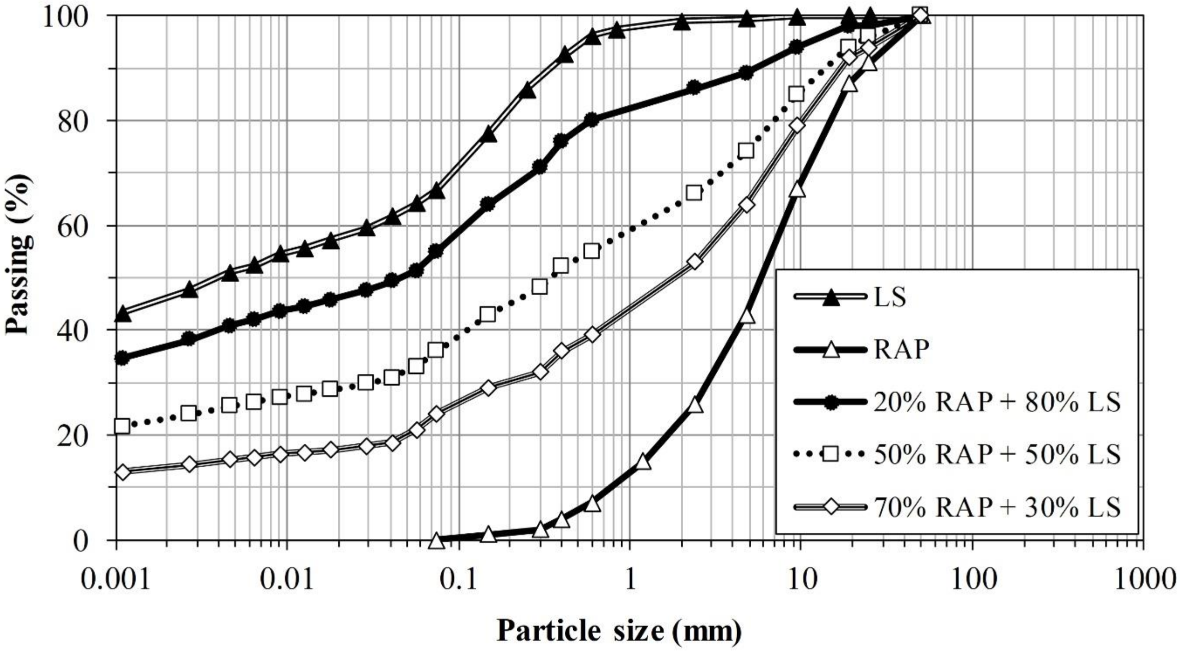


Figure 1. Grain size distributions of the materials (RAP and LS) and mixtures of RAP and LS

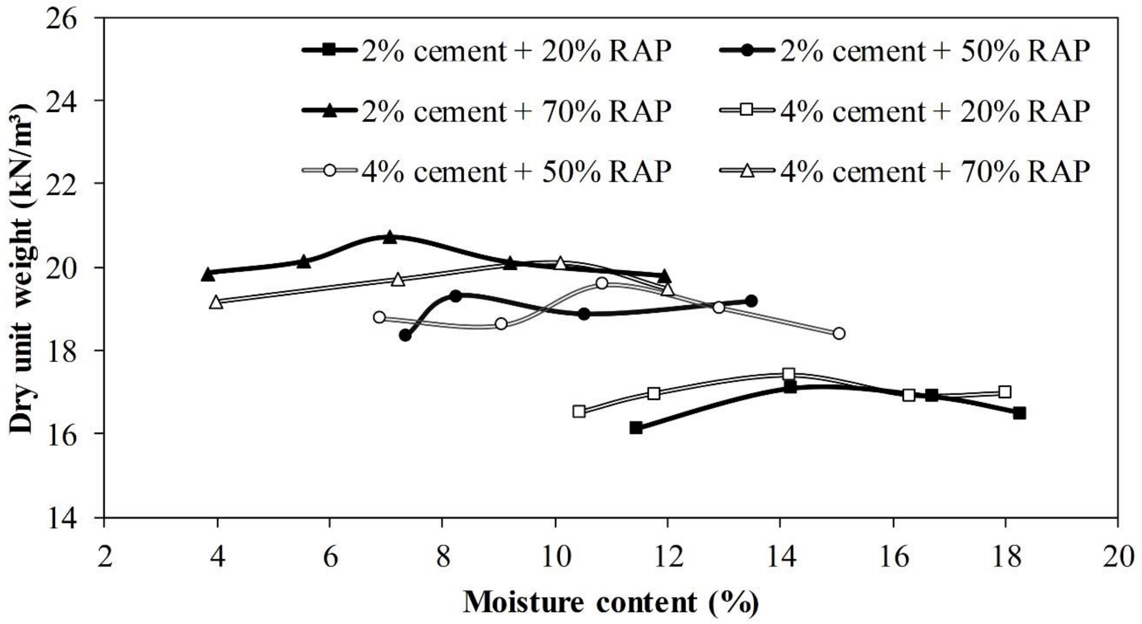


Figure 2. Compaction curves of the cement-treated mixtures of RAP and LS

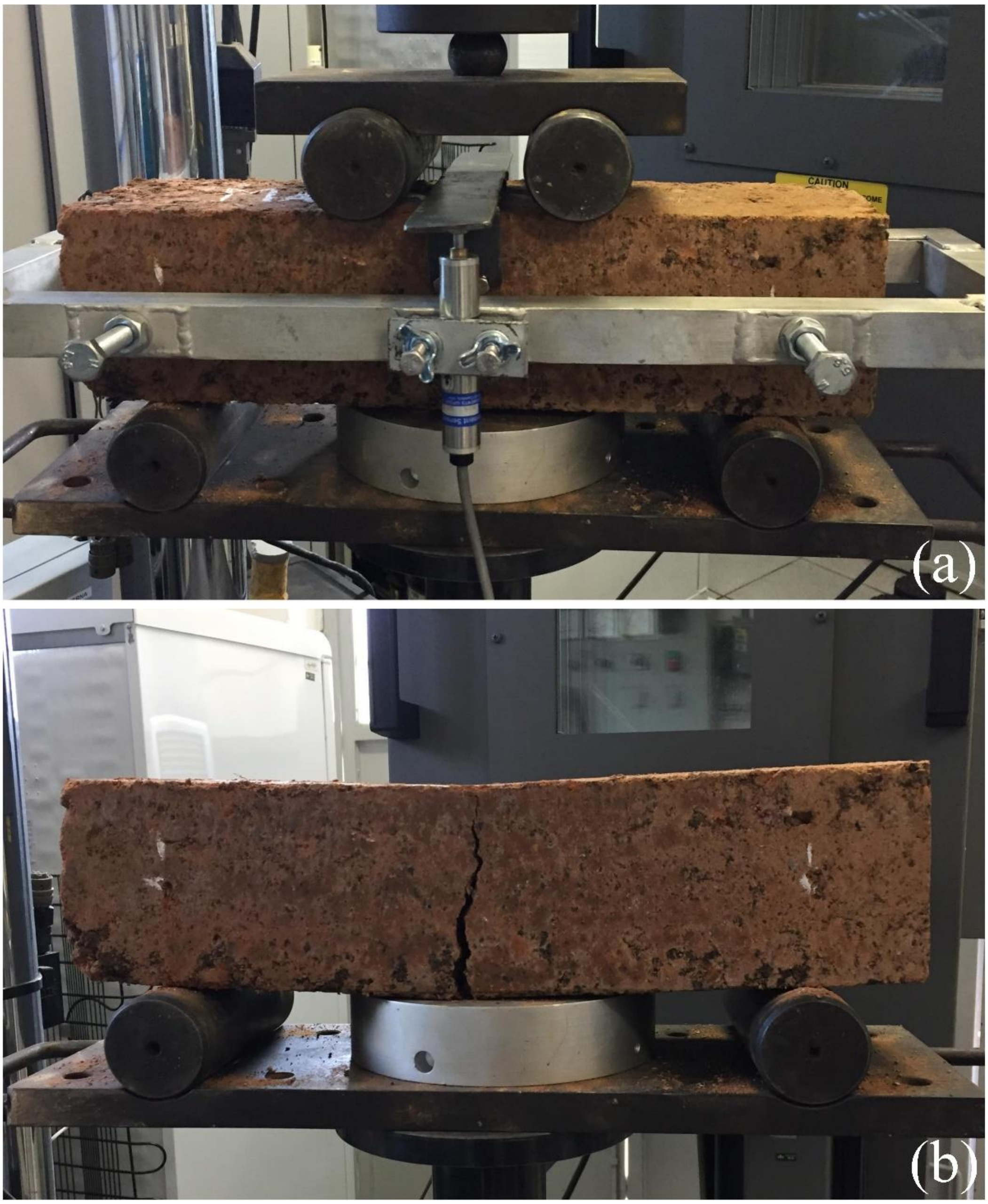


Figure 3. Flexural tests: (a) four-point bending configuration and (b) specimen failure

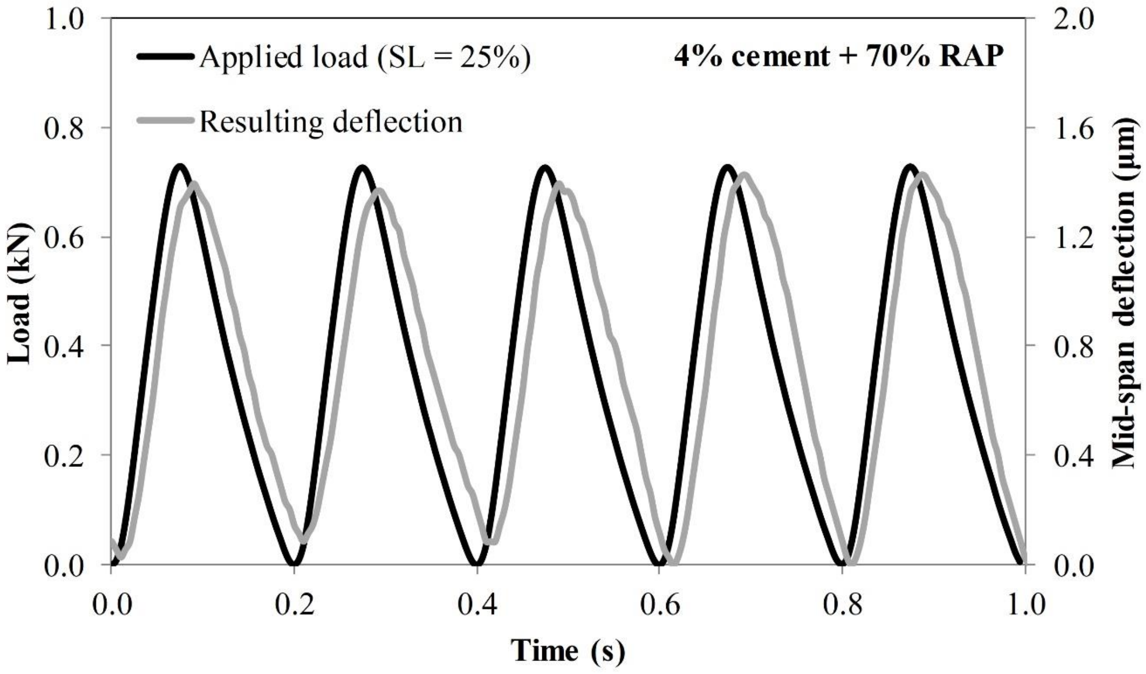


Figure 4. Flexural cyclic load pulse

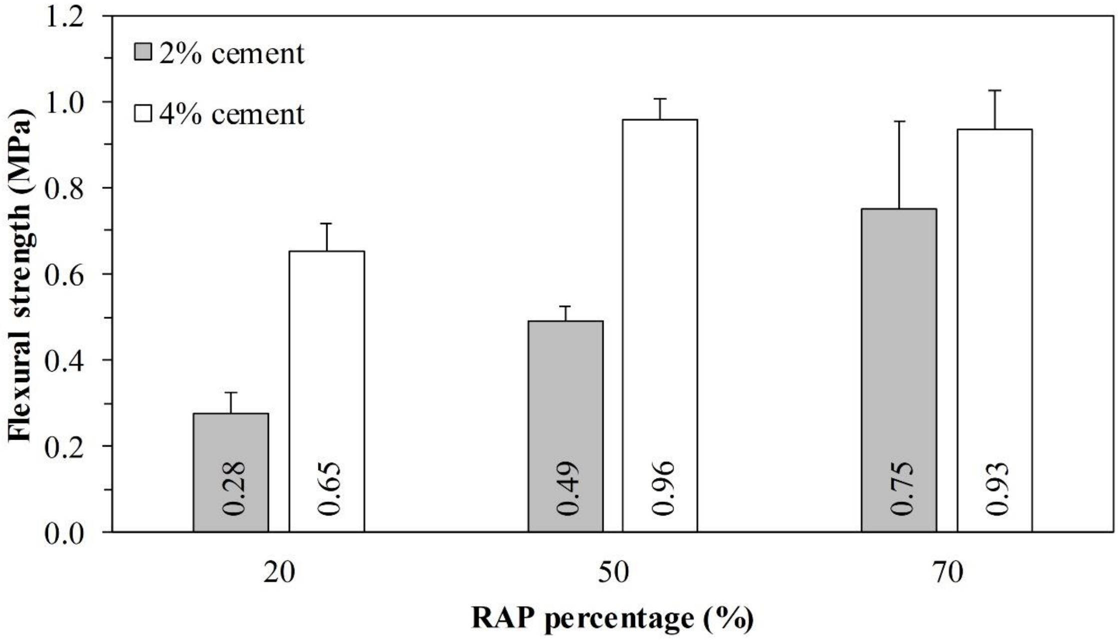


Figure 5. Effects of cement content and RAP percentage on the flexural strength

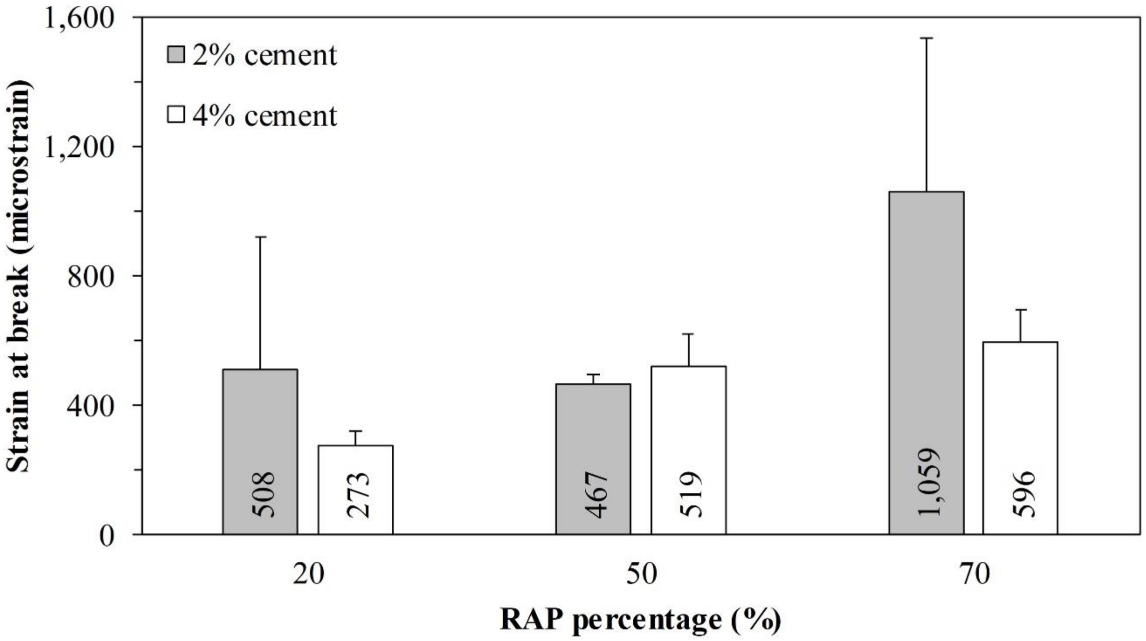


Figure 6. Effects of cement content and RAP percentage on the strain at break

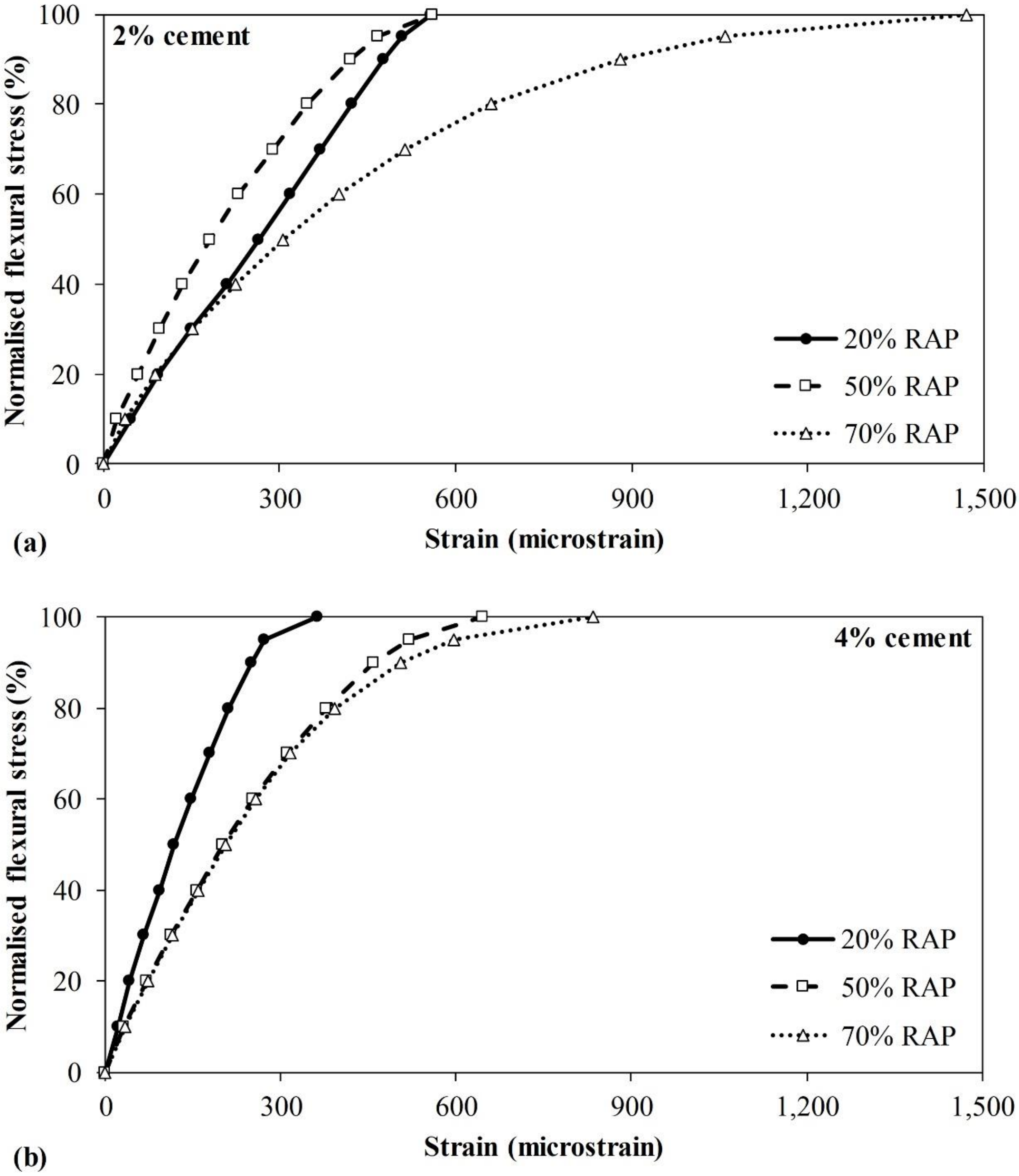


Figure 7. Stress-strain relationships of mixtures with cement contents of (a) 2% and (b) 4%

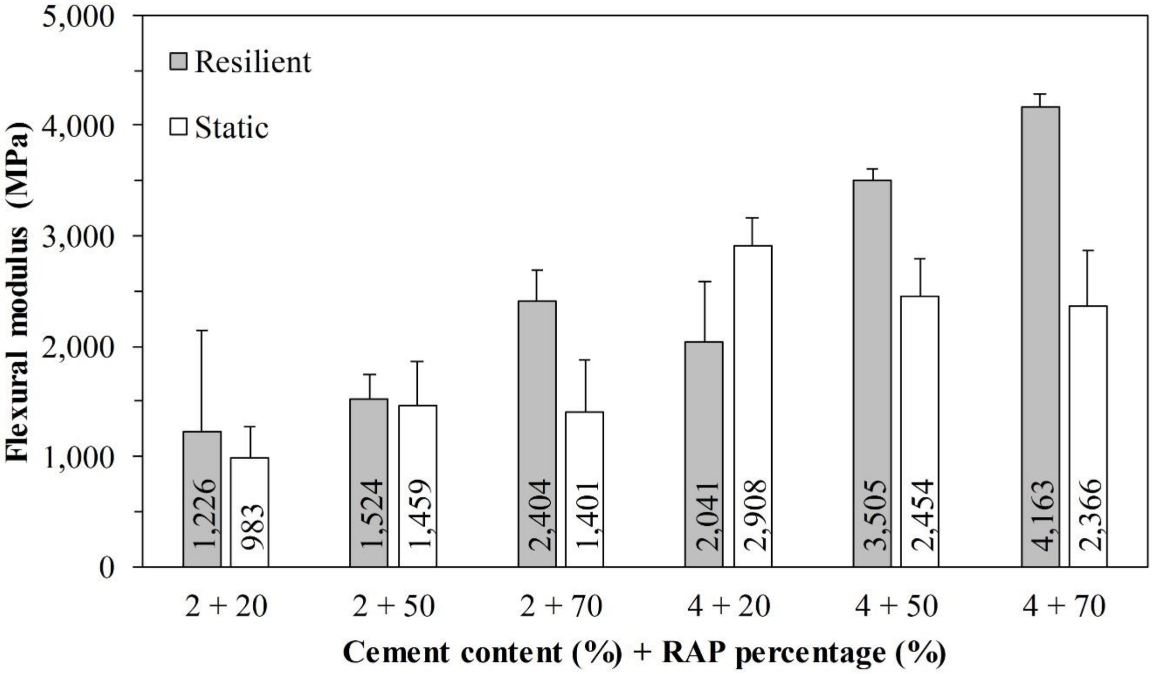


Figure 8. Resilient and static moduli as a function of cement content and RAP percentage

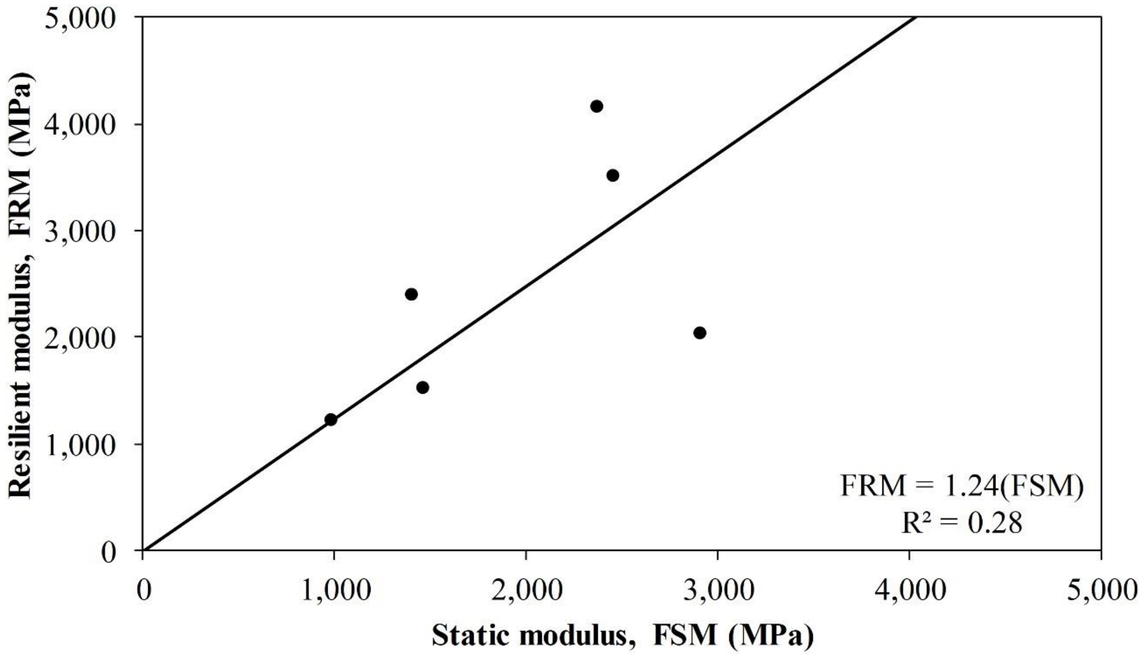


Figure 9. Relationship between flexural resilient and static moduli

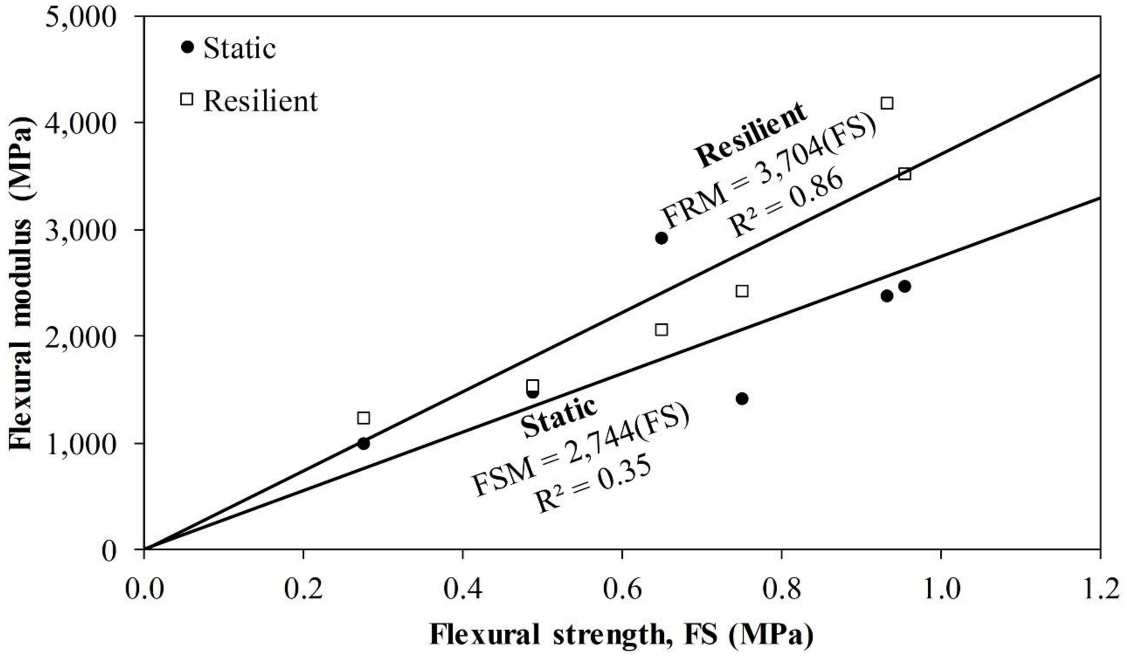


Figure 10. Relationships between flexural moduli (resilient and static) and flexural strength

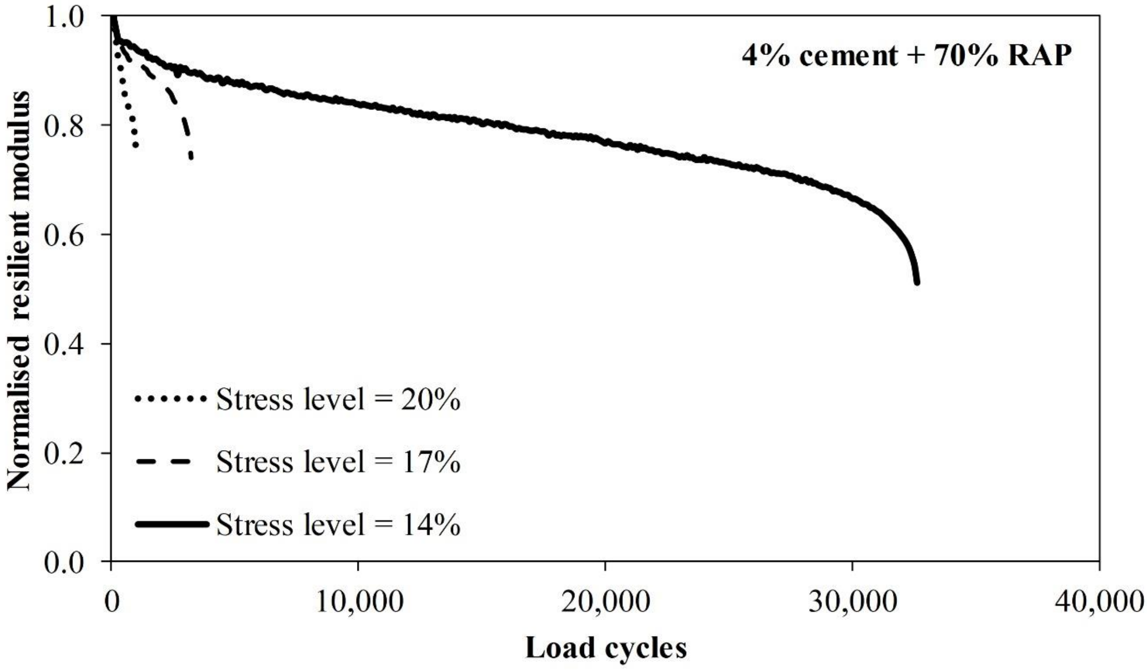
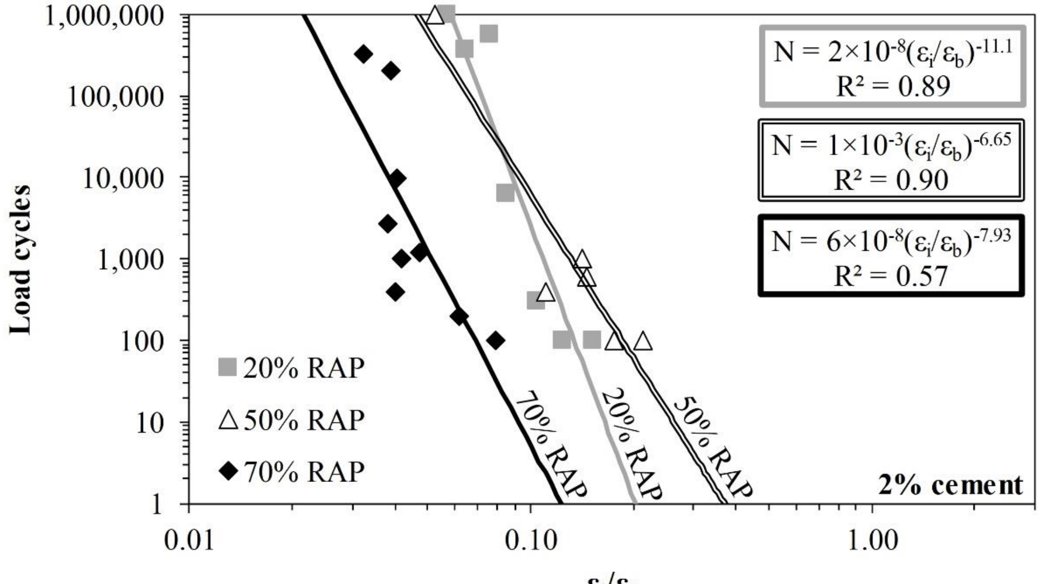
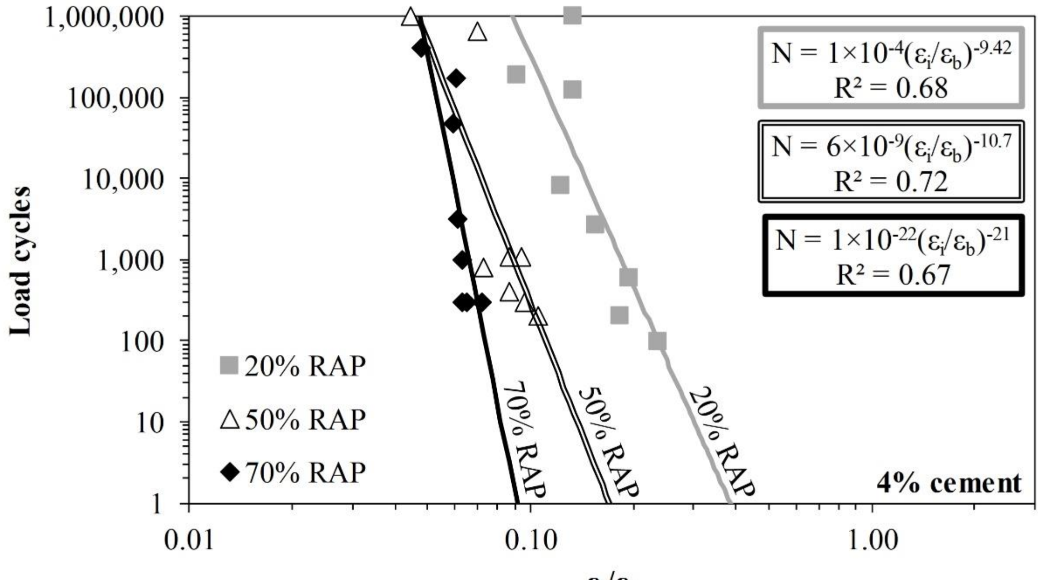


Figure 11. Damage evolution patterns for different stress levels



(a)



(b)

Figure 12. Strain-based fatigue relationships for mixtures with cement contents of (a) 2% and (b) 4%

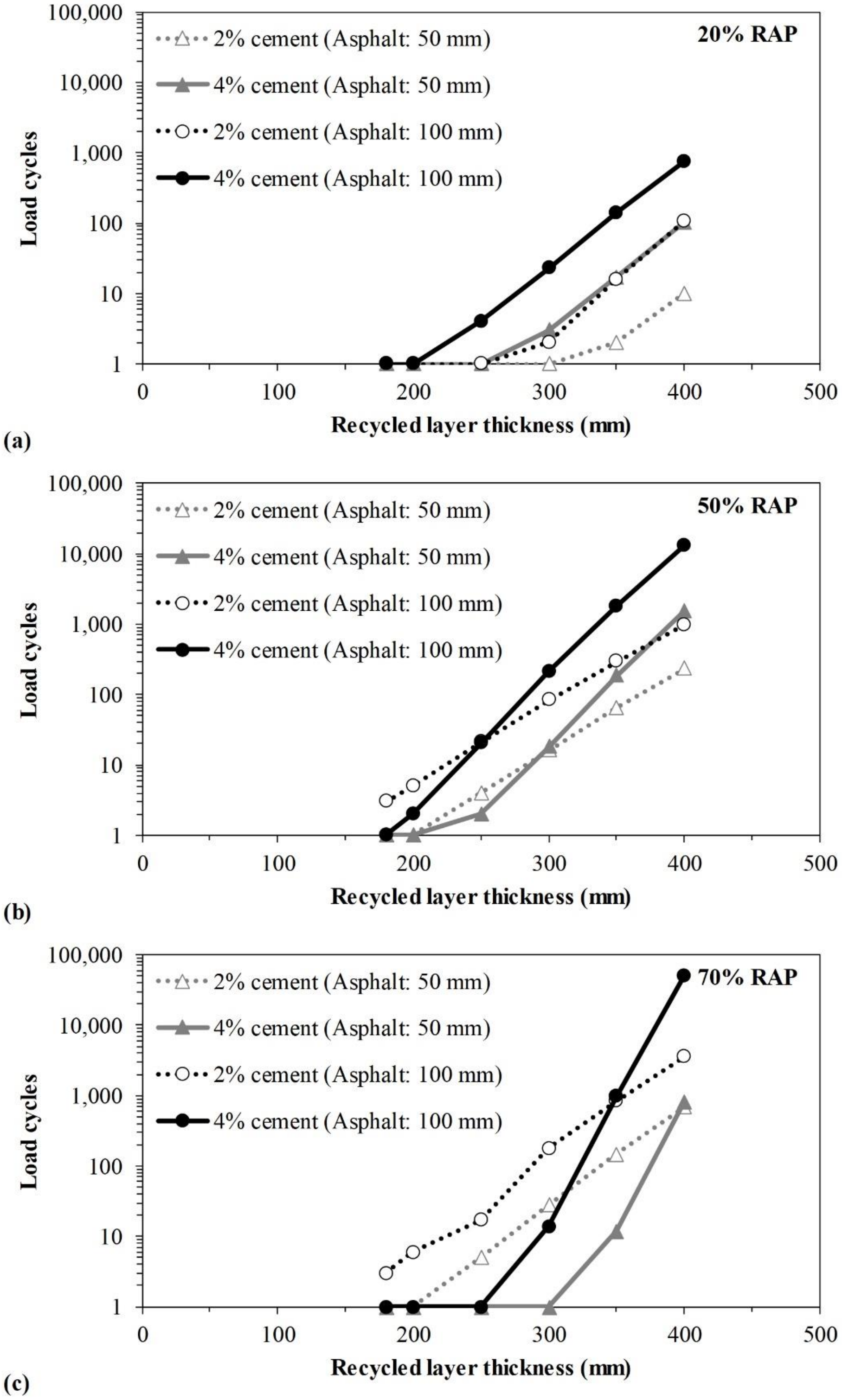


Figure 13. Fatigue lives as a function of the thicknesses of asphalt wearing course and recycled base layer: (a) 20% RAP, (b) 50% RAP and (c) 70% RAP

5 FLEXURAL BEHAVIOUR OF LIGHTLY CEMENT STABILISED MATERIALS: SOUTH AFRICA AND BRAZIL

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Abstract:

This paper focuses on the flexural behaviour of lightly cement stabilised materials. An experimental programme was developed to analyse the effects of compaction method, test setup and displacement rate on the static behaviour, to characterise the laboratory fatigue behaviour of South African materials and to compare South African and Brazilian materials. Similar flexural properties were achieved using vibration and compression compaction methods. The South African setup leads to lower strength and stiffness values, but to a better prediction of the field modulus. The static properties are not affected by the displacement rate. The strain at break is not affected by any tested variable, being the most meaningful property for design. Strain-based models provide a better fatigue life prediction than the stress-based models. Laboratory models lead to a fatigue life shorter than the South African field transfer functions. Similarities were observed between South African and Brazilian materials, which suggests comparable fatigue performances.

Keywords: lightly cement stabilised material; flexural behaviour; strain at break; fatigue; South Africa; Brazil

1 Introduction

Lightly cement stabilised materials (LCSM) are being increasingly used in the construction and rehabilitation of pavements due to the increase in traffic demand and load (Sounthararajah et al., 2017). These materials are the product of soil stabilisation with cementitious binders (e.g. cement and lime) that outperform granular materials and undergo fatigue when subjected to traffic loading. Thus, fatigue is the main design criteria for long-term performance of cement stabilised layers (Jitsangiam et al., 2016).

The four-point bending test (4PBT) is often used to characterise the behaviour and to obtain design parameters of LCSM. The test measures the stress states to which stabilised layers are subjected, such as the development of tensile and compressive stresses at the bottom and top of the beam, respectively (Fu et al., 2009). Although researchers have investigated the flexural behaviour of LCSM since the 1970s (Otte, 1972; Otte, 1978; Pretorius, 1970), no universal test method was established. The method used to perform the tests can affect the results and consequently the pavement design. For instance, results obtained in South Africa may greatly differ from those obtained in Brazil, even if the same materials are tested.

In this regard, a literature review of previous studies showed that the main differences between flexural test methods for LCSM are: (a) the compaction method used to produce the beams; (b) the test setup (i.e. beam section and span between support rollers); and (c) the static test mode (i.e. under load, stress or displacement control) and the rate of load application (speed). Furthermore, no laboratory fatigue characterisation of South African materials was found in the literature. The South African fatigue models were based on accelerated pavement testing (APT), while most of the fatigue models used worldwide (e.g. Brazil) were developed essentially based on flexural cyclic tests.

Considering the mentioned differences between the test methods for flexural characterisation of LCSM used in South Africa and internationally, especially in Brazil, the objectives of the research reported were: (a) to analyse and quantify the effects of compaction method, test setup and displacement rate on the flexural static behaviour; (b) to characterise the laboratory fatigue behaviour of South African LCSM; and (c) to compare the flexural behaviour of South African and Brazilian LCSM.

2 Literature review

2.1 Compaction method

Several compaction methods to produce specimens for flexural tests are reported in the literature. In South Africa, vibratory table compaction in conjunction with surcharges placed over the material was used in the past (Otte, 1972; Otte, 1978). This method was based on the work by Pretorius (1970) and the same kind of compaction method was recently used by other authors (Linares-Unamunzaga et al., 2018), although vibrating hammers are preferred nowadays (Farhan et al., 2019; Farhan et al., 2015; Katsakou & Kolias, 2007; Kolias, 1996a; Kolias, 1996b; Kolias et al., 2001; Mbaraga et al., 2014). In a way similar to what is done in the United States (Mandal et al., 2018; Mandal et al., 2016; National Cooperative Highway Research Program, NCHRP, 2014) and elsewhere (Biswal et al., 2018a; Biswal et al., 2018b; Thichês, 1993), some South African researchers also used the modified AASHTO hammer (Liebenberg, 2002; Liebenberg, 2003; Liebenberg & Visser, 2003; Liebenberg & Visser, 2004; Mgangira, 2011; Robroch, 2002) and even a protocol was developed (Council for Scientific and Industrial Research, CSIR, 2011). In Brazil, beams are produced by means of press compression (Castañeda López et al., 2018; Fedrigo et al., 2018; Nascimento & Albuquerque, 2018, Paiva et al., 2017), mainly based on the Australian experience (Arulrajah et al., 2015; Disfani et al., 2014; González et al., 2013).

No study regarding the effect of the compaction method on the flexural behaviour of LCSM was found in the literature. However, Otte (1972) reported that beams produced using vibratory table compaction have the same density as those produced using press compaction but are more homogeneous and stiffer (ultrasonic test). The author then suggested that vibratory compaction is more representative of field compaction. This statement was confirmed by other authors, although based only on unconfined compressive strength (UCS) tests (Ji et al., 2019; Jiang & Fan, 2013).

2.2 Test setup

Most commonly a 100 mm × 100 mm section beam and a 300 mm span between support rollers was used for the test setup (Farhan et al., 2019; Farhan et al., 2015; Jia et al., 2018; Katsakou & Kolias, 2007; Kolias, 1996a; Kolias, 1996b; Kolias et al., 2001; Xiao et al., 2017). Recent Brazilian studies used the same test setup (Castañeda López et al., 2018; Fedrigo et al., 2018; Nascimento & Albuquerque, 2018; Paiva et al. 2017) in accordance with the Australia and

United States experience (Arulrajah et al., 2015; Austroads, 2008; Disfani et al., 2014; González et al., 2013; Litwinowicz & Brandon, 1994; Mandal et al., 2018; Mandal et al., 2016; NCHRP, 2014). On the other hand, the South African setup differs from the rest of the world, using a 75 mm × 75 mm section beam and a span of 420 mm (CSIR, 2011; Liebenberg, 2002; Liebenberg, 2003; Liebenberg & Visser, 2003; Liebenberg & Visser, 2004; Mgangira, 2011; Otte, 1972; Otte, 1978; Robroch, 2002). Its development was based on the following (Otte, 1972): (a) the ratio of span to depth (height) must not be less than 5; (b) the ratio of the minimum beam dimension to the maximum aggregate size should exceed 3; and (c) reduction of the influence of shear force on the deflection.

Mbaraga et al. (2014) reported that flexural strength and stiffness increase with the use of wide spans and small heights, as well as if smaller aggregates are used as the maximum particle size. This agrees with the statements made by Otte (1972), although the South African setup was not compared with others reported elsewhere. It is worth emphasising that tests using 75 mm × 75 mm (3 inches × 3 inches) section beams were reported elsewhere, including in Brazil, but in conjunction with different spans (Biswal et al., 2018a; Biswal et al., 2018b; Ceratti, 1991; Gnanendran & Paul, 2016; Paul & Gnanendran, 2012; Paul & Gnanendran, 2015; Paul et al., 2015; Pretorius, 1970).

2.3 Test mode and rate of application

The flexural static test mode (i.e. under load, stress or displacement control) and the rate of load application (speed) also differ from one region to another. South African studies were performed using displacement control and the displacement rates varied from 1.0 to 1.8 mm/min (CSIR, 2011; Liebenberg, 2002; Liebenberg, 2003; Liebenberg & Visser, 2003; Liebenberg & Visser, 2004; Mbaraga et al., 2014; Mgangira, 2011; Otte, 1972; Otte, 1978; Robroch, 2002). In the rest of the world, tests are often performed using load or stress control (Arulrajah et al., 2015; Austroads, 2008; Ceratti, 1991; Disfani et al., 2014; González et al., 2013; Jitsangiam et al., 2016; Katsakou & Kolias, 2007; Kolias, 1996a; Kolias, 1996b; Kolias et al., 2001; Linares-Unamunzaga et al., 2018; Nascimento & Albuquerque, 2018; Paiva et al., 2017; Pretorius, 1970; Sounthararajah et al., 2018; Sounthararajah et al., 2017; Sounthararajah et al., 2016), although a few authors used displacement control (Farhan et al., 2019; Farhan et al., 2015; Gnanendran & Paul, 2016; Paul & Gnanendran, 2012; Paul & Gnanendran, 2015; Paul et al., 2015).

In Brazil, the tests are performed under stress control by applying a rate of 0.69 MPa/min (Castañeda López et al., 2018; Fedrigo et al., 2018), in accordance with the United States experience (Mandal et al., 2018; Mandal et al., 2016; NCHRP, 2014). Paul & Gnanendran (2012) reported that the stress rate of 0.69 MPa/min is equivalent to a displacement rate of 0.5 mm/min. The authors also verified that the flexural strength and stiffness increase with increases in the displacement rate, although they had not tested the rates used in South Africa.

2.4 Cyclic tests

There are also differences related only to the flexural cyclic (or fatigue) tests. A literature survey of the above-mentioned studies showed that generally the factors that vary most are the applied loading frequency and stress levels. Moreover, the tests are often carried out in a stress-controlled mode, applying a haversine load pulse and terminated when the specimen fails. Note that no laboratory fatigue characterisation of South African LCSM was found in the literature. The fatigue transfer functions included in the South African mechanistic pavement design method (South African National Roads Agency Limited, SANRAL, 2014; Theyse et al., 1996) are based on the extensive data collected and analysed by Otte (1972; 1978) and De Beer (1985; 1990) using APT. On the other hand, most of the fatigue models used internationally, including in Brazil, were developed based on laboratory results.

3 Experimental programme

The study can be divided into three stages, in accordance with the defined study objectives. In the first stage, the flexural static characterisation (modulus of rupture, strain at break and stiffness) of the studied LCSM was carried out in specimens produced with three compaction methods (vibratory table, press and hammer), using two test setups (100 mm × 100 mm × 300 mm and 75 mm × 75 mm × 420 mm) and applying three displacement rates (0.5, 1.0 and 1.8 mm/min). The effect of the independent variables was individually evaluated at this stage (a detailed description is provided in Table 3, Section 3.3.1). The second stage comprised the flexural cyclic characterisation (modulus, degradation, fatigue life) of the studied materials. In the third and final stage, the flexural static and cyclic behaviour of the studied South African materials was compared to that of Brazilian materials. The testing conditions used in the first and second stages were selected in order to allow these comparisons.

The experimental programme is described in detail in the following sections, including the mechanistic pavement analysis done in this study.

3.1 Materials

Two natural gravels (G6 materials) were stabilised for the laboratory tests. Table 1 and Figure 1 show their characteristics and grain size distributions, respectively. The weathered granite was obtained from a quarry located near Johannesburg. The burnt shale was used as parent material to produce the LCSM base layer of an experimental section located on the R104 road near Pretoria. This material was collected from the shoulder of the mentioned road, where it was used without stabilisation.

Table 1

Figure 1

Mix designs to obtain C3 materials (UCS between 1.5 and 3 MPa) were performed by evaluating the initial consumption of stabiliser (ICS), UCS and indirect tensile strength (ITS). The granite was tested using 2.5%, 3.5% and 4.5% of portland cement with fly ash and slag addition and strength class of 32.5 MPa (RoadCem CEM II/B-M (V-S) 32.5 N). To reduce its Plasticity Index, the shale was pre-treated with 1% of hydrated lime (Calcitic Road Lime) and then tested with 1%, 2% and 3% of the same cement as that used to stabilise the granite. Table 2 shows the resulting mix design characteristics. The shale characterisation and its mix design were done by Theyse (2013). The UCS of both materials achieved values higher than 3 MPa, but they were still considered C3 as the materials were G6 (Paige-Green, 2014).

Table 2

3.2 Specimens

Beams with sections of 100 mm × 100 mm (500 mm long) and 75 mm × 75 mm (450 mm long) were prepared. For the granite, solids were mixed and water at OMC was added while continuing mixing. For the shale, lime and water were added and mixed 24 hours prior to adding and mixing cement (Theyse, 2013). The material was compacted in three equal layers to achieve 97% MDD (CSRA, 1985) using one of the following methods: (a) vibratory table in conjunction with surcharges of approximately 50 kg (Figure 2(a)); (b) compression with a hydraulic press

(Figure 2(b)); and (c) modified AASHTO hammer (56 blows per layer). To improve the bond between layers, their surface was scarified. The specimens were wrapped in plastic sheets (Figure 2(c)) and cured at a temperature of 25 ºC for 28 days.

Figure 2

To reduce variability, specimens were discarded and remoulded when the following requirements were not met: (a) dry density lower than 97% of the MDD; and (b) effective moulding moisture content deviating by more than 1% of the OMC. A total of 30 beams with a section of 100 mm × 100 mm (21 from granite and 9 from shale) and 12 beams with a section of 75 mm × 75 mm (9 from granite and 3 from shale) were produced. Slabs were also taken from the shale LCSM of the R104 road experimental section (Figure 3(a)) and 6 beams with an age of 5.5 years (3 of each size) were sawed (Figure 3(b)).

Figure 3

3.3 Testing procedures

A hydraulic press with a capacity of 250 kN was used for static and cyclic tests. The actuator was controlled using a 20 kN load cell. 4PBT was used and the mid-span deflection was measured using two linear variable differential transducers (LVDTs).

3.3.1 Static tests

Flexural static tests were carried out in a displacement-controlled mode to evaluate the effects of the compaction method, test setup (Figure 4) and displacement rate on modulus of rupture (MOR, also known as flexural strength), strain at break (strain corresponding to MOR), stiffness and UCS (using cubes sawed from the tested beams). Eq. (1) was used to calculate the flexural stress. The flexural tensile strain was calculated using Eq. (2). The flexural static modulus (stiffness) was determined from the stress-strain relationships (secant modulus corresponding to 40% of MOR).

$$\sigma_i = \frac{P_i * L}{w * h^2} \quad (1)$$

$$\varepsilon_i = \frac{108 * h * \delta_i * 10^6}{23 * L^2} \quad (2)$$

Where σ_i (MPa) is the stress corresponding to force P_i (N); ε_i ($\mu\varepsilon$) is the tensile strain corresponding to the average deflection δ_i (mm); L is the length between supporting rollers, and; w and h are the average width and height (mm), respectively.

Figure 4

The effect of each independent variable was evaluated individually using Analysis of Variance (one-way ANOVA) while the levels of the remaining two were kept constant (Table 3). The variables and their levels were defined based on the literature review (Section 2). Three specimens were tested for each condition. The length between the loading rollers was equal to 1/3 of the span between supporting rollers.

Table 3

3.3.2 Cyclic tests

Flexural cyclic (fatigue) tests were carried out in a stress-controlled mode. Specimens were subjected to 3 Hz haversine cyclic loading (equipment maximum allowable frequency). A preload of 0.1 kN was maintained during the tests. The applied stress levels were 70%, 80% and 90% of the MOR. The tests were terminated after the failure of the specimens. Initial cyclic (resilient) modulus and initial strain were determined for the first load pulse because some specimens failed after only a few cycles. Some beams failed during the preload stage and no data was collected (ideally more sensitive equipment was needed).

The test characteristics were defined to allow comparisons with Brazilian materials. The vibratory table compaction was used since it was the easiest way to produce the beams and due to the reasons reported in Section 4.1. The 100 mm × 100 mm × 300 mm setup was used. The MOR used to determine the stress levels was the average obtained using a displacement rate of 0.5 mm/min since it is equivalent to the stress rate of 0.69 MPa/min (Paul & Gnanendran, 2012) and due to the reasons reported in Section 4.3.

3.4 Mechanistic pavement analysis

In order to compare field and laboratory modulus values, the modulus of the pavement layers of the R104 road experimental section was back-calculated using the Falling Weight Deflectometer (FWD) data collected by Theyse (2013) 28 days after construction. GAMES software (Maina & Matsui, 2004) was used to run the analysis. A structure consisting of 100 mm of C3 (observed when collecting the cement stabilised shale slabs), 400 mm of G7 (100 mm of neat G6 plus 300 mm of selected G7) and G7 subgrade was considered. The respective Poisson's ratios were 0.25, 0.35 and 0.44.

A mechanistic pavement analysis was also performed to compare the results obtained using the cement stabilised shale strain-based fatigue model and the South African transfer functions. Everstress software (Washington State Department of Transportation, WSDOT, 2005) was used to run the analysis. The structure of the R104 road experimental section was used in the analysis. The properties considered for the stabilised shale were those obtained in this study and those historically used in South Africa. Two 20 kN wheel loads and tire pressures of 560 kPa and 800 kPa were considered.

4 Static tests results

According to the detailed programme shown in Table 3, the following sections present the effects of compaction method, test setup and displacement rate on the static tests results.

4.1 Effect of the compaction method

Figure 5 shows the stress-strain relationships for the individual beams. Figure 6 shows the flexural static properties as a function of the compaction method. The values are the average of three specimens and the error bars represent the standard deviation (SD). The highest values of MOR (Figure 6(a)), UCS (Figure 6(b)) and modulus (Figure 6(d)) as well as the lowest value of strain at break (Figure 6(c)) were obtained using vibratory table compaction, which is followed by the press compaction and then by the hammer compaction. The obtained values of strain at break and modulus were higher than those recommended in South Africa for the design of C3 layers (SANRAL, 2014; Theyse et al., 1996). The UCS values obtained for the beams compacted using vibratory table were higher than those suggested for C3 materials (CSRA, 1985; Theyse et al., 1996).

Figure 5

Figure 6

Table 4 shows the results of the One-Way ANOVA (significance level of 0.05). A multiple comparison of means was also performed (Tukey's test) and groups were defined. Means that do not share a group letter are significantly different. The effect of the compaction method on strain at break and modulus was not significant. The compaction method effect on the MOR was significant but there was no difference between the means obtained using vibratory table and press. The highest effect due to the compaction method was on the UCS, which allows the deduction that the compression behaviour is more affected by the compaction than the tensile behaviour.

Table 4

4.2 Effect of the test setup

Figure 7 shows the stress-strain relationships of the specimens of granite (28 days) and shale (28 days and 5.5 years) obtained using the two test setups. Figure 8 shows the flexural static properties as a function of the test setup for the two materials. The shale was stronger and stiffer than the granite, possibly due to the lower ICS value and the higher amount of stabiliser. The average values of all properties decreased with the use of the 75 mm × 75 mm × 420 mm setup. The opposite trend was reported by Mbaraga et al. (2014). However, this could be an effect of the interaction between maximum aggregate size and beam section instead of an effect of the setup itself since larger aggregates can generate weak spots in a smaller specimen.

Figure 7

Figure 8

The effect of the curing time can be observed since all properties of the shale increased with age. The obtained values of strain at break (Figure 8(c)) were higher than suggested for the design of C3 layers (SANRAL, 2014; Theyse et al., 1996). The same was observed for the UCS values (Figure 8(b)). Furthermore, other authors reported that the UCS of concrete increases as the cube size decreases (Leung & Ho, 1996; Van Schalkwyk & Kearsley, 2018; Yi et al., 2006). Although the mentioned authors have not used 75 mm cubes, this was also observed for the shale after 5.5 years. On the other hand, the UCS of both materials after 28 days decreased with the decrease in cube size. This fact was also reported by Tokyay & Özdemir (1997) for concrete cubes of the same sizes and age as those used in this study. All these facts show that no correction factor can be suggested.

The modulus values (Figure 8(d)) obtained using the 75 mm × 75 mm × 420 mm setup were the closest to the recommended for design (SANRAL, 2014; Theyse et al., 1996). The back-calculated moduli for the C3 base (cement stabilised shale), G7 subbase and G7 subgrade were 1331 MPa (SD = 639 MPa), 241 MPa (SD = 95 MPa) and 167 MPa (SD = 36 MPa), respectively. The modulus obtained using the 75 mm × 75 mm × 420 mm setup (2908 MPa) was the closest to the field one (1331 MPa), although being more than twice the latter.

The results of One-Way ANOVA and comparison of means are presented in Table 5. Regardless of the material or age, the test setup effect on MOR and modulus was significant and there were significant differences between the means. On the other hand, the effect of the test setup on strain at break and UCS (75 mm cubes versus 100 mm cubes) was not statistically significant (although there were differences between the average values of UCS as shown in the comments presented above).

Table 5

4.3 Effect of the displacement rate

The stress-strain relationships of the specimens are presented in Figure 9. Figure 10 shows the static properties as a function of the displacement rate. The average values of MOR (Figure 10(a)) and modulus (Figure 10(d)) increased and the strain at break (Figure 10(c)) decreased with an increase in the displacement rate, which was also reported by Paul & Gnanendran (2012). There were no evident changes in the UCS as the regions from where cubes were cut are not affected by the loading. The average values of strain at break and UCS were higher than the suggested for C3 layers design. The average modulus obtained for a displacement rate of 0.5 mm/min was close to the recommended for the design of C3 layers (CSRA, 1985; SANRAL, 2014; Theyse et al., 1996), which suggests that this rate might be the most adequate.

Figure 9

Figure 10

The results of One-Way ANOVA and comparison of means are presented in Table 6. Regardless of the tested property, there were no significant effects and all the means were categorised into the same group. Therefore, for the tested levels, the displacement rate does not have a statistical effect on the flexural static behaviour.

Table 6

5 Cyclic tests results

The cyclic (fatigue) tests results are summarised in Table 7 and are analysed in the next sections. No statistical analysis was made due to the limited number of specimens.

Table 7

5.1 Cyclic (resilient) modulus

Figure 11 shows a comparison between the moduli obtained for both materials. The average cyclic (resilient) modulus values (considering all specimens) were comparable to the static modulus values. This suggests that static tests could be used to determine the design modulus since they are easier to undertake than the cyclic tests. Castañeda López et al. (2018) reported similar trends for Brazilian materials. For the shale (Figure 11(b)), the laboratory moduli were four times higher than the field modulus.

Figure 11

5.2 Degradation

Figure 12 shows the degradation of the specimens in terms of normalised modulus (ratio of the modulus at any load cycle to the initial modulus) against normalised number of cycles (ratio of any number of cycles to the number of cycles at failure). The typical pattern of LCSM showing three damage phases (González et al., 2013; Jia et al., 2018; Mandal et al., 2018; Mandal et al., 2016) was only observed for specimens subjected to the lower stress level (70%). For the others, the stiffness reduced according to a slow and nearly constant rate from the beginning until the end of the test. Most specimens failed when their stiffness reached approximately 50% of the initial stiffness.

Figure 12

5.3 Fatigue models

Figure 13 shows the fatigue models obtained for the granite and shale. The figure shows similar models for both materials. The strain-based models (Figure 13(a)) provide a better prediction of the fatigue life than the stress-based models (Figure 13(b)), as the variability is less (coefficient of determination, R^2, and standard error of estimate, SEE). Pretorius (1970) observed similar trends for soil-cement. Figure 13(a) shows that the laboratory models lead to a fatigue life shorter than the South African transfer functions (TF) (SANRAL, 2014; Theyse et al., 1996), regardless of the reliability level (RL). The maximum allowable strain for the laboratory models is approximately half of the strain at break, while it can be much higher for the transfer functions. These facts indicate that the laboratory tests are more destructive than the field tests (APT), possibly because in the field the LCSM is part of a structure and it is supported by another layer.

Figure 13

5.4 Mechanistic pavement analysis results

Table 8 shows that the laboratory model obtained for the cement stabilised shale resulted in only one load repetition for all the analysed conditions. The number of load repetitions obtained using the transfer functions decreased with the South African historic parameters and with the increase of tire pressure and RL. The difference between laboratory and field results ranged from 10^4 to 10^6 load repetitions, confirming the comments of Section 5.3.

Table 8

6 Comparison between South African and Brazilian materials

In the past, several Brazilian researchers studied the mechanical and fatigue behaviour of cement stabilised materials (Balbo, 1993; Ceratti, 1991; Trichês, 1993). However, the use of this kind of material still is limited in Brazil. Use is increasing because full-depth reclamation with cement is becoming a popular pavement rehabilitation technique. In this regard, two Brazilian materials (Fedrigo et al., 2017; Castañeda López et al., 2018) were selected according to the 7-day UCS range for C3 (CSRA, 1985; Theyse et al., 1996). These materials are cement stabilised (2%) mixtures of aggregates (80% and 50%) and reclaimed asphalt pavement, RAP

(20% and 50%). The strain at break values and the fatigue models were re-evaluated since the mentioned authors used a different approach as the one used in this study to analyse them. For the South African materials, the used properties were those obtained using the test characteristics mentioned in Section 3.3.2.

Figure 14 shows the properties of the South African and Brazilian materials and the statistics are presented in Table 9. No statistical analysis was made for the cyclic (resilient) modulus due to the different number of specimens tested. The effect of the material was significant for all analysed properties, but some means were not significantly different (Tukey's test). The shale and the 20% RAP mixture were categorised into the same groups for strain at break (group B) and static modulus (group A), two of the main design parameters. This fact associated with the similarity between their fatigue models (Figure 15) could suggest that a South African cement stabilised shale and a Brazilian cement stabilised recycled material with up to 20% RAP would have comparable fatigue performances. It is not possible to only compare the fatigue behaviour of different materials by observing their fatigue models since they might have different strength and stiffness.

Figure 14

Table 9

Figure 15

7 Conclusions

The following are conclusions based on the data analysed in this paper (they may be only valid for the materials used in this research and for similar materials):

- Statistically, similar flexural properties are obtained using vibratory table and press compaction methods. The compressive behaviour is more affected by the compaction method than the tensile behaviour. Considering the laboratory results, vibratory rollers are indicated for the compaction of lightly cement stabilised layers.
- The South African setup leads to lower flexural strength and stiffness values. However, this can be an effect of the interaction between maximum aggregate size and beam

section instead of the setup itself. This setup also leads to a modulus value that is closer to the observed in the field, although the laboratory value is still much higher.

- The flexural static properties are not statistically affected by the testing displacement rate.
- The strain at break is not statistically affected by compaction method, setup and displacement rate. Therefore, it is the most adequate property for design.
- The average cyclic (resilient) modulus is similar to the average static modulus. Therefore, static tests could be used to determine the design modulus. This fact is important for practical use since static tests are more common and cheaper to execute than cyclic tests.
- The strain-based models provide a better prediction of the fatigue life than the stress-based models.
- The laboratory models lead to a fatigue life shorter than the South African transfer functions. A mechanistic pavement analysis confirmed this fact, showing a difference in the results ranging from 10^4 to 10^6 load repetitions.
- Statistics revealed similarities between the strain at break and modulus (two main design parameters) of a South African cement stabilised shale and a Brazilian cement stabilised recycled material. Their laboratory fatigue behaviours were also similar. These facts indicate that the mentioned materials could have comparable performances.

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Material	Weathered granite	Burnt shale
Liquid Limit, LL	28	25
Plasticity Index, PI	7	9
Linear Shrinkage (%)	3.5	6.5
Grading modulus, GM	2.27	2.46
HRB classification	A-2-4(0)	A-2-4(0)
TRH 14 classification*	G6	G6
Optimum moisture content, OMC (%)**	7.4	6.6
Maximum dry density, MDD (kg/m^3)**	2106	2202
Swell (%)**	0.29	0.19
California Bearing Ratio, CBR (%)**	78	58

*Committee of State Road Authorities, CSRA, 1985; **Modified AASHTO compaction (American Association of State Highway and Transportation Officials, AASHTO, 2010); Shale results by Theyse (2013)

Table 1. Characteristics of the natural gravels

Material	Weathered granite	Burnt shale
Initial consumption of stabiliser (%)	1.5	1.0
Lime content (%)	0.0	1.0
Cement content (%)	2.5	3.0
Unconfined compressive strength (MPa)	3.82*	3.21**
Indirect tensile strength (MPa)	0.42*	0.27**

*Accelerated curing, which successfully predicts the 7-day strength (CSRA, 1986; Paige-Green, 2014); **7 days; Shale results by Theyse (2013)

Table 2. Final mix design characteristics of the LCSM

Evaluated effect	Independent variable levels			Material (age)
	Compaction method	**Test setup (mm × mm × mm)**	**Displacement rate (mm/min)**	
Compaction method	Vibratory table Press Hammer	100 × 100 × 300	0.5	Granite (28 d)
Test setup (mm × mm × mm)	Vibratory table	100 × 100 × 300 75 × 75 × 420	0.5	Granite (28 d) Shale (28 d) Shale (5.5 y)
Displacement rate (mm/min)	Vibratory table	75 × 75 × 420	0.5 1.0 1.8	Granite (28 d)

Table 3. Detailed programme for the static tests

Property	p-value	Compaction method	N	Mean	Grouping
MOR (MPa)	0.047	Vibratory table	3	0.507	A
		Press	3	0.445	A and B
		Hammer	3	0.354	B
UCS (MPa)	0.002	Vibratory table	3	3.512	A
		Press	3	2.495	B
		Hammer	3	1.918	B
Strain at break (με)	0.606	Vibratory table	3	309.6	A
		Press	3	282.5	A
		Hammer	3	249.7	A
Modulus (MPa)	0.529	Vibratory table	3	4043	A
		Press	3	3680	A
		Hammer	3	3381	A

Table 4. Statistical analysis of the compaction method effect on the static properties

Material	Property	p-value	Test setup (mm × mm × mm)	N	Mean	Grouping
Granite 28 d	MOR (MPa)	0.002	100 × 100 × 300	3	0.51	A
			75 × 75 × 420	3	0.23	B
	UCS (MPa)	0.37	100 × 100 × 300	3	3.51	A
			75 × 75 × 420	3	3.27	A
	Strain at break (με)	0.684	100 × 100 × 300	3	269	A
			75 × 75 × 420	3	250	A
	Modulus (MPa)	0.011	100 × 100 × 300	3	4043	A
			75 × 75 × 420	3	2071	B
Shale 28 d	MOR (MPa)	0.012	100 × 100 × 300	3	0.89	A
			75 × 75 × 420	3	0.44	B
	UCS (MPa)	0.719	100 × 100 × 300	3	5.73	A
			75 × 75 × 420	3	5.53	A
	Strain at break (με)	0.085	100 × 100 × 300	3	395	A
			75 × 75 × 420	3	314	A
	Modulus (MPa)	0.005	100 × 100 × 300	3	4949	A
			75 × 75 × 420	3	2908	B
Shale 5.5 y	MOR (MPa)	0.034	100 × 100 × 300	3	1.13	A
			75 × 75 × 420	3	0.71	B
	UCS (MPa)	0.675	100 × 100 × 300	3	8.36	A
			75 × 75 × 420	3	8.76	A
	Strain at break (με)	0.102	100 × 100 × 300	3	673	A
			75 × 75 × 420	3	498	A
	Modulus (MPa)	0.009	100 × 100 × 300	3	6494	A
			75 × 75 × 420	3	3350	B

Table 5. Statistical analysis of the test setup effect on the static properties

Property	p-value	Displacement rate (mm/min)	N	Mean	Grouping
MOR (MPa)	0.289	0.5	3	0.23	A
		1	3	0.24	A
		1.8	3	0.32	A
UCS (MPa)	0.285	0.5	3	3.27	A
		1	3	3.93	A
		1.8	3	3.99	A
Strain at break (με)	0.332	0.5	3	269	A
		1	3	242	A
		1.8	3	199	A
Modulus (MPa)	0.283	0.5	3	2071	A
		1	3	2666	A
		1.8	3	3237	A

Table 6. Statistical analysis of the displacement rate effect on the static properties

Material	Stress level, SL (%)	Stress, σ_i (MPa)	Initial strain, ε_i ($\mu\varepsilon$)	Strain level (%)	Initial Modulus, M_i (MPa)	Number of cycles, N	UCS (MPa)
Granite 28 d	70	0.36	58	23	5432	40,260	3.77
			63	25	5130	154,102	3.59
	80	0.41	133	53	2883	12	3.91
			103	41	3551	419	4.03
	90	0.46	133	53	3222	1493	3.74
			68	27	6402	30,408	4.06
Shale 28 d	70	0.62	75	19	5806	146,435	5.83
	80	0.71	137	35	4799	297	6.40
	90	0.80	158	40	4772	131	6.89

Table 7. Summary of cyclic tests results

Tire pressure (kPa)	LCSM parameters	Fatigue model	Number of load repetitions, N
560	This study (Modulus = 1331 MPa; Strain at break = 395 με)	Laboratory	1
		TF RL = 50%	2,061,281
		TF RL = 80%	1,334,320
		TF RL = 90%	1,246,226
		TF RL = 95%	944,031
	South African mechanistic pavement design method (Modulus = 2000 MPa; Strain at break = 125 με)	Laboratory	1
		TF RL = 50%	96,232
		TF RL = 80%	62,581
		TF RL = 90%	58,530
		TF RL = 95%	44,226
800	This study (Modulus = 1331 MPa; Strain at break = 395 με)	Laboratory	1
		TF RL = 50%	1,406,492
		TF RL = 80%	910,982
		TF RL = 90%	850,984
		TF RL = 95%	644,430
	South African mechanistic pavement design method (Modulus = 2000 MPa; Strain at break = 125 με)	Laboratory	1
		TF RL = 50%	38,099
		TF RL = 80%	24,811
		TF RL = 90%	23,215
		TF RL = 95%	17,528

Table 8. Fatigue lives predicted for the cement stabilised shale layer

Property	**p-value**	**Country**	**Material**	**N**	**Mean**	**Grouping**
MOR (MPa)	0.000	South Africa	Granite	3	0.51	B
			Shale	3	0.89	A
		Brazil	20% RAP	3	0.26	C
			50% RAP	3	0.32	B and C
Static modulus (MPa)	0.005	South Africa	Granite	3	4043	A and B
			Shale	3	4949	A
		Brazil	20% RAP	3	3950	A and B
			50% RAP	3	2900	B
Strain at break (με)	0.012	South Africa	Granite	3	250	A and B
			Shale	3	395	A and B
		Brazil	20% RAP	3	169	B
			50% RAP	3	470	A

Table 9. Statistical analysis of South African and Brazilian LCSM flexural properties

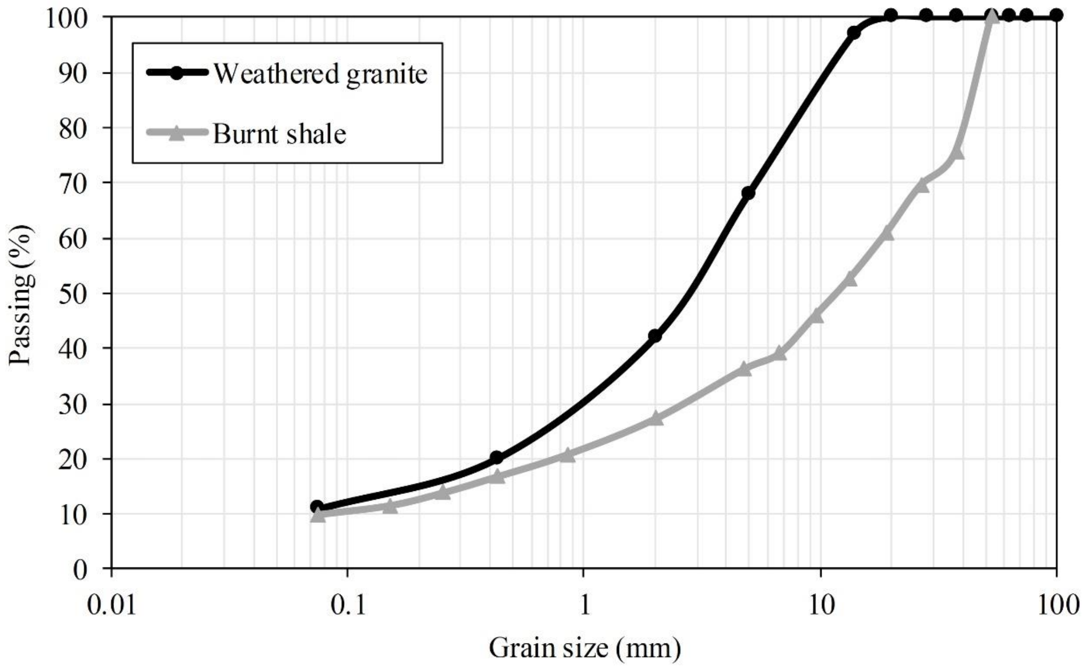

Figure 1. Grain size distributions of the natural gravels

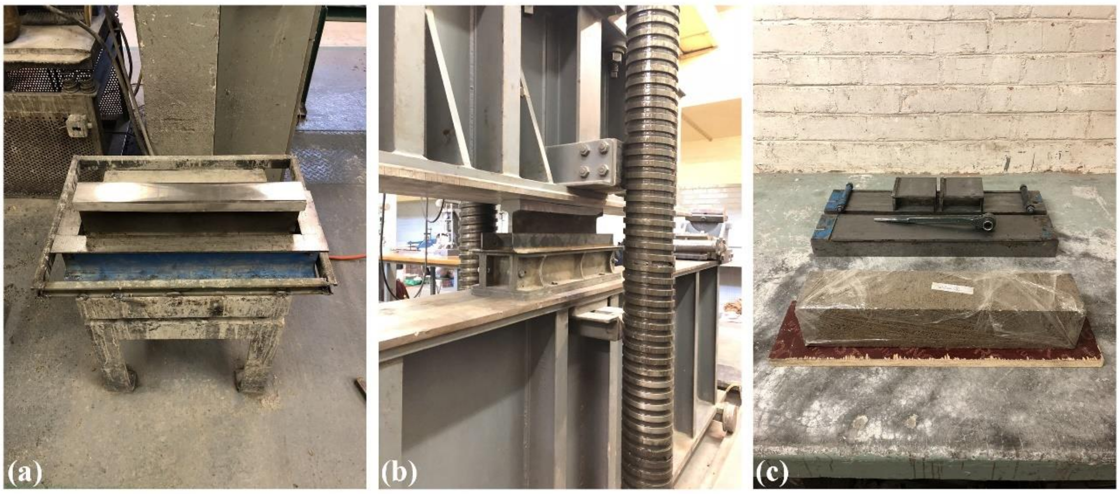

Figure 2. Laboratory specimens: (a) vibratory table compaction; (b) press compaction; and (c) demoulded specimen wrapped in plastic sheets

Figure 3. Field specimens: (a) slabs taken from the LCSM; and (b) beams cut from a slab

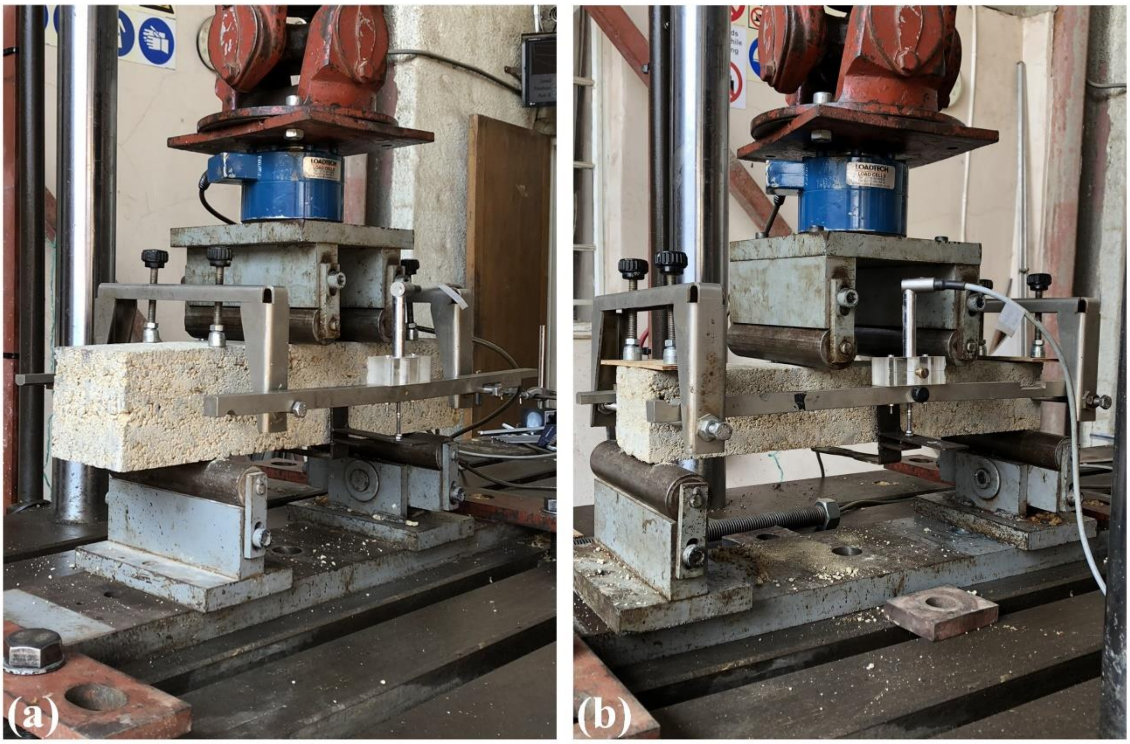

Figure 4. Test setups (height × width × span): (a) 100 mm × 100 mm × 300 mm; and (b) 75 mm × 75 mm × 420 mm

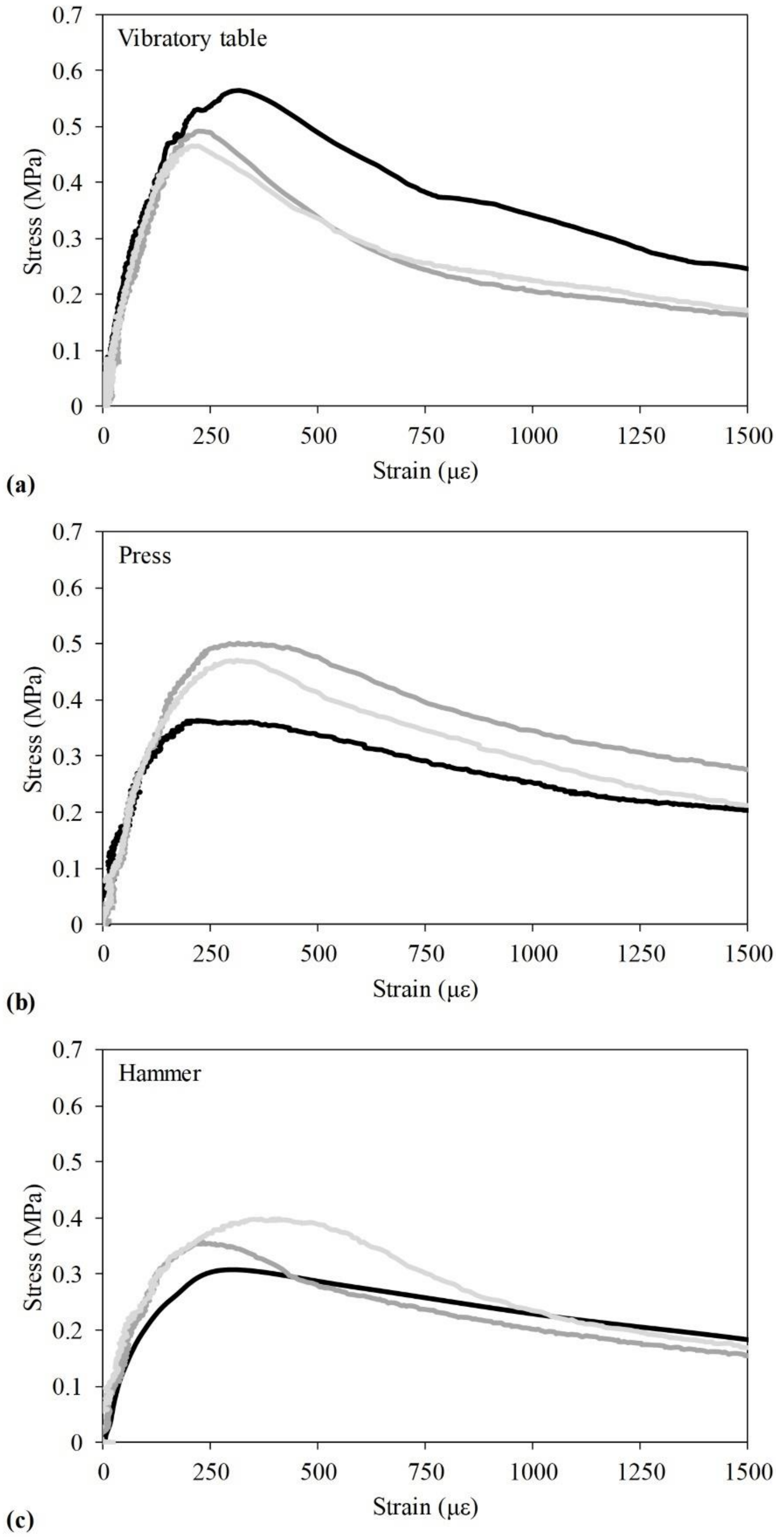

Figure 5. Stress-strain relationships for each sample as a function of the compaction method: (a) vibratory table; (b) press; and (c) hammer

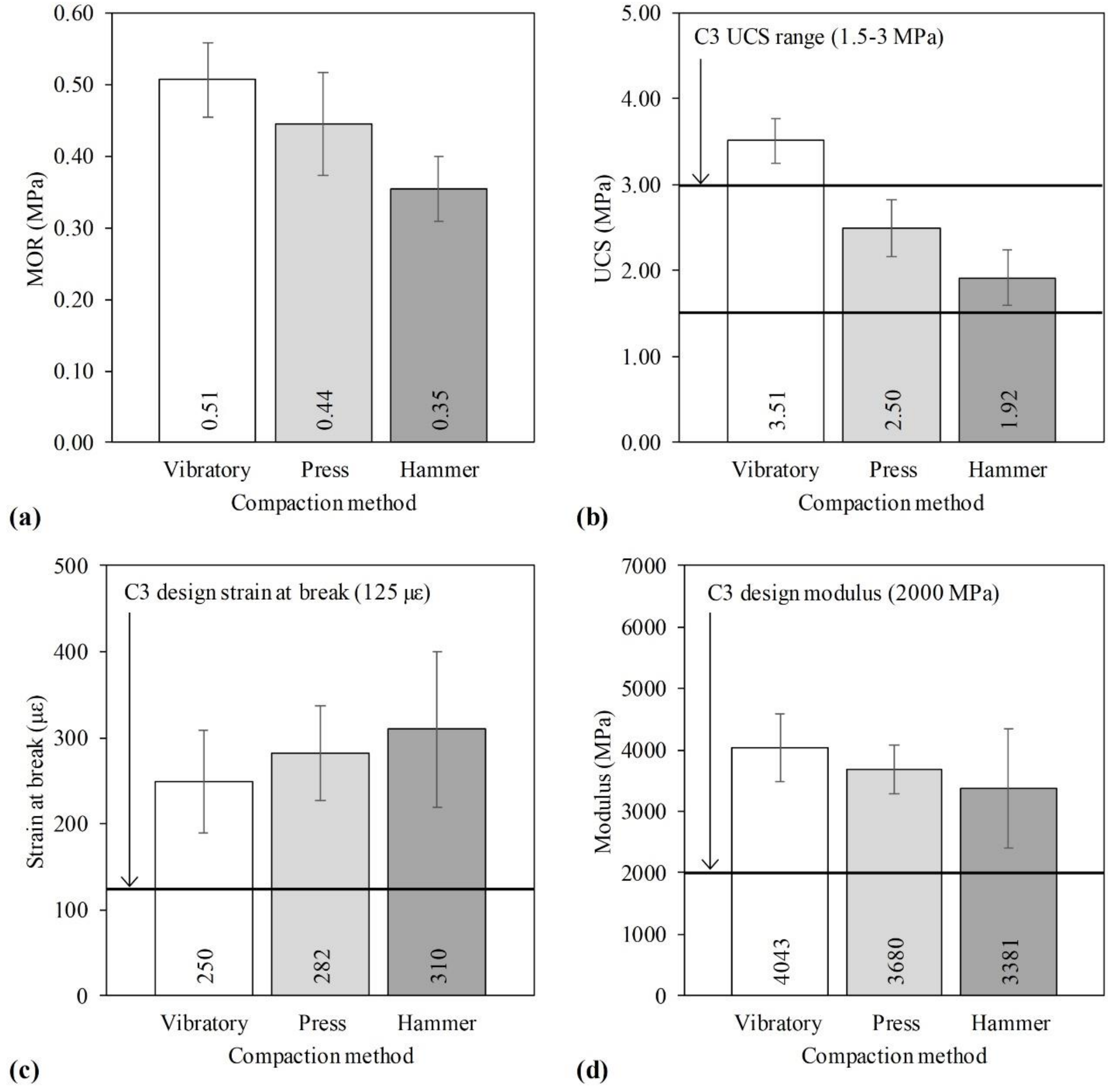

Figure 6. Flexural static properties as a function of the compaction method: (a) MOR, (b) UCS; (c) strain at break; and (d) modulus

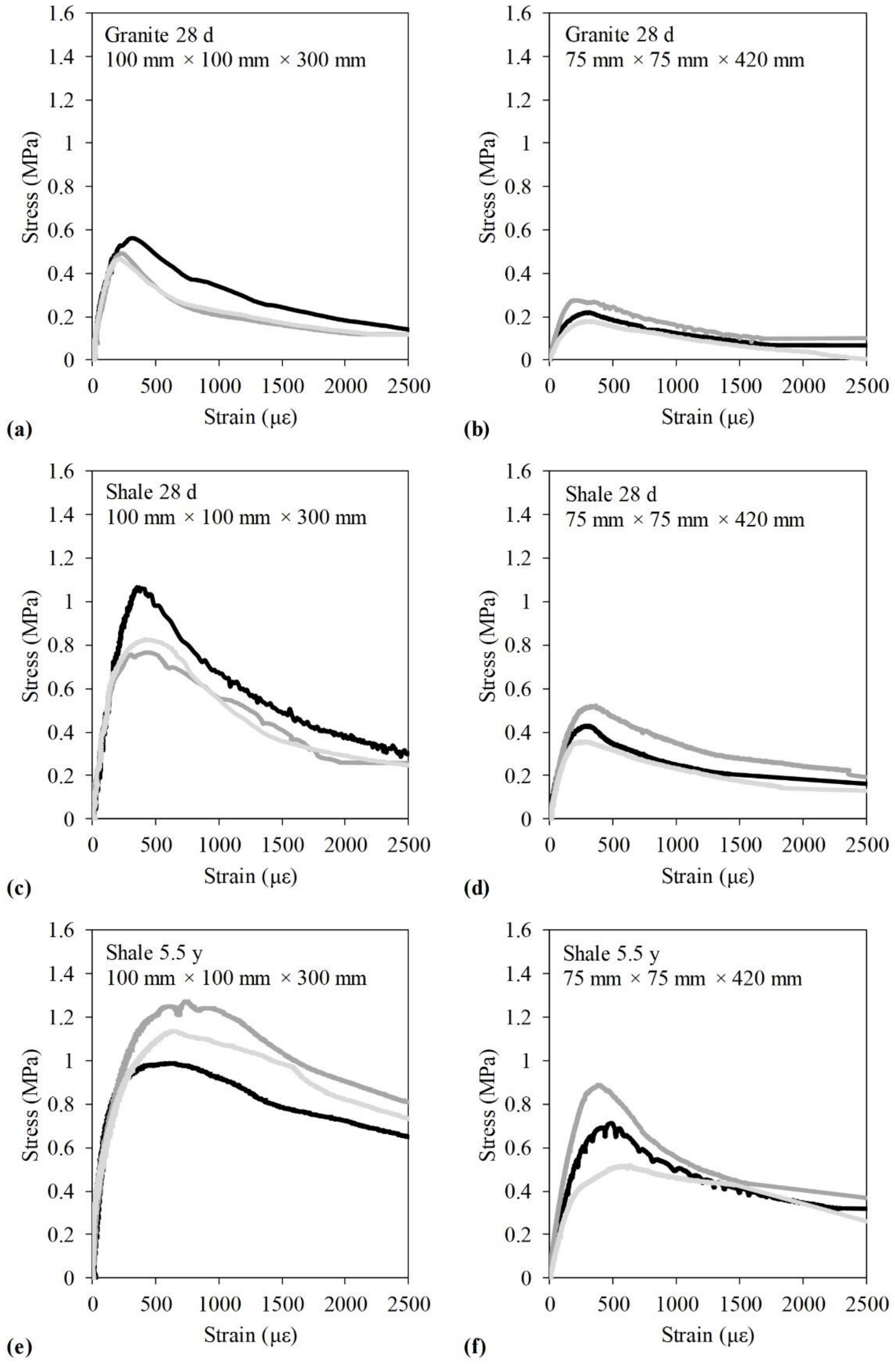

Figure 7. Stress-strain relationships for each sample as a function of the test setup for the granite (28 days) and shale (28 days and 5.5 years)

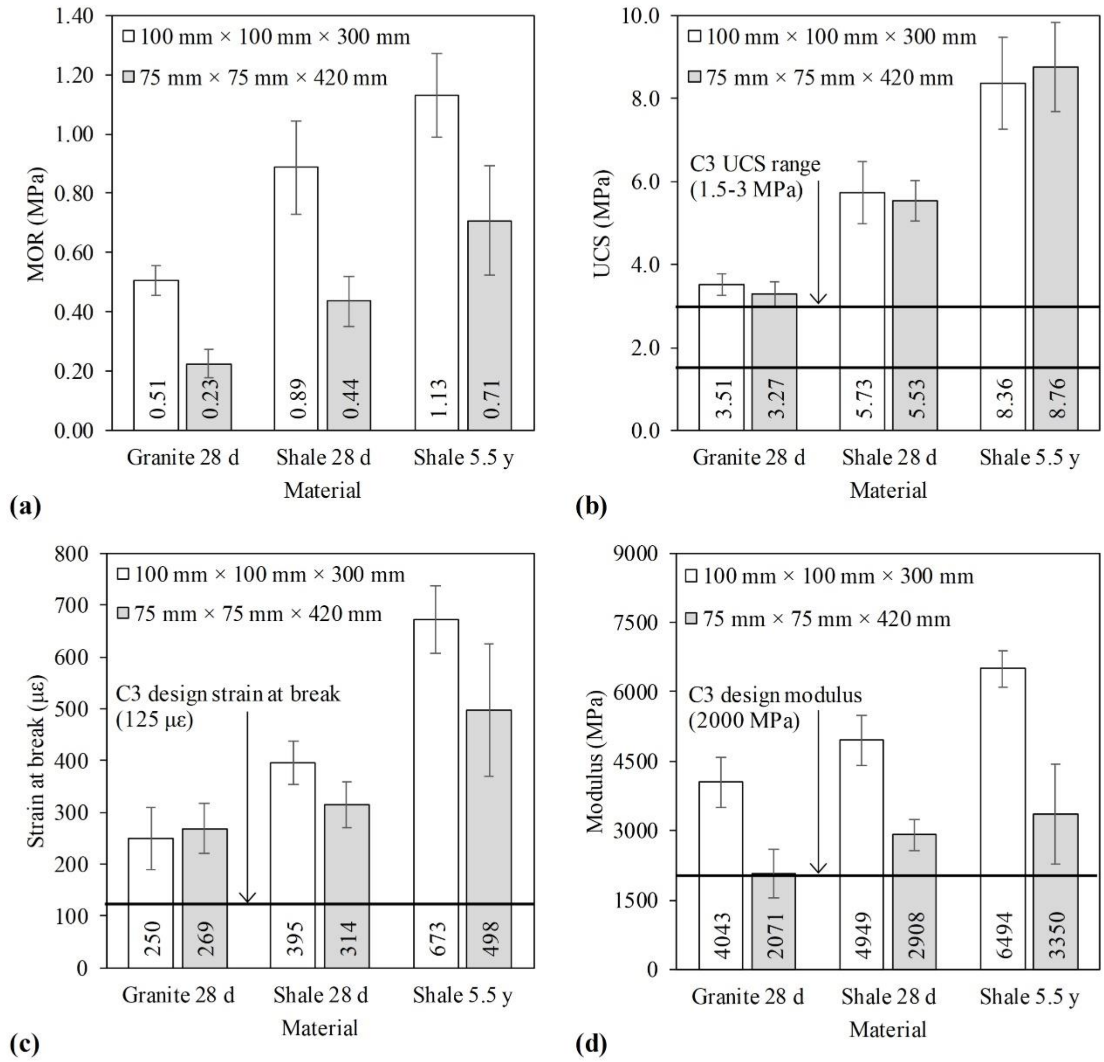

Figure 8. Flexural static properties as a function of the test setup for the granite and shale: (a) MOR, (b) UCS; (c) strain at break; and (d) modulus

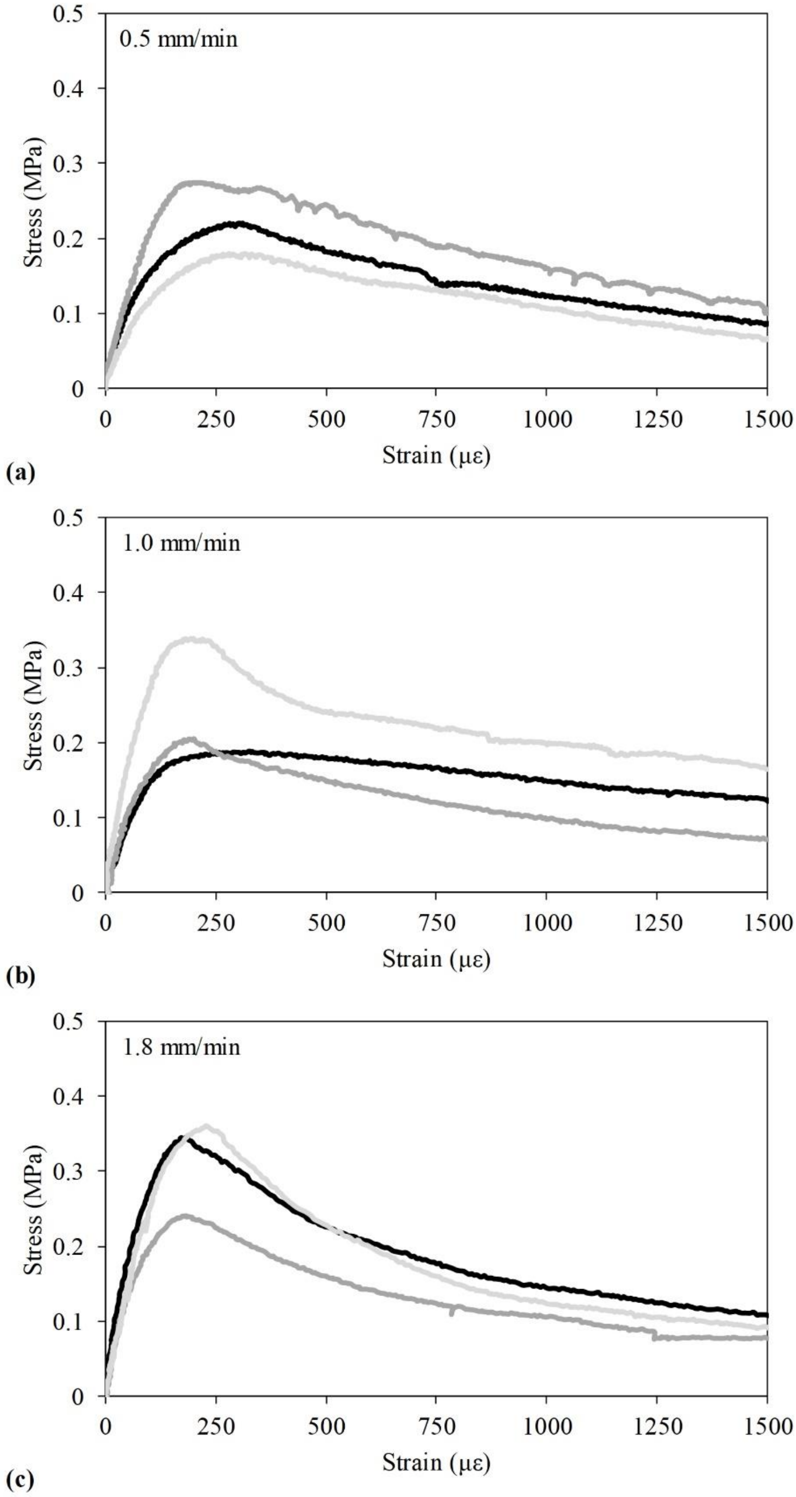

Figure 9. Stress-strain relationships for each sample as a function of the testing displacement rate: (a) 0.5 mm/min; (b) 1.0 mm/min; and (c) 1.8 mm/min

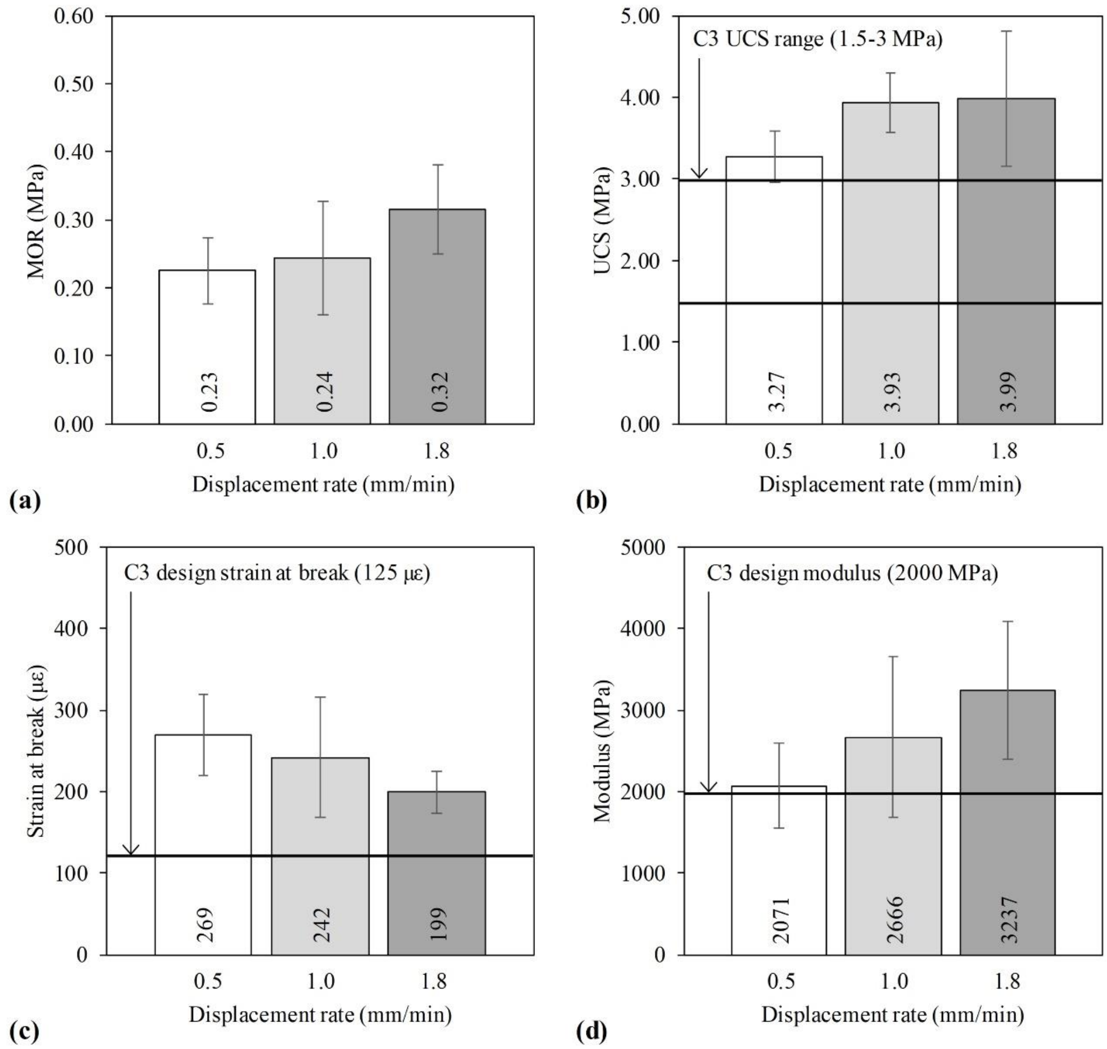

Figure 10. Flexural static properties as a function of the testing displacement rate: (a) MOR, (b) UCS; (c) strain at break; and (d) modulus

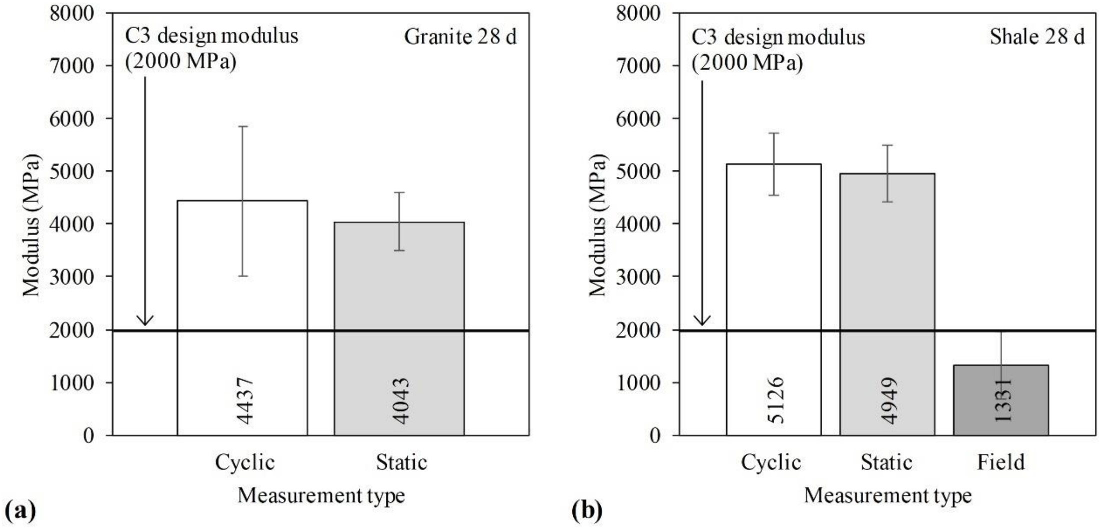

Figure 11. Modulus values for different measurement types: (a) granite; and (b) shale

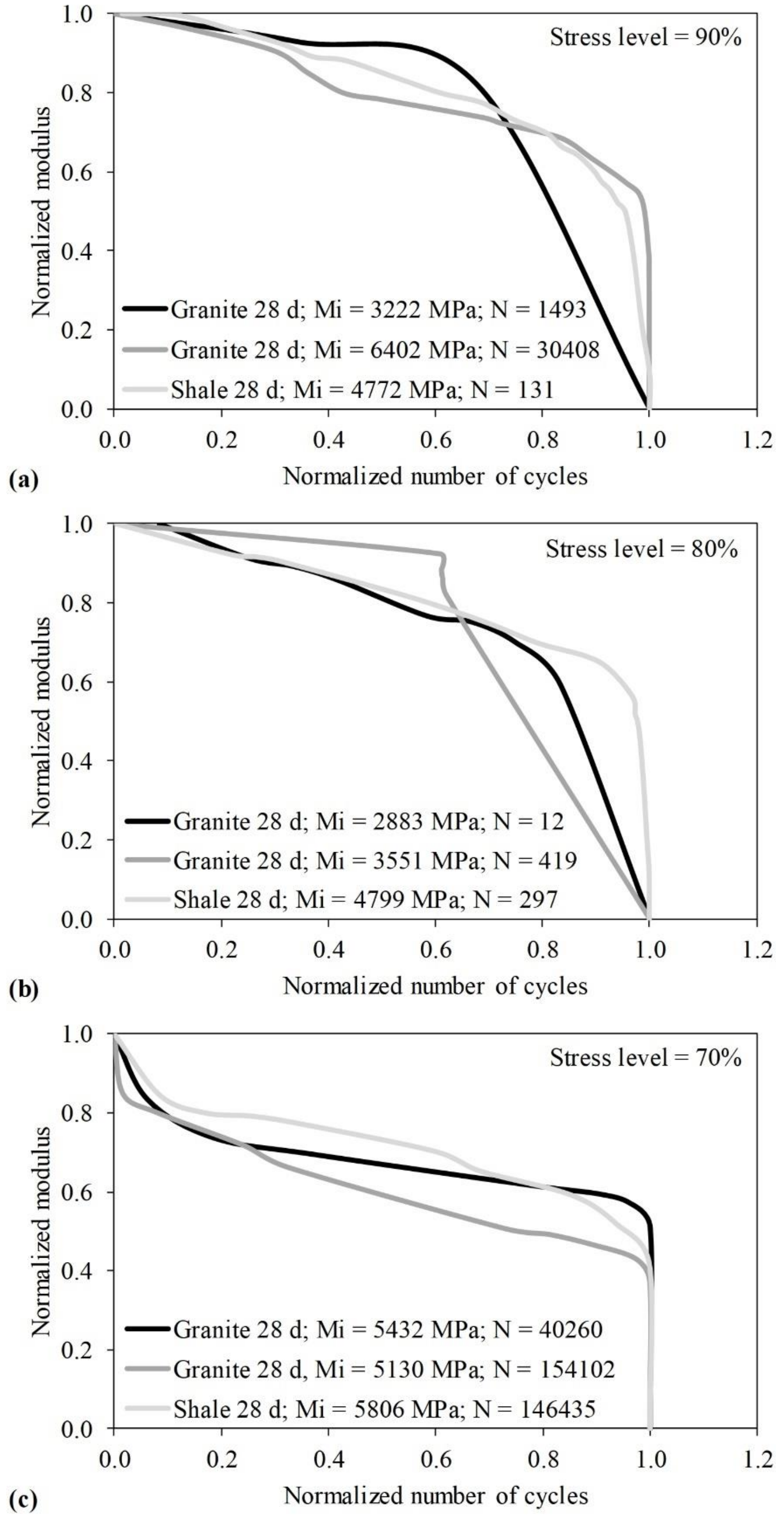

Figure 12. Degradation of each specimen: (a) 90% SL; (b) 80% SL; and (c) 70% SL

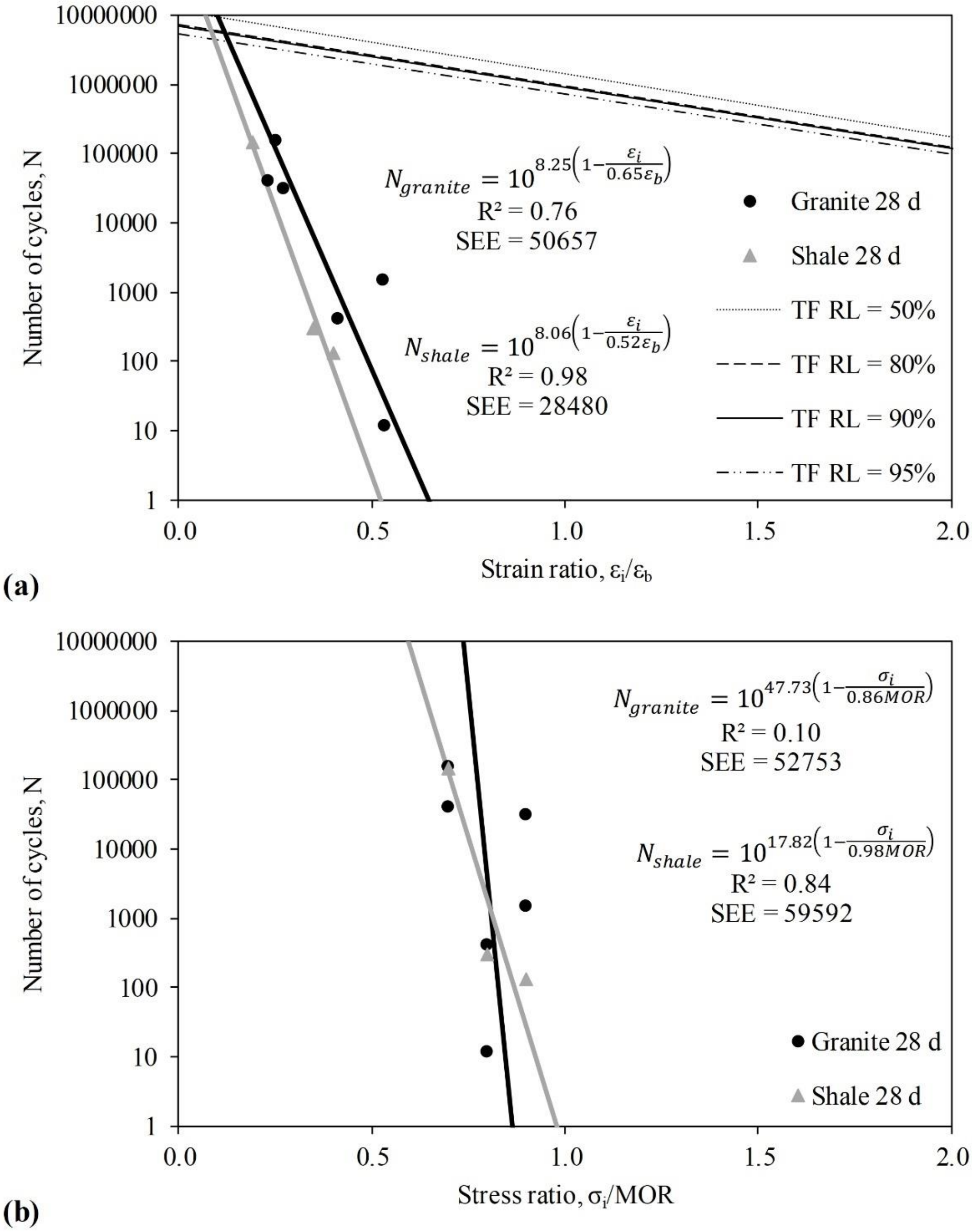

Figure 13. Fatigue models for the granite and shale: (a) strain ratio; and (b) stress ratio

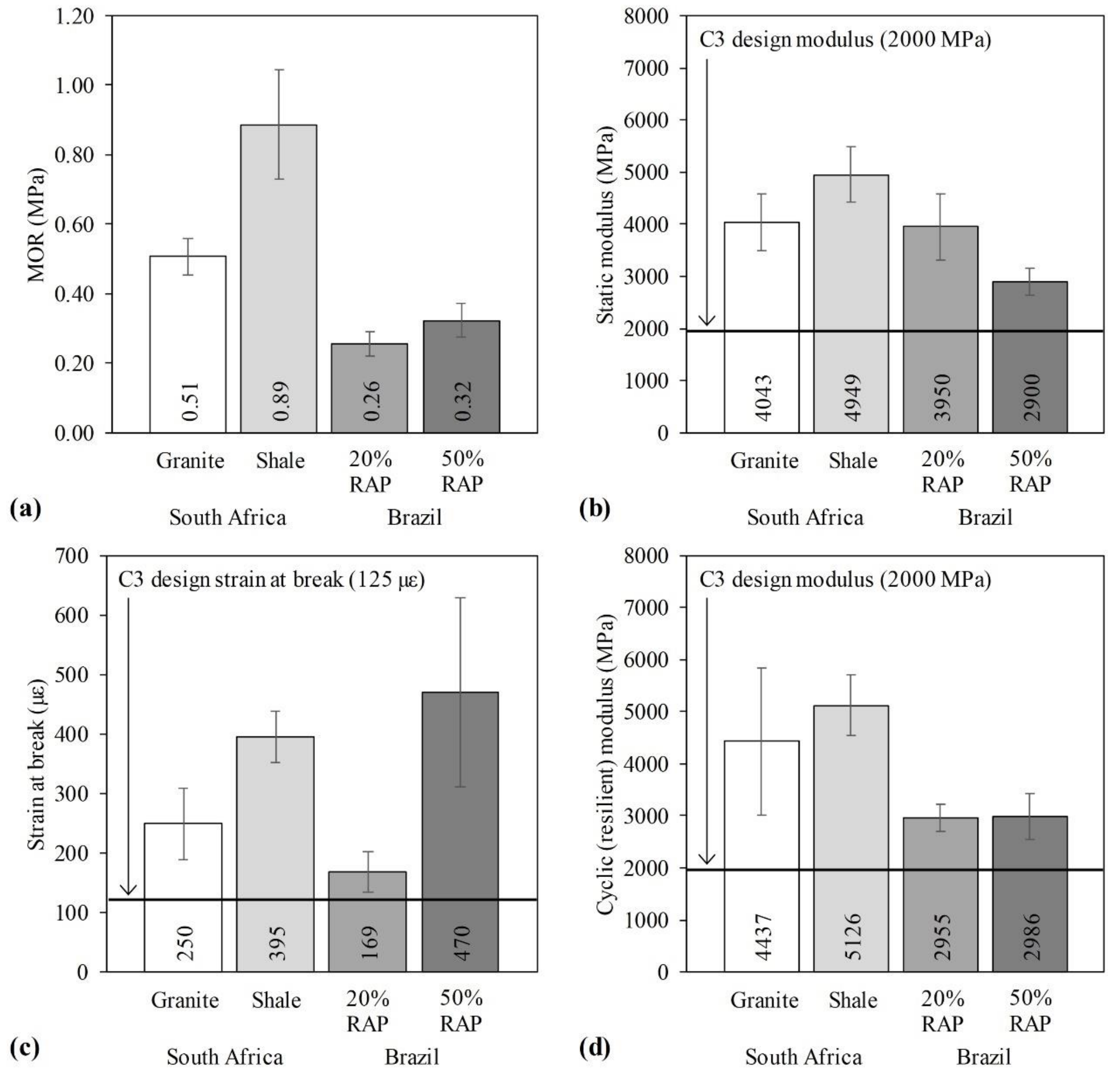

Figure 14. Comparison between South African and Brazilian LCSM: (a) MOR; (b) static modulus; (c) strain at break; and (d) cyclic modulus

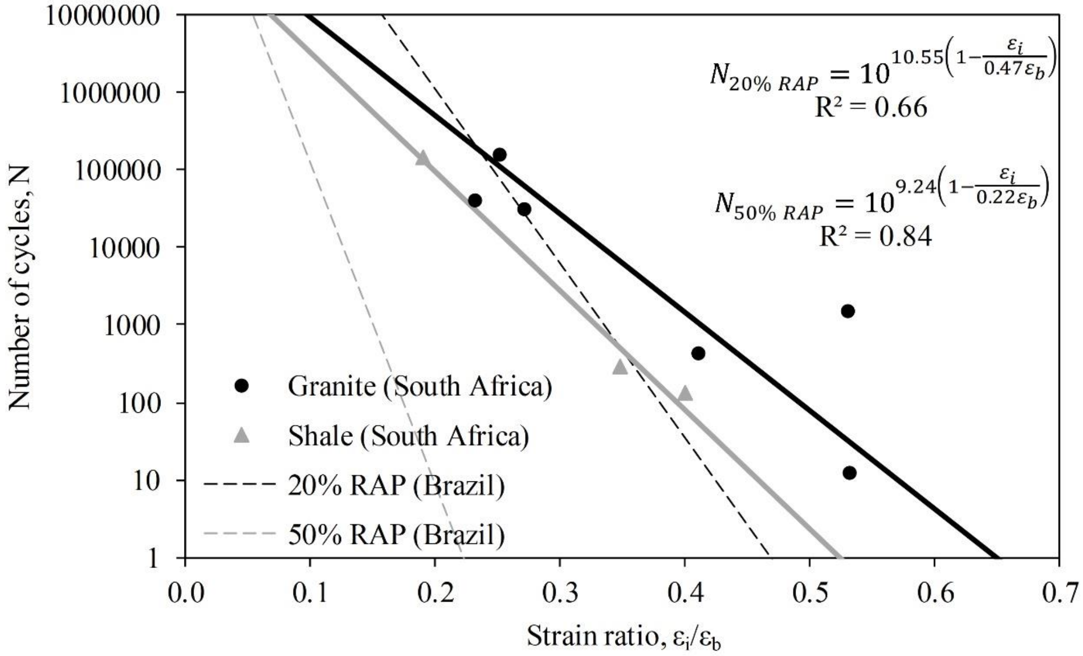

Figure 15. Laboratory fatigue models of South African and Brazilian LCSM

6 CONCLUDING REMARKS

This chapter presents the summary and conclusions of each paper, the thesis main contributions and recommendations for further research.

6.1 SUMMARY AND CONCLUSIONS

Concerning the literature review on full-depth reclamation of pavements with Portland cement (which generates cold recycled cement-treated mixtures):

- The technique is used worldwide with several benefits. While mix design methods are well established internationally, structural design methods still need to be adapted in most countries (e.g. Brazil). Analysis of data from previous research identified general trends regarding the behaviour of such materials. However, there are also knowledge gaps for future research, which, if addressed, might promote FDR-PC application.

Regarding the effects of the asphalt binder present in RAP (reclaimed asphalt pavement) on strength and stiffness of cold recycled cement-treated mixtures:

- The type, content and ageing of the asphalt binder present in RAP affect the mechanical behaviour of cement-treated mixtures of RAP and graded crushed stone (GCS), and hence mix and structural designs. The asphalt binder type and the interaction between asphalt binder type and content have significant effects on the indirect tensile strength. The asphalt binder content, type, and ageing, as well as the interaction between asphalt binder type and ageing, have significant effects on the indirect tensile resilient modulus. The asphalt binder content effects on the mechanical behaviour may be related to its significant effect on the compactability (dry density and voids in the mixture).

About the flexural behaviour of cold recycled cement-treated mixtures of RAP and lateritic soil (LS):

- Generally, strength, stiffness and flexibility of such mixtures increase with RAP percentage. Therefore, it is possible to in situ recycle old asphalt pavements with LS base layers and thick asphalt wearing courses. The flexural strength increases with cement content and RAP percentage; the obtained values (0.28-0.96 MPa) are comparable to those of cement-treated mixtures of RAP and GCS reported elsewhere. The strain at break decreases with cement content and increases with RAP percentage; the obtained values (273-1,059 microstrain) are generally higher than those of cement-treated aggregates with or without RAP reported elsewhere. The flexural modulus increases with cement content independently of the loading condition (static and cyclic) and the resilient modulus increases with RAP percentage. Stiffness values (983-4,163 MPa) are generally lower than those of cement-treated aggregates with or without RAP reported elsewhere.

- It was obtained fatigue relationships for such mixtures, until now not reported elsewhere. The fatigue relationships exponent values (6.65-21) are comparable to those of cement-treated aggregates with or without RAP reported elsewhere. Fatigue progressive failure of such mixtures seems to be affected by the stress level. The low fatigue resistance of some mixtures could be explained by insufficient cement contents or by micro-cracking due to drying shrinkage. Adequate mix design and field curing by maintaining the moisture condition could improve the recycled pavement performance. Mechanistic analyses indicate that the fatigue life of such recycled base layers increases with the thicknesses of asphalt wearing course and base layer. The effect of cement content depends on the layers' thickness. Mixtures with a high RAP percentage generally perform better, confirming that pavements with thick asphalt layers could be in situ recycled.

The following are conclusions based on tests on lightly cement stabilised materials (LCSM) performed in South Africa and Brazil:

- Vibratory and compression compaction methods result in similar flexural properties for LCSM. Therefore, using heavy vibratory rollers is advised. The testing displacement rate does not statistically affect flexural properties. The South African test setup leads to lower flexural strength and stiffness values. This setup also leads to modulus values that are closer to those observed in the field. The strain at break is preferred for structural design, since it is not statistically affected by compaction method, setup and displacement rate.

- It was obtained laboratory fatigue relationships for South African LCSM, until now not reported elsewhere. Strain-based relationships provide a better prediction of fatigue life than stress-based relationships. Such relationships lead to fatigue lives shorter than those obtained using South African transfer functions (based on field tests). A mechanistic pavement analysis confirmed this fact, showing a difference in the results ranging from 10^4 to 10^6 load repetitions.
- Statistical analyses revealed similarities between the strain at break and modulus (two of the main design parameters) of South African LCSM and Brazilian cement-treated mixtures of RAP and GCS. Their laboratory fatigue behaviours were also similar. These facts indicate that such materials could have comparable performances.

6.2 MAIN CONTRIBUTIONS TO THE KNOWLEDGE

This research advanced on the knowledge of mechanical and fatigue behaviour of cold recycled cement-treated mixtures. The following summarises the most meaningful findings of this research:

a) The characteristics of the asphalt binder present in RAP affect the mechanical behaviour of cold recycled cement-treated mixtures and must be taken into account when designing mixtures and structures.
b) It is possible to recycle pavements with lateritic soil base layers overlaid by thick asphalt layers without compromising mechanical and fatigue behaviour of the recycled layer.
c) It was obtained laboratory fatigue relationships of cement-treated mixtures of reclaimed asphalt pavement and lateritic soil, until now not reported elsewhere.
d) Strain at break is the preferred property for pavement structural design because it is not affected by the characteristics of the flexural test.
e) It was obtained laboratory fatigue relationships for South African cement stabilised materials, until now not reported elsewhere.

6.3 RECOMMENDATIONS FOR FURTHER RESEARCH

Based on the testing and conclusions presented in this thesis, the following are recommendations for further research:

- To enhance the quality of RAP produced in laboratory since it is essential for assessing the behaviour of recycled mixtures; a suggestion is constructing test sections with asphalt mixtures produced with different asphalt binder types and contents (but with the same aggregates and grain size distribution). These pavements could then be subjected to traffic and climate effects and milled at different ages to produce RAP materials that would be even more representative of field RAP. It would also be interesting to verify the ageing effect by testing the asphalt binder after its extraction and recovery.
- To reproduce the experiment performed in Chapter 3 using different cement contents, RAP percentages and curing times, since these variables may affect RAP asphalt binder effect.
- To verify the effect of asphalt binder characteristics (type, content and ageing) on the flexural behaviour of cold recycled cement-treated mixtures with RAP produced in the laboratory; this could be useful for pavement structural design, because of the similarities between flexural test and field stress states.
- To monitor the long-term performance of cold recycled cement-treated layers. Comparing it with the laboratory behaviour could help to develop/calibrate mechanistic-empirical structural design methods. Even though it is harder than in the laboratory, varying cement and RAP contents in test sections could help to understand their effect on the behaviour of such materials. Accelerated pavement testing is an interesting option for such studies.
- To compare more Brazilian and South African cement-treated materials using laboratory tests and mechanistic analyses to verify if there are similarities between their behaviour. Since both countries have similar weather and economy, it could be helpful making use of the South African advanced experience on cement-treated materials.
- To evaluate the viscoelastic behaviour of cold recycled cement-treated mixtures. Although such materials show little sensitivity to temperature and frequency, this might change for different ranges of cement and RAP, since the asphalt binder present in RAP shows time and temperature dependence. Cold recycled cement-treated mixtures with RAP produced in laboratory could help to identify the effect of asphalt binder characteristics (type, content and ageing) on the viscoelastic behaviour.

- To study the effect of different cement types on strength and stiffness of cold recycled cement-treated mixtures, including green types of cement with reduced environmental impact.
- To evaluate economic and environmental issues of full-depth reclamation/recycling with Portland cement using life-cycle cost analysis and life cycle assessment, respectively. Such studies could emphasise the advantages of the technique, helping to make it a standard choice.
- To investigate the durability of cold recycled cement-treated mixtures, since there are only a few studies on this matter and several effects remain unknown (e.g. RAP content and existing base material).

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APPENDIX I

This appendix presents the comments (C) made by the examiners and the responses (R) provided by the author.

ALEX T VISSER (UP)

C: Since the papers have more than one author, the examiner inquired the author about his contributions to the research performed and to the knowledge.

R: The author of the thesis is also the first and corresponding author of all papers. He was the only author that worked in all parts of the studies, including testing, analysis and writing (draft, review and editing). The author included this information and the main contributions to the knowledge in the thesis document (Chapters 1 and 6, respectively).

C: The examiner inquired the author about the reason for using different tire pressures in the mechanistic analysis performed in Chapter 4 (560 kPa) and Chapter 5 (800 kPa). The examiner mentioned that tire pressures of 560 kPa were measured forty years ago, while nowadays higher pressures are often measured. He explained that tire pressure affects the surface layer and the upper part of the base, while tire load affects the lower part of the pavement structure. Therefore, by using a low tire pressure, one may not be recognising the damage which comes to the surface layer and the upper part of the cement stabilised base.

R: In Chapter 4, the author used the same tire pressure as that used by Castañeda López et al. (2018) for comparison purposes. The mentioned authors performed similar analyses as those presented in Chapter 4. Besides, even though other countries (e.g. France and South Africa) use higher tire pressures, 560 kPa is often used in Brazil. In Chapter 5, the author used pressures commonly used in Brazil and South Africa, since the paper compares Brazilian and South African methods and materials. Furthermore, based on the AASHTO road test, most fatigue (and other distress types) models are expressed in terms of the number of repetitions of an 80 kN load single axle with dual tires and tire pressure of 560 kPa.

YVES GEORGE FRANÇOIS JEAN BROSSEAUD (IFSTTAR)

C: Chapter 3: Since the author used a curing time of 28 days in Chapters 4 and 5, the examiner inquired the author about the reason for using a curing time of 7 days in Chapter 3.

R: Chapter 4 and 5 report fatigue test results obtained after 28 days of curing. The literature states that most of the cement reactions occur until this age, then it was expected that curing would not affect fatigue behaviour. However, we are running tests after 365 days of curing to identify curing time effect. Chapter 3 reports the effects of RAP asphalt binder type, content and ageing. Due to the investigation of three factors and to the extremely long time necessary to produce RAP, the author had to fix some factor levels. However, some standards recommend a curing time of 7 days for such materials. The author included a recommendation for further research in light of the comment of the examiner.

C: Chapter 3: The examiner inquired the author about the reasons for choosing RAP produced in laboratory instead of field RAP.

R: Comparing the behaviour of cold recycled cement-treated mixtures containing RAP from different sources (with different asphalt binder type and content, aggregate type, grain size distribution, age and so on) would not lead to reliable results. Moreover, the literature states that laboratory manufactured RAP is important for assessing the recyclability of asphalt mixtures. Chapter 3 already describes these facts.

C: Chapter 3: The examiner inquired the author if a 5 mm thick layer is sufficient to identify ageing of the asphalt binder.

R: The author explained that asphalt binder extraction and testing was not feasible due to the extremely long time necessary to produce RAP in the necessary amount with that purpose. However, results proved the soundness of the alternative asphalt binder ageing method. Besides, the ageing effect must have been even stronger for the asphalt binder in loose mixtures, since the thickness of asphalt binder film that coats aggregates is thinner than 5 mm. Chapter 3 already describes these facts.

C: Chapter 3: The examiner inquired the author if the interaction between natural rock and asphalt binder could affect ageing. The examiner mentioned that some pavements constructed in Ghana had early cracking problems due to the presence of manganese in the rock used to produce the asphalt mixture, which changed its ageing properties.

R: Literature states that the adhesion between rocks and asphalt binder may change due to chemical characteristics. However, inert rocks (e.g. a basalt as that used in the study) would probably not react with asphalt binder. It could be a problem when using residues and waste materials, as in the mentioned case in which the rocks contained manganese (probably a mining waste).

C: Chapter 3: The examiner inquired the author about his opinion regarding the softening point values obtained for the conventional asphalt binder before and after ageing (RTFOT and oven). The examiner point of view is that this asphalt binder has a specific evolution regarding ageing (probably it did not age).

R: The author highlighted that the softening point did not change much, but the penetration did. The latter shows an increase in the stiffness of the conventional asphalt binder due to ageing. However, it aged less than the rubber asphalt binder, a fact that is not common, corroborating with the examiner statement.

C: Chapter 3: The examiner inquired the author about the ageing characteristics of asphalt binder recovered from RAP. The examiner mentioned the results of one of his studies in which the softening point average values were 50 and 70 for the virgin and recovered binder, respectively.

R: The author explained that asphalt binder extraction and testing was not feasible due to the extremely long time necessary to produce RAP in the necessary amount with that purpose. However, it is a recommendation for further research (included in Chapter 6).

C: Chapter 3: The examiner inquired the author about the type of polymer used to modify the asphalt binder and the importance of describing the characteristics of the aggregates.

R: Chapter 3 already presents the characteristics of the asphalt binders. SBS (styrene-butadiene-styrene) was used to modify the asphalt binder. The aggregate used in the study was basalt. As presented in the text, the properties of the basalt aggregates followed the required limits used in Brazil. Other laboratory colleagues used this aggregate (Barros, 2017; Barros et al., 2018; Godoi, 2017; Godoi et al., 2017; Mensch, 2017) and reported its characterisation.

C: Chapter 3: The examiner inquired the author if he would recommend using a cement content of 2% since such a low content tends to increase variability.

R: As presented in the text, the levels of cement content and RAP percentage follow the maximum limits reported in the most recent Brazilian standard for full-depth reclamation with cement, which are 3% and 50%, respectively. However, one would have to increase the cement content to decrease the asphalt binder effect, but tests are needed to prove this. Regarding variability, in this case, low values of standard deviation were achieved for strength and stiffness as presented in the figures.

C: The examiner inquired the author about the reason for not using a reference mixture without RAP.

R: Cold recycling with cement is a technique performed in situ, then most of the times RAP will be part of the mixture. Moreover, for the studied RAP percentages, results lead to the conclusion that RAP percentage affects the mixture behaviour.

C: Chapter 3: The examiner inquired the author if there is any effect of asphalt binder ageing.

R: As explained in the text, asphalt binder ageing affected the resilient modulus. Probably because the binder became stiffer and then the same happened to RAP and cold recycled mixture.

C: Chapter 4: Since literature states that increasing RAP contents usually reduce strength and stiffness, the examiner inquired the author why the opposite happened in this case.

R: The text already enumerates possible reasons for that phenomena. However, the main reason is that, in this case, RAP particles are stronger than lateritic soil particles. For most cases reported in the literature, the base layer material is crushed stone, which is stronger than RAP.

C: The examiner inquired the author about the reason for using a stress level range between 10% and 90% to perform the fatigue tests. The examiner also asked if one can call "fatigue test" a test that does not achieve at least 1 million cycles.

R: In Chapter 4, the stress levels varied between 10% and 40% and some tests reached 1 million cycles. Stress levels as high as those reported in the literature (50-90%) could not be achieved, possibly because of insufficient cement content or drying shrinkage problems, as explained in the text. In Chapter 5, technical problems led to the use of higher stress levels (70-90%), showing the necessity of more sensitive equipment. However, the performed tests are still fatigue tests, since one can observe the degradation of the specimen even for failures after a few hundred cycles. Countries like Australia often use stress levels as high as those used in Chapter 5 for testing cement-treated materials. Furthermore, in this research, the tests were carried out in a stress-controlled mode, which causes more damage to the specimen than strain-controlled tests (like in France), and consequently is harder to attain 1 million cycles.

C: The examiner mentioned that low cement contents lead to materials with low stiffness and high heterogeneity, which increases variability. To reduce variability, one needs to increase the number of specimens. The examiner then inquired the author if it is really necessary to carry out fatigue tests for such materials.

R: Fatigue tests need to be performed even for materials treated with low cement contents (called lightly cement stabilised in Australia and South Africa). Although sometimes the failure criteria for such materials can be crushing at the top of the layer (also called top-down compressive fatigue in the USA), fatigue is always a concern. In this case, the author reduced the number of specimens per stress level aiming at increasing the tested stress levels. This procedure is accepted in Australia and leads to fatigue relationships with lower variability (higher coefficients of determination and standard error of estimate). Anyways, the obtained laboratory fatigue relationships are not ready to be used in pavement design, not only because of variability but because of the differences in comparison to field fatigue relationships.

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C: Chapter 3: The examiner inquired the author about the temperatures used for the production of the asphalt mixtures (RAP).

R: As mentioned in the text, the temperatures used for mix and compaction were the same as those used by other researchers that worked with the same asphalt binders and aggregates (Barros, 2017; Barros et al., 2018; Godoi, 2017; Godoi et al., 2017; Mensch, 2017).

C: Chapter 3: The examiner explained that for asphalt mixtures, the optimum content of rubber asphalt binder is usually higher than that of conventional asphalt binder. Then he inquired the author about the reason for using the same contents for all asphalt binders.

R: The author used the same levels for asphalt binder content to make possible identifying its effect. As mentioned in the text, the contents followed those recommended by Brazilian standards for asphalt concrete.

C: Chapter 3: The examiner inquired the author about the reason for using a 5 mm thick layer for the ageing of the asphalt binder.

R: The author explained that asphalt binder extraction and testing were not feasible due to the extremely long time necessary to produce RAP in the necessary amount with that purpose. The final chapter of the thesis cites this as a recommendation for further research. Using a 5 mm thick layer was an experimental limitation. Due to the high viscosity of rubber asphalt binder, it was not feasible to reach a thickness of less than 5 mm. Therefore, the author decided to subject the asphalt binders to the same ageing condition. However, results proved the soundness of the alternative asphalt binder ageing method. Besides, the ageing effect must have been even stronger for the asphalt binder in loose mixtures, since the thickness of asphalt binder film that coats aggregates is thinner than 5 mm.

C: The examiner inquired the author about the curing method generally employed in the field.

R: Brazilian standards state that an emulsion curing membrane should be applied over the recycled layer to maintain its moisture. This procedure was also observed by the author when visiting recycling services.

C: Chapter 3: The examiner inquired the author about carrying out indirect tensile strength tests after resilient modulus tests. Since this procedure may damage the specimens, the examiner stated that the author should perform a statistical analysis to guarantee the integrity of the specimens.

R: The results of strength showed low values of standard deviation, confirming its low variability. In light of the examiner’s comment, the author performed a statistical analysis (one-way ANOVA) which resulted in not statistically significant effects (p-value > 0.05) due to resilient modulus testing for all mixtures.

C: Chapter 4: The examiner inquired the author about the degree of compaction for specimen acceptation (>95%) and the homogeneity of the specimens.

R: The author explained that it was not possible to achieve a degree of compaction of 100%, especially for mixtures with high RAP percentages. Therefore, the author decided to reduce the minimum degree of compaction to 95%. The author also mentioned that he had similar problems with other compaction methods (Chapter 5). In Chapter 5, the minimum degree of compaction (97%) followed South African standards for lightly cement stabilised materials (C3). A possible explanation could be that the conventional compaction test (Proctor and AASHTO Modified) is not adequate for such materials. In this case, compaction tests for such materials should follow other methods (e.g. vibrating hammer method by ASTM D7382-08). Regarding heterogeneity, one can often identify it near the interface between layers.

C: Chapter 4: Since fatigue tests are often carried out with a frequency of 1 Hz (based on the effect of a vehicle passing over the pavement structure), the examiner inquired the author about the reason for using 5 Hz, which is more aggressive.

R: The author defined the frequency as the maximum that the equipment could apply while maintaining the load cycle form, leading to sooner test termination. A literature survey showed that most researchers use 2 Hz to test cement stabilised materials, but some countries (e.g. China, France and Greece) often use higher frequencies.

C: Chapter 4: The examiner argued that the author determined the resilient modulus as the average value between the 50th and 100th load cycles, but some specimens failed after a few hundred cycles. The examiner then inquired the author if one could not be losing part of the specimen degradation by using this procedure, explaining why some specimens do not show the typical three damage phases.

R: The author agreed with the examiner and explained that the pneumatic equipment used in Chapter 4 takes about 50 cycles to achieve a predetermined stress level. This problem did not happen in the study of Chapter 5 since the author used hydraulic equipment.

C: Chapter 4: The examiner inquired the author why he stated that crushing could be the main design criteria for the studied materials.

R: The author stated that the low fatigue resistance of some mixtures is due to a cement content that was not enough to produce bound materials with fatigue properties. The author should have performed a test to identify the minimum cement content (e.g. initial consumption of stabiliser). However, for comparison purposes, the author used the same amounts of cement and RAP as those used by Castañeda López et al. (2018) for cement-treated mixtures of RAP and crushed aggregates. Chapter 4 already reports these facts.

C: Chapter 4: The examiner argued that, in the mechanistic analysis, it would be interesting to investigate the fatigue behaviour of the asphalt wearing course since it could fail before the base layer. Besides, the author should also evaluate the subbase. Since the resilient modulus of the subbase material depends on the stress state, it could behave worse than the subgrade material, which would lead to high tensile stresses at the bottom of the base layer.

R: The author agreed with the examiner and stated that a crushing evaluation at the top of the base would also be interesting for further research.

C: Chapter 4: The examiner stated that the mixture with 4% of cement and 70% of RAP has a phase angle of approximately 36°. This viscoelastic behaviour allows analysis using other approaches (e.g. viscoelastic continuum damage theory).

R: The author agreed with the examiner and highlighted that there are plans for dynamic modulus tests aiming at identifying the viscoelastic behaviour of such mixtures. The last chapter of the thesis also reports this as a recommendation for further research.

C: Chapter 5: The examiner inquired the author if the lower values of strength and stiffness obtained using the South African test setup could be an effect of the maximum aggregate size, especially for the shale (50 mm).

R: The author stated he removed shale grains bigger than 25 mm before specimen preparation. However, Chapter 5 reports that the decreasing in strength and stiffness could be an effect of the interaction between maximum aggregate size and beam section since larger aggregates can generate weak spots in a smaller specimen. Even so, all materials showed the same effect, which corroborates that its cause can be the test setup.

C: Chapter 5: The examiner inquired the author about the reason for using such a high range of stress levels as 70-90%.

R: A literature survey stated that the stress levels used worldwide vary between 40% and 95%. However, the main reason for such a high range of stress levels was the fact that the used hydraulic equipment was not sensitive enough to apply lower stress levels.

C: Chapter 5: The examiner inquired the author about his observation on the effect of compaction being stronger for compressive than for tensile behaviour. The examiner explained this fact as a possible anisotropy created by the compaction. Tests carried out with cubes in the same and opposite direction as that used for compaction could lead to a conclusion.

R: The author agreed with the examiner since specimens are stronger if compacted and tested in the same direction.

C: Chapter 5: The examiner inquired the author if one could calculate the fatigue life of a pavement layer just by multiplying the fatigue life obtained using a laboratory relationship by a shift factor.

R: The author stated that it would not be correct. The performed mechanistic analysis resulted in different fatigue lives using field and laboratory relationships. Furthermore, the main difference between these two kinds of relationships is the slope; laboratory relationships are steeper, indicating that laboratory tests are more destructive than field tests, possibly because in the field the material is supported by another layer.

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C: The examiner inquired the author about what one still needs to study regarding the materials of pavements that could be recycled using cement.

R: The author stated that there still research to be done. For instance, it is necessary to study cold recycled cement-treated mixtures with other types of base materials, especially tropical aggregates and soils due to their particular characteristics that can have different effects on the mixture behaviour.

C: The examiner commented about the different types of cement produced in Brazil and suggested evaluating their effect on the behaviour of the recycled mixtures. This study could lead to the production of a specific binder for pavement recycling, following the examples of other countries (e.g. France and South Africa).

R: The author agreed with the examiner and added that could also be interesting to study alternative types of green cement with reduced environmental impact. The last chapter of the thesis reports these facts as a recommendation for further research.