Computational mechanics evaluation of the structural capacity and raveling processes in Permeable Friction Courses (PFC)

[Ph.D. Dissertation]

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July 2020
“What you get by achieving your goals is not as important as what you become by achieving your goals.”

— Henry David Thoreau
EXECUTIVE SUMMARY

In the last decades, researchers in the pavement engineering field have become more aware of the importance of developing new technologies with durable and eco-friendly materials and techniques that bring safety and comfort to the road users. Among these technologies, Permeable Friction Course (PFC) mixtures are among the most popular, due to their multiple benefits. PFC are hot mix asphalt materials used as thin (i.e. 2-6 cm) surface layers over conventional pavements, which are characterized for having an open gradation that results in high air void (AV) contents (i.e. between 15 -25%) (Kandhal 2002, Putman and Kline 2012). This characteristic makes these mixtures highly permeable, which brings several safety and environmental benefits, such as reduction of hydroplaning risks, reduction of backsplash and spray from vehicle tires, improvement of mark visibility during raining events, and reduction of road noise pollution (Huddleston et al. 1991, Cooley et al. 2009).

Despite these benefits, the principal distress affecting the durability of PFC mixtures is the loss of aggregates from the surface—a phenomenon known as raveling— which is caused by the repetitive pass of vehicles and intensified by the presence of moisture and other environmental factors. Since the internal strength and durability of PFCs depend on the stone-on-stone contact network within the coarse aggregate fraction of the mixtures (Alvarez et al., 2010, 2018), the assessment of the structural capacity of PFC mixtures and the evaluation of the material failure process that occurs at the material located at the stone-on-stone aggregate contacts or at the aggregate-mortar interface (cohesive or adhesive failure) (Mo et al. 2007) become topics of interest.

The main objectives of this dissertation are to use 2-Dimensional (2D) micromechanical computational models of PFC layers to assess the internal strength (i.e. structural capacity) of these mixtures, to identify the factors that determine the structural contribution of these layers to the pavement structure, and to quantify the effect of several mechanical and environmental conditions on the initiation and progression of raveling in PFCs.

To achieve these objectives, a preliminary set of computational models were developed using Finite Element (FE) modeling and X-ray Computer Tomography (CT). Results from these preliminary models show that PFC layers do contribute to the pavement structural capacity but that this contribution strongly depends on different microstructural and geometrical characteristics. The initial models were also subjected to the pass of a wheel load under different operational conditions (i.e. load magnitude, vehicle speed and braking conditions and external temperature), material combinations (i.e., volumetric properties, air void content) and pavement properties (i.e. thickness of the PFC and properties of the underneath layers). These results also showed that raveling is mainly a fracture Mode I (i.e. opening mode) process, highly influenced by the volumetric properties of the mixture (e.g. total air void and binder content), and loading conditions (e.g. extremely high load, breaking over the pavement and low speed).

The results of these preliminary FE PFC models evidenced the necessity of working with PFC microstructures that better represent the internal mechanical behavior of these mixtures. For this reason, this dissertation proposes a novel methodology to generate random 2D PFC microstructures that
efficiently represent the mechanical behavior of a 3-Dimensional PFC mixture, while providing the possibility of conducting simulations at lower computation costs when compared to 3D models with similar geometry. This methodology can be used to generate multiple random microstructures of any PFC mixture to conduct computational probabilistic and statistical studies of the functionality, durability and mechanical response of PFC mixtures under different field conditions.

Moreover, the literature review and the results of the preliminary models showed a lack of information on how climate-related factors (mainly the presence of air, temperature and moisture) impact the mechanical properties and durability over time. Consequently, an experimental plan was proposed and developed to assess the combined effects of aging and moisture on the linear viscoelastic and fracture properties of the asphalt mortar asphalt (i.e. mixture of asphalt binder and the aggregate particles passing sieve size #16, 1.18 mm (Caro, Masad, Airey, et al. 2008)) present at the stone-on-stone contacts within PFC mixtures. To achieve this goal, loose asphalt mortar of a typical PFC was aged at two different conditions (short- and long-term). Afterwards, the mortar was compacted, and the testing specimens were subjected to different dry-wet-dry moisture vapor cycles. The linear viscoelastic and fracture properties of the mortar were determined through Dynamic Mechanical Analyzer (DMA) and Semi-Circular Bending (SCB) tests. The experimental results showed that short- and long-term aging impacted the dynamic modulus of the mortar, while moisture had a negligible effect on this property. Also, the coupled effects of long-term aging and dry-wet-dry moisture cycles reduced the fracture energy and some additional fracture parameters of the asphalt mortar between 2 and almost 10 times with respect to the short-term aging condition in dry state. These experimental data provided an initial insight to the change in material properties caused at the stone-on-stone contacts in PFC mixtures during their service life due to environmental factors. The obtained data also offer a reliable input data as part of more accurate FE computational models.

The new methodology to generate 2D PFC microstructures and the material properties obtained from the experimental plan were used to evaluate the raveling susceptibility of PFC mixtures after several service years (i.e. approx. 6 years) using FE with realistic field operation conditions (e.g. different traffic and material degradation conditions). These FE models included Cohesive Zone Elements (CZE) to incorporate fracture mechanics principles as part of the quantification of the degradation occurring at the PFC mortar contacts. The results from these models showed that raveling susceptibility of a PFC layer after around 6 service years could be 10% under average loading conditions, while the raveling susceptibility of the same PFC layer under extreme loading conditions (i.e. extreme load magnitude, vehicle breaking over the pavement, low vehicle speed and considerations of both environmental and fatigue degradation on material properties) could raise up to 90%.

Future studies on this topic include, among other topics, the evaluation of PFCs degradation after including adhesive fracture properties at the aggregate-mortar interface, and the assessment of the mechanical response of different design approaches for PFC mixtures after considering the uncertainty induced by the heterogeneity of the PFC microstructures (i.e. computational analysis of multiple randomly generated PFC microstructures as part of reliability-based design approaches).
First of all, I would like to thank my advisor Silvia Caro. Silvia, thank you for your time, patience and advises you gave to me in the last years. All this work is thanks to you. Thank you for being more than my advisor, for being a role model and an unconditional friend. I will certainly miss working with you.

I would like to thank Professor Yong-Rak Kim. Thank you for your guidance and support to this work and thank you for the opportunity to work and live in Lincoln, it was a great experience.

I would like to thank my family. Thanks to my Nonna, my parents Carlos H. and Mary, and my brother Felipe for your patience and unconditional support. Thank you for loving me and always reminding me that I can achieve huge things. Thank you for teaching me the importance of discipline and always be an honorable person. This work is dedicated to you and to my grandfather Carlos Elias, who is now in heaven.

I would like to thank professor Nicolas Estrada for his guidance and time. His support was fundamental in the development of this work.

I wish to thank the current and past members of the research group GeoSI. Thank you for your support and friendship, especially thanks to Daniel Castillo and Eduardo Rueda.

I thank my former coworkers of the Sustainable Infrastructure and Materials Research Group at University of Nebraska-Lincoln for their support and friendship, specially to Jamilla, Shayan, Keyvan, Behrad and Mahdieh.

I would like to thank the Fulbright commission and the Administrative Department of Science, Technology and Innovation of Colombia (Colciencias) for their financial support, and for giving me the opportunity to live this wonderful experience called a Ph.D.

Finally, I would like to thank the Universidad de los Andes for being a stunning organization of which I made part for the last 10 years and of which I always be part.
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<tr>
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<th>Definition/Description</th>
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<tbody>
<tr>
<td>2D</td>
<td>Two-dimensional</td>
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<tr>
<td>3D</td>
<td>Three-dimensional</td>
</tr>
<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
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<tr>
<td>ACL</td>
<td>Average Contact Length</td>
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<td>AI</td>
<td>Angularity Index</td>
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<td>ARB</td>
<td>Asphalt Rubber Binder</td>
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<td>$A_{lig}$</td>
<td>Ligament Area</td>
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<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
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<tr>
<td>ATPB</td>
<td>Asphalt Treated Permeable Base</td>
</tr>
<tr>
<td>AV</td>
<td>Air Voids</td>
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<tr>
<td>BC</td>
<td>Binder Content</td>
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<tr>
<td>CN</td>
<td>Coordination Number</td>
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<tr>
<td>COV</td>
<td>Coefficient of Variation</td>
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<td>CRI</td>
<td>Cracking Resistance Index</td>
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<tr>
<td>CT</td>
<td>Computer Tomography</td>
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<td>DE</td>
<td>Discrete element Method</td>
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<td>DH</td>
<td>Drop Height</td>
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<tr>
<td>DMA</td>
<td>Dynamic Mechanical Analysis</td>
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<tr>
<td>DSR</td>
<td>Dynamic Shear Rheometer</td>
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<tr>
<td>ESAL</td>
<td>Equivalent Single Axle Load</td>
</tr>
<tr>
<td>FA</td>
<td>Floating Aggregates</td>
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<td>FDOT</td>
<td>Florida Department of Transportation</td>
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<tr>
<td>FE</td>
<td>Finite Element method</td>
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<td>FHWA</td>
<td>Federal High Way Administration</td>
</tr>
<tr>
<td>$F_l$</td>
<td>Flexibility Index</td>
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<td>Florida Porous Friction Course Mixture</td>
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<tr>
<td>$G_f$</td>
<td>Fracture Energy</td>
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<tr>
<td>GR</td>
<td>Granite</td>
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<td>HMA</td>
<td>Hot-mix Asphalt</td>
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<td>Indirect Tensile</td>
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<td>Modified Saturation Ageing Tensile Stiffness test</td>
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<td>NCAT</td>
<td>National Center of Asphalt Technology</td>
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<tr>
<td>NCHRP</td>
<td>National Cooperative Highway Research Program</td>
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<td>NMAS</td>
<td>Nominal Maximum Aggregate Size</td>
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<td>Optimum Binder Content</td>
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<td>Open Graded Friction Courses</td>
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<td>Porous Asphalt</td>
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<td>Probabilistic density function</td>
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<td>Performance Grade</td>
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<tr>
<td>PGA</td>
<td>Porous Graded Asphalt</td>
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<tr>
<td>$P_{\text{max}}$</td>
<td>Peak Load</td>
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<td>PMA</td>
<td>Polymer Modified Asphalt</td>
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<td>RH</td>
<td>Relative Humidity</td>
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<td>R.E.</td>
<td>Remaining Energy</td>
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<td>Raveling Index</td>
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<td>Short-term Aged conditioning</td>
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<td>Styrene-Butadiene-Styrene</td>
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<td>SCB</td>
<td>Semi Circular Bending test</td>
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<td>Superpave Gyatory Compactor Research Association for Underground Transportation Facilities</td>
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<td>Tensile Strength Ratio</td>
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<td>Texas A&amp;M Transportation Institute</td>
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<tr>
<td>UBG</td>
<td>Unbounded Granular Base</td>
</tr>
<tr>
<td>U.S.</td>
<td>United States</td>
</tr>
<tr>
<td>$W_f$</td>
<td>Work of Fracture</td>
</tr>
</tbody>
</table>
Academics working in civil engineering have become more interested in the search for new technologies that enhance the benefits provided by traditional infrastructure systems. In the case of pavement engineering, different innovative surface technologies have been developed, among which Permeable Friction Course (PFC) mixtures are the most popular. These mixtures, which are placed over conventional flexible pavements, are characterized for having an open gradation and, consequently, a high air void (AV) content, which allows quick water drainage from the pavement surface (Huber 2000). PFC mixtures are well recognized for offering several safety and environmental benefits. In terms of their safety advantages, PFCs minimize hydroplaning-related accidents and enhance users’ visibility under rainy conditions (Nicholls and Carswell 2001, Dell’Acqua et al. 2011). The most relevant environmental benefit is related to the reduction of noise pollution produced by the repeating pass of vehicles over the surface of the pavement (Freitas et al. 2009). Due to their positive impacts, during the use of PFC mixtures have become popular in several states in the United States and in several European countries during the last 30 years (Hernandez-Saenz et al. 2016).

Despite these benefits, the main drawbacks of PFCs are related with their long-term functionality and durability. In terms of functionality, the capability of PFCs to reduce noise and drain water (i.e. permeability) gets affected through time due to clogging. In terms of durability, the loss of aggregates from the PFC surface due the repetitive load passes and by the presence of moisture, a phenomenon called raveling, strongly impacts and reduces the service life of these mixtures. In general, clogging and raveling are phenomena that progress rapidly and enhance the appearance of other distresses, increasing the long term costs of asphalt pavements with PFCs (Huber 2000, Alvarez et al. 2006, Cooley et al. 2009).

This dissertation aims at contributing to the current state of knowledge related to the internal strength of PFC mixtures and the degradation mechanisms causing raveling. To achieve this goal, several computational micromechanics models and an accompanying experimental plan are proposed and presented in this document. Initially, 2-Dimensional (2D) PFC microstructure geometries were obtained from X-ray Computer Tomography (CT) images. These 2D images were used as input parameters in computational mechanical models develop in the finite element (FE) software Abaqus®. These models were used to obtain an initial first quantification of the internal strength and overall structural contribution of these layers, as well as the degradation processes within the internal stone-on-stone contact network of PFCs which might cause raveling.

The results obtained in these initial computational approaches evidenced the need of working with 2D PFC microstructure geometries that could better represent the mechanical behavior of actual 3-Dimensional (3D) PFCs, and evidenced the absence of reliable available data on the viscoelastic and fracture properties of the asphalt mortar located at the stone-on-stone contacts. Consequently, a novel methodology to randomly generate 2D PFC microstructures that better represent the mechanical behavior
of actual PFC mixture was proposed. In addition, an experimental plan was designed and conducted in order to obtain the viscoelastic and fracture properties of the asphalt mortar (i.e. mixture of asphalt binder and the aggregate particles passing sieve size #16, 1.18 mm (Caro, Masad, Airey, et al. 2008)) located at the stone-on-stone contacts of PFCs affected by different environmental conditions, as those expected in the field.

The microstructure geometries obtained using the methodology proposed and the data collected from the experimental tests were used in the last stage of this work as input parameters in new computational micromechanics models of the mechanical response of PFC mixtures under realistic in-service conditions. The results from these new simulations permitted to identify the road conditions and the properties and characteristics of the PFC mixtures that promote or prevent raveling. This information could be efficiently used to improve the long-term durability of PFC mixtures.

The initial section of this dissertation presents the objectives of this research. Next, a comprehensive review of the recent Finite Element (FE) computational works conducted to characterize raveling processes are presented, followed by a detailed description of the different computational models and experimental plan developed, as well as the main findings. Finally, the last section summarizes the results obtained and describes some guidelines for future work.

1.1. General objective

The main objective of this dissertation is to evaluate the mechanical response of PFC mixtures and their susceptibility to raveling, through the design and implementation of diverse computational mechanics models.

1.2. Specific objectives

The specific objectives of this proposal are:

- Design and implement FE computational models to quantify the structural integrity and contribution of PFC mixtures over conventional pavement structures.

- Evaluate and analyze the influence of the micromechanical characteristics of PFC mixtures on their mechanical response.

- Design and conduct an experimental plan to determine the changes in the mechanical and fracture properties of the stone-on-stone contacts within PFCs when subjected to environmental conditions.

- Conduct FE computational mechanics models that use representative geometries of the microstructure of the mixtures in combination with fracture principals to study the susceptibility and evolution of raveling within PFC mixtures subjected to realistic field operation conditions.
CHAPTER II
LITERATURE REVIEW

This chapter provides a general overview of the definition, advantages and disadvantages of PFC mixtures. This is followed by a brief summary of the structural contribution of PFC and most relevant computational FE works developed to evaluate degradation processes related to raveling within these mixtures and the main challenges presented in this topic. The information herein presented would permit to clearly identify how this dissertation constitutes a relevant contribution in the study of PFC mixtures using computational models.

2.1. Definition, advantages and challenges of PFC mixtures

PFC, also called Porous Graded Asphalt (PGA), Open-Graded Friction Courses (OGFC) or Porous Asphalt (PA) — are open-graded hot mix asphalt materials. These mixtures are characterized for having higher total air void (AV) contents, which are between 18% and 25%, and a higher binder content in comparison with conventional dense-graded hot-mix asphalt (HMA) (Cooley et al. 2009). PFC mixtures are frequently placed as thin layers —usually between 2 to 6 cm— over conventional flexible pavements. Due to their high AV content and coarse aggregate fraction, these mixtures bring several safety and environmental benefits.

In terms of safety, their high AV content allows water to drain down under rainy events. This condition reduces hydroplaning (Dell’Acqua et al. 2011) and wet-weather splash and spray during rainy events up to 90-95% in comparison with HMA mixtures, improving the visibility in 2.7-3.0 times with respect to conventional mixtures (Nicholls 1997, Rungruangvirojn and Kanitpong 2010). The use of PFC mixtures as part of the pavement structure has also proven to reduce glare at night and in wet weather, showing enhancements in mark visibility and improvements in the friction of the pavements surface (Huddleston et al. 1991, Lefebvre 1993). PFC mixtures also bring some other benefits to the driver, such as higher average operation speeds and traffic capacity under rainy weather (Cooley et al., 2009), improvements in the pavement smoothness (Bennert et al. 2005), and fuel consumption reduction (Khalid and Pérez 1996).

The environmental benefits are related to the reduction in the pollutants in storm water runoff and in the tire/pavement noise levels. In general, the presence of PFC mixtures has shown a 90% reduction in the total suspended solids presented in the water runoff in comparison with HMA mixtures (Eck et al. 2010), and a reduction of 3 to 6dB in the noise produced by the repeating pass of the vehicles in comparison with HMA mixtures (Freitas et al. 2009, Chen et al. 2018) and between 5.5 and 10.5dB when compared with Portland Cement Concrete layer surfaces (Kandhal 2004). According to Hernandez-Saenz et al. (2016), PFC’s environmental benefits are the major reason for using this type of mixtures in European countries, while safety benefits are the main reason for using PFC mixtures in the United States.
Beyond their multiple benefits, there are several challenges associated with the functionality and durability of PFC mixtures, which typically increase the overall maintenance cost of flexible pavements that include this surface layer.

In terms of functionality, which refers to the capacity of the PFC to maintain its beneficial properties through the service life of the pavement (Huber 2000, Mallick 2000), the principal difficulty is related to clogging. This phenomenon consists in the sedimentation of pollutants within the pavement surface, which reduces the permeability and noise reduction capability of the mixtures (Kandhal 2004, Arámbula-Mercado 2014).

On the other hand, durability is related to the capacity of PFCs to resist distress and/or failure (Huber, 2000; Mallick et al., 2000). Although PFC mixtures can experience fatigue and permanent deformation (Alvarez et al. 2006, Yildirim et al. 2007, Cooley et al. 2009, Wang and Wang 2011), the principal phenomenon related to durability in these materials is raveling. This phenomenon refers to the loss of aggregates from the top surface of the PFC layer due to the abrasion caused by traffic, and it is usually aggravated by the presence of moisture (Huddleston et al. 1991, Poulilakos and Partl 2009, Arámbula-Mercado et al. 2016, Zhang and Zhen 2017). Raveling in PFC pavements progresses rapidly and accelerates the appearance of other distresses, causing a reduction in the overall service condition of the pavement, an increase in the cost of maintenance activities, and a reduction in the road network capacity and serviceability (Alvarez et al. 2006, Mo et al. 2007, Cooley et al. 2009, Wu, Yu, et al. 2020). Table 1 summarizes some typical service life values of PFC mixtures reported in two literature review works conducted by Alvarez et al. (2006) and Arámbula-Mercado et al., (2016).

<table>
<thead>
<tr>
<th>Typical Service Life (years)</th>
<th>Type of Mixture</th>
<th>Country</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 or more</td>
<td>PFC (in more than 70% of the states)</td>
<td>United States</td>
<td>(Mallick., 2000)</td>
</tr>
<tr>
<td>13</td>
<td>Rubber-modified PFC (Arizona)</td>
<td>United States</td>
<td>(Huber, 2000)</td>
</tr>
<tr>
<td>15</td>
<td>PFC (Wyoming)</td>
<td>United States</td>
<td>(Huber, 2000)</td>
</tr>
<tr>
<td>6 to 10</td>
<td>PFC (in more than 45% of the states)</td>
<td>United States</td>
<td>(Yildirim et al., 2007)</td>
</tr>
<tr>
<td>7 to 10</td>
<td>PA</td>
<td>United Kingdom</td>
<td>(Nicholls and Carswell 2001)</td>
</tr>
<tr>
<td>7</td>
<td>PA</td>
<td>Denmark</td>
<td>(Vejtekniik Institut 2005)</td>
</tr>
<tr>
<td>8 to 12</td>
<td>PA</td>
<td>France</td>
<td>(Huber, 2000)</td>
</tr>
<tr>
<td>8 to 12</td>
<td>PA</td>
<td>Germany</td>
<td>(Dell’Acqua et al. 2011)</td>
</tr>
<tr>
<td>9 to 16</td>
<td>PGA</td>
<td>Netherlands</td>
<td>(van der Ven 2010, Voskuilen and Elzinga 2010)</td>
</tr>
<tr>
<td>5 to 18</td>
<td>PA</td>
<td>Netherlands</td>
<td>(Huurman et al. 2010a)</td>
</tr>
<tr>
<td>12</td>
<td>PGA</td>
<td>Netherlands</td>
<td>(Voskuilen et al. 2004)</td>
</tr>
<tr>
<td>8</td>
<td>Two-layer PGA</td>
<td>Netherlands</td>
<td>(Hofman et al. 2005)</td>
</tr>
<tr>
<td>Typical Service Life (years)</td>
<td>Type of Mixture</td>
<td>Country</td>
<td>Reference</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>--------------------------</td>
<td>---------</td>
<td>-------------------------------------</td>
</tr>
<tr>
<td>10</td>
<td>PGA</td>
<td>Spain</td>
<td>(Ruiz et al. 1990)</td>
</tr>
<tr>
<td>20</td>
<td>SBS modified PGA</td>
<td>Japan</td>
<td>(Suzuki et al. 2010)</td>
</tr>
<tr>
<td>8 to 10</td>
<td>PGA</td>
<td>Japan</td>
<td>(Takahashi 2013)</td>
</tr>
<tr>
<td>15 to 17</td>
<td>Typical HMA</td>
<td>USA</td>
<td>(Von Quintus et al. 2005)</td>
</tr>
</tbody>
</table>

One of the main problems related to the durability of PFC mixtures is the difficulty to preserve the integrity of their microstructures during winter seasons. Due to their high AV contents, PFC mixtures freeze sooner and for longer periods than HMA mixtures (Fay and Akin 2013). Consequently, de-icers must be applied in a higher rate and more frequently than when using HMAs (van der Zwan 2011). According to a recent study, this seem to be the main reasons explaining why several of the northern states of Unite States have discontinued the use of PFC mixtures in the last three decades (Hernandez-Saenz et al., 2016).

In the last two decades, different experimental works have been conducted to improve PFC mixture design, prevent early raveling initiation, and enhance PFC service life (Alvarez et al., 2012; Zhang et al., 2016; Manrique-Sanchez et al., 2018; Arámula-Mercado et al., 2019). Several of these works have focused on the role of the properties of the individual components of the mixture (i.e. aggregates, binder and additives) on the durability of the PFC, such as the assessment of the morphological properties of the aggregates that improve the quality of the stone-on-stone contacts (Putman and Kline 2012); the use of regular and heavily polymer modified asphalt binders (Jenks 2017, Arámula-Mercado et al. 2019); the use of anti-stripping agents to enhance adhesive properties and prevent moisture damage (Gu et al. 2018); and the addition of mineral or cellulose fibers to prevent drain down and improve durability (Cooley et al. 2000, Alfonso et al. 2017).

Moreover, only few efforts on the evaluation of the structural contribution of PFC mixture were identified in the literature (Moore, 1989; Van Der Zwan et al., 1990; Van Heystraeten and Moraux, 1990; Timm and Vargas-Nordcbeck, 2012; Wang. et al., 2014; Dylla and Hansen, 2015). It is believed that the combination of the typical thin thickness of these layers, which are usually determined by the hydrological design and the gradation of the mixture (Ruiz et al. 1990, Mallick et al. 2000), and their limited service life, which is typically between 4 and 12 years (Table 1), are two of the main reasons explaining why the presence of these layers is not usually included when assessing the overall structural capacity of pavement structures. Considering that raveling is a material failure process that occurs at the asphalt mortar located at the stone-on-stone aggregate contacts or at the aggregate-mortar interface (cohesive or adhesive failure) (Mo et al. 2007), and that the internal strength and durability of PFCs depend on the stone-on-stone contact network within the coarse aggregate fraction of the mixtures (Alvarez et al., 2010, 2018), the study of the internal strength and hence the structural contribution of PFC mixtures become a topic of interest. As explained in the next section, there are not conclusive results about the structural capacity of PFCs layers and the factors that determine such structural contribution to the pavement. For this reason, the assessment of the structural contribution of PFCs layer to flexible pavements became one of the general objectives of this dissertation.

There have also been numerous computational mechanics efforts to understand the physical and mechanical modes of failure that can lead into PFC raveling (Mo et al. 2010, 2014, Arámula-Mercado et al., 2018).
et al. 2016, 2019). Most of these works have been implemented in finite element (FE) simulations, and have quantified the stress and strain states at the stone-on-stone contacts or mortar-mortar contacts under different loading and environmental conditions. These studies have shown that PFCs become more susceptible to raveling with extremely high or low temperatures (Huurman et al. 2010a). However, a main challenge of these computational works is the absence of reliable data on the fracture properties of the asphalt mortars located at the stone-on-stone contacts, and the degradation of those properties through the service life of PFC.


- Incompatibility between binder and aggregates.
- Placement of the mixtures at low ambient temperatures.
- Inadequate compaction.
- Insufficient binder content.
- Early stop-and-go traffic.
- Binder drain down.
- Binder softening generated by oil and fuel drippings.

On the other hand, there is still a lack of information about how climate-related factors (mainly the presence of air, temperature and moisture) impact the mechanical properties and durability of PFCs over time. It is well recognized that the properties of the asphalt binder in the mixtures evolve with aging, which is defined as the chemical reactions caused by the presence of air, temperature and pressure in the binder through time, and that lead to changes in the rheological, physical and mechanical properties of the material (Traxler 1961, Petersen et al. 1993, Herrington 1998). Additionally, the presence of moisture within asphalt mixtures deteriorates the adhesive bonds between the aggregates and asphalt binder, and the cohesive bonds between the mortar contacts through a degradation process called moisture damage (Wright 1965, Kiggundu and Roberts 1988, Caro, Masad, Bhasin, et al. 2008, Ban et al. 2013).

Due to the high AV content of PFCs, it is postulated that the combined influence of climate-related processes in PFCs would be more significant than in conventional dense-graded HMA mixtures. Although several works have assessed the isolated influence of moisture damage (e.g. Poulakakos and Partl (2009) and Varveri et al. (2016)) or aging (e.g. Hagos (2008) and Frigio et al. (2016)) on PFCs mixtures, there are still limited studies on the effects of coupled climatic conditions on the mechanical properties of these materials. Besides, most of the works in this area have been restrained to regular dense-graded HMAs (Das et al. 2015, López-Montero and Miró 2016, Yang et al. 2016). The only two works that were identified during this literature review in this topic that have been conducted on PFCs, are those developed by Arámbula-Mercado et al., (2019) and Wu et al. (2020). Arámbula-Mercado et al., (2019) subjected unaged and aged PFC specimens to the Indirect Tensile Strength test (IDT) after having conditioned the specimens to one freeze/thaw cycle. The authors computed the ratio of the IDT strength in wet and dry conditions (i.e. tensile strength ratio, TSR) of unaged and aged PFC mixtures,
finding that the IDT strength increased with aging an average of 17.5%, and that the IDT strength decreased under wet condition in a range of 10 to 29%. In contrast, Wu et al. (2020) designed a new device (M-SATS) to subject PFC mixtures to coupled aging and moisture conditions. The authors aged the PFC specimens in both short- and long-term, and included several moisture conditioning procedures. The PFC specimens were subjected to wheel tracking and three-point bending beam fatigue tests. The results showed that short-term aging increased the stiffness of the PFCs; while long-term aging led to adhesive failure between the binder and the aggregates. The results also showed that the coupled aging and moisture conditioning decreased the rutting resistance of the specimens in 30 and 50% for short- and long-term aging, respectively, when compared to the dry condition. Finally, moisture was observed to play an insignificant role in the fatigue performance of long-term aged specimens. Overall, these recent efforts highlight the importance of evaluating the influence of coupled climatic effects on the mechanical response of PFC mixtures.

Due to their importance for the development of this dissertation, the following two sections explore with more detail the results of existing studies on the structural integrity and strength of PFC mixtures and on computational mechanics models developed to assess the degradation of these mixtures due to raveling.

2.2. Structural contribution of PFC mixtures

Most works dealing with the structural contribution of PFC mixtures have evaluated the incorporation of the structural capacity of PFCs as part of the AASHTO-93 pavement design methodology. For example, in 1989 the California Department of Transportation (Caltrans) in cooperation with the U.S. Department of Transportation and the Federal Highway Administration (FHWA), funded a study to evaluate the structural properties of asphalt treated permeable base (ATPB) and PFC layers (Moore 1989) using deflection measurements. According to the results, the ATPB had a gravel-equivalent factor \( G_f \) –which is analogous to the structural coefficient used in the AASHTO-93 pavement design methodology—equal to that of a regular HMA, whereas the deflection measurements were not conclusive for the PFC layers, mainly due to their thinner thickness (i.e. 2 cm). However, based on laboratory tests and on the similarity of the resilient moduli between the ATPB and the PFC layers (i.e. 972.16 and 1075.6 MPa, respectively), the teamwork also suggested a \( G_f \) value for the PFC layer equal to that used for regular HMAs (Moore 1989). Using the structural coefficient of conventional HMA for these permeable mixtures has been also implemented in Spain (Ruiz et al. 1990), and recommended by other organizations, such as the Oregon Department of Transportation (Kandhal 2002). Moreover, the FHWA advised the use of the PFC structural coefficient proposed by Hansen (2008), which ranges between 0.40 and 0.42 (Dylla and Hansen 2015), being similar to those recommended by AASHTO (1993) for regular HMA materials (i.e. 0.44).

Other studies have shown that the inclusion of the structural capacity of the PFC layer as part of existing mechanical-empirical design methods could extend the fatigue life of HMA layers and reduce the potential for subgrade rutting. Wang. et al. (2014), for example, determined the dynamic modulus of different PFCs and included them into the Mechanical Empirical Pavement Design Guide (MEPDG) trial design software. The selected pavement structures were subjected to 20,000 average annual daily truck traffic at different temperatures, and fatigue degradation and subgrade rutting were estimated based
on the mechanical response of the layers. For this specific case, the results showed that the inclusion of the structural capacity of the PFC could extend the fatigue resistance of the HMA in more than 500 million loading cycles at a temperature of 0°C, and that the surface rutting of the pavement might get reduced in a range of 0.3 to 1.0 mm at the end of the service life of the structure.

Nevertheless, full-scale and laboratory tests have suggested that the actual structural capacity of a PFC layer might be 30 to 79% that of a regular HMA (Van Heystaeten and Moraux, 1990; Timm and Vargas-Nordbeck, 2012), which raises important questions about current assumptions regarding the actual capacity of these layers. Moreover, some works about PFCs in the Netherlands have applied mechanical-empirical design methods to evaluate the effect of the initial stiffness modulus, temperature, aging and stripping characteristics of PFCs to determine their structural contribution on regular pavement structures (Gerritsen et al. 1987, Rijkswaterstaat 1989, Van Der Zwam et al. 1990, Hopman et al. 1998). The results showed that, depending on the thickness of the structure, the structural contribution of PFCs was expected to be near 50% that of a regular HMA. Moreover, the MEPDG simulations conducted by Wang et al. (2014) suggested that, although the structural contribution of the PFC varied with the selected climate, at intermediate temperatures near 21 °C, and for the specific loading conditions evaluated in the study, a PFC layer of 3.8 cm was equivalent to an HMA layer of 2.5 cm; evidencing that the contribution of PFCs is only a portion of that provided by regular HMAs.

As noticed, existing works suggest that the PFCs do contribute to the structural capacity of flexible pavements, but this contribution is not necessarily equivalent to those of regular HMA layers. This also suggests that there is a lack of understanding of the origin of the structural contribution of PFC mixtures and the role of the geometrical and microstructural characteristics of these mixture over their structural contribution to the pavements.

2.3. Computational models to assess raveling in PFC mixtures

The necessity to identify and better understand the mechanisms associated with the initiation and progression of raveling in PFC mixtures has motivated the development of several numerical modeling efforts. Most of these efforts have been developed using FE models, although other modeling techniques such as artificial neural networks and Discrete Elements (DE) have also been used for similar purposes (Miradi and Molenaar 2004, Alvarez, Mahmoud, et al. 2010). Considering that this research proposal focuses on FE techniques, the literature review herein presented only includes studies that have evaluated raveling processes in PFC mixtures using this type of computational models.

The majority of these FE models have been conducted at Delft University of Technology in the Netherlands, with the collaboration, in some specific cases, of Wuhan University of Technology in China. The Netherlands has more than 90 percent of the primary road networks surfaced with PFCs (Voskuilen 2011), which explains this country’s necessity to gain fundamental insight about the performance and deterioration mechanisms of these materials. Consequently, a series of research projects focused on the behavior of PFCs have been sponsored by the Road and Hydraulics Division of the Dutch Ministry of Transportation from 2007 to 2010 (Mo et al. 2007).

The main Dutch project sponsored by this agency was the DWW-2923, which is commonly referred to as Life Optimization Tool for Porous Asphalt (LOT). The LOT project resulted in a tool that combines
experimental work and computational modeling to estimate the fatigue life expectancy of PFC mixtures. This tool allows for the evaluation of different material’s properties and operation conditions required to improve the raveling resistance of PFC mixtures (Mo et al. 2010, 2014). The initial objective of this project was to analyze the physical mechanisms of raveling. To achieve this goal, different 2D and 3D PFC-FE models were developed using the commercial software Abaqus®. Some of the latest versions of FE models using LOT have permitted the prediction of the type of failure that could result in raveling (i.e. loss of adhesion between the aggregate and the binder surface contact, or loss of cohesion between the binder material at the stone-on-stone contact areas), and the number of the tire cycles required to develop each type of failure (Huurman et al. 2010c, Mo et al. 2014). Details of these works are explained in this section.

Before the LOT project officially started, however, the researchers made some initial attempts to understand the initiation of raveling in PFC mixtures. These models made part of what could be considered the pre-LOT project. Thus, the first FE raveling model in PFC materials was reported in 2006 using the 3D Computer Aided Pavement Analysis (CAPA-3D) FE platform, which is an in-house FE software developed at Delft University (Mo et al., 2007). This model aimed at studying the magnitude and location of the critical stresses that were developed in a simplified 2D PFC system (Figure 1) under different moving tire loads. As observed in Figure 1, the aggregates in the PFC model were represented by one-size polygons and were assumed to be coated by a thin film of mastic (i.e. combination of asphalt binder and fillers, passing sieve # 200). This model provided important initial insights about raveling, since it concluded that this phenomenon was mainly a low temperature problem. In addition, it was observed that the high shear stresses quantified at the interfacial zones (i.e. aggregate-asphalt mastic material) could generate adhesive failure. It was also observed that the characteristics of the particle skeleton were a parameter that highly influenced the appearance of raveling.

Figure 1. 2D model using polygonal particles covered by a constant mastic film (modified after Mo et al. 2007).

At the end of 2006, and still as part of the pre-LOT program, an initial comparison between three-dimensional (3D) and 2D models of PFCs was conducted. The principal objective was to study the
potential of the 2D and 3D microstructures to develop raveling by comparing the mechanical responses of both systems using FE modeling. The 2D model illustrated in Figure 1 was modified by changing the packing particle angle (i.e. the initial model counted with 2 contacts per contact face, the new 2D model counted with 4 contacts per contact face) with the objective of evaluating the influence of this parameter over the overall behavior of the PFC system. The 3D model was composed by an array of one-size spheres coated with a thin film of mastic, as observed in Figure 2.

![3D model (top view)](image1)

![3D model (bottom view)](image2)

**Figure 2.** 3D meso-scale model (modified after Mo et al. 2008).

The stresses and strains obtained from the mortar contact areas and the mortar-aggregate interfaces zones of the models were initially evaluated using the Mohr-Coulomb-like failure envelope and von Mises equivalent stress to evaluate adhesive and cohesive failure, respectively. These parameters permitted to compare the response of the mortar phase in both models under the mechanical load applied by the moving wheel. Since the direct measurement of the tensile strength of the asphalt-stone adhesion that could represent real traffic loading was not available, a damage accumulation model that used the stresses developed at the asphalt-stone adhesion (i.e. adhesive failure) during the passage of the rolling wheel was used to calculate the potential to raveling of the system (Mo et al. 2006).

\[ \dot{D} = \left( \frac{\sigma_e}{\sigma_0} \right)^n \text{ for } \sigma_e > 0, D = 0 \text{ for } \sigma_e < 0 \text{ with } \sigma_e = \sigma_n + \frac{|\tau|}{\tan \theta} \]

[1]
where \( \dot{D} \) is the rate damage, \( \sigma_e \) the equivalent tensile stress (MPa), \( \sigma_n \) the normal stress (MPa), \( \sigma_0 \) the maximum tensile strength (MPa), \( \tau \) the shear stress (MPa), \( \phi \) the internal friction angle (º) and \( \tau \) represents time.

In the same way, the life expectancy of the mixtures according to the von Mises equivalent stress of the mortar-mortar contacts was calculated using the following fatigue life model:

\[
N = k\sigma^{-n}
\]

where \( k \) and \( n \) are constants of the mortar material and \( \sigma \) is the von Mises equivalent stress (MPa).

The results showed that the von Mises equivalent stress was not a proper quantity to evaluate mortar or cohesive failure, mainly because this parameter overestimated the magnitude of the stresses in the mortar-mortar contact zones. The results also showed the presence of high stresses within the mortar-mortar contacts (i.e. 5.38-5.97 MPa, in comparison with the tensile strength of the mortar which was lower than 10 MPa). The use of these results in the fatigue life model provided a short service life estimation of the mixtures, which did not agree with the results observed in field. For this reason, the Mohr-Coulomb like failure envelope was proposed as an alternative to analyze the response of the mortar in future works. In addition, the damage accumulation model, which used the stresses developed at the asphalt-stone interface to evaluate adhesive degradation, did not show a congruent result in terms of the service life of the PFC models. In the case of the 3D PFC model, the life expectancy of this PFC model was 70 cycles, which did not agree with PFCs mixtures performance in the field.

However, an important result of the analyses conducted as part of this project was that the particle packing had a strong influence in the overall response of the PFC model. For example, the 2D model with a higher number or contacts per contact face showed Mohr-Coulomb stresses that were 18 times higher than the 2D model, which had 2 contacts per contact face. Also, the Mohr-Coulomb stresses of the 3D model resulted to be 2.6-3.5 times higher than those obtained in the 2D model. This might be explained by the differences on the effective loading per unit contact area between the two models. In general, these results suggested that the inclusion of aggregates of different sizes, shapes, textures, and packing geometries were important in the evaluation of the response of the system (Mo et al. 2008).

Following the results obtained in the previous works, the research group initiated the LOT project in 2007. Several efforts were conducted as part of this work, including models with different geometries to represent the PFC microstructure, some of them similar to those previously evaluated: i) a 2D idealized model, ii) a 3D idealized model, and iii) a 2D image-based model (Huurman et al. 2010b). At this point, the authors migrated from using CAPA-3D to Abaqus®. In the case of the 3D models, the researchers represented the aggregates with spheres, similar to their previous works (Figure 2), and with circles in the case of the 2D models, unlike the works described before that used polygons. Figure 3 illustrates one of the 2D idealized models that used one-size circle particles. On the other hand, the 2D image-based models were obtained using photos from in-service PFC mixtures. The transformation of the photos to the image vectors required for the FE models was performed manually. Figure 4 illustrates one of these models. As explained by the authors, when using real images it was sometimes difficult to differentiate between clogging and the actual asphalt mortar (Huurman et al. 2010b, 2010c).
From these three proposed models with different geometries, the 2D idealized model (Figure 3) was defined as the default model of LOT, due to its low computational costs and acceptable accuracy. To estimate the consequences of this decision, five PFC LOT models were compared among them. The selected models were: i) one 2D idealized model with circles, ii) one 3D idealized model with spheres, and iii) three 2D image-based models obtained from different photos. The comparison between the response of the 3D and 2D idealized models allowed the estimation of the effects of neglecting the third dimension. In the same way, the comparison between the response of the 2D image-based and the 2D idealized models allowed to estimate the effects of neglecting the actual stone shape. The results showed that although the life expectancy of the compared models was always different, all of them had a coherent or realistic life expectancy in terms of their magnitude.

The evaluation of the mechanical response of these five models was done after subjecting the PFC microstructures to the pass of four rolling wheels of 50 kN at a speed of 79.2 km/h. A total of eight material combination (i.e. aggregate-mortar combination with aged, unaged and water diffusion conditions) at a temperature of 10°C were considered in the calculations (Huurman et al., 2010c). Because raveling could be caused by adhesive or cohesive failure, the models evaluate these two types of degradation phenomena. On one side, the models counted with adhesive zones in two or three coarse aggregates of the PFC systems, which were composed by cohesive elements (i.e. cohesive zone modeling, CZM) type COH2D4 in Abaqus®, with a thickness of 0.01 mm. These elements only had two degrees of freedom (e.g. normal and tangential traction components) (Huurman and Mo 2008, Mo et al. 2010). Opposite to most FE approaches, the CZM was only used in this work to easily determine the
normal and shear stresses at these locations, instead of using it to represent actual fracture within the
PFC samples. These normal and shear stresses at the interfaces were translated into an equivalent tensile
stress using the Mohr-Coulomb failure criterion, which was used to estimate the life expectancy of the
PFC mixtures based on the damage accumulation model proposed by Mo et al. (2006). In this case, the
value of $\sigma_0$ of the mortars, used as an input parameter in the damage accumulate model, was obtained
from an experimental procedure that consisted in subjecting samples of mortar located between two stone
columns to tensile stresses (Huurman and Mo, 2008).

On the other hand, cohesive failure was evaluated using quadrilateral elements (i.e. CPE4R) in
Abaqus®. The cohesive failure evaluation was also made at the mortar-mortar contacts of the aggregates
located at the surface of the PFC models. opposite from the cohesive evaluation conducted by Mo et al.
(2008), this evaluation was made through a new mortar fatigue model based on the dissipated energy
concept, using the stresses and strains of the mortar response. The fatigue model used the following
equation:

$$N_f = \left(\frac{W_{\text{initial-cycle}}}{W_0}\right)^{-b}$$  \[3\]

where $b$ is a material constant, $W_0$ is a reference energy (MPa) and $W_{\text{initial-cycle}}$ is the dissipated energy
per cycle in initial phase (MPa).

The value of the $W_{\text{initial-cycle}}$ was not determined through experimental procedures but through the area
of stress-strain hysteresis loops obtained from the numerical models.

Some of the main conclusions obtained from the efforts conducted as part of the LOT project include
(Huurman and Mo, 2008; Huurman et al., 2010c):

- Unlike to what was found by Mo et al. (2008), the comparison between the 2D and 3D
  idealized models showed that the mortar contacts were stronger in the 3D model in
  comparison to the 2D ones. It was also observed that the stress levels at the adhesive zone of
  the two models were similar, which suggested that the 2D idealized models could be used
  instead of 3D models.
- The models that used geometries obtained from actual images consistently showed a shorter
  service life in comparison to the models that use arrays of symmetric figures to represent the
  coarse aggregates (i.e. circles, spheres). The authors attribute these results to the strong
  variability among the stresses and strains obtained at each location in the same model.
- Although the life expectancy of the 2D image-based models resulted to be shorter than that
  of the 2D and 3D idealized models, the overall life expectancy obtained from all models in
  this project in terms of its magnitude appears to be realistic.
- In all cases, the results showed that raveling could be caused by cohesive or adhesive fatigue
damage depending on the material evaluated.
- The results from the material combination with water ingress condition were not conclusive.
• In most cases, the effects of having an aged material combination enhance the chance of the PFC mixtures to develop raveling.

In addition, the results from the 2D idealized LOT FE models were compared against a full-scale test conducted at the STUVA (Research Association for Underground Transportation Facilities) test center in Germany (Huurman and Mo, 2008). The calibration consisted in testing several PFC pavement layers (i.e. with the same material combination used in the LOT FE models) in an indoor Accelerated Pavement Test (APT) test machine. The indoor condition maintained the PFC pavement layers at a constant temperature of 10°C. The circular track of the APT machine allowed the application of the same load used in the models (i.e. wheel of 50 kN at a speed of 80km/h at the PFC surface). After 700,000 load cycles, the PFC texture was measured through laser techniques and visual inspections. The calibration of the model was achieved since the results revealed a good agreement between the life expectancy and the damage measured in the STUVA tests. The results proved that a long-life expectancy obtained from 2D idealized LOT models correlated with minor damage in the STUVA tests, and that the short life expectancy according to LOT was correlated to much more damage in the STUVA tests.

Equivalent 2D PFC idealized FE models as those used in LOT have been later used by other authors (Huurman et al., 2010b; Mo et al., 2010; Klutz et al., 2013). Mo et al. (2010), for example, used LOT to evaluate the influence of a wide range of temperatures over the response of different mortar materials. The results showed that raveling could initiate under the range of temperatures evaluated, but that this development is critical at extreme temperatures (i.e. low and high). The authors also found that adhesive failure is predominant at low temperatures, while cohesive failure is predominant at high temperatures. These results also showed that mortar aging made PFC mixture less susceptible to raveling under high temperatures and more susceptible to raveling at low temperatures. Besides, Huurman et al. (2010b) used the LOT model to explain the rapid development of raveling in PFCs subjected to low temperatures. The authors found that winter damage was mainly generated by a strong reduction of the relaxation potential of the aged mortar at these temperatures. These authors also used LOT to evaluate the raveling performance at high temperatures, and they found that even though PFCs are also vulnerable to raveling at these conditions, the development of raveling is more aggressive under the presence of low temperatures. Based on these results, Klutz et al. (2013) decided to use LOT to evaluate four different highly modified aged binders that could mitigate the evolution of raveling under extreme temperatures. The results showed that the highly Styrene-butadiene-styrene block (SBS) polymer-modified binders had a very good performance under low and high temperatures.

On a different effort, Zhang et al., (2016) and by Zhang and Zhen (2017) recently used 2D image-based geometries obtained through X-ray Computer Tomography (CT) techniques to evaluate the durability of PFC mixtures using FE models. The work conducted by Zhang et al., (2016) in collaboration with Delft University of Technology and Wuhan University of Technology, evaluated the implications of using a surface treatment (i.e. a cationic rapid setting bituminous emulsion) over an in-service PFC mixture as a preventive maintenance procedure. For this purpose, two 2D PFC-FE models with the same realistic structure obtained from X-ray CT were developed in Abaqus (Figure 5). The properties of the asphalt mortar were modified in the two models to represent a PFC with no treatment and a PFC with the surface treatment. The von Mises stresses were computed in only four points of analysis within the microstructure of the mixture after subjecting the system to the pass of a wheel load.
These values were used to calculate the dissipated energy per cycle of the mortar, and the results showed that, in all cases, the von Mises stresses and the dissipated energy were 25% higher, in average, for the PFC-FE model with no surface treatment. Similarly, the shear dissipated energy resulted to be 10% higher in the model that had no treatment than in the treated one, suggesting that the use of a surface treatment as a preventive maintenance procedure could potentially extend the service life of PFC mixtures.

![Figure 5. PFC image-based geometry model (modified after Zhang et al. (2016)).](image)

Zhang and Zhen (2017) used a similar procedure that in their previous work (Zhang et al., 2016) to quantify the influence of mortar aging in the susceptibility of PFC mixtures to raveling. In this case, two PFC mixtures with different gradation and mortar materials (i.e. unmodified and SBS modified bitumen) were evaluated using 2D FE models with image-based geometries (equivalent to those presented in Figure 5). In the first part of the work, cylindrical mortar specimens of 10 mm of length and 6 mm of diameter were aged in an oven at 165°C for 2 h and then in a Pressure Aging Vessel at 100°C for 80 h under an air pressure of 2.1 MPa. Afterwards, the specimens were subjected to temperature and frequency sweep tests in order to obtain their mechanical properties. The properties of the unaged mortar were also obtained for comparison. As usual, the model was subjected to the pass of a rolling wheel and the response of the material was used to estimate the shear fatigue life of the PFC systems through the energy ratio dissipated by the mortar. The results showed that the aged mortar fabricated with unmodified binder was 5% more susceptible to fracture than the unaged material, while the aged SBS modified mortar resulted to be less susceptible to raveling than the unmodified one. Contrary to what was expected, the aged mortars dissipated less energy than the unaged mortars, resulting in higher predicted fatigue lives in the aged materials. These final results showed that it was necessary to modify the current shear fatigue law used as part of these models, in order to obtain valid and reliable results. In summary, Table 2 presents all the FE raveling models described in this section, including their objectives and some of their limitations.
Table 2. Recent numerical studies of PFC Materials

<table>
<thead>
<tr>
<th>Reference</th>
<th>Modeling Approach</th>
<th>Objectives</th>
<th>Limitations</th>
</tr>
</thead>
</table>
| Mo et al., (2007)  
“Investigation into stress states in porous asphalt concrete on the basis of FE-modeling” | 2D FE | Study the mechanical response of a PFC mixture at the micromechanical scale, emphasizing the causes of raveling and analyzing the stress concentration at the adhesive interface among the aggregates and the mastic material. | The geometry of the microstructure was simplified (aggregates were modeled as regular polygons). No actual deterioration processes at the interfaces were simulated. The impact of environmental factors on reducing raveling resistance was not considered. |
| Mo et al., (2008)  
“2D and 3D meso-scale finite element models for raveling analysis of porous asphalt concrete” | 2D and 3D FE | Study the mechanical response of PFC mixtures, especially the response associated with the potential of the mixture to raveling using a fatigue life law based on the damage generated by the pass of a wheel. This model was the initial part of the LOT project. | The geometry of the microstructure was simplified (aggregates were modeled as circles or spheres). No actual deterioration processes at the contacts were simulated. The constant materials of the fatigue life and damage accumulation models were obtained from the literature and not obtained for this specific project. |
| Huurman et al., (2010b)  
“Porous Asphalt raveling in cold weather conditions” | 2D FE | Study the mechanical response of the mortar within PFC mixtures under low temperatures and high tire loads. LOT was used to determine the life expectancy of the PFC mixtures through the damage accumulation model proposed. | The geometry of the microstructure was simplified (aggregates were modeled as circles). Only adhesive failure was considered. |
| Mo et al., (2010)  
“Investigation into material optimization and development for improved raveling resistant porous asphalt concrete” | 2D FE | Study the influence of temperature changes over the aged SBS modified mortar mechanical response of PFC mixtures. This model used LOT to predict the life service of the models. The models considered adhesive and cohesive failure as a cause of raveling. | The geometry of the microstructure was simplified (aggregates were modeled as circles). |
| Kluttz et al., (2013)  
“Highly modified bitumen for prevention of winter damage in OGFCs” | 2D FE | Evaluate the response of four highly modified mortars within PFC mixtures under winter weather conditions using the LOT model. The model considered adhesive and cohesive zones where the presence of high stress was an indicator of raveling. | The geometry of the microstructure was simplified (aggregates were modeled as circles). |
| Zhang et al., (2016)  
“Preventive maintenance of” | 2D FE | Evaluate the resistance of PFC to raveling after applying a surface treatment material. The stresses and strains obtained from a model Mo et al., (2008) suggested that the use of von Mises stresses was not a reliable parameter to evaluate the susceptibility of a |
<table>
<thead>
<tr>
<th>Reference</th>
<th>Modeling Approach</th>
<th>Objectives</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>porous asphalt concrete using surface treatment technology”</td>
<td>with realistic geometry were computed to obtain the von-Mises stresses and the dissipated energy by the mortar.</td>
<td>PFC mixture. In addition, no actual damage criterion was used. Only one PFC microstructure was evaluated, even though the results are highly dependent on the microstructures used.</td>
<td></td>
</tr>
<tr>
<td>Zhang and Zhen (2017) “Quantification of bituminous mortar ageing and its application in raveling evaluation of porous asphalt wearing courses”</td>
<td>2D FE Evaluate the PFC mixtures’ raveling evolution considering the changes in the mechanical properties of the asphalt mortar caused by oxidation. The authors used realistic PFC geometries obtained from actual images. The resulting energy ratio was used to determine the fatigue life of the mortars.</td>
<td>Only three points within the PFC model were selected for evaluation. Raveling can develop in other locations of the mortar that are not analyzed. Only two PFC microstructures were used, even though the results are highly dependent on the microstructure characteristics of the mixture used.</td>
<td></td>
</tr>
</tbody>
</table>

The previous FE models demonstrate that a considerable progress has been achieved to study raveling processes in PFC mixtures. However, there are still some additional variables and considerations that need to be accounted for in order to better characterize this phenomenon, such as the coupled influence of aging and moisture in the susceptibility of PFC mixtures to develop of raveling, as well as of other relevant climatic factors. In addition, the implementation of image-based models brings new questions about the influence of the geometrical characteristics of those geometries over the magnitude and variability of the overall mechanical response of the mixture. Finally, the inclusion of fracture mechanics-based models within the FE geometries could offer a more accurate presentation of raveling within these mixtures, providing new information about the evolution of this degradation process. All these aspects are considered as part of this dissertation.
CHAPTER III
NUMERICAL ASSESSMENT OF THE STRUCTURAL CAPACITY OF PFC

As mentioned in Chapter I, existing works suggest that the structural contribution of PFCs to flexible pavement structures is not negligible but also that it is not necessarily equivalent to those of regular HMA layers. Moreover, there is still a lack of understanding regarding the origin of such structural resistance and the role of the geometrical characteristics of the microstructure of these mixtures to the overall capacity of PFC systems. The main objective of this Chapter is to provide new and valuable information about this topic by assessing the structural contribution of PFC layers in conventional flexible pavements using computational mechanics.

To achieve this goal, a computational layer that follows a typical PFC used in the United States (US) with three different thickness (i.e. 2, 4 and 6 cm) and with two different AV contents (i.e. 20 and 25%) was selected and located on top of typical asphalt pavements. The PFCs were modeled in the commercial FE software Abaqus® as bidimensional layers. The analysis consisted in quantifying the mechanical response of the pavement structures with PFCs after the pass of a moving wheel load over the surface of the pavement. The maximum tensile strain ($\varepsilon_t$) at the bottom of the asphalt layers (i.e. fatigue control) and the maximum vertical compressive strain ($\varepsilon_z$) on top of the subgrade (i.e. rutting control) were selected as the evaluation parameters. These mechanical indicators were compared against those obtained from the control pavements (i.e. without PFCs), which permitted to isolate the mechanical contribution of the PFC layers. Furthermore, since the stone-on-stone contact network developed within the coarse aggregate fraction of PFCs is recognized as a key component responsible of the structural capacity of these mixtures (Kandhal 2002, Alvarez, Mahmoud, et al. 2010, Alvarez et al. 2018), the analysis included a complete assessment of the influence of several characteristics of PFC microstructures on the mechanical contribution of these layers.

3.1. Materials and pavement structures

3.1.1. PFC mixtures

The PFC mixture selected for this study corresponds to a ‘type FC-5’ mixture specified by the Florida Department of Transportation (FDOT). This PFC, herein named ‘FL-PFC’ mixture, is considered a typical PFC mixture used in the United States (U.S.), as reported by Cooley et al. (2009) and Arámbula-Mercado et al. (2016). The mixture is composed by Limestone (LS) aggregates, PG 76-22 binder modified with polymers (PMA), 0.4% of mineral fibers per total weight and 0.5% of liquid antistripping by binder weight. The optimum binder content (OBC) of the mixture is 6.5% by total weight (Arámbula-Mercado et al. 2016). Table 3 presents the gradation and other properties of this PFC. As explained in a

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1 Most of this work can be found in the paper “Numerical Assessment of the Structural Contribution of Porous Friction Courses (PFC)” by Manrique-Sanchez and Caro (2019).
later section, the microstructures of the FL-PFC used in the FE models of this work were obtained from images captured through X-Ray Computed Tomography (CT) techniques on cylindrical FL-PFC specimens prepared in the laboratory for that purpose at the two different target air voids (i.e. 20 and 25%).

Table 3. Design of the selected FL-PFC mixture used in Chapter III.

<table>
<thead>
<tr>
<th>Sieve</th>
<th>FDOT FC-5 Specification Limits</th>
<th>Cumulative pass [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm (3/4”)</td>
<td>100%</td>
<td>100</td>
</tr>
<tr>
<td>12.5 mm (1/2”)</td>
<td>85% - 100%</td>
<td>92.7</td>
</tr>
<tr>
<td>9.5 mm (3/8”)</td>
<td>55% - 75%</td>
<td>69.4</td>
</tr>
<tr>
<td>4.75 mm #4</td>
<td>15% - 25%</td>
<td>22</td>
</tr>
<tr>
<td>2.36 mm (#8)</td>
<td>5% - 10%</td>
<td>8.7</td>
</tr>
<tr>
<td>1.18 mm (#16)</td>
<td>–</td>
<td>4.7</td>
</tr>
<tr>
<td>600um (#30)</td>
<td>–</td>
<td>3.6</td>
</tr>
<tr>
<td>300um (#50)</td>
<td>–</td>
<td>2.5</td>
</tr>
<tr>
<td>150um (#100)</td>
<td>–</td>
<td>2.5</td>
</tr>
<tr>
<td>75um (#200)</td>
<td>2% - 4%</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Aggregate type | Limestone
Binder type | PMA
OBC (%) | 6.5
Antistripping (%) | 0.5 by weight of binder
Mineral fibers (%) | 0.4 by weight of mix

3.1.2. FL-PFC mastic

Mastics are herein defined as the combination of asphalt binder and fillers (passing sieve # 200). The study of this material resulted necessary because the proposed FE-PFC models assume that a thin film of mastic coats each coarse aggregate, a condition that is expected in any asphalt mixture. Therefore, the stone-on-stone contacts in these open coarse mixtures are in fact mastic-on-mastic contacts. Following the recommendations of previous works (Alvarez, Ovalles, et al. 2012, Hesame et al. 2012, Underwood and Kim 2014), the mastic samples were prepared using a volume ratio of 0.33 filler/binder. The samples were subjected to sweep temperature and frequency oscillatory torsional shear characterization tests (i.e. dynamic shear rheometer or DSR tests) under a shear strain controlled condition of 1.0% (Arámbula-Mercado et al. 2016). The results from these tests were herein used to construct the shear complex modulus master curves of the mastic at a reference temperature of 30°C, as observed in Figure 6.
3.1.3. Flexible pavement structures

Three conventional flexible pavements were selected in this study, over which the different FL-PFC layers were placed. The following layers compose these structures: i) an HMA layer, ii) an unbounded granular base (UGB) layer, and iii) the subgrade. The pavements were named as Pavement A, Pavement B and Pavement C. The thickness of the layers and their linear elastic material properties are listed in Table 4. Pavement A corresponds to a low traffic AASHTO (1993) pavement design; Pavement B follows the design of a trial pavement built in the research facilities of Texas A&M Transportation Institute (TTI); and Pavement C corresponds to an actual pavement structure that is currently in-service between Jacksonville and Tyler, Texas.

Table 4. Linear elastic properties of the materials in the selected flexible pavements (E refers to the elastic modulus and v to the Poisson’s ratio).

<table>
<thead>
<tr>
<th>Layer</th>
<th>Pavement A</th>
<th>Pavement B</th>
<th>Pavement C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E [MPa]</td>
<td>v [-]</td>
<td>Thickness [cm]</td>
</tr>
<tr>
<td>HMA</td>
<td>2,448</td>
<td>0.32</td>
<td>14</td>
</tr>
<tr>
<td>UGB</td>
<td>231</td>
<td>0.35</td>
<td>46</td>
</tr>
<tr>
<td>Subgrade</td>
<td>28</td>
<td>0.40</td>
<td>N/A*</td>
</tr>
</tbody>
</table>

*Not applicable

3.2. Modeling methodology

3.2.1. FL-PFC microstructure geometry

As previously explained, the structural capacity of PFC mixtures is strongly dependent on the contact network developed within the coarse aggregate fraction. Therefore, it is important to count with a
realistic geometrical representation of the microstructural geometries of the FL-PFC. In this study, the FL-PFC geometries used in the numerical models were obtained from X-Ray CT images of specimens of the FL-PFC mixture described in Table 5. These specimens were compacted using the Superpave Gyratory Compactor (SGC) at two different target AV contents: 20 and 25%. Sections of vertical cuts of the CT scans having between 12 to 14 cm wide were selected to generate 2D representations of the PFC microstructures at both AV contents and three thicknesses: i) 2 cm, ii) 4 cm and iii) 6 cm. Due to the heterogeneous distribution of the AVs within the FL-PFC specimens, it was not possible to obtain 2 cm FL-PFC microstructures that satisfied the target AV content of 25%. Typically, the images obtained from the scans of the 25% PFC specimen had AV contents that ranged between 20 and 30%, and most of these 2 cm microstructures presented weak stone-on-stone contact networks, which generated high numerical instabilities when modeled in FE (Figure 7a and 7b). Thus, it was decided to exclude FL-PFC microstructures of 2 cm of thickness and 25% AV content from this work. Finally, to capture the variability among the microstructural models of the same mixture, three different FL-PFC microstructures or ‘numerical replicates’ were obtained from the PFC scans for each combination of thickness and AV content – except for the 2 cm FL-PFC thickness microstructures with 25% AV – for a total of 15 microstructures.

Figure 7. Examples of 2 cm FL-PFC sections of the 25%AV PFC specimen with corresponding AV content of: (a) 22.9%, and (b) 29.9%.

Figure 8 illustrates the three replicates of the FL-PFC microstructures evaluated for the different thicknesses and both AV contents. Although it is not observable in these figures, all coarse aggregates are coated by a thin film of mastic. The thickness of this film was estimated numerically using the gradation and OBC of the mixture and after assuming a uniform coating thickness in all particles, which resulted in a value of 0.015 mm.

In an initial stage of the work, each FL-PFC microstructure was characterized in terms of the following parameters:

- Number of aggregates.
- Number of contacts.
- Number of floating aggregates; which are defined as aggregates that do not contribute to the contact network and, therefore, do not transmit stresses within the microstructure.
- Total length of the contacts between aggregates.
- Principal orientation of the normal vectors of the contacts, a parameter that influences the load path of two connected grains (Chang and Lio 1994). The orientation of the normal vector of each contact was computed counterclockwise with respect to the horizontal axis. Then, the principal orientation of the normal vectors of the contacts was defined as the average of all the orientations of the normal vectors within the contacts.

- The probabilistic density function (pdf) that better fits the number of contacts per aggregate. These pdfs resulted to be normal distributions in all cases. This parameter is related with the Coordination Number (CN), which is defined as the ratio between the total number of contacts and the total number of aggregates (i.e. average number of contacts per aggregate). Larger values of CN indicate an overall better connectivity. Overall, the CN is considered a good indicator of the network connectivity of the PFC microstructure.

These characteristics for the microstructures analyzed are summarized in Table 5 and Table 6.

Figure 8. FL-PFC microstructures: (a) 20% AV and 2 cm thickness, (b) 20%AV and 4 cm thickness, (c) 20% AV and 6 cm thickness, (d) 25%AV and 4 cm thickness, and (e) 25% AV and 6 cm thickness.
Table 5. AV content, number of aggregates, contacts and floating aggregates of the selected FL-PFC layers.

<table>
<thead>
<tr>
<th>Target AV</th>
<th>PFC thickness</th>
<th>Replicate</th>
<th>AV content</th>
<th>Aggregates No.</th>
<th>Contacts No.</th>
<th>Floating aggregates %</th>
<th>Avg.*</th>
</tr>
</thead>
<tbody>
<tr>
<td>20%</td>
<td>2 cm</td>
<td>1</td>
<td>20.4%</td>
<td>58.0</td>
<td>80.0</td>
<td>6.9%</td>
<td>6.3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>20.7%</td>
<td>79.0 68.0</td>
<td>114.0 96.7</td>
<td>6.3%</td>
<td>7.4%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>21.2%</td>
<td>67.0</td>
<td>96.0</td>
<td>8.9%</td>
<td></td>
</tr>
<tr>
<td>4 cm</td>
<td></td>
<td>1</td>
<td>20.2%</td>
<td>119.0</td>
<td>143.0</td>
<td>16.8%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>19.9%</td>
<td>133.0 133.7</td>
<td>175.0 181.7</td>
<td>16.5%</td>
<td>14.7%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>19.6%</td>
<td>149.0</td>
<td>227.0</td>
<td>10.7%</td>
<td></td>
</tr>
<tr>
<td>6 cm</td>
<td></td>
<td>1</td>
<td>20.1%</td>
<td>223.0</td>
<td>292.0</td>
<td>8.9%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>19.8%</td>
<td>182.0 213.0</td>
<td>248.0 306.7</td>
<td>10.9%</td>
<td>8.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>20.7%</td>
<td>234.0</td>
<td>380.0</td>
<td>4.7%</td>
<td></td>
</tr>
<tr>
<td>25%</td>
<td>4 cm</td>
<td>1</td>
<td>24.2%</td>
<td>146.0</td>
<td>208.0</td>
<td>7.5%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>25.2%</td>
<td>170.0 149.7</td>
<td>216.0 201.0</td>
<td>18.2%</td>
<td>9.6%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>24.2%</td>
<td>133.0</td>
<td>179.0</td>
<td>3.0%</td>
<td></td>
</tr>
<tr>
<td>6 cm</td>
<td></td>
<td>1</td>
<td>25.4%</td>
<td>177.0</td>
<td>242.0</td>
<td>7.9%</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>24.1%</td>
<td>213.0 201.7</td>
<td>312.0 286.7</td>
<td>8.4%</td>
<td>7.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>24.7%</td>
<td>215.0</td>
<td>306.0</td>
<td>5.1%</td>
<td></td>
</tr>
</tbody>
</table>

* Avg.: average obtained from three FL-PFC replicates.

Data in Table 5 shows that the AV content of each replicate is successfully near the target AV content (20% and 25%) with an average of 20.3% and 24.6%, respectively. Also, it is observed that the percentage of floating aggregates ranges between 3.0 to 18.2%. In average, the FL-PFCs replicates with 20% AV and 4 cm thickness have the maximum percentage of floating aggregates. It is important to notice that the amount of floating aggregates is also related to the nature of the technique used to construct the models. For example, Kim et al., (2016) concluded that the original gradation of HMA mixtures can be affected when they are obtained through CT scans; even more when 2D instead of 3D samples are used, as in this work. This suggests that several of the floating aggregates identified in these 2D microstructures could belong to more complex 3D contact networks that could lead to higher structural contributions. Nevertheless, the computational models conducted using these microstructures are considered a valid initial approximation to quantify the structural impact of these layers on regular flexible pavements; specially since previous works have demonstrated that 2D numerical simulations on these mixtures offer qualitative valid results (Huurman and Mo 2008, Mo et al. 2008).

An analysis of the data in Table 5 also shows that there is a direct correlation between the PFC thickness, the number of aggregates, and number of contacts of the replicates. As the PFC thickness increase, the number of aggregates and contacts also increase, with an estimated linear correlation factor of 0.93.
Table 6. Total contact length, principal orientation of contacts and parameters of the normal pdf of contacts per aggregate of the selected FL-PFCs.

<table>
<thead>
<tr>
<th>Target AV</th>
<th>PFC thickness</th>
<th>Replicate</th>
<th>Total contact length [mm]</th>
<th>Principal orientation of contacts [º]</th>
<th>Parameters of a normal pdf fitting the number of contacts per aggregate**</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Avg.</td>
<td>Avg.</td>
</tr>
<tr>
<td>20%</td>
<td>2 cm</td>
<td>1</td>
<td>161.3</td>
<td>85.8</td>
<td>86.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>225.2</td>
<td>86.5</td>
<td>86.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>195.6</td>
<td>86.1</td>
<td>86.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>430.1</td>
<td>87.0</td>
<td>87.1</td>
</tr>
<tr>
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<td></td>
<td>2</td>
<td>556.1</td>
<td>87.5</td>
<td>87.8</td>
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<td>626.6</td>
<td>87.8</td>
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<td>1</td>
<td>846.8</td>
<td>88.1</td>
<td>87.6</td>
</tr>
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<td>760.9</td>
<td>88.1</td>
<td>87.8</td>
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<td></td>
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<td>912.9</td>
<td>87.6</td>
<td>87.8</td>
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<tr>
<td>25%</td>
<td>4 cm</td>
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<td>464.0</td>
<td>87.5</td>
<td>87.4</td>
</tr>
<tr>
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<td>350.5</td>
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<td>87.4</td>
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<tr>
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<td></td>
<td>1</td>
<td>693.6</td>
<td>87.7</td>
<td>87.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>593.9</td>
<td>87.9</td>
<td>87.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>616.3</td>
<td>87.9</td>
<td>87.8</td>
</tr>
</tbody>
</table>

* Avg.: average obtained from three FL-PFC replicates.

** μ and σ are the mean and standard deviation values of the normal pdf distributions.

On the other hand, data in Table 6 shows that the average principal orientation of contacts for all FL-PFCs replicates was 87.3±0.7º with a coefficient of variation (COV) of 0.8%, and the average mean of the normal pdf of the contacts per aggregates was 2.9±0.2 for all replicates, with a COV of 7.5%. The low COVs (<10%) of the principal orientation of contacts and of the contacts per aggregate indicate that all PFC-microstructures are a good representation of the internal 2D contact networks of the whole FL-PFC mixture. For this reason, it is expected that the principal orientation of contacts and the contacts per aggregates parameters would not impact the structural contribution of the different FL-PFC layers. Also, an analysis of the data in Table 6 shows that there exists a direct linear correlation of 0.90 between the PFC thickness and the total length of the contacts between aggregates.

In summary, it is observed that all FL-PFC microstructures had similar principal orientation of contacts and number of contacts per aggregate, and that the number of aggregates, number of contacts and total contact length are directly related with the thickness of the PFC.

**3.2.2. FE pavement model geometry**

The FE models representing the pavement structures in Abaqus® had two main components. The first component is the FL-PFC layer that was analyzed at a microstructural level and consists of coarse aggregates, air voids and mastic. The second component is the pavement structure located below the FL-PFC layer (i.e. HMA, UGB and subgrade layers), which was considered to be composed of continuous
layers with isotropic and homogeneous materials. In all pavement structures, the layers were considered fully bonded.

To avoid boundary effects and obtain a proper representation of the deflection bowl under the loading wheel, the subgrade layer and the total horizontal length or total width of the control pavement structures (i.e. without FL-PFCs) were defined based on recommendations provided in the literature (Duncan et al. 1968, Olidis and Hein 2004) and on the results of a sensitivity analysis. The sensitivity analysis consisted in quantifying the change in the maximum tensile ($\varepsilon_t$) and vertical compression ($\varepsilon_z$) strains extracted from the FE models when modifying the subgrade thickness and the pavement length gradually until observing a convergence in the strain values. This analysis permitted to conclude that a subgrade with a thickness of 100 cm and a horizontal length of the model of 360 cm were enough to avoid border effects.

As an example of the pavement models, Figure 9 presents the final geometry of Pavement C and the model geometry of this pavement with a FL-PFC layer of 2 cm, from where it is also possible to observe the selected boundary conditions. The same procedure was used with the other FL-PFC layers in the three pavement structures.

![Figure 9](image)

**Figure 9.** (a) Geometry of the model for Pavement C and (b) for Pavement C with a 2 cm FL-PFC layer.

Figure 10 exemplifies the global FE mesh used in the models, with a detail of the mesh used for the FL-PFC layers. This image corresponds to a portion of the FE model of Pavement A, and it includes the HMA and the FL-PFC layer. The coarse aggregates were discretized using 3-node linear elements (i.e. type CPE3R in Abaqus®), while the mastic was discretized using 4-node bilinear elements (i.e. type CPE4R in Abaqus®). After conducting a sensitivity analysis to define the size of the mesh, the aggregates were meshed using a seed of 1.5 cm and the mastic with a seed of 0.0075 cm. As a result, the FL-PFC layers had approximately $6 \times 10^6$ elements. All the other pavement layers were modeled with a total of 77,300 CPE4R elements using a 0.5 cm seed.
Figure 10. Global FE mesh for a typical FL-PFC layer and a portion of the HMA layer in Pavement A.

3.2.3. Loading conditions

Each model was subjected to a moving wheel load over the pavement at a speed of 96.5 KPH, which is considered a typical speed on roads with PFCs (FDOT 2018). The running wheel represents the load of half of a standard axle of 8.2 ton with a pressure contact \((q)\) of 0.83 MPa and a radius \((r)\) of 12.5cm. The wheel applied a vertical force \((F_y)\) of 41.0 kN and a frictional force representing an average rolling friction force \((F_x)\) of 0.84 kN, which was computed based on the recommendations provided by Milne et al. (2004). Figure 11 illustrates the vertical and frictional force of the running wheel over the pavement, and the selected zones in the model from where the maximum tensile and vertical compression strains (i.e. \(\varepsilon_t\) and \(\varepsilon_z\)) were obtained.

Figure 11. (a) Moving wheel load passing over Pavement C, and (b) zones of analysis for the maximum tensile, \(\varepsilon_t\), and vertical compressive strain, \(\varepsilon_z\) (positive values represent compression).
3.3. Mechanical response of the components of the FE model

3.3.1. FL-PFC aggregates and pavement layers

The coarse aggregate fraction of the FL-PFC mixtures was modeled as a linear elastic material with a modulus of 30,000 MPa and a Poisson’s ratio of 0.25 (Rummel 1991, Schultz 1993). As explained before, all the layers of the pavement, except for the FL-PFCs, were also modeled as linear elastic, isotropic and homogenous materials (Table 6).

3.3.2. Mechanical properties of the mastic materials in the FL-PFC

The mastic in the FL-PFC layers was modeled as a linear viscoelastic material. The rheological information from Figure 6 was transformed from the frequency to the time domain to determine the Prony series of the relaxation modulus of the material. The general formulation of the Prony Series is expressed by Eq. 4 (Fabrizio and Morro 1992),

\[ G(t) = G_{\infty} + \sum_{j=1}^{n} G_j e^{-\frac{t}{\rho_j}} \]  

where \( G(t) \) corresponds to the shear relaxation modulus, \( G_{\infty} \) is the long-term equilibrium modulus, \( G_j \) is the \( j^{th} \) Prony series parameter, \( t \) is time and \( \rho_j \) is the \( j^{th} \) relaxation time. The obtained parameters of the Prony series of the FL-PFC mastic at a temperature of 30°C are summarized in Table 7.

<table>
<thead>
<tr>
<th>( j )</th>
<th>( \rho_j ) [s]</th>
<th>( G_j ) [Pa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00001</td>
<td>2.30x10^8</td>
</tr>
<tr>
<td>2</td>
<td>0.001</td>
<td>2.95x10^7</td>
</tr>
<tr>
<td>3</td>
<td>0.01</td>
<td>4.86x10^6</td>
</tr>
<tr>
<td>4</td>
<td>0.1</td>
<td>8.91x10^5</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>2.17x10^5</td>
</tr>
<tr>
<td>6</td>
<td>10</td>
<td>4.13x10^4</td>
</tr>
<tr>
<td>7</td>
<td>100</td>
<td>8.94x10^3</td>
</tr>
<tr>
<td>8</td>
<td>1,000</td>
<td>2.59x10^3</td>
</tr>
<tr>
<td>9</td>
<td>10,000</td>
<td>5.95x10^2</td>
</tr>
</tbody>
</table>

\( G_{\infty} \) [MPa] 1.00

3.4. Analysis of results

A total of 48 models were evaluated, which correspond to the conventional pavements without FL-PFC layers (i.e. models from which the ‘control strains’ were obtained) and all possible combinations between the three types of pavements, the PFC thickness (i.e. 2, 4 and 6 cm for 20% AV, and 4 and 6 cm for 25% AV), the types of PFC volumetric designs (i.e. 20 and 25% AV), and the replicates of the microstructures (i.e. three geometries for each thickness and AV content). The following sections present the results from the numerical models and the corresponding analysis.
3.4.1. *Structural contribution of the FL-PFC layers*

Table 8 summarizes the percentage reduction of the tensile and vertical strains in comparison to the control strains obtained from the 45 pavement models containing FL-PFC layers. As observed in this table, there was an increase in the overall structural capacity of the structures in all models that included FL-PFC layers, and this contribution—computed as the reduction in the critical strains within the structure in comparison to the control strains—ranged between 0.3% and 15.8%.

Moreover, the results show that Pavement A—which has the layers with the lowest moduli—showed the smallest strains reductions with the inclusion of FL-PFCs, with an average of 5.7% in tensile strains and 6.0% in compressive strains. Pavement B—which has the layers with intermediate moduli—showed average strain reductions of 6.6% in tensile strains and 5.8% in compressive strains. Finally, Pavement C—which has the layers with highest moduli—showed average strain reductions of 6.6% in tensile strains and 5.5% in compressive strains. These results suggest that the overall structural contribution of the FL-PFCs resulted to be similar among pavement structures. When studying the data in detail, however, it is observed that the percentages of strain reduction in Pavement A were slightly smaller than those in Pavements B and C (5.2% and 2.9% lower, respectively), and that Pavement B presented an average of 2.4% higher reductions in the strain values than Pavement C.

To better visualize these results, Figure 12 shows the average strain reductions obtained for the three replicates conducted in each FL-PFC microstructure, with the corresponding variability. It is noticeable that the average strains reductions in the pavements increased with the increase in the thickness of the FL-PFCs. As it was expected, the 2 cm FL-PFC layers had the lowest structural contribution, with an average strain reduction of 1.7% and 2.6% for $\varepsilon_t$ and $\varepsilon_z$ in comparison with the control strains, while the 4 cm FL-PFC layers had an intermediate contribution with an average strain reduction of 5.3% and 4.7% for $\varepsilon_t$ and $\varepsilon_z$, and the 6 cm FL-PFC layers had the highest contribution with an average strain reduction of 10.6% and 9.0% for $\varepsilon_t$ and $\varepsilon_z$. These results are mainly due to the fact that the number of aggregates and the number of contacts increase with the thickness of the PFCs layers as explained before (Table 5), which confirms that there is a strong correlation between these parameters.
Table 8. Reduction in strains with respect to the control strains due to the inclusion of a FL-PFC layer.

<table>
<thead>
<tr>
<th>AV</th>
<th>Thickness layer</th>
<th>Pavement A</th>
<th>Pavement B</th>
<th>Pavement C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>et [-]</td>
<td>ez [-]</td>
<td>et [-]</td>
<td>ez [-]</td>
</tr>
<tr>
<td>20%</td>
<td>2 cm</td>
<td>0.8% 4.2%</td>
<td>0.8% 2.7%</td>
<td>0.6% 2.0%</td>
</tr>
<tr>
<td></td>
<td>0.3% 1.7%</td>
<td>1.6% 1.8%</td>
<td>1.5% 1.7%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.3% 4.2%</td>
<td>4.0% 2.7%</td>
<td>3.2% 2.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Avg.</td>
<td>1.2% 3.4%</td>
<td>2.1% 2.4%</td>
<td>1.8% 1.9%</td>
</tr>
<tr>
<td></td>
<td>Std.</td>
<td>1.0% 1.4%</td>
<td>1.6% 0.5%</td>
<td>1.3% 0.2%</td>
</tr>
<tr>
<td>20%</td>
<td>4 cm</td>
<td>4.7% 5.0%</td>
<td>3.1% 3.9%</td>
<td>3.8% 4.0%</td>
</tr>
<tr>
<td></td>
<td>9.2% 6.7%</td>
<td>9.4% 6.9%</td>
<td>9.9% 6.7%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.3% 2.7%</td>
<td>2.4% 2.9%</td>
<td>1.4% 2.6%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Avg.</td>
<td>5.4% 4.8%</td>
<td>5.0% 4.6%</td>
<td>5.0% 4.4%</td>
</tr>
<tr>
<td></td>
<td>Std.</td>
<td>3.5% 2.0%</td>
<td>3.8% 2.1%</td>
<td>4.4% 2.1%</td>
</tr>
<tr>
<td>20%</td>
<td>6 cm</td>
<td>8.6% 8.6%</td>
<td>9.4% 7.6%</td>
<td>10.3% 7.7%</td>
</tr>
<tr>
<td></td>
<td>5.6% 5.9%</td>
<td>7.1% 5.9%</td>
<td>7.3% 5.5%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.9% 8.7%</td>
<td>10.8% 8.8%</td>
<td>11.6% 8.1%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Avg.</td>
<td>7.7% 7.7%</td>
<td>9.1% 7.4%</td>
<td>9.7% 7.1%</td>
</tr>
<tr>
<td></td>
<td>Std.</td>
<td>1.8% 1.6%</td>
<td>1.9% 1.4%</td>
<td>2.2% 1.4%</td>
</tr>
<tr>
<td>25%</td>
<td>4 cm</td>
<td>2.7% 4.3%</td>
<td>4.4% 4.3%</td>
<td>4.2% 3.9%</td>
</tr>
<tr>
<td></td>
<td>3.3% 5.0%</td>
<td>5.4% 5.2%</td>
<td>4.7% 4.9%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.8% 1.6%</td>
<td>2.3% 2.9%</td>
<td>1.7% 2.2%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Avg.</td>
<td>2.6% 3.6%</td>
<td>4.0% 4.1%</td>
<td>3.5% 3.7%</td>
</tr>
<tr>
<td></td>
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<td>0.8% 1.8%</td>
<td>1.6% 1.2%</td>
<td>1.6% 1.4%</td>
</tr>
<tr>
<td>25%</td>
<td>6 cm</td>
<td>12.1% 9.1%</td>
<td>11.6% 9.0%</td>
<td>12.2% 8.9%</td>
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<td>10.4% 9.3%</td>
<td>10.8% 8.7%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Avg.</td>
<td>11.7% 10.5%</td>
<td>12.6% 10.7%</td>
<td>13.0% 10.3%</td>
</tr>
<tr>
<td></td>
<td>Std.</td>
<td>3.5% 2.6%</td>
<td>2.7% 2.7%</td>
<td>2.6% 2.6%</td>
</tr>
</tbody>
</table>

* Avg.: average obtained from three FL-PFC replicates

** Std.: standard deviation obtained from three FL-PFC replicates
Figure 12. Results of the pavement structures with 20% and 25% AV FL-PFC mixtures with respect to their control strain: (a) maximum ε_t reduction in Pavement A, (b) maximum ε_z reduction in Pavement A, (c) maximum ε_t reduction in Pavement B, (d) maximum ε_z reduction in Pavement B, (e) maximum ε_t reduction in Pavement C, and (f) maximum ε_z reduction in Pavement C.
Another important observation from the results in Table 8 and Figure 12 is that the reduction in strains produced by the presence of FL-PFCs is highly dependent on the specific PFC-microstructure evaluated. This is corroborated by the high COV (> 30%) of the strain reductions among replicates. This result could be mainly attributed due to the internal properties of the individual FL-PFCs, as explained next.

To better analyze the influence of the geometrical and microstructural characteristics of the selected FL-PFC layers (Table 5 and Table 6) on the overall structural capacity of the pavements (Table 8 and Figure 12), individual linear regression models were constructed between the maximum $\varepsilon_t$ and $\varepsilon_z$ strain reductions obtained from the FE models and several of these characteristics (i.e. AV, PFC thickness, number of aggregates and contacts and total contact length between aggregates). A linear model was selected because the coefficient of determination ($R^2$) obtained from this type of regression model was observed to be higher than with several other models (e.g. pure quadratic, quadratic, polynomial of higher order, etc.). The linear regression models were constructed using a confidence level of 95%, and the $p$-value (i.e. the probability of finding a linear regression between the FL-PFC characteristic analyzed and the maximum strain reductions) was the index selected to determine if there was statistical significance between each geometrical or microstructural characteristic of the FL-PFC layers and the maximum reductions obtained for $\varepsilon_t$ and $\varepsilon_z$. The results of the $p$-values obtained from these regressions are listed in Table 9. Parameters with $p$-values higher than $5 \times 10^{-2}$ are not statistically significant (i.e. those parameters do not influence the structural capacity of the pavements), while parameters with $p$-values smaller than $5 \times 10^{-2}$ (values in italic in Table 9) do contribute to this structural capacity. It is worth mentioning that, for the purposes of this analysis, the number of contacts and the number of aggregates did not include floating aggregates. Since by definition these aggregates do not transmit stresses within the microstructure and, consequently, do not contribute to the structural capacity of the FL-PFC layer.

### Table 9. Statistical significance of the characteristics of the FL-PFC layers in the overall structural capacity of the pavement structures (values in italic are statistically significant).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Pavement A</th>
<th>Pavement B</th>
<th>Pavement C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\varepsilon_t$ reduc.</td>
<td>$\varepsilon_z$ reduc.</td>
<td>$\varepsilon_t$ reduc.</td>
</tr>
<tr>
<td>Number of aggregates*</td>
<td>$p$-value</td>
<td>$p$-value</td>
<td>$p$-value</td>
</tr>
<tr>
<td>Number of contacts*</td>
<td>$1.6 \times 10^{-3}$</td>
<td>$2.1 \times 10^{-3}$</td>
<td>$3.6 \times 10^{-4}$</td>
</tr>
<tr>
<td>PFC thickness</td>
<td>$3.6 \times 10^{-3}$</td>
<td>$3.2 \times 10^{-3}$</td>
<td>$6.7 \times 10^{-4}$</td>
</tr>
<tr>
<td>Total contact length</td>
<td>$4.1 \times 10^{-4}$</td>
<td>$1.3 \times 10^{-3}$</td>
<td>$2.5 \times 10^{-4}$</td>
</tr>
<tr>
<td>AV</td>
<td>$6.6 \times 10^{-3}$</td>
<td>$2.2 \times 10^{-2}$</td>
<td>$5.7 \times 10^{-3}$</td>
</tr>
<tr>
<td></td>
<td>$3.6 \times 10^{-1}$</td>
<td>$9.7 \times 10^{-1}$</td>
<td>$8.3 \times 10^{-1}$</td>
</tr>
</tbody>
</table>

*a Without including floating aggregates.

Data in Table 9 show that the parameters that have a statistical influence on the structural capacity of the pavements (i.e. parameters with $p$-value lower than $5 \times 10^{-2}$) are: i) number of aggregates, ii) number of contacts, iii) PFC thickness, and iv) total contact length. As an example of such relationship, Figure 13 illustrates the linear regression between the average number of aggregates per thickness (i.e. 2, 4 and 6 cm) of all 20% AV FL-PFC layers evaluated and the average maximum reductions of $\varepsilon_t$ and $\varepsilon_z$ obtained for Pavement $B$. The $R^2$ of these regressions indicates an acceptable fitting between both parameters (i.e. 0.99 for $\varepsilon_t$ and $\varepsilon_z$ reductions, respectively).
Figure 13. Linear regressions between the average number of aggregates per PFC thickness and the average maximum $\varepsilon_t$ and $\varepsilon_z$ strain reductions for all 20% AV PFC layers in Pavement B.

On the other hand, results from Table 9 show that – for the specific FL-PFCs evaluated in this work– the AV content is not a statistically significant factor contributing to the structural capacity of the pavements. This corroborates why some PFC layers having 25% AV presented higher structural contributions than PFC layers having 20% AV (Table 8 and Figure 12).

It should be noted that although assuming a continuum and homogenous PFC layer with a specific dynamic modulus in these FE models would imply a lower computational cost than including realistic PFC microstructures, such approach will shadow the actual nature of the structural behavior of these layers and, more importantly, it will neglect the observed uncertainty derived by the variability of the mechanical response of the different FL-PFC numerical replicates.

3.4.2. Reduction in the thickness of the pavement layers

Since the addition of FL-PFC layers enhanced the structural capacity of the pavement structures, they could be used to reduce the thickness of the other layers of the pavement structure, with the objective of preserving the original overall structural capacity of the pavement.

Pavement B, that presented intermediate results in terms of the structural contribution of the FL-PFCs, was selected to assess this aspect. The exercise consisted on individually reducing the thickness of the HMA or UGB layers of this pavement without any FL-PFC, until the growth in the mechanical tensile and vertical strains of interest were equivalent to the mechanical strain reductions obtained with the inclusion of the FL-PFC layers (Table 8). In other words, the principle of the analysis was to compensate
the structural capacity provided by the FL-PFC layers with a reduction in the thickness of the HMA or UBG layers.

The numerical results show that, in all cases, the reduction in the thickness of the HMA or UBG layers of this pavement consistently produced higher values of $\varepsilon_z$ than of $\varepsilon_t$. Therefore, $\varepsilon_z$ was selected as the critical strain parameter for this exercise. The results in Table 8 show that, for this pavement, a 2, 4 and 6 cm FL-PFC layer with 20 or 25% AV—with the gradation specified in Table 3 and the microstructural and geometrical characteristics showed in Tables 5 and 6—could be used to reduce, in average, 0.5, 1.0, and 3.0 cm thickness of the HMA layer, respectively, and 1.0, 3.0, and 6.0 cm thickness of the UGB layer, respectively. More specifically, it is observed that the structural contribution of the different FL-PFC layers with 20 and 25% AV and different thicknesses is equivalent to a reduction of 0.5 to 4.0 cm in the HMA layer and of 1.0 and 9.0 cm in the UGB layer. These results are in good agreement with those showing that the structural contribution of PFC layers is only a fraction of that of regular HMA layers (Han 2008, Wang. et al. 2014, Dylla and Hansen 2015), and could be considered an initial validation of these numerical results.

Table 10. Maximum thickness reduction in the layers of Pavement B due to the addition of the FL-PFC layers.

<table>
<thead>
<tr>
<th>Target AV</th>
<th>PFC thickness</th>
<th>Maximum reduction in the thickness of the layer due to the presence of a PFC</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>HMA [cm]</td>
<td>UGB [cm]</td>
</tr>
<tr>
<td>20%</td>
<td>2 cm</td>
<td>Avg.* 0.5</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Std.** 0.0</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>4 cm</td>
<td>Avg. 1.3</td>
<td>2.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Std. 0.6</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>6 cm</td>
<td>Avg. 2.2</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Std. 0.3</td>
<td>1.0</td>
</tr>
<tr>
<td>25%</td>
<td>4 cm</td>
<td>Avg. 1.2</td>
<td>2.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Std. 0.3</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>6 cm</td>
<td>Avg. 3.3</td>
<td>6.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Std. 0.6</td>
<td>1.9</td>
</tr>
</tbody>
</table>

* Avg.: average obtained from three FL-PFC replicates
** Std.: standard deviation obtained from three FL-PFC replicates

Even though data in Table 10 provide a broad idea of the range of layer thickness reductions that could be achieved with the use of FL-PFC layers, it should be noted that it is not possible to postulate general guidelines of typical thickness reduction of a pavement layers as a direct function of the thickness and AV of the PFC layer. This is due to the fact that these values strongly depend on other microstructural variables of the PFC, and on the specific pavement structure under analysis.

Given the influence of the microstructural characteristics of the PFCs on the overall stiffness of these mixtures, a comparison between such stiffness and that of regular HMAs could be also used to estimate the relative structural contribution of these layers. The authors have numerically determined that the dynamic modulus for this FL-PFC mixture with 20% AV is approximately 2,000 ± 250 MPa at 30°C. This result was obtained after applying oscillatory loading on seven 2D numerical specimens (10 cm by
of this mixture. Considering that a typical HMA at the same temperature might have a dynamic modulus between 3,000 and 4,000 MPa, the dynamic moduli of the PFCs studied correspond, approximately, to 50-66% of the modulus of a regular HMA. This value is comparable with previous works that have showed that PFCs moduli could be between 30 and 79% of an HMA (Van Heystraeten and Moraux, 1990; Timm and Vargas-Nordcbeck, 2012, Gerritsen et al., 1987; Rijkswaterstaat, 1989; Van Der Zwam et al., 1990; Hopman et al., 1998).

Finally, it is important to note that PFC layers have short service lives in comparison to regular HMA layers, mainly due to the initiation and progression of raveling. For this reason, although these numerical analyses demonstrate that the presence of PFCs could be used to reduce the thickness of the pavement layers, it is not recommended to modify them unless there is certainty about the durability of the PFC layer and/or there are strict PFC-related maintenance and/or rehabilitation strategies that guarantee the continuous presence and effective contribution of these mixtures throughout the service life of the pavement.

3.5. Conclusions and recommendations

The main objective of the modeling work performed in this chapter was to quantify the magnitude and factors that impact the structural contribution of PFC layers in conventional flexible pavements through the application of computation mechanics using realistic geometries of this type of mixtures. After conducting a total of 64 FE simulations in Abaqus®, it was demonstrated that the FL-PFC layers do contribute to the structural capacity of the three pavements evaluated concluding that:

- The differences in the structural contribution of the FL-PFCs are strongly related with different geometrical and microstructural characteristics of the PFC microstructures. In fact, while all FL-PFC layers presented similar number of contacts per aggregate (i.e. average 2.87 ± 0.2) and principal orientation of the contacts (i.e. 87.3 ± 0.7°), the PFC thickness, the number of aggregates and the number of contacts – without considering floating aggregates— were observed to be critical in impacting the structural contribution of these layers.

- The numerical results conducted on Pavement B showed that the use of the 2, 4 and 6 cm FL-PFC layers with 20 and 25% AV provided a structural contribution equivalent to 0.5–3.0 cm of the HMA layer or to 1.0–9.0 cm of the UGB layer. It is important to stress that if the thickness of any layer of a flexible pavement is reduced due to the placement of a PFC layer, it is necessary to guarantee that the service life of the PFC is similar to that of regular HMA layers and/or that the project has efficient management strategies to maintain or replace the PFC layer when necessary. If these conditions cannot be guaranteed, thickness reductions of other structural pavement layers are not recommended.

Finally, future works in this topic should include extensive laboratory and field efforts on a variety of PFCs and pavement structures. This would permit to continue assessing the actual structural contribution of these layers in regular pavements and to further corroborate the numerical results obtained in this study.
CHAPTER IV
FIRST APPROXIMATION TO FE RAVELING MODELING

The objective of this chapter is to contribute to the state of knowledge of raveling through the implementation of FE models that investigate the role of different material-related and traffic-related conditions in affecting the propensity of PFC mixtures to raveling. To accomplish this goal, 2D micromechanical FE models were implemented in the commercial software Abaqus®, building on some of the results already achieved in the past works presented in Chapter II, and proposing a new energy-based criterion to estimate PFC mixtures’ susceptibility to raveling.

Specifically, FE modeling was used to compare the susceptibility to raveling of six different PFC computational mixtures located on top of three conventional pavements. Since the stone-on-stone contacts provides the main strength source in these mixtures to resist raveling (Huber, 2000; Alvarez, Epps Martin, et al., 2010; Alvarez, Mahmoud, et al., 2010), the geometry of the PFC layer plays a relevant role in the quality of the FE model, as already demonstrated in some of the models described in the literature review chapter (Mo et al., 2007, 2008; Huurman et al., 2010a) and in the initial models presented in Chapter III. For this reason, and similar to the works conducted by Zhang et al. (2016) and by Zhang and Zhen (2017), X-ray CT techniques were used to obtain more realistic geometrical configurations of the PFC microstructures.

The extracted geometries were located on top of a flexible pavement and subjected to the load imposed by a moving wheel load. The PFCs mixtures were then exposed to different internal (i.e. material properties, AV content, binder content) and external (i.e. load magnitude, friction forces, wheel velocities, and pavement structural resistance) conditions. Then, a new energy-based criterion in combination with probabilistic theory was proposed to conduct an initial quantification of the influence of each factor in the chances of the mixtures to experience raveling. Next, a description of the materials and mixtures used as part of this Chapter is presented, followed by a description of the model geometry, the cases of study and the methodology of analysis. A summary of the main findings is also presented at the end of this section.

4.1. Materials and mixtures

4.1.1. PFC mixtures

The six mixtures comply with the Florida Department of Transportation (FDOT) design manual recommendations for PFC mixtures type FC-5 (FDOT 2018). Two of these mixtures contained Granite

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2 The initial version of the FE simulations presented in this chapter made part of the project BDR74-977-04 executed by the Texas A&M Transportation Institute (TTI) in collaboration of Universidad de los Andes, and funded by the Florida Department of Transportation (FDOT). Most of this work can be also found in the paper “Numerical modelling of ravelling in porous friction courses (PFC)” by Manrique-Sanchez et al., (2018).
(GR) aggregates, while the other four were composed by Limestone (LS) aggregates. The mixtures we fabricated using one of the following binders: i) SBS modified PG 76-22 (PMA), or ii) asphalt rubber PG-76-22 (ARB). Besides, all mixtures included 0.4% mineral fiber by weight of the mixture to prevent drain down, and 0.5% antistripping agent by weight of the binder. The antistripping agent was hydrated lime in the granite mixtures, added at a dose of 1% by weight of aggregate, while for the limestone mixtures a liquid antistripping was used at a dose of 0.5% by weight of binder. The liquid antistrip was blended with the binder at the terminal.

Table 11 summarizes the properties of the mixtures. As it can be noticed, there are only three types of gradations among the six mixtures, corresponding to Mixtures 1 and 2, Mixtures 3 and 4, and Mixtures 5 and 6. However, it is noteworthy that the gradations of Mixtures 1 and 2 and Mixtures 3 and 4 are very similar, except for the aggregates smaller than 1.18 mm (i.e. the fine aggregate matrix or mortar of the mixture).

According to previous studies (Massahi et al. 2016), Mixtures 1 and 2, that contained limestone aggregates, performed well in the field in terms of raveling. These two mixtures are identical in terms of their gradation and aggregate source, but they differ in their type of asphalt binder. According to the same study, Mixture 3, also with limestone aggregates, showed poor raveling performance in the field. This mixture has a different gradation that Mixture 1 but contains the same asphalt. There are no records regarding the raveling performance of mixtures 4 to 6.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>FDOT FC-5 Specification Limits</th>
<th>Mixture</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0mm (3/4&quot;)</td>
<td>100%</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.5mm (1/2&quot;)</td>
<td>85% - 100%</td>
<td>90</td>
<td>93</td>
<td>95</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5mm (3/8&quot;)</td>
<td>55% - 75%</td>
<td>66</td>
<td>69</td>
<td>72</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.75mm</td>
<td>15% - 25%</td>
<td>24</td>
<td>23</td>
<td>19</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.36mm</td>
<td>5% - 10%</td>
<td>10</td>
<td>9</td>
<td>9</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.18mm</td>
<td>–</td>
<td>8</td>
<td>5</td>
<td>6</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>600um</td>
<td>–</td>
<td>7</td>
<td>4</td>
<td>4</td>
<td>9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>300um</td>
<td>–</td>
<td>6</td>
<td>3</td>
<td>3</td>
<td>9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>150um (#100)</td>
<td>–</td>
<td>5</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>75um (#200)</td>
<td>2% - 4%</td>
<td>3.5</td>
<td>3</td>
<td>2</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aggregate Type</td>
<td>Limestone</td>
<td>Granite</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Binder Type</td>
<td>PMA</td>
<td>ARB</td>
<td>PMA</td>
<td>ARB</td>
<td>PMA</td>
<td>ARB</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Optimum Binder Content (OBC) (%)</td>
<td>7.1</td>
<td>6.1</td>
<td>5.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anti-Strip (%)</td>
<td>0.5 by weight of binder</td>
<td>1.0 by weight of aggregate</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mineral Fiber (%)</td>
<td>0.4 by weight of mix</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.1.2. Mastic materials

Similar to previous works (Mo et al., 2007, 2008; Huurman et al., 2010c), these models assume that each aggregate in the mixture is coated by a uniform thin film of mastic material. Therefore, the mechanical performance of the mastics is a required input parameter for the FE model. In order to obtain this information, Dynamic Shear Rheometer (DSR) tests were performed to characterize the linear viscoelastic material properties of the mastics present in each mixture (Arámbula-Mercado et al., 2016). Data in Table 11 shows that there are four different mastics that result from the combination of both aggregates (LS and GR) and binders (PMA and ARB). Consequently, the mastics were named based on the binder-aggregate combination; i.e. PMA-LS, for example, is the mastic prepared with limestone and PMA binder, which corresponds to the mastic in Mixtures 1 and 3. All mastic samples were prepared using a volume ratio of 0.33 filler/binder, following the recommendation stated in previous works (Alvarez, Ovalles, et al. 2012, Hesame et al. 2012, Underwood and Kim 2014). DSR tests included temperature and frequency sweep tests that were conducted at Texas A&M University under strain-controlled conditions on the four mastics (Arámbula-Mercado et al., 2016), and the results were used to construct shear complex modulus master curves at a reference temperature of 30ºC, as observed in Figure 14.

![Figure 14. Master curves of the mastics (T_{Reference}=30ºC).](image)

In addition, the mastics were tested using the Wilhelmy Plate (WP) to obtain the Surface Free Energy (SFE) of these materials (Hill 2015). The SFE is defined as the amount of energy required to create one unit of area of the material in vacuum conditions. Based on this definition, the work of cohesion, or the energy require to fracture the material, corresponds to two times its SFE (Bhasin 2006). This quantity is relevant in this work because raveling is a phenomenon related to the loss of stone-on-stone or mastic-mastic contacts. Therefore, the energy required to fracture a mastic-mastic contact, computed from the SFE of these materials, could be efficiently used as an initial estimator of the propensity of a PFC system to raveling, as it will be explained later. The results of the SFE properties of the mastics are summarized in Table 12.
Table 12. SFE of the mastics (Hill, 2015).

<table>
<thead>
<tr>
<th>Mastic Binder Type</th>
<th>Aggregate</th>
<th>SFE (mJ/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMA-LS PMA</td>
<td>LS</td>
<td>14.63</td>
</tr>
<tr>
<td>PMA-GR PMA</td>
<td>GR</td>
<td>15.62</td>
</tr>
<tr>
<td>ARB-LS ARB</td>
<td>LS</td>
<td>21.96</td>
</tr>
<tr>
<td>ARB-GR ARB</td>
<td>GR</td>
<td>15.62</td>
</tr>
</tbody>
</table>

4.2. Characteristics of the FE computational models

4.2.1. Geometry

The initial raveling FE models consisted of a pavement structure composed by three layers: i) a PFC layer, ii) a representative base layer, which is equivalent to all other pavement layers beneath the PFC, and iii) a subgrade. The representative base and the subgrade layers were assumed to be a continuum media with linear elastic material properties. This simplification reduced the computational cost of the simulations and permitted to focus the analysis on the response of the PFC layer.

The thickness of the PFC layer was of 20 mm, which was determined based on typical thicknesses used in Florida for these mixtures. Besides, the equivalent base of the pavement structure resembles some of the most common pavements in this state for arterial roads, with a total thickness of 600 mm (FDOT 2018). Finally, the subgrade was assigned a thickness of 400 mm. The horizontal length of the model was 600 mm. To avoid undesired boundary effects, only the mechanical response of the central part of the PFC layer was analyzed.

The PFC layer was considered at the micromechanical level and it was simplified as a three-phase material: i) coarse aggregates, ii) mastic, and iii) air voids, and it was modeled using realistic microstructure geometries. As explained before, these microstructures were obtained from X-ray CT and image from compacted specimens of the mixtures that were fabricated in the laboratory. The 2D vertical images of the PFC geometries used in the models were fully characterized in terms of their AV content and of their number and length of the stone-on-stone contacts (Table 13). Due to the resolution of the tomographer used, it was not possible to identify the mastic films coating the aggregates. Then, similar to the procedure used in the previous chapter, a thin film of mastic with constant thickness was assigned to each aggregate, which was computed using the effective asphalt content and aggregate gradation of each mixture, and after assuming a spherical shape for all aggregate particles. Thus, for Mixtures 1 and 2, the obtained mastic film thickness was of 17 μm, for Mixtures 3 and 4, it was of 10 μm, and for Mixtures 3 and 4 it was of 15 μm. Figure 15 summarizes the geometry of a typical FE model, while Figure 15 shows the general geometry of the FE models implemented in this chapter, as well as the mastic film coating the aggregates (i.e. red lines).
Figure 15. Geometry of the initial raveling PFC-FE models.

Figure 16 illustrates the typical global FE mesh used in these models. This figure also shows in detail some coarse aggregates, from where it is possible to observe again the mastic phase coating the particles. The subgrade and the equivalent base pavement were modeled using a total of 5,226 3-node linear elements (i.e. type CPE3 in Abaqus®). The aggregates were also modeled as CPE3 elements, while the mastic was discretized using 4-node bilinear elements (type CPE4R in Abaqus®). To ensure a higher precision into the results, the mastic was meshed with a seed of 0.075 mm, while the aggregates were meshed with a larger seed of near 30 mm. The PFC layer counted with a total of 431,485 elements but only those located at the mastic-mastic contact zones in the upper central part of the model were analyzed (i.e. near 3,500 elements, depending on the mixture).
4.2.2. Constitutive response of the components of the model

In these models, the aggregates were modeled as linear elastic materials with a modulus of 50,000 MPa for GR and 27,000 MPa for LS, both with Poisson’s ratio of 0.3 (Rummel 1991). The equivalent base and the subgrade were also modeled as linear elastic materials, with moduli ranging between 400 and 645 MPa for the equivalent base, and between 100 and 150 MPa for the subgrade. The values of the mechanical properties of these layers were modified to evaluate their influence on the raveling susceptibility of the PFC layer. The mastic materials were modeled as linear viscoelastic materials, using the rheological information presented in Figure 14 at 30ºC.

4.2.3. Cases of study

The initial part of the simulations consisted on evaluating the overall mechanical response of the six PFC mixtures presented in Table 1 to the load conditions imposed by a rolling wheel passing over the pavement. This moving wheel represents half of a single-axle load of 49.7 kN, a value that was identified to be a typical loading magnitude in highways in Florida (LTPPBind 2015). The tire-pavement interaction was assumed to have a constant contact pressure \( (q) \) of 0.88 MPa, which resulted in a contact radius \( (a) \) of 9.5 cm. The wheel passed over the surface of the structure at an average speed of 88 km/h, which corresponds to the minimum speed allowed for the FC-5 type of mixtures in the state of Florida (FDOT 2018).

As mentioned before, Mixtures 1, 2, 3 and 4 have similar gradations. Likewise, Mixtures 4 and 5 have similar gradations too. For this reason, only four PFC geometries –that represent the six PFC mixtures evaluated at an AV content of 20% and 25%– were selected. Figure 17 presents the four basic PFC microstructures used for the simulations, and Table 13 summarizes their corresponding stone-on-stone properties. It should be highlighted that even though these geometries where obtained from images of specimens prepared in laboratory, the actual contact characteristics of the mixtures are different from those listed in Table 13. This is due to the fact that the authors believed that a 2D representation of the mixture does not fully capture the actual 3D skeleton of the stone-on-stone contact interactions (Huurman et al. 2010b). Nevertheless, since this study consists on comparing the results of equivalent 2D simulations, this condition is not considered a factor of concern for the purposes of these initial models.
Figure 17. Model geometries for (a) Mixtures 1, 2, 3 and 4 with 20% AV, (b) Mixtures 1, 2, 3 and 4 with 25% AV, (c) Mixtures 5 and 6 with 20% AV, and (d) Mixtures 5 and 6 with 25% AV.

As a first attempt to characterize the stone-on-stone network within the coarse aggregate fraction of the PFC mixtures, the number of contacts per aggregate and the percentage of aggregate perimeter in contact with other aggregates were selected to characterize the microstructures of the PFCs. In each case, the measurements were adjusted to a probability density function or pdf. The results from Table 13 show that, in average, the number of contacts per aggregate is 24% higher in the 20% AV content mixtures than in the 25% AV content mixtures. Similarly, the percentage of aggregate perimeter in contact with other aggregates resulted to be 29% higher for the mixtures with 20% AV content than for those with 25% AV content. At first sight, these results suggest that, due to their stone-on-stone contact network, the PFC mixtures with 20% AV content might be more resistant to raveling.

Initially, the mechanical response of the six different PFC mixtures with 20% AV content under the same loading conditions was evaluated. Thus, the influence of the type of mixture in the raveling susceptibility of the mixtures was analyzed.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>AV</th>
<th>Total particles analyzed</th>
<th>Number of contacts per aggregate</th>
<th>Percentage of aggregate perimeter in contact with other aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>pdf distr. Parameters</td>
<td>pdf distr. Parameters</td>
</tr>
<tr>
<td>Mixtures 1, 2, 3 and 4</td>
<td>20%</td>
<td>59</td>
<td>Gamma</td>
<td>μ = 3.39, σ = 1.44</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>56</td>
<td>Gamma</td>
<td>μ = 2.32, σ = 1.20</td>
</tr>
<tr>
<td>Mixtures 5 and 6</td>
<td>20%</td>
<td>70</td>
<td>Normal</td>
<td>μ = 3.23, σ = 1.58</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>77</td>
<td>Weibull</td>
<td>μ = 2.71, σ = 1.28</td>
</tr>
</tbody>
</table>
In the second portion of the simulations, Mixtures 1 and 6, which significantly differ in their gradation and materials, were selected to further evaluate the impact of different internal (i.e. mix-related) and external (i.e. load conditions and structural capacity of the pavement) factors in the propensity of these mixtures to develop raveling processes. The internal factors included BC and total AV content of the mixtures; while the external factors included the wheel load magnitude, load speed, friction forces, and the structural capability of the pavement beneath the PFC layer. The parameters were evaluated once at a time while keeping the others constant, in a parametric way, and the results were compared to quantify the specific effect of each variable in the susceptibility of the mixture to raveling. The values that were kept constant in the different simulations make part of what was called the ‘base models’. Table 14 summarizes the internal and external parameters evaluated. The values in italic in this table correspond to those used as base parameters.

Table 14. Internal and external parameters evaluated in the first models.

<table>
<thead>
<tr>
<th>Type</th>
<th>Parameter</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal</td>
<td>BC (%)</td>
<td>4.1, 5.6, and 7.1</td>
</tr>
<tr>
<td></td>
<td>AV (%)</td>
<td>20 and 25</td>
</tr>
<tr>
<td>External</td>
<td>PFC layer thickness (mm)</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Vertical load magnitude (kN)</td>
<td>24.8, 66.7, and 83.7</td>
</tr>
<tr>
<td></td>
<td>Vertical load radius (mm)</td>
<td>95, 110, and 120</td>
</tr>
<tr>
<td></td>
<td>Friction force magnitude (kN)</td>
<td>0.621 and 19.38</td>
</tr>
<tr>
<td></td>
<td>Load speed (km/h)</td>
<td>48, 88.5, and 113</td>
</tr>
<tr>
<td>Pavement</td>
<td>Equivalent base (MPa)</td>
<td>400, 520, and 645</td>
</tr>
<tr>
<td>structural</td>
<td>subgrade (MPa)</td>
<td>50, 100, and 150</td>
</tr>
<tr>
<td>capacity</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 18 presents an example of the results obtained in one of these models. As expected, the internal stresses distributions within the microstructure of the PFC changed as a function of the different variables that were included as part of the analysis. In this case, the model represents a typical PFC FC-5 Florida mixture (Mixture 1), with a wheel of 24.8 kN that passed over the pavement at a speed of 88 km/h when the PFC had a mean temperature value of 30°C.
4.3. Methodology for data analysis

4.3.1. Dissipate energy criterion

Since raveling is expected to be a function of the weakening of the mastic-mastic contacts, only the elements in these zones were analyzed (i.e. about 3,500 finite elements per model). However, the stresses and strains were obtained in global Cartesian coordinates and they did not correspond to the directions of the expected modes of failure responsible for the actual fracture at the stone-on-stone contact: i) Mode I or opening mode, and ii) Mode II or shear mode (Figure 19). Therefore, the stresses associated with these modes of failure, which are the maximum tensile and shear stresses, were computed using the Mohr’s Circle. In this way, the principal tensile stress and strain, $\sigma_{t_{\text{max}}}$ and $\varepsilon_{e_{\text{max}}}$, were related to Mode I of failure, while the maximum shear stress and strain, or $\tau_{\text{max}}$ and $\gamma_{\text{max}}$, were related to Mode II of failure.
Since the mastics were modeled as viscoelastic materials, the magnitude of the stresses and strains were equally relevant when analyzing the potential of the material to fracture. Therefore, the *dissipated energy* of the mastic-mastic contacts elements generated during the pass of the wheel load was proposed as a raveling susceptibility criterion. The dissipated energy was calculated as the area within the curve of the maximum tensile (i.e. Mode I) and shear (i.e. Mode II) stresses and strains. Figure 20 shows a typical maximum tensile stress vs. tensile strain curve for Mode I of failure for a mastic element located at the stone-on-stone contact.

![Diagram showing fracture modes I and II in a stone-on-stone contact.](image)

**Figure 19.** Fracture modes I and II in a stone-on-stone contact.

![Graph showing dissipated energy at a mastic stone-on-stone contact element for Mode I of failure.](image)

**Figure 20.** Dissipated energy at a mastic stone-on-stone contact element for Mode I of failure.
4.3.2. Quantification of the raveling potential

Although the total dissipated energy at each mastic contact element provides important information on the mechanical response of the PFC layer under different conditions, this quantity does not provide direct information of the PFC susceptibility to raveling. This is due to the fact that the occurrence of raveling does not only depend on the magnitude of the total dissipated energy by the element but also on the amount of energy required to produce fracture in the mastic. As expected, this last property is unique for each mastic material. Since there are not standardized fracture experiments on mastics that could provide reliable information on the fracture properties of these materials, the cohesive bond energy computed based on the SFE of the mastics was used for these purposes. With this information, a new indicator of raveling susceptibility, named ‘Raveling Index’ or ‘R.I.’, was defined as:

\[
R. I_{\text{dry}} = \frac{\text{Dissipated energy} \left( \frac{J}{m^3} \right)}{\text{Cohesive bond energy} \left( \frac{J}{m^2} \right)}
\]  

[5]

Thus, R.I. was selected as the raveling parameter to analyze the results of the initial numerical simulations. As observed in Equation 4, larger values of R.I. (i.e. larger ratio values between the dissipated energy and the cohesive bond energy) indicate a higher susceptibility of the PFC mixture to fracture at the stone-on-stone contacts and, therefore, a higher susceptibility to raveling.

It is important to mention that Masad et al. (2010) demonstrated that the theoretical work of adhesion or cohesion computed through SFE measurements is several orders of magnitude smaller than the actual fracture energy measured for any material in the laboratory. Despite this and taking into account that the R.I. components have different units, the R.I. should only be understood as a dimensionless index, whose purpose is to provide valid comparative analyses of the susceptibility to raveling of different mastic–mastic contacts.

To represent the overall response of the mixtures evaluated, the R.I. for Mode I and Mode II of failure for all the mastic elements located at the stone-on-stone contact zones were fitted to proper pdfs. The best fit for the R.I. results for all simulation was a Weibull distribution, with a confidence level of 95%. Figure 21 illustrates the Weibull pdf distribution for Mode I for the six PFC mixtures analyzed in this chapter.

However, since raveling is a fracture-related phenomenon, it constitutes a classical problem of extreme values. This means that the quantification of the raveling susceptibility should not be focused on the mean values of these pdfs but, instead, on the values located at the ‘tail’ of those distributions, where fracture is more probable to occur. After analyzing the data of these distributions, it was determined that R.I. values larger than 5.0x10^{-3} for Mode I and Mode II were relevant for characterizing raveling, since they capture the behavior of the tail of all pdf distributions by presenting a probability of exceedance between 7 and 10% in all cases.
Based on this information, all pdf distributions were truncated at a value of R.I. of $5.0 \times 10^{-3}$ to capture the behavior of critical values of R.I. The procedure for truncating a distribution consists in transforming the values of the pdf lower than the truncation value, $x_0$, into 0, and the values higher or equal to $x_0$ in a new value of the pdf affected by constant $k$, which includes the probability of presenting smaller values of $x_0$ in the pdf distribution evaluated. This procedure is described by the following equations (Benjamin and Cornell 1970):

$$f_T(y) = \begin{cases} 0, & y < x_0 \\ kf_x(y), & y \geq x_0 \end{cases}$$  \[6\]

$$k = \frac{1}{[1 - F_X(x_0)]}$$  \[7\]

where $f_T(y)$ is the truncated pdf, $x_0$ is the truncated value, and $F_X(x_0)$ is the probability of having a value smaller than $x_0$.

For all cases, the mean ($\mu_T$) and the standard deviation ($\sigma_T$) of the truncated pdf distributions for R.I. were computed using the traditional definitions. The truncated mean R.I. plus one standard deviation ($\mu + \sigma$)$_T$ was selected as the evaluation parameter to rank the susceptibility of each mixture to raveling. Larger values of this value are related with higher probabilities of fracture in the mastic elements located at all the stone-on-stone contacts. This parameter does not only provide the mean R.I. value of the truncated distribution (i.e. the most probable value of R.I. that is larger than a critical value) but also relevant information about the dispersion of the R.I among the coarse aggregate contact elements, which means that it captures the uncertainty associated with the occurrence of this phenomenon.
Considering that both quantities have different units, the R.I. should be understood as a dimensionless index which purpose is to provide valid comparative analyses of the susceptibility to raveling of different mastic-mastic contacts.

### 4.4. Results and discussion

Figure 22 presents the raveling susceptibility of the six mixtures evaluated in the initial part of this chapter using the base conditions presented in Table 14, expressed as the mean plus one standard deviation of the truncated pdf distribution of R.I., \((\mu+\sigma)_T\). The results showed that the raveling parameters for Mode I were, on average, 30% higher than those for Mode II, suggesting that Mode I is the main mechanism of fracture causing raveling in these mixtures. The results also show that Mixtures 5 and 6 are the least susceptible to raveling under the simulated conditions, while Mixtures 3 and 4 are the most susceptible, with an average difference between their \((\mu+\sigma)_T\) of R.I. in Mode I of failure of 62%. Mixtures 1 and 2, which reported a good performance in the field, presented low-medium levels of raveling susceptibility, respectively, while Mixture 3 that performed poorly in the field, presented the highest levels of raveling susceptibility among all mixtures. This supports the interpretation of R.I as a valid measurement of raveling susceptibility for PFC.

![Figure 22. Raveling susceptibility of the Mixtures 1—6 evaluated with 20% AV content.](image)

In terms of the influence of the materials, the mixtures containing LS aggregates (Mixtures 1, 2, 3 and 4) were observed to be more susceptible to raveling, presenting a difference near 60% in their \((\mu+\sigma)_T\) of R.I., in comparison with the mixtures with GR aggregates (Mixtures 5 and 6). In all cases, mixtures with ARB asphalt binder showed a lower susceptibility to raveling when compared with the mixtures with the same gradation but with PMA binder (i.e. Mixture 1 vs. Mixture 2, Mixture 3 vs. Mixture 4 and Mixture 5 vs. Mixture 6). This suggests that the material combination less susceptible to raveling, in
mechanical terms, is the combination of granite aggregates and ARB binder (Mixture 6). It is important to mention, however, that these models did not incorporate the moisture susceptibility of the materials, in which case granite aggregates usually present higher chances to develop adhesive failure, and consequently, to present other mechanisms of raveling that have not been considered in this study.

Since the stone-on-stone contact properties define the strength of a PFC mixture, it was expected that a higher number of contacts and larger lengths of contacts per aggregate would improve the resistance of PFC mixtures to raveling. Based on the mixture characterization presented in Table 13, it is observed that Mixtures 1, 2, 3 and 4 have a slightly higher number of contacts and larger contacts lengths per aggregate. However, the results from the simulations show that these mixtures presented a higher \((\mu+\sigma)_T\) of R.I., which would make them more susceptible to raveling in comparison with Mixtures 5 and 6. However, the stone-on-stone characteristics between the selected geometries at 20% AV are very similar since the mean difference for the number and length of contacts per aggregate is near 5%. After considering that the characterization of the mixtures is based on a probabilistic analysis where the Coefficient of Variation COV (i.e. \(\sigma/\mu\)) for each parameter (i.e. number of contacts per aggregate and perimeter in contact per aggregate) is close to 50%, it is not possible to obtain reliable conclusions regarding existing relationships between the raveling evaluation parameter and the microstructure properties of the mixtures at this time.

As mentioned in the previous section, due to their material composition (i.e. PMA-LS and ARB-GR) and gradation, the geometries of Mixture 1 and 6 were selected as two representative cases to analyze –through 24 different FE simulations– the influence of different internal and external factor in the promotion of raveling.

Figure 23 present the raveling susceptibility of Mixtures 1 and 6 with different internal conditions (i.e. three different BC and two different AV content) subjected to the base load and pavement conditions. As noticed in Figure 23a and Figure 23b, the BC that generates the lower value of \((\mu+\sigma)_T\) R.I. is 7.1% for Mixture 1 and 5.6% for Mixture 6. These values correspond to the OBC of each mixture, which suggests that the use of any BC different than the optimum would increase the raveling susceptibility of the PFC mixtures. Likewise, Figure 23c and Figure 23d show that a higher AV content increases the raveling susceptibility of the mixtures in 22.8%, in average. In these particular cases, the number of contacts per aggregate and the percentage of the aggregate in contact with other aggregates had an inverse correlation with the susceptibility to raveling, i.e. at a higher number of contacts and aggregate percentage in contact, a lower R.I. is generated.
Figure 23. Raveling susceptibility of Mixture 1 and 6 under different internal conditions: (a) BC for Mixture 1, (b) BC for Mixture 6, (c) AV content for Mixture 1, (d) AV content for Mixture 6.

On the other hand, Figure 24 shows the susceptibility of raveling of Mixtures 1 and 6 under different external conditions. As expected, the higher values of $(\mu+\sigma)_{T}$ R.I. for both mixtures were obtained with a high magnitude of the vertical force –related to the axle weight–, a lower wheel speed, a high friction force (i.e. braking conditions) –which can be interpreted as a car braking over the pavement– and a weak pavement structure. The pavement structure bellow the PFC, however, presented the lowest effects over the susceptibility of the PFC mixtures, with differences of $(\mu+\sigma)_{T}$ R.I. below 4% among the cases studied.
Figure 24. Raveling susceptibility of Mixtures 1 and 6 under different external conditions: (a) $F_x$, Mixture 1, (b) $F_x$, Mixture 6, (c) wheel speed Mixture 1, (d) wheel speed Mixture 6, (e) $F_x$ Mixture 1, (f) $F_x$ Mixture 6, (g) pavement structure capacity Mixture 1, and (h) pavement structure capacity Mixture 6.
Finally, Table 15 presents the rank of the internal and external factors that showed to have the highest influence in the initiation of raveling within these mixtures, under the operation conditions presented in Table 14. These data show that the parameter with the highest influence in raising the susceptibility of the PFC mixtures to raveling is the total AV content. This parameter can be linked to the quality of compaction processes during the construction of the layers, a process that is difficult to control in these mixtures. As shown before, an increase in 5% in this parameter caused the highest impact in promoting raveling among all parameters analyzed. Likewise, there are three factors associated with the traffic conditions that may increase raveling initiation: i) vehicle braking, ii) low-speed traffic, and iii) very high-load traffic. Finally, the BC content of the mixture was found to be the fourth parameter that increases the propensity of PFCs to raveling.

<table>
<thead>
<tr>
<th>Ranking of factors promoting raveling</th>
<th>Mixture 1 (µ+σ) of R.I.</th>
<th>Mixture 6 (µ+σ) of R.I.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Air void content (AV)</td>
<td>High 0.0836</td>
<td>High 0.1226</td>
</tr>
<tr>
<td>2 Friction force</td>
<td>High 0.0425</td>
<td>High 0.0302</td>
</tr>
<tr>
<td>3 Traffic speed</td>
<td>Low 0.0287</td>
<td>Low 0.0128</td>
</tr>
<tr>
<td>4 Binder content (BC)</td>
<td>Low 0.0274</td>
<td>High 0.0127</td>
</tr>
<tr>
<td>5 Vertical force</td>
<td>High 0.0259</td>
<td>High 0.0118</td>
</tr>
<tr>
<td>6 Pavement Structure Capacity</td>
<td>Weak 0.0255</td>
<td>Weak 0.0113</td>
</tr>
</tbody>
</table>

It is relevant to highlight that the simulations were made on the basis of the four geometries presented in Figure 17. This condition limits the results to these particular microstructures and to their microstructural geometry characteristics presented in Table 13. Considering some incongruences presented between the PFC mixtures microstructural geometry characteristics and the susceptibility to raveling presented by the six mixtures evaluated (see Figure 22), further evaluations of the quality of the 2D microstructural geometry characteristics of the PFC mixtures is required. This aspect is analyzed in later chapters of this dissertation.

4.5. Conclusions and recommendations

The following conclusions were obtained from the FE numerical models performed to assess raveling in PFC mixtures:

- Raveling is mainly a mechanical surface-contact problem associated with Mode I of failure.
- The mixture design was observed to play a main role in the promotion or prevention of raveling in PFC materials. When increasing the AV content, the quality of the skeleton of the mixture influenced the mechanical response of the mastic-mastic contact elements and, consequently, the susceptibility of the mixtures to raveling (i.e. 17.04% less contacts and contact lengths, in average, when the AV content is increased). The final quality of these
microstructures is determined by three main conditions: i) the gradation of the mixture, ii) the compaction process that determines the final AV of the PFC, and iii) the binder content.

- The results suggest that it may be convenient to limit the maximum AV content of the mixture in the field, probably by making it dependent on PFC layer thickness or on certain characteristic property related to its gradation (e.g. on its Nominal Maximum Aggregate Size, NMAS).
- The results also showed that any binder content differing from the design OBC increased the chances of the mixtures to develop raveling.
- It was proven that PFC mixtures are more prone to develop raveling under low speed traffic conditions and in zones where the vehicles are forced to brake frequently, which justifies current recommendations provided by several highway agencies regarding the use of these mixtures only in high-speed roads.
- After considering that the characterization of the mixtures is based on a probabilistic analysis where the COV for each parameter (i.e. number of contacts per aggregate and perimeter in contact per aggregate) is close to 50%, it is not possible at this time to obtain reliable conclusions regarding existing relationships between the raveling evaluation parameter and some of the properties of the microstructure of the mixtures.

It is important to notice that the numerical results reported are highly linked to the selected PFC geometries. Therefore, it is important to further study the effect of the 2D FE model geometry of the microstructure of the PFCs, including the role of the PFC microstructure characteristics and the variability of these parameters, on the susceptibility of these mixtures to raveling. Also, it seems important to evaluate the impact of other parameters in the durability of the mixture, including the mixes susceptibility to raveling along their service life by incorporating the degradation caused by climatic effects to the asphalt material located at the stone-on-stone contacts. These observations constitute the basis for the activities proposed as part of Chapters V and VI, which are presented in the following sections.
CHAPTER V
RANDOM GENERATION OF PFC GEOMETRIES
THROUGH DE GRAVIMETRIC METHODS

The conclusions obtained from Chapters II, III and IV highlight the relevance of the microstructural geometry of PFC when using FE models to assess the mechanical response of these mixtures. In Chapter III, it was also mentioned that the 2D PFC microstructural geometries obtained from X-Ray CT images might underestimated the 3D stone-on-stone contact network existing within the PFC sample from where the images were obtained.

Based in this information, this Chapter exemplifies how a 2D PFC microstructure obtained from images of a 3D sample could not properly resemble certain 3D PFC microstructural parameters. This is achieved by using a set of spheres to represent the coarse aggregate particles of a PFC. Afterwards, this Chapter presents a novel methodology to obtain random 2D PFC microstructures that efficiently represent the mechanical behavior of a 3D PFC mixture while providing the possibility of conducting simulations at lower computation costs when compared to 3D models with similar geometry.

The proposed methodology uses a Microstructure Generator (MG) (Castillo et al. 2015) to randomly produce a set of aggregates with controlled gradation and morphological properties, and the DE software LMG90 –developed by Université de Montpellier (Dubois and Jean 2003)– to apply gravimetric conditions to such a set of particles. The proposed gravimetric modeling approach to generate 2D PFC microstructures was calibrated by subjecting the microstructures to a computational dynamic modulus (|E*|) test using FE and comparing the results with actual laboratory test results. The application of the proposed methodology opens the possibility of conducting different probabilistic and statistical studies that would be difficult to pursue using 3D computational models or through experimental work, which will lead to a more comprehensive understanding of the mechanisms that control the functionality, durability, and mechanical response of PFC mixtures.

5.1. Challenges of 2D image-based PFC microstructures

The most common way to obtain 2D image-based PFC microstructures is by using X-ray CT images of PFC laboratory compacted specimens. This technique has been widely used to model dense-graded hot mix asphalt materials, where the coarse aggregates are embedded in a phase of mastic or asphalt mortar (i.e. mixture of asphalt binder and the aggregate particles passing sieve size #16, 1.18 mm (Caro, Masad, Airey, et al. 2008)). However, in the case of PFC mixtures, where the coarse aggregates –typically particles retained in the sieve #4, 4.75 mm– create a 3D contact network, this approach might underestimate their actual stone-on-stone contact density (Manrique-Sanchez and Caro, 2019).

The work of this Chapter was summarized in the article “Random generation of PFC microstructures through DEM gravimetric methods” by Laura Manrique-Sanchez, Silvia Caro, Nicolas Estrada, Daniel Castillo and Allex E. Alvarez. This paper is currently under review.
To exemplify how 2D image-based PFC microstructures may misrepresent the stone-on-stone contact network of an actual PFC mixture, a 3D granular microstructure that simulates a simplified aggregate skeleton of a PFC was randomly created using the DE LMGC90 software. This 3D granular microstructure was used to obtain different image-cuts that simulate the 2D image-based PFC microstructures obtained from X-ray CT applied to PFC specimens.

The 3D granular microstructure was created using a set of 406 spheres that emulates the coarse-aggregate particles of a PFC, and it complies with the typical PFC mixture gradation presented in Table 16. The set of aggregates was subjected to gravimetric forces using frictionless particles in order to create a stone-on-stone contact network that was as dense as possible. The final 3D granular microstructure measures 60x60x70 mm, which ensures a representative volume element, since these dimensions are three times the Nominal Maximum Aggregate Size (NMAS) of the PFC mixture (Kim et al. 2010).

Table 16. Gradation selected to represent the PFC mixture (FDOT 2018).

<table>
<thead>
<tr>
<th>Sieve</th>
<th>FDOT Specification Limits</th>
<th>Cumulative pass [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm (3/4&quot;)</td>
<td>100%</td>
<td>100</td>
</tr>
<tr>
<td>12.5 mm (1/2&quot;)</td>
<td>85% - 100%</td>
<td>92.7</td>
</tr>
<tr>
<td>9.5 mm (3/8&quot;)</td>
<td>55% - 75%</td>
<td>69.4</td>
</tr>
<tr>
<td>4.75 mm (#4)</td>
<td>15% - 25%</td>
<td>22</td>
</tr>
<tr>
<td>2.36 mm (#8)</td>
<td>5% - 10%</td>
<td>8.7</td>
</tr>
<tr>
<td>1.18 mm (#16)</td>
<td>–</td>
<td>5.7</td>
</tr>
<tr>
<td>0.60 mm (#30)</td>
<td>–</td>
<td>4.7</td>
</tr>
<tr>
<td>0.30 mm (#50)</td>
<td>–</td>
<td>3.7</td>
</tr>
<tr>
<td>0.15 mm (#100)</td>
<td>–</td>
<td>2.1</td>
</tr>
<tr>
<td>0.075 mm (#200)</td>
<td>2% - 4%</td>
<td>2.1</td>
</tr>
</tbody>
</table>

A total of six image-cuts were extracted from the 3D microstructure: three image-cuts in the horizontal or X-axis direction, and three image-cuts in the transversal or Y-axis direction (Figure 25). Next, the 3D and the 2D image-based microstructures were compared using two basic microstructural parameters that were previously used in Chapter III:

- Coordination number (CN), defined as the ratio between twice the total number of contacts and the total number of aggregate particles (i.e. the average number of contacts per aggregate particle). Overall, the CN is considered a good indicator of the contact network (i.e. the network of stone-on-stone contacts) of the PFC microstructure. As explained in previous chapters, larger values of this parameter indicate better aggregate connectivity and therefore a better mechanical response and higher resistance to disintegration (i.e. resistance to raveling) (Alvarez et al. 2018).
- Number of floating aggregates (FA), defined as aggregate particles that have one contact or no contacts, or that do not contribute to the contact network (i.e. that do not transfer stresses within the microstructure).
Figure 25 (a) 3D granular microstructure and (b) example of a vertical image-cut obtained from the 3D granular microstructure. The origin of the coordinate system is in the center of the 3D granular microstructure.

Table 17 summarizes the CN and FA microstructural parameters of the 3D granular and the 2D image-based microstructures. The 3D microstructure presented a CN of 4.59, while the 2D image-based microstructures presented an average CN of 0.085. This means that the CN of the 2D image-based microstructures is, on average, 98% lower than the CN of the 3D system. This difference is mainly due to the fact that 91.5% of the aggregates in the image-based microstructures were FA, as those illustrated in Figure 1b. This behavior was observed in the six image-based microstructures analyzed, proving that 2D microstructures obtained through image cuts tend to underestimate the actual stone-on-stone contact density of a 3D microstructure. It would therefore be expected that if these 2D microstructures were used as part of computational numerical models, their mechanical response would not resemble that of actual PFC mixtures.

<table>
<thead>
<tr>
<th>Microstructural parameter</th>
<th>3D microstructure</th>
<th>2D image-based microstructures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Avg*</td>
<td>Std**</td>
</tr>
<tr>
<td>CN [-]</td>
<td>4.59</td>
<td>N.A</td>
</tr>
<tr>
<td>FA [%]</td>
<td>6.79%</td>
<td>N.A.</td>
</tr>
</tbody>
</table>

*Average, **Standard deviation, N.A: not applicable for this case

Considering that these 2D image-based microstructures do not represent the contact network of a 3D microstructure, the methodology of generating 2D microstructures using gravimetric methods is an interesting alternative. Ten 2D microstructures were generated using the gravimetric method in order to explore whether they could better simulate critical microstructural parameters related to the mechanical
response of a 3D microstructure. To do so, frictionless disks aggregates –that followed the gradation presented in Table 1– were randomly located in a predefined space measuring 60x100 mm, as observed in Figure 26a, and subjected to gravity forces using the LMGC90 software, resulting in contact networks such as those presented in Figure 26b.

These 2D microstructures were characterized in terms of their CN and FA and compared with the parameters of the 3D microstructure (Table 18). The results show that the 2D gravimetric microstructures have an average CN value of 3.69, which is 19.5% smaller than the CN in the 3D microstructure. This is similar to the difference between the CN that sphere and disk packings are expected to exhibit in isostatic conditions (i.e. 6 in 3D and 4 in 2D), showing that these systems are indeed comparable. In terms of FA, the 2D microstructures obtained through gravimetric methods presented 2.32% FA, while the 3D microstructure presented a total FA of 6.79%. These values are remarkably lower when compared to the average 91.47% FA value obtained in the 2D image-based microstructures. This demonstrates that the 2D microstructures obtained with gravimetric methods have microstructural properties that are closer to those of the 3D microstructure, when compared to the 2D image-based geometries. Thus, the gravimetric DEM method can generate 2D microstructures that simulate the microstructural parameters of an actual 3D microstructure and, consequently, present an equivalent mechanical response. The following section describes the new proposed methodology to generate and calibrate 2D PFC microstructures using the gravimetric DEM method.

<table>
<thead>
<tr>
<th>Microstructural parameter</th>
<th>3D microstructures</th>
<th>2D gravimetric microstructures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Avg*</td>
<td>Std**</td>
</tr>
<tr>
<td>CN [-]</td>
<td>4.59</td>
<td>N.A.</td>
</tr>
<tr>
<td>FA [%]</td>
<td>6.79%</td>
<td>N.A.</td>
</tr>
</tbody>
</table>

*Average, **Standard deviation, N.A: not applicable for this case
5.2. Proposed methodology to generate 2D PFC microstructures

Figure 27 illustrates the general flowchart of the methodology proposed to generate 2D microstructures of PFC mixtures that better simulate 3D PFC microstructures. This methodology consists in combining three different numerical techniques: i) the MG software (Castillo et al. 2015), ii) the DEM software LMGC90, and iii) the FEM software Abaqus®.

The MG software is a computational tool that generates random 2D dense-graded asphalt microstructures composed by coarse aggregates that follow specific gradation, orientation, and shape parameters. In this chapter, the MG software is used to produce sets of coarse aggregate particles of a PFC mixture with controlled morphological properties. The LMGC90 DEM software is then used to apply gravimetric forces to the set of coarse aggregate particles generated using the MG. The result of this process is the coarse-aggregate fraction of a 2D PFC microstructure. It is worth mentioning that the final arrangement of the coarse aggregates depends on the input parameters used in LMGC90, meaning that these parameters need to be calibrated. Finally, the coordinates of the 2D PFC microstructures are imported to the FE software Abaqus®, where the 2D PFC microstructures can be subjected to different mechanical tests after coating each coarse aggregate by a thin film of asphalt mortar instead of asphalt mastic, as in the previous chapters, since the use of asphalt mortar is considered a more realistic representation of the actual material existing at the coarse-aggregate contacts. In this case, the PFC mixtures were subjected to computational dynamic modulus tests, or $|E^*|$ tests, to calibrate the
mechanical behavior of the generated 2D PFC microstructures using the experimental results of the PFC mixture used as a reference.

Although the methodology herein proposed can be used to generate multiple 2D microstructures of any PFC mixture, this work uses a specific PFC to illustrate the different stages of this technique. The selected PFC complies with the gradation presented in Table 16, which is the same used in previous chapters. The mixture is composed of sandstone aggregates, uses a neat asphalt binder with 60-70 (1/10 mm) penetration with an Optimum Binder Content (OBC) of 6.5%, and its target AV content is 20%. The aggregate gradation and volumetric properties of this mixture were used to conduct the computational generation of multiple 2D PFC microstructures.

In sum, the proposed method uses three different computation tools, and it requires two different calibration processes: i) calibration of the morphological properties of the individual aggregate particles produced by the MG software, and ii) calibration of the input DEM parameters. These processes are described in detail in the following sections.

5.3. Calibration of the morphological properties of the coarse aggregate particles

The morphological properties of the aggregate particles used in pavement engineering include their angularity, form, and texture (Masad et al. 2001, Chandan et al. 2004, Al-Rousan et al. 2005, Singh et al. 2013). Since texture is a difficult parameter to represent in 2D modeling, the MG software does not consider it when generating individual aggregate particles. The angularity and form properties are controlled in the MG software through both the Angularity and Form indexes, or AI and FI, originally proposed by Masad et al. (2001) and Al-Rousan et al. (2005), respectively, and later adapted by Castillo et al. (2018). The AI measures the accumulated change in the orientation of the segments on the perimeter of the particle. Larger values of AI correspond to more angular particles. The AI parameter is calculated by the MG following the equation below (Castillo et al. 2018):

\[
AI = \frac{1}{N-1} \sum_{i=1}^{N-1} |G_i - G_{i+1}|
\]

where \(N\) is the defined number of points located on the 2D perimeter of the aggregate particle and \(G_i\) is the orientation of the \(i\)th perimeter segment of the particle measured in radians from an arbitrary horizontal line.

On the other hand, FI measures the incremental change in the distance from the geometrical center to each vertex of the particle in all directions. For example, the FI of a circle is zero and larger values of FI correspond to more elongated particles. The FI parameter is computed using the equation proposed by Masad et al. (2001):

\[
FI = \sum_{\theta=0}^{\theta=360-\Delta\theta} \frac{|R_{\theta+\Delta\theta} - R_\theta|}{R_\theta}
\]

where \(R_\theta\) is the distance or ‘radius’ from the center of the aggregate particle to its perimeter at an angle \(\theta\) in degrees from the horizontal axis, and \(\Delta\theta\) is the increment of the angle at which the perimeter of the
aggregate particle would be divided for the analysis. The MG software defines $\Delta \theta = 4^\circ$, which means that the perimeter of the aggregate particles is divided into 90 equal sections. To illustrate the influence of the AI and FI parameters over the final morphological properties of the generated aggregate particles, Figure 28 presents four particles with different AI and FI parameters.

![Random aggregates created with different AI and FI parameters](image)

Figure 28. Random aggregates created with different AI and FI parameters: (a) FI=1.56, AI=0.056, (b) FI=1.56, AI=0.15, (c) FI=1.13, AI=0.085, and (d) FI=2.21, AI=0.085.

The MG software controls the final AI and FI indexes of the generated particles using three main parameters: i) the number of vertices of the aggregate particle ($verts$: $v_{\text{Min}}$ and $v_{\text{Max}}$), ii) the radius from the center of the particle to the vertices ($r$: $r_{\text{Min}}$ and $r_{\text{Max}}$), and iii) an elongation factor ($elong$), which multiplies the horizontal coordinates of the polygon by a random value ‘$f$‘. The elongation factor creates realistic aggregate particle shapes, like the ones of the aggregates used in asphalt mixtures. The number of vertices follows a uniform probability density function (pdf) limited by $v_{\text{Min}}$ and $v_{\text{Max}}$; the radius of the aggregate particles follows a beta pdf between $r_{\text{Min}}$ and $r_{\text{Max}}$; and the elongation factor follows a beta pdf limited by $f=1$ (no elongation) and $f=1+elong$. Once the aggregate particles are created, the MG rotates the coarse aggregates with respect to the horizontal using a factor that follows a uniform pdf between $-45^\circ$ and $45^\circ$, and randomly locates each aggregate particle in a predefined space. For additional details about the method to control the morphological properties of a set of aggregate particles in the MG software, please refer to Castillo et al. (2018).

To obtain aggregate particles with the desired morphological properties of typical PFCs, the parameters previously indicated were calibrated. First, ten vertical images were obtained from two PFC specimens reconstructed using X-ray CT images, since they provide a fair representation of the morphological properties of the aggregate particles. Thus, the AI and FI parameters of a total of 5,148 coarse aggregates were computed and adjusted to the pdf that provided the best fit. The best fit for the AI data was a normal distribution, while the best fit for the FI data was a log-normal distribution.

Then, ten sets of 500 aggregates that followed the gradation in Table 16 were created using the MG software. The number of vertices, radius, and elongation factor of these aggregate particles were modified until obtaining pdfs of AI and FI that followed the pdfs from the image analyses. As a result of the calibration process, the minimum and maximum number of vertices per aggregate were set at 7 and 15, $r_{\text{Min}}$ and $r_{\text{Max}}$ were set at 0.38 and 0.6 mm, and the $elong$ factor was set at 2.6 ($f=3.6$). Figure 29a illustrates the normal pdf of AI obtained from the images and the fit obtained from the calibration analysis. Figure 29b illustrates equivalent results for FI.
5.4. Calibration of the input DE parameters

The LMGC90 DE software uses the following input parameters for the gravimetric simulation: i) the drop height (\(DH\)) of the aggregates, ii) the friction coefficient (\(\Phi\)), iii) the restitution coefficient (i.e. the amount of energy dissipated in collisions) of the aggregate particles, and iv) the density of the aggregates. The microstructural characteristics and, consequently, the mechanical behavior of the final array of particles depend on these input parameters. As such, the input parameters of the DE software should be calibrated.

The selected approach for the calibration process consisted in changing the DE input parameters and subjecting each of the resulting microstructures to a \(|E^*|\) computational test in Abaqus®. These computational results were compared to the actual values of \(|E^*|\) obtained from PFC samples tested in the laboratory until obtaining similar results.

The experimental results of \(|E^*|\) were obtained after testing four cylindrical samples of the selected PFC mixture. The testing specimens were fabricated using the Superpave Gyratory Compactor and their final dimensions were 100 mm in diameter and 150 mm in height. Since the internal structure of the PFC mixtures is highly heterogenous (Alvarez et al., 2010b), the PFC specimens were tested twice (once per each side) with a Universal Test Machine MTS after applying a constant cyclic compressive force of 1,190 N at a frequency of 4, 10, and 16 Hz, following the standard E1876-15 (2015). Table 19 summarizes the \(|E^*|\) laboratory results.
Table 19. Laboratory results of the $|E^*|$ [MPa].

<table>
<thead>
<tr>
<th>PFC Specimen</th>
<th>4 Hz</th>
<th>10 Hz</th>
<th>16 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1,200</td>
<td>1,477</td>
<td>1,506</td>
</tr>
<tr>
<td>2</td>
<td>1,363</td>
<td>1,667</td>
<td>1,606</td>
</tr>
<tr>
<td>3</td>
<td>1,367</td>
<td>1,669</td>
<td>1,738</td>
</tr>
<tr>
<td>4</td>
<td>1,247</td>
<td>1,504</td>
<td>1,536</td>
</tr>
<tr>
<td>Avg*</td>
<td>1,294</td>
<td>1,579</td>
<td>1,596</td>
</tr>
<tr>
<td>Std**</td>
<td>118</td>
<td>148</td>
<td>176</td>
</tr>
<tr>
<td>COV***</td>
<td>9.1%</td>
<td>9.3%</td>
<td>11.0%</td>
</tr>
</tbody>
</table>

*Average, **Standard deviation, ***Coefficient of variation

To initiate the calibration process, three random sets of coarse aggregates with controlled morphological properties were generated using the MG software, using the PFC gradation described in Table 16. The coarse-aggregate particles were randomly located in a predefined space measuring 100 by 300 mm, as shown in Figure 30.

The three sets of coarse aggregates were imported into the LMGC90 software, and the $\Phi$ and $DH$ variables were modified during the application of gravimetric forces. Only these two variables were modified since previous studies determined that these were the most influential factors in the final microstructural characteristics of 2D PFC microstructures (Torres and Caro, 2016). Four different $\Phi$ values (i.e. 0 (frictionless), 0.35, 0.40, and 0.70) and three $DH$ values (i.e. 0, 10, and 15 mm) were selected for the analysis. The restitution parameter was set at 0.3, while the density of the aggregates was set initially at 1,000 g/mm$^3$, as suggested by Torres and Caro (2016). This process resulted in 36 different
microstructures with dimensions of 100 by 150 mm. As an example, Figure 31 shows four microstructures created after subjecting the third set of aggregate-particles to gravimetric forces at a \(DH\) of 15 mm and at the four \(\Phi\) values assessed. As illustrated in this figure, although the four resulting microstructures used the same set of aggregate particles and a constant \(DH\) value, their internal microstructural parameters, such as the AV content, are different.

![Figure 31](image)

Figure 31. Resulting microstructures obtained with the third MG random set of coarse aggregate particles with a \(DH\) of 15 mm, and \(\Phi\) values of 0.00, 0.35, 0.40, and 0.70.

The coordinates of the 36 PFC microstructures generated with the different DE input parameters were then imported to the FEM software Abaqus\(^\circ\). The PFC microstructures modeled in Abaqus\(^\circ\) have three main components: i) coarse aggregates, ii) asphalt mortar, and iii) air voids. The asphalt mortar uniformly coats each aggregate particle. The thickness of this film was numerically estimated again using the gradation and optimum binder content of the PFC mixture of reference, and after assuming a uniform film thickness in all particles. This estimation resulted in a value of 60 \(\mu\)m. Figure 32 illustrates the three material components of the 2D PFC models.

In terms of the mechanical response of these components in the FE model, the coarse aggregates were considered linear elastic materials with a modulus of 10,000 MPa (Blair 1955) and a Poisson’s ratio (\(v\)) of 0.25. The asphalt mortar was modeled as a linear viscoelastic material. The rheological properties of the mortar were obtained after applying frequency and temperature sweep tests to small cylindrical testing specimens measuring 125 mm in diameter and 500 mm in height using a solid geometry in a TA200ex rheometer (Kim et al. 2003). The details of the fabrication of this type of asphalt mortar samples can be found elsewhere (Caro et al. 2012, 2015). The dynamic modulus results were transformed from the frequency to the time domain to determine the Prony series of the relaxation modulus of the
material. The Prony series representing the shear relaxation modulus of a viscoelastic material is expressed according to Eq. 4 presented in Chapter III.

Table 20. Prony series of the reference PFC asphalt mortar at 25°C.

<table>
<thead>
<tr>
<th>$i$</th>
<th>$\rho$ [s]</th>
<th>$G$ [Pa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.01</td>
<td>$1.37 \times 10^9$</td>
</tr>
<tr>
<td>2</td>
<td>0.10</td>
<td>$4.13 \times 10^8$</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>$3.13 \times 10^8$</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>$1.00 \times 10^7$</td>
</tr>
<tr>
<td>5</td>
<td>100</td>
<td>$8.51 \times 10^4$</td>
</tr>
<tr>
<td>6</td>
<td>1,000</td>
<td>$9.85 \times 10^3$</td>
</tr>
<tr>
<td>7</td>
<td>10,000</td>
<td>$9.98 \times 10^2$</td>
</tr>
<tr>
<td>8</td>
<td>100,000</td>
<td>$1.00 \times 10^2$</td>
</tr>
<tr>
<td>9</td>
<td>1,000,000</td>
<td>$1.00 \times 10^1$</td>
</tr>
<tr>
<td>10</td>
<td>10,000,000</td>
<td>$1.00 \times 10^{-1}$</td>
</tr>
<tr>
<td>11</td>
<td>100,000,000</td>
<td>$1.00 \times 10^{-2}$</td>
</tr>
<tr>
<td>12</td>
<td>1,000,000,000</td>
<td>$1.00 \times 10^{-3}$</td>
</tr>
<tr>
<td>13</td>
<td>10,000,000,000</td>
<td>$1.00 \times 10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>$G_0$ [Pa]</td>
<td>$2.10 \times 10^9$</td>
</tr>
<tr>
<td></td>
<td>$\nu$ [-]</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>$G_\infty$ [Pa]</td>
<td>800</td>
</tr>
</tbody>
</table>

Figure 33a presents a typical FE model in which the displacement was fully restricted at the bottom of the PFC microstructure, and Figure 33b illustrates the global mesh used in the models. After conducting a sensitivity analysis of the mesh size, the coarse aggregates were meshed using a seed of 15 mm and the mortar was meshed using a seed of 0.75 mm. The coarse-aggregate particles were discretized using 3-node linear elements (i.e. type CPE3R in Abaqus®) and the mortar was discretized using 4-node bilinear elements (i.e. type CPE4R in Abaqus®). On average, a total of 67,000 elements were used per
model, a considerably smaller number of elements when compared to a 3D model with similar aggregate geometry.

![Figure 33. FEM PFC model geometry to obtain their dynamic modulus: (a) load and boundary conditions and (b) global mesh.](image)

The resulting value of $|E^*|$ was computed based on the amplitude of the stress and strains signals. Figure 34 shows an example of the stress vs. strain curve of a PFC microstructure generated using the second set of aggregates (Figure 7b) $\Phi=0.35$ and of $DH=0$ mm and tested at a cyclic compressing load of 1,190 N at 10 Hz. The $|E^*|$ of this computational PFC microstructure was 1,694 MPa.

![Figure 34. Stress vs. strain curve, numerically obtained from a dynamic axial moduli test performed over a PFC microstructure generated using a contact friction of 0.35 and a drop height of 0 mm.](image)
After conducting all the PFC FEM simulations, several correlations between some microstructural parameters of the PFCs and the resulting $|E^*|$ were analyzed. The selected microstructural parameters are:

- AV content
- FA; which in this case also included those aggregates that do not transmit stresses even when in contact with other particles.
- CN, without considering aggregates classified as FA.
- Average length of the contacts between aggregates (ACL), computed as the ratio between the total length of the contacts between aggregates and the number of aggregates.
- The principal orientation of the normal vectors of the contacts ($\theta_N$). The orientation of the normal vector of each contact was computed counterclockwise from the horizontal axis. The principal orientation of the normal vectors of the contacts was defined as the average of all the orientations of the normal vectors within the contacts. The relevance of this parameter is that it influences the load path of two connected aggregate particles (Chang and Lio 1994).

Figure 35 illustrates the Von Mises stress (i.e. equivalent stress) path distribution of four out of the 36 different 2D PFC microstructures tested at a frequency of 10 Hz. These microstructures were obtained for the first set of aggregates (Figure 30a) by varying the $\Phi$ values (i.e. 0, 0.35, 0.4, and 0.7), after maintaining a constant $DH$ of 0 mm. This figure shows that as the $\Phi$ parameter increases, the AV content increases, directly affecting the stress path distribution and the $|E^*|$ of the PFC microstructures. In addition, Figure 35d highlights a set of floating aggregates.

Figure 36 presents the correlation between the selected microstructural parameters and the values of $|E^*|$ obtained at 10 Hz. It should be highlighted that floating aggregates were not considered in the computation of CN, ACL, and $\theta_N$. It was found that, in all cases, there is an inverse linear correlation between the $|E^*|$ and the AV content (Figure 36a), while there is a direct correlation between the $|E^*|$ and the CN and ACL (Figure 36b and c). In contrast, it was found that there is no correlation between the $|E^*|$ and $\theta_N$ (Figure 36d). Since the $\theta_N$ influences the load path of two connected aggregate-particles and the PFC microstructures are usually subjected to vertical loads, an average $\theta_N$ of 90°, like the one obtained, is desired. It was also found that there is an inverse linear correlation index of 0.99 between the CN and AV content. These correlations are in good agreement with previous works (Alvarez, Mahmoud, et al. 2010, Manrique-Sanchez and Caro 2019), and they further corroborate that the mechanical performance of PFCs depends on their geometrical microstructural parameters. It is noteworthy that the correlations presented in Figure 36 could be used as a guide to calibrate the mechanical responses of different PFC.
Figure 35. Stress path distribution of four PFC microstructures. The PFC microstructures had a DH of 0 mm and Φ coefficients of: (a) 0.00, (b) 0.35, (c) 0.40, and (d) 0.70.

Table 21 summarizes the average computational \( |E^*| \) of the resulting 2D PFC microstructures using the three sets of initial coarse aggregates (Figure 30) with different values of Φ and DH during the gravimetric simulations. These results correspond to a loading frequency of 10 Hz. Values in bold and italics in this table correspond to Φ and DH combinations that produce PFC microstructures that successfully represent the mechanical performance of the samples tested in the laboratory at 10 Hz (i.e. \( |E^*| = 1,579 \pm 148 \text{ MPa} \)). It must be noted that intermediate contact frictions (e.g. 0.35 and 0.4) are close to the friction coefficient between real particles (Mitchell and Soga 2005).

Table 21 shows that PFC microstructures generated with a Φ coefficient of 0.35 and DH values of 0, 10, and 15 mm and those generated with Φ coefficient of 0.4 and DH values of 10 and 15 mm result in values of \( |E^*| \) similar to the experimental result at 10 Hz. In general, these PFC microstructures have an average AV content of 20.9% – which correspond to the target AV content of the PFC mixtures evaluated in laboratory –, an average CN of 3.16, and an ACL of 2.72 mm (Figure 36). Nevertheless, to conclude that these sets of input parameters are correct, the numerical \( |E^*| \) results at other frequencies should also coincide with the experimental results at other loading conditions. Therefore, the PFC microstructures obtained with these combinations of Φ and DH were tested in FEM again at loading frequencies of 4 and 16 Hz.
The data in Table 22 show that the only PFC microstructures that match the experimental modulus at these additional frequencies correspond to the combination of 0.4 for $\Phi$ and 15 mm for $DH$. The PFC microstructures generated with these input parameters had average $|E^*|$ values that were 13.5%, 0.6%, and 0.7% higher than the experimental $|E^*|$ values at 4, 10, and 16 Hz, respectively. Considering the
heterogeneity of the contact network of the PFC mixtures and the acceptable differences between the numerical and experimental results, particularly for frequencies equal to or higher than 10 Hz, it can be concluded that it is possible to generate random 2D PFC microstructures using DE gravimetric methods with similar mechanical responses as that of actual PFC mixtures.

Table 22. Average FE results of the $|E^*|$ [MPa] of PFC microstructures generated with $\Phi=0.4$ and $DH=15$ mm at a load frequency of 4, 10, and 16 Hz.

<table>
<thead>
<tr>
<th>Initial coarse aggregate set</th>
<th>4 Hz</th>
<th>10 Hz</th>
<th>16 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1,426</td>
<td>1,527</td>
<td>1,543</td>
</tr>
<tr>
<td>2</td>
<td>1,517</td>
<td>1,611</td>
<td>1,625</td>
</tr>
<tr>
<td>3</td>
<td>1,465</td>
<td>1,571</td>
<td>1,586</td>
</tr>
<tr>
<td>Avg*</td>
<td>1,469</td>
<td>1,570</td>
<td>1,585</td>
</tr>
<tr>
<td>Std**</td>
<td>46</td>
<td>42</td>
<td>41</td>
</tr>
<tr>
<td>COV***</td>
<td>3.1%</td>
<td>2.7%</td>
<td>2.6%</td>
</tr>
</tbody>
</table>

Overall, these results demonstrate that the methodology proposed to generate random 2D PFC microstructures through gravimetric methods can –in terms of computational and time costs– efficiently represent the 3D stiffness at different loading frequencies applied to PFC mixtures. The value of this methodology is that it can be used to generate multiple random PFC microstructures in order to conduct computational probabilistic and statistical studies on the mechanical response of PFC mixtures under different field conditions, a task that would be expensive or even impossible to perform in the laboratory, in the field or with full 3D models.

5.5. Conclusions and recommendations

This chapter proposed a novel methodology to randomly generate 2D PFC microstructures that are able to capture the mechanical response of actual 3D PFC mixtures. The 2D method proposed combines the MG software, to generate random coarse-aggregate particles with controlled morphological properties, and the LMGC90 DE software, to generate the coarse aggregate fraction of 2D PFC microstructures using gravimetric techniques. In this case, the resulting 2D arrays of coarse aggregate particles were then coated by a film of asphalt mortar, imported to the FE software Abaqus®, and subjected to computational $|E^*|$ tests. Overall, from this chapter is concluded that:

- The geometries obtained from 2D cuts of 3D microstructures provide inaccurate internal networks.
- After the proposed calibration process, it was demonstrated that the DE model produces microstructures with $|E^*|$ values that produce an acceptable match for the experimental results for the three different load frequencies evaluated (4, 10, and 16 Hz). Depending on the problem...
under consideration, a similar approach could be used to calibrate other mechanically-related properties of these mixtures.

- The results demonstrate that the proposed technique is a powerful tool to generate random 2D PFC microstructures.

The proposed methodology can be used to generate multiple random microstructures of any PFC mixture to conduct computational probabilistic and statistical studies of the functionality, durability and mechanical response of PFC mixtures under different field conditions. In particular, this methodology was used in Chapter VII to generate 2D PFC microstructures to evaluate the raveling susceptibility of PFC mixtures using FE with realistic field operational conditions (e.g. different traffic speeds, load magnitudes, temperature conditions, etc.) and mortar material properties. Other potential applications include the assessment of the mechanical response of different design approaches for PFC mixtures, after considering the uncertainty induced by the heterogeneity of the PFC microstructures (i.e. computational analysis of multiple randomly generated PFC microstructures as part of reliability-based design approaches).
From previous chapters, it was concluded that there is still a lack of information on how climate-related factors (mainly the presence of air, temperature and moisture) impact the mechanical properties and durability of PFCs over time. The objective of this Chapter is to assess the combined effects of aging and moisture on the viscoelastic and fracture properties of the asphalt mortar present at the stone-on-stone contacts within PFCs. To achieve this goal, loose asphalt mortar of a typical PFC was aged at two different conditions (short- and long-term). Afterwards, the mortar was compacted, and the testing specimens were subjected to different dry-wet-dry moisture vapor cycles. Next, the linear viscoelastic and fracture properties of the mortar were determined through Dynamic Mechanical Analyzer (DMA) and Semi-Circular Bending (SCB) tests. Thus, the changes in the linear viscoelastic and fracture properties between the unaged and aged specimens after each moisture cycle were used to quantify the relative effect of aging and moisture on the mechanical response of the asphalt mortar (Aragão and Kim 2012, Kim et al. 2015).

These experimental data provide an initial insight to the change in performance caused in the stone-on-stone contacts in PFC mixtures during their service life due to environmental factors and are used as input parameters in computational micromechanical models presented in Chapter VII.

6.1. Materials

Similar to previous Chapters, the selected PFC corresponds to a type ‘FC-5’ mixture specified by the Florida Department of Transportation (FDOT 2018), and it could be considered a ‘typical’ PFC mixture currently used in the United States (Cooley et al. 2009, Hernandez-Saenz et al. 2016). This mixture is composed of limestone aggregates, PG 76-22 polymer-modified binder (PMA), 0.3% of cellulose fibers per total weight and 0.5% of liquid antistripping agent by binder weight. The OBC of the mixture was determined as 6.5% by total weight (Arámbula-Mercado et al. 2016, FDOT 2018). This information is summarized in Table 23.

The aggregate gradation of the asphalt mortar corresponds to the fine fraction of the PFC gradation (Figure 37). The binder content of the mortar was estimated based on the OBC of the PFC full mix, following existing recommendations (Masad et al., 2006; Caro et al., 2008). Following the same procedure explained in previous chapters, the aggregates from the full mix were assumed to be spheres.

4 The work of Chapter VI is summarized in the article “Coupled Effects of Ageing and Moisture on the Fracture Properties of Permeable Friction Courses (PFC)” by Laura Manrique-Sanchez, Silvia Caro and Yong-Rak Kim, International Journal of Pavement Engineering (DOI: 10.1080/10298436.2020.1784417).
homogeneously coated by a thin film of binder. Using the entire gradation of the mixture (i.e. number of particles per each size of aggregates) and the total available asphalt binder, it was determined that the thickness of the binder that coats each aggregate was approximately $7\mu m$. This value was used to estimate the volume and weight of the asphalt binder present in the mortar phase of the mixture, which resulted in a value of 13.4% of binder content by total weight of asphalt mortar.

Table 23. Design of the selected PFC mixture and the corresponding design of the asphalt mortar.

<table>
<thead>
<tr>
<th>Sieve</th>
<th>FDOT FC-5 Specification Limits</th>
<th>PFC full mix cumulative pass [%]</th>
<th>Mortar cumulative pass [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm (3/4&quot;)</td>
<td>100%</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>12.5 mm (1/2&quot;)</td>
<td>85% - 100%</td>
<td>85.5</td>
<td>100</td>
</tr>
<tr>
<td>9.5 mm (3/8&quot;)</td>
<td>55% - 75%</td>
<td>58.2</td>
<td>100</td>
</tr>
<tr>
<td>4.75 mm (#4)</td>
<td>15% - 25%</td>
<td>21.0</td>
<td>100</td>
</tr>
<tr>
<td>2.36 mm (#8)</td>
<td>5% - 10%</td>
<td>8.6</td>
<td>100</td>
</tr>
<tr>
<td>1.18 mm (#16)</td>
<td>–</td>
<td>5.7</td>
<td>100</td>
</tr>
<tr>
<td>0.60 mm (#30)</td>
<td>–</td>
<td>4.7</td>
<td>82.46</td>
</tr>
<tr>
<td>0.30 mm (#50)</td>
<td>–</td>
<td>3.7</td>
<td>64.91</td>
</tr>
<tr>
<td>0.15 mm (#100)</td>
<td>–</td>
<td>2.1</td>
<td>36.84</td>
</tr>
<tr>
<td>75um (#200)</td>
<td>2% - 4%</td>
<td>2.1</td>
<td>36.84</td>
</tr>
</tbody>
</table>

Aggregate type: limestone  
Binder type: PMA  
OBC (%): 6.5  
Antistripping (%): 0.5 by weight of binder  
Cellulose fibers (%): 0.3 by weight of mix

Figure 37. (a) PFC full mix gradation with cellulose fibers, and (b) mortar gradation with cellulose fibers.
6.2. Environmental conditioning and mechanical testing

Aging and moisture damage in asphalt materials are phenomena that occur in the field at the same time but at different rates and intensities. However, when conducting laboratory characterization, each phenomenon is typically treated independently. To couple these two phenomena, this work selected a sequential process that consisted in first aging the loose asphalt mortar in two states (short- and long-term) –before compaction and prior to any moisture conditioning– and then compacting the mortar, preparing the required testing specimens and subjecting them up to three dry-wet-dry cycles. The ‘dry’ state refers to a condition where the mixture has a low relative humidity (RH) of 0-4%, and the ‘wet’ state to a RH condition of 97-100%. DMA and SCB specimens were tested at the end of each dry-wet-dry cycle in both aging conditions to assess the individual and combined impact of the climatic processes. The following sections explain these procedures.

6.2.1. Aging conditioning

The short-term aging (SA) state of the PFC asphalt mortar aims at simulating the asphalt aging and binder absorption during the production of the asphalt mixtures and the construction of the pavement structure. Following the recommended procedure AASHTO-R-30 (2002), the SA state consisted in subjecting the loose asphalt mortar to two hours at the compaction temperature \( T_c = 160^\circ\text{C} \) in the oven. On the other hand, the long-term aging (LA) procedure aims at simulating the properties of the mortar after several service years. In this case, the asphalt mortar was aged in a loose state following the recommendations of the NCHRP-09-54 project (Kim et al. 2018), which reported that aging processes under loose conditions are more homogenous and induces a higher oxidation rate than when aging is conducted on compacted specimens, as specified by the AASHTO-R-30 standard. Considering that the service life of PFC mixtures is typically between 7-12 years (Huber 2000, Yildirim et al. 2007, Cooley et al. 2009), it was decided to let the loose mortar mix during 10 days in an oven at 95°C. According to Kim et al. (2018), this procedure emulates 8-10 years of asphalt aging in the field in regular dense-graded HMA. Since the AV content of the PFC mixture is around 20%, the rate of oxidation of the PFC in field is expected to be noticeably higher than regular dense-graded HMAs. Thus, this procedure is assumed to properly simulate the state of the mortar after several years of in-field service (e.g. around 8 years or less).

6.2.2. Fabrication of the testing specimens

After aging, the loose asphalt mortar in both states was compacted using the Superpave Gyratory Compactor (SGC) at a target AV of 7.5%. It should be noted that the AV content in a regular PFC ranges between 15 and 20%, but the mortar located among the macro-voids in the microstructure of the mixture is expected to have a much lower value, similar to that in dense graded mixtures. Thus, an AV content of 7.5% was selected since it is not only a logical value for mortar materials within these mixtures but also because it is enough to enable moisture diffusion within the specimens in a reasonable period of time.

Afterwards, the specimens for DMA and SCB testing were fabricated. The final geometry and dimensions of the specimens are illustrated in Figure 38. In the case of the DMA test, small mortar cylinders of 50 mm in height and 12.5 mm in diameter, shown in Figure 38, were cored from the
compacted SGC specimens (150 mm in diameter and 80 mm in height), as presented in Figure 39. The SCB testing specimens were obtained after cutting disks with 25 mm in thickness from compacted SGC cylinders that were 150 mm in diameter and 170 mm in height (Figure 40a). A total of 5 disks with a thickness of 25 mm were obtained per specimen (Figure 40b). Next, the disks were cut in half and a 15 mm notch was generated in the middle of the planar base of each specimen, resulting in 10 SCB specimens with 150 mm in diameter, 75 mm tall and a 15 mm notch (Figure 40c).

Figure 38. Sketch of the mortar testing specimens: (a) small cylinders for DMA testing, and (b) Semi-circular specimen for SCB testing. All units are in mm.

Figure 39. Fabrication of DMA testing specimens: (a) SGC asphalt mortar cylindrical specimen, and (b) cored mortar cylinders (50 mm in height and 12.5 mm in diameter).
6.2.3. *Moisture conditioning*

The mortar specimens in both SA and LA conditions were subjected to three dry-wet-dry cycles (i.e. C1, C2 and C3) at a controlled room temperature of 24°C ± 2°C. One full dry-wet-dry cycle consists in taking a specimen in a steady-state dry condition (in an environment with RH ≈ 0-4%, control condition), to a steady-state environment of high-moisture vapor (RH ≈ 100%), and then again to a steady-state dry condition (RH ≈ 0-4%). The objective of these moisture conditioning cycles was to assess the impact of changes in RH that could occur in the field on the mechanical characteristics of the PFC asphalt mortar. Also, since several dry-wet-dry cycles were subsequently applied to the specimens, the mechanical testing results are expected to show if there are irreversible effects caused by changes in the internal moisture condition of the mortar.

The two RH states were obtained after applying the principle of water vapor equilibrium technique using silica gel (RH ≈ 0-4%) and distilled water (RH ≈ 100%) (ASTM E104-02, 2012). More specifically, as observed in Figure 41 and Figure 42, the compacted DMA and SCB mortar specimens were placed in isolated plastic containers with distilled water or silica gel, where the RH was controlled and monitored using a relative humidity sensor (Honeywell HIH4602C). The DMA and SCB specimens were located within the containers but were not in contact with the silica gel or the distilled water; instead, they were placed on top of a metallic or acrylic ‘grills’, as observed in both figures.
Figure 41. Moisture vapor conditioning system for DMA mortar testing specimens: (a) dry conditioning, and (b) wet conditioning.

Figure 42. Vapour conditioning system for SCB mortar testing specimens (right figures: top view of SCB specimens within the plastic container).

The RH level within the mortar specimens was controlled using the gravimetric method, in which the specimens were weighted every 2 days until approaching a steady state condition. A specimen was considered to be near a steady state (i.e. dry or wet) when the difference in weight of two consecutive measurements was lower than 0.01 gram. As an example, Figure 43 shows the cumulative weight change of a SCB specimen conditioned to one dry-wet-dry cycle. The first part of this graph shows that the SCB
specimen—that was under room conditions of around RH ≈ 30%—loses weight after being subjected to dry conditioning. Then, the SCB specimen was subjected to wet conditioning, causing an increase in weight in the specimen. Afterwards, the sample was dried again, completing one dry-wet-dry cycle. In average, 60 days were needed to complete one dry-wet-dry cycle, which means that approximately 180 days (6 months) were required to complete three cycles. It is worth mentioning that this moisture conditioning approach is preferred over submerging the specimens in water, as the only process occurring in a controlled RH environment is moisture vapor diffusion within the specimen. This approach has been successfully used to evaluate the influence of partial saturation condition on the mechanical properties of dense-graded asphalt mixtures (Rueda et al. 2017, 2019).

![Cumulative weight change of a short-term aged SCB specimen subjected to one moisture conditioning dry-wet-dry cycle.](image)

6.2.4. Linear viscoelastic properties of the asphalt mortar specimens

DMA testing was used to quantify the rheological properties of the mortar specimens (Kim et al. 2002, 2003, 2004, Masad et al. 2006, Caro et al. 2012, 2015). The mortar cylinders were prepared for testing in both aging conditions after being subjected to each dry-wet-dry cycle. To do so, metallic holders were attached using an epoxy to each end of the cored mortar specimen (Figure 44a). Then, the cored specimens were placed in the rheometer (TA-series AR 2000) (Figure 44b) and subjected to a frequency sweep test in a range of frequencies from 0.01 to 30 Hz, using a constant shear strain of 1x10^{-4} % under a controlled room temperature (24±2 ºC). The shear strain value was selected after assuring that the specimens were within the linear viscoelastic region, which was determined through additional strain sweep tests. The temperature was kept constant at 25ºC to maintain the required moisture conditions within the specimen (Rueda et al. 2017, 2019). Since the DMA test is non-destructive within the linear viscoelastic range, the same 10 DMA specimens (i.e. five replicates for the SA and LA, respectively) were moisture vapor conditioned and tested after completing each dry-wet-dry cycle.
6.2.5. Fracture properties of the asphalt mortar specimens

Once the SCB specimens in short and long-term aged states were moisture vapor conditioned at each dry-wet-dry cycle, they were placed and tested in a loading frame (TA Electro Force 3200), as shown in Figure 45. Each SCB specimen was sit on two supports that restrained vertical displacement Figure 45a (Nsengiyumva and Kim 2019). A monotonic downward load at a displacement rate of 0.5 mm/min was applied until sample failure. During the test, the force vs. load point displacement (LPD) curve was recorded. Then, the planar area of the fractured faces of the specimen, also known as the ligament area ($A_{lig}$) and shown in Figure 45b, was quantified to compute several fracture indicators.

![Figure 44](image)

Figure 44. (a) Attached clips or holders to the cored mortar specimens, and (b) setup of the mortar specimen in the rheometer.

![Figure 45](image)

Figure 45. (a) SCB setup before testing, and (b) ligament area ($A_{lig}$) of a fracture SCB specimen obtained after testing.

Figure 46 exemplifies a force vs. LPD curve resulting from a long-term aged SCB specimen after two dry-wet-dry cycles. From these data, several fracture-related quantities can be obtained such as: (i) the peak load ($P_{max}$), which is the maximum load recorded before fracture, (ii) the work of fracture ($W_f$),
which corresponds to the total area below the force vs. LPD curve, and (iii) the pre and post peak slopes
($m_1$ and $m_2$), which are the slopes of the initial part of the curve and after its peak.

![Figure 46. Fracture-related parameters obtained from the force vs. LPD curve for a long-term aged specimen subjected to two dry-wet-dry cycles.]

To analyze the coupled effects of aging and moisture on the fracture properties of the SCB mortar specimens, four fractured-related parameters were selected: (i) the peak load ($P_{\text{max}}$), (ii) the fracture energy ($G_f$), (iii) the cracking resistance index ($CRI$), and (iv) the flexibility index ($FI$).

The fracture energy ($G_f$) is the total dissipated energy during the fracture process and is usually expressed in kJ/m$^2$. It is computed as the ratio between the work of fracture ($W_f$) and the ligament area ($A_{\text{lig}}$):

$$G_f = \frac{W_f}{A_{\text{lig}}}$$  \[9\]

The $CRI$, which was proposed by Kaseer et al., (2018), quantifies the amount of energy required to fracture a SCB specimen and is obtained after normalizing the fracture energy ($G_f$) by the peak load ($P_{\text{max}}$):

$$CRI = \frac{G_f}{P_{\text{max}}}$$  \[10\]

If two mixtures have the same $G_f$ but different $P_{\text{max}}$, the $CRI$ will show that one mixture is more brittle than the other. A lower $CRI$ value is related to a lower resistance to fracture (due to more brittleness), while a higher $CRI$ indicates a better resistance to fracture (more ‘ductile’ mixtures).

Finally, the $FI$ was proposed by the Illinois Center for Transportation with the objective of measuring the intermediate temperature cracking resistance of asphalt mixtures (Ozer et al. 2016). $FI$ is defined as the fracture energy ($G_f$) divided by the post-peak slope ($m_2$) at the inflexion point:

$$FI = \frac{G_f}{|m_2|} \times 10$$  \[11\]
The inclusion of $m_2$ in the computation of $FI$ gives an initial insight of the speed at which the crack propagates after reaching the peak load. Even though this index captures post-cracking processes, one limitation is the difficulty to compute $m_2$, especially when the load-LPD curve does not follow a continuous trend or when the mixture fractures at the peak load (i.e. brittle behavior) (Zhou et al. 2017, Kaseer et al. 2018).

6.2.6. Summary of the experimental plan

Table 24 lists the experimental plan of this study. In summary, after aging and moisture vapor conditioning of the PFC asphalt mortar specimens, DMA and SCB tests were conducted. The specimens in short- and long-term aging states were tested after being dry conditioned (RH ≈ 0-4%, control condition), and after completing one, two and three dry-wet-dry cycles (i.e. C1, C2, or C3). Five replicates were tested per each case to account for the potential sample-to-sample variability, which lead to a total of 40 SCB and 10 DMA specimens. Considering the size and number of specimens, 6 plastic containers were used to complete the moisture conditioning of the SCB specimens, and 2 plastic containers were used for the DMA specimens (as shown in Figure 41 and Figure 42).

Table 24. Experimental plan (1 cycle corresponds to one dry-wet-dry vapor conditioning process).

<table>
<thead>
<tr>
<th>Aging state</th>
<th>RH Condition</th>
<th>SCB Test</th>
<th>DMA Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short-term</td>
<td>Dry (RH ≈ 0-4%, control condition)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Short-term</td>
<td>Cycle 1 (C1)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Short-term</td>
<td>Cycle 2 (C2)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Short-term</td>
<td>Cycle 3 (C3)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Long-term</td>
<td>Dry (RH ≈ 0-4%, control condition)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Long-term</td>
<td>Cycle 1 (C1)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Long-term</td>
<td>Cycle 2 (C2)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Long-term</td>
<td>Cycle 3 (C3)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Total specimens per test</td>
<td>40</td>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

6.3. Analysis of results

6.3.1. Linear viscoelastic properties of the asphalt mortar

Figure 47 presents the DMA frequency sweep test results. Data in this figure correspond to the dynamic shear modulus ($|G^*|$) of the conditioned mortar specimens under different frequencies (from 0.01 to 30 Hz in 17 steps), at a temperature of 25°C. The values in the graph correspond to the average of the results obtained for five replicates at each testing condition (i.e. short and long-term aging after each moisture conditioning cycle). It is noteworthy that temperature sweep tests were not conducted in order to avoid
internal moisture condition changes within the specimen. The specimens were named based on their aging state (i.e. SA or LA) and moisture condition (i.e. dry, C1, C2, or C3). For example, a specimen in short-term aged condition and tested after two dry-wet-dry cycles was named as ‘SA_C2’.

![Figure 47](image-url)

**Figure 47.** DMA test results at 25°C for the short-term (SA) and long-term (LA) aged conditioned specimens under different moisture vapour conditions.

**Effect of moisture vapor**

Figure 47 shows the effects of dry-wet-dry cycles in both SA and LA states on the dynamic moduli. In average, the shear dynamic moduli of the SA and LA aged specimens decreased in 7.4% and 6.6% after each dry-wet-dry cycle. Nonetheless, the coefficient of variation (COV) among the 5 replicates tested in each condition (aging and moisture vapor) was approximately 13%. Therefore, a statistical \(t\)-test was performed to determine if there were statistical differences among the average dynamic modulus values obtained from the DMA replicates at each moisture condition cycle under the same aging states. The \(t\)-test was conducted to corroborate or reject the null hypothesis (\(H_0\)) that the mean of two pairs of results (\(\mu_1\) and \(\mu_2\)) were the same (i.e. \(H_0: \mu_1 - \mu_2 = 0\)). The test was performed among the different moisture vapor conditions (i.e. Dry-C1, Dry-C2, Dry-C3, C1-C2, C1-C3, and C2-C3) for SA and LA aged specimens, at a confidence level of 90%. Statistical test results are shown in Table 3. Considering that each DMA test had 17 data points, the null hypothesis is rejected if the \(t\)-test result is lower than -1.33 or higher than 1.33. Results presented in Table 25 indicate that the mean dynamic modulus results of the dry cases are not statistically different from the mean values of the moisture vaped cases, in both SA and LA conditions.

The statistical results in Table 25 may be explained, although further proof is needed, by the fact that the low strain-level conditions imposed on the specimens during this test do not allow to identify any potential cohesive or adhesive degradation caused by moisture in the internal microstructure of the specimens. This does not mean that there was not any damage generated in the microstructure of the mortars due to the imposed moisture cycles but that the test is not able to capture it. Other authors have
also reported that the specific moisture conditioning process selected in a study may not be severe enough to result in changes in the dynamic modulus (López-Montero and Miró 2016).

Table 25. *t*-test results from DMA specimens.

<table>
<thead>
<tr>
<th>Result pairs</th>
<th>Short-term (SA)</th>
<th>Long-term (LA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry-C1</td>
<td>-0.61</td>
<td>0.53</td>
</tr>
<tr>
<td>Dry-C2</td>
<td>-0.08</td>
<td>-0.34</td>
</tr>
<tr>
<td>Dry-C3</td>
<td>0.60</td>
<td>0.74</td>
</tr>
<tr>
<td>C1-C2</td>
<td>0.56</td>
<td>-0.94</td>
</tr>
<tr>
<td>C2-C3</td>
<td>1.11</td>
<td>1.12</td>
</tr>
<tr>
<td>C1-C3</td>
<td>1.21</td>
<td>0.28</td>
</tr>
</tbody>
</table>

It is also important to consider that the binder content of the mortar specimens is 13.4%, which is a high amount in comparison with full HMA or PFC mixtures. Given the low moisture diffusion coefficient of asphalt binders (Arambula *et al.* 2009), a slower moisture diffusion in the mortar specimens is expected when compared to full HMA mixtures. In fact, after 25 days in wet conditioning (i.e. RH≈100%)—when the specimens were expected to approach a wet steady state according to the criterion specified in Section 3.2—the degree of saturation of the DMA specimens were approximately 65%. The degree of saturation is defined as the ratio of the volume of water to the volume of void and was computed according to Equation 4, where $S_a$ is the degree of saturation of the DMA sample after being moisture vapor conditioned; WWV is the weight gained during the moisture vapour conditioning; $\rho_w$ is the water density at room temperature (≈25°C); and AV is the total volume of air voids in the DMA specimens (Muni Budhu 2000):

$$S_a = \left( \frac{WWV}{\rho_w} \right) \times AV \times 100$$  \[12\]

This suggests that the criterion used to determine the steady state condition of the specimen (i.e. a change of less than 0.01 gram between two consecutive measurements) may not reflect a ‘true’ steady state condition of the material. According to a previous study (Vasconcelos *et al.* 2010), asphalt mortars may continue gaining moisture slowly for months or even years. Thus, it is hypothesized that if the mortar specimens had been subjected to longer periods of moisture vapor conditioning, the produced adhesive or cohesive damage would have been manifested through changes in the dynamic moduli. Finally, this PFC mixture was designed with polymer modified binder, antistripping agent and limestone aggregates, a combination of materials that have proven to be moisture resistant and perform well under actual field conditions (Jenks 2017, Watson *et al.* 2018). For these reasons, the asphalt mortar might not present significant changes in its viscoelastic properties when subjected to the three dry-wet-dry cycles.
Effect of the aging conditioning:
In order to investigate the effects of aging on the viscoelastic properties of the mortar, $t$-tests were performed with the dynamic moduli of the SA and LA cases only in the dry condition. No additional $t$-tests were done between both aging states for the moisture-conditioned specimens, since it was already demonstrated that the dynamic modulus results are not statistically different among dry-wet-dry cycles. The $t$-test concluded that the mean values of dynamic shear modulus of the specimens in SA and LA condition are statically different with a $t$-score of -4.02. In general, it was found that the LA mortar specimens were stiffer than the SA specimens in all moisture states by 52.8%, 41.8% and 35.7% for low, medium and high frequencies (i.e. 1, 10 and 30 Hz), respectively.

6.3.2. SCB test results
Figure 48 presents the load vs. LPD curves obtained for the short and long-term aged SCB specimens in dry condition. As noticed, there is a clear difference in the shape of the curves between the two levels of aging and, therefore, in their fracture-related properties. Figure 49 summarizes the mean values and variability of the fracture-related parameters (i.e. peak load $P_{\text{max}}$, fracture energy $G_f$, cracking resistance index (CRI) and flexibility index (FI)) obtained from five replicates after each moisture cycle under either aging condition. The same nomenclature adopted for the DMA tests was used to present the SCB results (SA and LA for short and long-term aged conditions, and dry, C1, C2 and C3 for moisture conditions in dry-wet-dry cycles).

![Figure 48](image)
In general, Figure 48 and Figure 49 show that there is a clear difference between LA and SA results, in which the LA cases experience higher force peaks and faster/earlier fracture than the SA cases. In addition, the LA cases present a clearer trend with respect to the moisture condition. All four fracture-related parameters in this aging state decrease with increasing cycles of moisture conditioning, while this was not the case for the SA specimens. When computing the COV of the fracture parameters of the dry specimens in both aging conditions, it was found that the variability was between 5-7% for \( P_{\text{max}} \), 17-23% for \( G_f \), 13-19% for \( CRI \), and 33-46% for \( FI \). Table 26 summarizes the COV of each fracture parameter under the different aging and moisture vapor conditions. These data show that the COV tends to increase after each dry-wet-dry cycle, especially for \( FI \). This could be explained by the fact that the addition of a new random variable (i.e. dry-wet-dry moisture cycles through moisture diffusion) to an existing process increases the uncertainty of the combined process (Coleman et al. 1999). In this case, the new variable is moisture diffusion, which is a physical phenomenon with different sources of uncertainty that increases.
the total variability of the experimental results. Data in Figure 49 and Table 26 were used to analyze the effect of the aging and moisture conditioning on the SCB fracture results, as explained in the following sub-sections.

Table 26. COV of the fracture-related properties among SCB replicates at each aged and moisture vapor condition.

<table>
<thead>
<tr>
<th></th>
<th>Short-term aging (SA)</th>
<th>Long-term aging (LA)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P_{\text{max}}$</td>
<td>$G_f$</td>
</tr>
<tr>
<td>Dry</td>
<td>4.5%</td>
<td>23.4%</td>
</tr>
<tr>
<td>C1</td>
<td>4.5%</td>
<td>5.7%</td>
</tr>
<tr>
<td>C2</td>
<td>14.0%</td>
<td>19.9%</td>
</tr>
<tr>
<td>C3</td>
<td>9.4%</td>
<td>14.0%</td>
</tr>
</tbody>
</table>

Effect of the moisture vapors conditioning

Figure 49 shows that SA specimens did not show a consistent trend in the magnitude of the fracture parameters with the varying moisture vapors conditioning cycles. This could be caused by the variability reported among SCB replicates (see Table 26). On the contrary, the fracture parameters in LA specimens tended to consistently decrease when they were subjected to additional dry-wet-dry cycles. The mean values of $P_{\text{max}}$, $G_f$ and $CRI$ decreased approximately 11.1%, 17.2% and 6.9%, respectively, after each dry-wet-dry cycle. Similar to the SA cases, $FI$ results did not show any consistent tendency with the moisture vapors conditioning cycles in the LA specimens. For both aging conditions, $FI$ showed the highest variability with COV values that ranged between 18.1 and 66.9%. Therefore, caution is required when using the mean value of this fracture parameter for comparing or ranking the fracture performance of different mixtures, or of mixtures subjected to different loading or environmental conditions.

Several $t$-tests were performed to identify if under a specific aging condition, the mean values of the fracture parameters were statistically different among moisture cycles. Since five replicates were tested per each case, the mean values between groups being compared are statistically different if the $t$-test result is smaller than -1.47 or higher than 1.47, at a confidence level of 90%. The $t$-test results are presented in Table 26. Values in italic in the table indicate cases presenting statistical difference.

Table 27. $t$-test results from SCB mortar specimens under different moisture cycles and under the same aging condition. Values in italic indicate statistical differences between moisture cycles.

<table>
<thead>
<tr>
<th>Result pairs</th>
<th>$P_{\text{max}}$</th>
<th>$G_f$</th>
<th>$CRI$</th>
<th>$FI$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SA</td>
<td>LA</td>
<td>SA</td>
<td>LA</td>
</tr>
<tr>
<td>Dry-C1</td>
<td>-7.64</td>
<td>2.36</td>
<td>-1.91</td>
<td>1.18</td>
</tr>
<tr>
<td>Dry-C2</td>
<td>-1.57</td>
<td>3.88</td>
<td>-1.05</td>
<td>3.14</td>
</tr>
<tr>
<td>Dry-C3</td>
<td>-3.36</td>
<td>7.30</td>
<td>-0.87</td>
<td>4.14</td>
</tr>
<tr>
<td>C1-C2</td>
<td>3.23</td>
<td>0.39</td>
<td>0.35</td>
<td>1.69</td>
</tr>
<tr>
<td>C1-C3</td>
<td>2.70</td>
<td>2.26</td>
<td>1.06</td>
<td>2.85</td>
</tr>
<tr>
<td>C2-C3</td>
<td>-1.00</td>
<td>2.49</td>
<td>0.36</td>
<td>1.87</td>
</tr>
</tbody>
</table>
Results in Table 27 show that, with some few exceptions that mainly correspond to $P_{\text{max}}$, the SA specimens did not present significant differences among the moisture conditioning cycles. On the contrary, with the exception of $FI$, the LA specimens presented significant differences in their fracture behavior under the different moisture vapor cycles. This proves that long-term aging increases the moisture sensitivity of the asphalt mortar. Also, a visual examination of the fracture faces of the SCB specimens showed that the damage in these cases was predominantly cohesive, since no evidence of stripping was observed.

These results are in agreement with the findings reported by Arámbula-Mercado et al. (2019). These authors aged PFC mixtures in the short- and long-term, and subjected them to IDT testing after one freeze-thaw cycle. The short-term aged specimens presented a reduction of 17% in the IDT strength after the freeze-thaw cycle, while the long-term aged specimens presented more than twice this reduction (i.e. 36%) after applying the same moisture conditioning, showing an increase in moisture susceptibility induced by aging.

The LA results show that $P_{\text{max}}$ decreased in an average of 11.1% after each cycle and $G_f$ decreased in an average of 17.2% after each cycle. At the end of C3, $G_f$ decreased 43% in comparison with the dry condition. Regarding $CRI$, the results did not show statistical differences after C1, but after C2 and C3 this parameter decreased an average of 21% in comparison to the dry condition. Overall, $G_f$ was the most susceptible parameter to the dry-wet-dry cycles.

The low fracture resistance of the PFC mortar in LA condition after the third dry-wet-dry cycle (C3) in comparison to the SA condition in dry state, might be a primary reason of the high raveling potential and durability issues of this type of open-graded mixtures. Indeed, the $CRI$ and $FI$ results of the LA mortar specimens subjected to C3 were 3.45 and 9.89 smaller than in the SA mortar samples in dry condition. During the service life of a pavement, PFC mixtures can be subjected to multiple moisture vapors and other moisture-related changes, including water infiltration, pore pressure due to saturated conditions and freeze thaw cycles, among others, which can further decrease their resistance to fracture. Moreover, if the mechanical deterioration due to the repetitive pass of vehicles (i.e. fatigue degradation) and more severe environmental conditions (i.e. freeze-thaw cycles or mortar erosion caused by liquid water flowing within the PFC microstructure) had been considered as part of this study, the results would have shown larger reductions in the fracture resistance of the mortar located at the stone-on-stone contact of the PFC mixtures.

**Effect of aging conditions**

As anticipated, the fracture parameters of the SA and LA specimens are dissimilar. For instance, LA specimens in dry condition showed a more brittle fracture behavior than SA specimens, as expected. Due to the high variability presented in Table 26, several $t$-tests were further conducted to verify if aging affects the fracture behavior of asphalt mortar under a constant moisture condition. Statistical $t$-test results are presented in Table 28. Values in bold and italic in the table indicate cases presenting statistical difference. Data in this table indicate that there are statistical differences between SA and LA conditions in almost all fracture parameters at all dry-wet-dry cycles.
Table 28. $t$-test results of SCB mortar specimens comparing the effect of the aging condition under different environmental conditions. Values in bold and italic indicate statistical differences between SA and LA conditions.

<table>
<thead>
<tr>
<th>Moisture condition</th>
<th>$P_{\text{max}}$</th>
<th>$G_f$</th>
<th>CRI</th>
<th>FI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>17.78</td>
<td>-1.13</td>
<td>-5.85</td>
<td>-4.69</td>
</tr>
<tr>
<td>C1</td>
<td>3.85</td>
<td>-5.60</td>
<td>-16.23</td>
<td>-8.04</td>
</tr>
<tr>
<td>C2</td>
<td>6.41</td>
<td>-4.60</td>
<td>-9.47</td>
<td>-5.45</td>
</tr>
<tr>
<td>C3</td>
<td>4.02</td>
<td>-6.64</td>
<td>-5.86</td>
<td>-2.90</td>
</tr>
</tbody>
</table>

The differences between LA and SA results for the dry, C1, C2 and C3 conditions were the following: i) 110.8, 37.7, 66.4 and 30.3% for $P_{\text{max}}$, ii) -17.6, -38.3, -47.8 and -56.0% for $G_f$, iii) -63.5, -54.7, -69.9 and -67.0% for CRI, and iv) -91.5, -86.1, -92.9 and -92.0% for FI. Although there are important differences in the fracture parameters between both aging conditions at all moisture states, the most sensitive was FI, with an average reduction of 90.6% between SA and LA under equivalent moisture conditions. In general, these results further demonstrate that the fracture resistance of PFC asphalt mortar decreases during its service life, and that the reduction is influenced by both aging and moisture cycles.

It is noteworthy that in the work conducted by Arámbula-Mercado et al. (2019), CRI and FI of a full PFC mixture decreased after applying a similar long-term aging procedure, under the same moisture condition. Specifically, the PFC evaluated in dry condition had a reduction of 46.0% in CRI and of 78.6% in FI due to long term aging. In the current study, the asphalt mortar in dry condition presented larger reductions in both parameters (63.5 and 91.5% for CRI and FI, respectively) but, in both cases, the reduction induced by aging in FI was larger than in CRI.

6.4. Conclusions and recommendations

This Chapter assessed the coupled effects of aging and moisture on the linear viscoelastic and fracture behavior of a PFC asphalt mortar. To achieve the goal, the PFC mortar specimens were aged at two different states (short and long term) and subjected to three different dry-wet-dry moisture vapor conditioning cycles. The PFC mortar specimens were then subjected to DMA and SCB testing. The following conclusions were obtained:

- Statistical $t$-tests indicated that the dry-wet-dry cycles did not significantly impact the dynamic shear modulus of the PFC mortar specimens in both aging states. On the contrary, under any moisture condition, the long-term aged PFC mortar specimens were between 35 and 53.0% stiffer than the short-term aged specimens, depending on the loading frequency.

- SCB test results showed that the effect of moisture vapor conditioning cycles had a minor effect on fracture behavior when the mortar was short-term aged. In contrast, long-term aged PFC mortar specimens were highly susceptible to moisture. LA specimens experienced higher peak and faster/earlier fracture than SA specimens. For example, $P_{\text{max}}$, $G_f$ and CRI in the long-term aged specimens decreased in an average of 11.1%, 17.2% and 6.9% after each moisture cycle.
Besides, after three dry-wet-dry cycles, $P_{max}$, $G_f$ and $CRI$ of the long-term aged specimens were 30, 43 and 21% smaller in comparison to the dry condition.

- The low fracture resistance of the PFC mortar in LA condition after the multiple dry-wet-dry cycles in comparison with the SA condition in dry state might be a primary reason to the high raveling potential and overall durability issues of PFC mixtures. For instance, $CRI$ and $FI$ in the LA mortar specimens subjected to three dry-we-dry cycles were 3.5 and 9.9 times less resistant to fracture than the SA mortar specimens in dry condition.

- The experimental results obtained in this study suggest that PFCs should not be evaluated in the laboratory only in a short-term aged condition when assessing the durability of the material. Other weather-related conditions should be included in the pre-testing of the material in order to capture its natural degradation.

The uniquely coupled climatic processes adopted in this Chapter are useful to assess the degradation of fracture resistance of a PFC mortar phase that can lead to stone-on-stone failure and, consequently, to the initiation of raveling. As it will be presented in Chapter VII, the rheological and fracture properties obtained from this work can be efficiently used as input parameters in computational micromechanical modeling. This computational modeling would permit to predict the behavior, performance and degradation of PFC mixtures in pavements under different operational-environmental conditions through the service life of the structures.
The main objective of this Chapter is to computationally evaluate the susceptibility to raveling of different PFC mixtures after several years of in-service conditions (i.e. > 6 years) using improved input parameters for material properties and enhanced methods to generate the geometry of PFC microstructures. To achieve this objective, this chapter brings together all the findings obtained in previous Chapters.

To start, this chapter collects the information from Chapter II, III and IV to design a FE pavement model that uses material properties that represent a PFC mixture subjected to in-service conditions. Moreover, this chapter uses the methodology proposed in Chapter V to randomly generate several 2D PFC microstructures that better represent the mechanical response of actual 3D PFC mixtures. Following the recommendation of Chapter III, the basic microstructural parameters (i.e. AV, CN and total contact length between aggregates) of these PFC geometries were obtained and the coarse aggregates from the PFC microstructures were assumed to be coated by asphalt mortar, as suggested in Chapter V. The material properties of the asphalt mortar are those obtained after subjecting this material to a long-term aged (LA) conditioning and three dry-wet-dry water vapor cycles (C3), as reported in Chapter VI.

Finally, zero thickness Cohesive Zone Elements (CZE) were included within the mortar-mortar contacts of the PFC, in order to count with a more realistic quantification of the initiation and progression of raveling during the service live of PFCs. The use of fracture mechanics through the incorporation of CZE to the model is considered superior than the initial approach followed in Chapter IV, which used the Raveling Index (R.I.) parameter.

The proposed FE pavement models were subjected to the pass of a rolling wheel at different load conditions. After this, the dissipated energy or the remaining fracture energy (R.E) of the CZE was obtained and analyzed. Next, the materials used in these new models, the modeling methodology and the main results are presented.

7.1. Materials and mixtures

7.1.1. PFC mixture

The mixture used in this chapter corresponds to the mixture used in Chapter VI (Table 23). This mixture is composed by Limestone (LS) aggregates, PG 76-22 binder modified with polymers (PMA), 0.4% of mineral fibers per total weight and 0.5% of liquid antistripping by binder weight. The optimum binder content (OBC) of the mixture is 6.5% by total weight (Arámbula-Mercado et al. 2016, FDOT 2018).
7.1.2. PFC mortar

The material properties of the asphalt mortar correspond to those obtained after subjecting the mortar to long-term aged conditions and three dry-wet-dry water vapor cycles (LA_C3) in Chapter VI. Figure 50 presents the dynamic shear modulus obtained through DMA testing at 25°C for the LA_C3 mortar.

![Figure 50. DMA test results at 25°C for the LA_C3 conditioned mortar.](image)

7.2. Model components and methodology

7.2.1. PFC microstructure geometry

In this case, the methodology proposed in Chapter V to randomly generate PFC microstructures using the MG generator and gravimetric methods through DE, was used to obtained five PFC layer geometries with the gradation described in Table 23. The input parameters for the morphology of the coarse aggregates and for the DE simulations are those determined in Sections 5.3 and 5.4. These microstructures were generated with a target AV of 20% and with dimensions of 25 cm wide and 4 cm thickness. The thin PFC layers were located in the FE models on top of a pavement structure to represent field conditions, as explained later in this Chapter.

Figure 51 illustrates the five random replicates of the PFC microstructures generated through gravimetric methods. All coarse aggregates are coated by a thin film of mortar of 0.060 mm. Each PFC microstructure was characterized in terms of the following parameters:

- AV content
- Number of aggregates.
- Total length of the contacts between aggregates.
- Coordination number (CN).

These characteristics for the microstructures are summarized in Table 29.
Due to the low variability (i.e. COV<10%) of the parameters presented in Table 29, it can be stated that the PFC microstructures generated could be considered replicates of the PFC microstructure, with an average AV content of 21.1%, 511 total coarse aggregates, a CN of 3.51 and a total length between contacts of 937.9 mm.

7.2.2. FE pavement model geometry

The FE pavement model in Abaqus® have three main components. The first one is the PFC layer that was analyzed at a microstructural level and which consists of coarse aggregates, mortar and air voids (a)
The second component is the pavement structure located below the PFC layer, which correspond to Pavement B, described Chapter III. This pavement is composed by HMA, UGB and subgrade layers; these layers were considered fully bonded. The third component is an equivalent PFC layer which is considered isotropic and homogeneous material. This equivalent PFC layers were located on top of the pavement next to the actual PFC microstructure and its purpose was to reduce the computational costs of the simulations while avoiding border effects.

To obtain a proper representation of the deflection bowl under the loading wheel, the thickness of the subgrade layer and the total horizontal length or total width of the pavement structure were defined based on recommendations provided in the literature (Duncan et al. 1968, Olidis and Hein 2004), and on observations of the results reported in Chapter III. Thus, it was concluded that a subgrade with a thickness of 100 cm and a horizontal length of the model of 425 cm were enough to avoid border effects. The final FE pavement model geometry is presented in Figure 52, from where it is also possible to observe the selected boundary conditions. Notice that the PFC microstructure is located in the upper central part of the pavement model with a length of 75 cm (See Figure 53). This is the only section of the pavement model where the mechanical impact of the pass of a wheel load is going to be analyzed.

Figure 52. FE pavement model geometry

Figure 53 exemplifies the global FE mesh used in the models, with a detail of the mesh used for the PFC microstructure and de HMA layer. The coarse aggregates and the mortar were discretized using 3-node linear elements (i.e. type CPE3R in Abaqus®). After conducting a sensitivity analysis, the aggregates were meshed using a seed of 15 mm and the mortar with a seed of 0.075 mm. The CZE were included as zero thickness elements between the CPE3R elements of the mortar-mortar contacts using the MIDAS-VT-Pre software developed by Zare-Rami and Kim (2019). These elements were included in Abaqus as 4-node two-dimensional cohesive element (i.e. COH2D4) with a longitude of 0.075 mm. As a result, the PFC layers had approximately 6.1x10^6 elements. All the other pavement layers were modeled with a total of 240,000 CPE4R elements using a 0.5 cm seed.
7.2.4. Constitutive response of the components of the model

PFC coarse aggregates and pavement layers
The coarse aggregates of the PFC mixtures were modeled as a linear elastic material with a modulus of 63,500 MPa and a Poisson’s ratio of 0.25 (Rami et al. 2017). All the layers of the pavement, except for the PFCs, were also modeled as linear elastic, isotropic and homogenous materials. The modulus of the PFC equivalent layers located next to the PFC microstructure were obtained after conducting computational dynamic test modulus on three different PFC layers of 15x10 cm, as explained in Chapter V and exemplified in Figure 33. The PFC equivalent layer was then modeled with an elastic modulus of 5,180 MPa and a Poisson’s ratio of 0.3.

As explained before, the pavement structure used in this case corresponds to the Pavement B described in Chapter III. All the layers were assumed linear elastics, isotropic and homogenous materials (Table 6).

Mechanical properties of the mortar materials in the PFC
The mortar in the PFC layers was modeled as a linear viscoelastic material. The rheological information from Figure 50 was transformed to the time domain to determine the Prony series of the relaxation modulus of the material at 25°C. The general formulation of the Prony Series is expressed by Eq. 4. The Prony series obtained from the LA_C3 mortar material is presented in Table 30.
Table 30. Prony series of the LA_C3 mortar at 25°C

<table>
<thead>
<tr>
<th></th>
<th>( p_j ) [s]</th>
<th>( G_j ) [Pa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.00x10^{-2}</td>
<td>8.62x10^8</td>
</tr>
<tr>
<td>2</td>
<td>1.00x10^{-1}</td>
<td>5.07x10^8</td>
</tr>
<tr>
<td>3</td>
<td>1.00</td>
<td>4.26x10^8</td>
</tr>
<tr>
<td>4</td>
<td>1.00x10^{1}</td>
<td>3.90x10^8</td>
</tr>
<tr>
<td>5</td>
<td>1.00x10^{2}</td>
<td>2.08x10^6</td>
</tr>
<tr>
<td>6</td>
<td>1.00x10^{3}</td>
<td>4.57x10^6</td>
</tr>
<tr>
<td>7</td>
<td>1.00x10^{4}</td>
<td>1.02x10^5</td>
</tr>
<tr>
<td>8</td>
<td>1.00x10^{5}</td>
<td>5.90x10^2</td>
</tr>
<tr>
<td>9</td>
<td>1.00x10^{6}</td>
<td>1.48x10^1</td>
</tr>
<tr>
<td>10</td>
<td>1.00x10^{7}</td>
<td>1.05</td>
</tr>
<tr>
<td>11</td>
<td>1.00x10^{8}</td>
<td>1.01x10^{-1}</td>
</tr>
</tbody>
</table>

\( G_0 \) [Pa] 2.65x10^9

\( E_0 \) [Mpa] 7406.1

Mechanical response of the CZE

The CZE used in this Chapter is based on the Cohesive Zone Modeling (CZM) methodology, which is a numerical technique commonly used to simulate crack initiation and evolution, and that was initially developed on the principles proposed by Barenblatt (1959) and Dugdale (1960). This technique has been commonly used for asphalt materials (Kim et al. 2006, 2015, Kim 2011) and assumes that there exists a process zone in front of the crack tip that supports certain level of traction, but that when the material reaches its maximum tensile strength (\( \sigma_{\text{max}} \)) in any of the classical modes of fracture (Mode I, II, or III), such resistance starts decreasing due to the stresses developed near the crack tip, up to the point where the material cannot support any additional load, the element physically disappears, and the crack propagates. As a consequence, the process zone advances, promoting the evolution of the cracking process (Xu and Needleman 1995). The energy required for this process to initiate corresponds to the critical fracture energy (\( G^c \)) of the material. In the models herein proposed, the CZE were located in the middle of the mortar-mortar contacts zones. In other words, similar to the models in Chapter IV, in these models is being assumed that raveling processes are mainly driven by the cohesive failure at the mortar contacts between coarse aggregates. Since adhesive degradation due to the presence of moisture is usually prevented in these mixtures with the use of antistripping agents, the prevalence of cohesive degradation as the main mechanisms driving raveling is a fair assumption.

The selected traction separation law for the cohesive elements assumes a linear elastic behavior before damage initiation, and an exponential decay function to represent the softening process occurring in the material until failure. Figure 54 presents the location of the cohesive elements and the selected traction separation law. From this figure, it can be notice that the part of the traction separation law before damage is complete elastic, in where \( K \) is the stiffness of the mortar. Once the material reaches the maximum tensile strength the damage evolution starts. Figure 55 illustrates the initiation and propagation of cracks within one PFC-FE model. A main result of this model is the degradation level of the CZE (SDEG) (i.e. a scalar between 0 and 1, where 0 means no degradation and 1 means that the element is completely
degraded) or the remaining energy (R.E.) which was initially proposed by Arámbula-Mercado et al. (2019) and is described by the following equation:

\[ R.E. = 1 - SDE \]  

Thus, when the SDEG reaches a value of 1, it means that the element dissipated all the energy (R.E =0), it is totally degraded, and it physically disappears from the model (i.e., crack initiation).

![Figure 54. Cohesive zones: (a) location, and (b) traction-separation law.](image)

![Figure 55. Crack propagation within the microstructure of a PFC mixture.](image)

Having reliable input parameters for the CZE based on the experimental results reported in Chapter VI is one of the most valuable contributions of this dissertation. In this case, the force Vs. LPD curve obtained from the LA_C3 mortar specimens subjected SCB tests were used to calibrate the CZM input parameters. The calibration consisted in representing the SCB test conducted in laboratory using FE modeling. This simulation was performed in Abaqus® following the mechanical response of the mortar
described in Table 30. In addition, a set of CZE was included in the central part of the computational SCB specimen, which was subjected to a loading rate of 0.5mm/min as described in Chapter VI. The input parameters of the CZE were changed until the force Vs. LPD curve of the simulation had an acceptable fit with the experimental results (Figure 56).

After several attempts, the CZE was calibrated using a traction separation law with a maximum tensile strength of 0.8 MPa and a fracture energy of 698.5 N-mm. The displacement at failure was defined at 1.5 mm and the exponential law parameter at 2. Additionally, these models assumed that the fracture properties for both modes of failure (mode I and mode II) were the same. Although this is a strong assumption, there are no currently available data to reliably estimate Mode II fracture parameters of this material. Nevertheless, this assumption is considered acceptable for the scope of this work.

![CZE calibration](image)

Figure 56. CZM calibration: (a) FE SCB test and (b) comparison of the force Vs. LPD curves obtained numerically and experimentally.

7.2.5. Loading conditions and cases of study

The FE models with the PFC microstructure located at the top center of the of the structure were subjected to different load conditions imposed by a rolling wheel passing on top of the pavement. As presented in Figure 57, the moving wheel is represented by two types of forces: i) the vertical force, $F_v$, which results from the axle weight, and ii) the friction force, $F_c$, which is usually modeled as a percentage of the vertical force.

The values of the vertical force were extracted from a load spectrum for a single-single axle load obtained from the Long-term Pavement Performance program (LTPP) database for typical high-volume roads in the U.S. (Selezneva et al. 2016). The pdf providing the best fit for these data was a normal distribution with $\mu=51.1$ kN and $\sigma=19$ kN. Therefore, to cover the load spectrum, two different vertical axle load cases for half an axle (i.e. one wheel) were evaluated: i) standard load ($\mu=25.57$ kN), and ii) extreme high load ($\mu+2\sigma=44.57$ kN). These values were used to define the friction force ($F_c$) and contact...
radius for the FE models. In terms of the friction force, two different cases were considered: i) an average rolling friction force, computed as 2.0% of the vertical force (Milne et al. 2004), and ii) a kinematic friction force representing a car braking on the pavement, computed as 78% of the vertical force (Tang et al. 2013). After assuming a contact pressure of 0.89 MPa, the radius of the wheel load was computed as 9.5 cm for the standard loading case and as 12.7 cm for the extreme high load case. Although PFCs are usually placed in high-speed roads, the impact of the wheel speed on the susceptibility of raveling is considered a load factor of interest. Thus, two vehicle speed cases were evaluated: i) low speed (50 km/h) and ii) high speed (100 km/h).

![Figure 57. Typical response of PFC FE pavement model subjected to a moving wheel (deformation scale factor: 100).](image)

Additionally, these simulations considered that the asphalt mortar is not only degraded by the presence of environmental conditions (aging and moisture damage) but also by the repetitive pass of traffic, or fatigue damage. The inclusion of this additional factor was simulated by reducing the fracture energy of the CZE in 25%. Therefore, two different cases of material degradation were considered: i) where the CZE have been degraded only due to environmental conditions (LA_C3 condition), and ii) where the CZE has been degraded not only by these environmental conditions but also by fatigue damage, in which case the CZE are assumed to have 75% of the total energy of the first case. The reduction of 25% on the fracture energy of the material for these simulations was arbitrarily selected; other less or more critical values could be considered as part of future analysis. When fatigue was considered, the maximum tensile strength ($\sigma_{\text{max}}$) of the traction separation law of the CZE was computed as 0.6 MPa, reducing the fracture energy ($G^c$) to 474.5 N-mm in comparison with the control condition which was computed with $\sigma_{\text{max}}=0.8$ MPa and $G^c=698.52$ N-mm. Notice that in this case, the fatigue damage was included in the traction separation law by changing only the $\sigma_{\text{max}}$, this decision was made based on the assumption that the mortar stiffness was not affected by fatigue damage. Future works can
include the analysis of the fatigue damage by changing also the stiffness of the mortar material. The input parameters of the CZE for the control and fatigue case are presented in Table 31.

Table 31. Input parameters of the CZE.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Control case</th>
<th>Fatigue damage case</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G^c$</td>
<td>100%</td>
<td>75%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>698.52</td>
<td>474.5</td>
<td>N-mm</td>
</tr>
<tr>
<td>$\sigma_{\text{max}}$</td>
<td>0.8</td>
<td>0.6</td>
<td>MPa</td>
</tr>
<tr>
<td>$K$</td>
<td>50,000</td>
<td>50,000</td>
<td>N/mm</td>
</tr>
</tbody>
</table>

In general, the analysis consisted in modifying the value of one parameter of the Control Case while keeping all other parameters constant. Table 32 summarizes the loading cases and parameters evaluated. In this table, values underlined correspond to the control case, while the values in italic correspond to those parameters changed from the Control Case. The Critical Case (G) correspond to the extreme conditions that would enhance the susceptibility to raveling of the PFCs. It considers a moving wheel at a low speed of 50 km/h, with an extreme high load and braking conditions, over a PFC that has been degraded by aging, moisture and fatigue damage. In particular, Figure 58 illustrates the von mises stress distribution within the third PFC microstructure replicate under the control case (A3) and the critical case (G3), when the wheel is on top of the microstructure. According to the stress path distribution observed in these two cases, it can be inferred that case G3 will be more susceptible to raveling damage.

Table 32. Description of the cases of study.

<table>
<thead>
<tr>
<th>Case ID</th>
<th>Case description</th>
<th>Vertical wheel load ($F_y$) [kN]</th>
<th>Frictional Wheel load ($F_x$) [kN]</th>
<th>Radius [cm]</th>
<th>Wheel Speed [km/h]</th>
<th>CZE Fracture Energy ($G^c$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Control</td>
<td>25.57</td>
<td>0.51</td>
<td>9.50</td>
<td>100</td>
<td>100%</td>
</tr>
<tr>
<td>B</td>
<td>Fatigue damage</td>
<td>25.57</td>
<td>0.51</td>
<td>9.50</td>
<td>100</td>
<td>75%</td>
</tr>
<tr>
<td>C</td>
<td>Low speed</td>
<td>25.57</td>
<td>0.51</td>
<td>9.50</td>
<td>50</td>
<td>100%</td>
</tr>
<tr>
<td>D</td>
<td>Braking</td>
<td>25.57</td>
<td>10.2</td>
<td>9.50</td>
<td>100</td>
<td>100%</td>
</tr>
<tr>
<td>E</td>
<td>Extreme high load</td>
<td>44.57</td>
<td>0.89</td>
<td>12.70</td>
<td>100</td>
<td>100%</td>
</tr>
<tr>
<td>F</td>
<td>Extreme high load and braking</td>
<td>44.57</td>
<td>17.83</td>
<td>12.70</td>
<td>100</td>
<td>100%</td>
</tr>
<tr>
<td>G</td>
<td>Critical case</td>
<td>44.57</td>
<td>17.83</td>
<td>12.70</td>
<td>50</td>
<td>75%</td>
</tr>
</tbody>
</table>
7.4. Analysis of results

After conducting a total of 35 simulations (i.e. five PFC microstructures replicates per each of the seven cases of study), the R.E. of all the CZE located at the mortar-mortar contacts was computed. Since an average of 20,000 CZE were registered in the mortar-mortar contacts per model, the R.E. data –per study case and model– was adjusted to a Lognormal pdf, which presented the best fit in all cases, with a confidence level of 95%. Figure 59 illustrates the pdfs obtained from this analysis. Data series in this figure include the case number and the replicate; thus, for example, data series ‘B5’ refers to the results of the fifth replicate of case B (Table 32). Also, the three vertical dotted lines represent where the values of the CZE registered a remaining energy less than 30, 20 and 10%; that is, the condition where the SDEG reached 70, 80 and 90% (i.e. higher propensity to initiate fracture).

From Figure 59 it can be observed that among all cases, the critical case (G) is the one with the overall smaller R.E, as expected. This case is followed by Case C, in which a standard rolling wheel pass over the pavement at a slower speed of 50 km/h. Cases C and G were the only two cases that considered the influence of low speed on raveling susceptibility, which confirms that this parameter is one of the most influential conditions in impacting raveling susceptibility of PFCs, as already proved in Chapter IV.
Considering the proximity between the R.E pdf curves of the study cases A, B, D, E and F, the percentage of elements with R.E. values less than 30% was computed and used to quantify the effect of each parameter on the susceptibility to raveling of the PFCs. This information is summarized in Table 33.

![Figure 59](image.png)

Figure 59. R.E. of each PFC microstructure and case of study after the pass of one moving wheel.

<table>
<thead>
<tr>
<th>Study Case</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PFC replicate</strong></td>
<td><strong>Control</strong></td>
<td><strong>Fatigue damage</strong></td>
<td><strong>Low speed</strong></td>
<td><strong>Braking</strong></td>
<td><strong>Extreme high load</strong></td>
<td><strong>Extreme high load and braking</strong></td>
<td><strong>Critical case</strong></td>
</tr>
<tr>
<td>1</td>
<td>10.92%</td>
<td>13.39%</td>
<td>50.25%</td>
<td>12.90%</td>
<td>19.71%</td>
<td>20.77%</td>
<td>86.76%</td>
</tr>
<tr>
<td>2</td>
<td>10.38%</td>
<td>14.43%</td>
<td>46.58%</td>
<td>11.27%</td>
<td>15.79%</td>
<td>19.54%</td>
<td>88.45%</td>
</tr>
<tr>
<td>3</td>
<td>10.46%</td>
<td>14.90%</td>
<td>47.85%</td>
<td>10.65%</td>
<td>19.25%</td>
<td>19.90%</td>
<td>82.45%</td>
</tr>
<tr>
<td>4</td>
<td>8.17%</td>
<td>11.34%</td>
<td>43.27%</td>
<td>11.68%</td>
<td>18.12%</td>
<td>22.30%</td>
<td>92.75%</td>
</tr>
<tr>
<td>5</td>
<td>10.28%</td>
<td>15.78%</td>
<td>42.82%</td>
<td>18.76%</td>
<td>20.03%</td>
<td>20.54%</td>
<td>98.08%</td>
</tr>
<tr>
<td><strong>Avg</strong></td>
<td>10.04%</td>
<td>13.97%</td>
<td>46.15%</td>
<td>13.05%</td>
<td>18.58%</td>
<td>20.61%</td>
<td>89.70%</td>
</tr>
</tbody>
</table>
Table 33 shows that after the pass of one moving wheel, the control case (A) presented an average of 10.0% of total CZE at the contacts with R.E.<30%. This value reveals the susceptibility to raveling of the PFCs due to the environmental conditions (i.e. aging and moisture) at which these mixtures are subjected during their service life. Although this value is not significative, it is considered relevant since it shows the raveling susceptibility of a PFC degraded by environmental conditions after just one-wheel pass.

Case B, which also includes the degradation of the properties of the asphalt mortar due to fatigue damage by reducing the fracture properties of the CZE, presents an average of 13.97% of the total CZE with R.E.<30%. This value is 3.97% higher than the control case and it corroborates that the inclusion of fatigue damage is relevant when considering the susceptibility to raveling of PCFs. It is worth noting that this result is highly dependent on the arbitrary decision of reducing the CZE energy in 25%; thus, further analysis on this topic should is required.

As previously mentioned, Case C is one of the cases with the highest influence in promoting raveling within the PFC mixtures. In this case, after one wheel pass at a speed of 50km/h, an average of 46.2% of the CZE remained with 30% or less of its energy. This result is in good agreement with the findings of Arámbula-Mercado et al. (2016) and the specifications of several DOTs that have restricted low speed circulation over pavements with PFC mixtures (e.g. FDOT 2018).

Case D, which includes vehicle braking conditions, showed that an average of 13.0% of the total CZE have R.E.<30%. This result is only 3.0% higher than the control case but exemplifies that wheel braking enhances raveling susceptibility. Considering the results from Case C, it is believed that if the braking would have not been only modeled as 78% $F_y$, but have also included a progressive speed reduction, the R.E. would have been smaller.

Case E and F assess the influence of extreme high loading (Case E) with vehicle braking conditions (Case F). The results show that an average of 18.58% and 20.61% of the total CZE presented a R.E.<30% for case E and F, respectively. These results almost double those of the control Case A, demonstrating the impact of extremely high loads on PFC degradation.

Finally, Case G, which collects all critical conditions that promote raveling in PFCs, presented an average of 89.7% CZE with R.E. < 30%. This result suggests that the damage caused by the different parameters here evaluate can be cumulative, and that when all these factors are simultaneously present in the pavement, only one critical moving wheel pass is required to boost raveling in PFC mixtures.

Since raveling is a phenomenon defined as the loss of aggregates from the PFC surface, it was concluded that a R.E. ≤ 30% is enough to quantify the potential to raveling initiation of the PFCs but not enough to state that the raveling has actually initiated. For this reason, the evaluation of the percentage of CZE with a R.E. lower than 20 and 10% was also conducted, and the results are presented in Table 34.
The results show that only Cases C and G, presented elements with a R.E. lower than 20 and 10%. This means that cases A, B, D, E and F enhance the susceptibility to raveling of these mixtures but are not enough to initiate fracture processes. This was expected since only one moving wheel pass was considered in the simulations.

In contrast, Case C and G presented an average of 31.78 and 73.6% CZE with R.E. <20%; and 10.17 and 32.21% CZE with R.E. <10%. This means that after one wheel pass, 10.17 and 32.21% CZE with R.E. <10% will start getting fractured. Considering the CZE length, the CN and the total contact length of the PFCs (Table 29), it can be concluded that after pass of the wheel with the load conditions of case C and G, the PFC mixtures will present an approximately average loss of 28 and 91 aggregate contacts, respectively. Considering the average contacts per aggregate in these PFC microstructures, this is equivalent to an average loss of 8 and 26 aggregates, respectively (i.e. 1.6 and 5.1% aggregate loss of the whole PFC mixtures).

7.5. Conclusions and recommendations

The main objective of this Chapter was to evaluate the susceptibility to raveling of different PFC mixtures after several years of in-service conditions through computational mechanics. To achieve this goal, this chapter integrates the information from Chapter III, IV, V and VI, and proposes a FE pavement model with a PFC surface layer that better represents the mechanical behavior of an actual PFC layer and counts with realistic and reliable material properties. In addition, zero thickness CZE were included at the mortar-mortar contacts of the PFC to make a more realistic quantification of the potential initiation and progression of raveling. The proposed FE pavement models were subjected to the pass of one moving wheel at different loading and material properties conditions. After this, the remaining fracture energy or R.E of the CZE was subtracted and analyzed. The main results from these models are:
• The control case (A) that includes average loading and material degradation conditions, presented an average of 10.0% of CZE with R.E.<30%. This 10% of CZE with R.E.<30% reveal the susceptibility to raveling of the PFCs due to the environmental conditions (i.e. aging and moisture) at which these mixtures are subjected during their service life.

• The fatigue damage case (B) resulted in an average of 13.97% of the total CZE with R.E.<30%. This value is 3.97% higher than the control case and gives a first insight of the relevance of including fatigue damage as a factor that increases the susceptibility to raveling.

• Low speed proved to be one of the most influential parameters on raveling susceptibility, since Cases C and G—the only two cases that included this parameter—were the ones with the lowest R.E. from all cases.

• Case D, that considered wheel braking, presented an average of 13.0% of the total CZE with R.E.<30%. This result is 3.0% higher than the control case and exemplifies a slight increase in PFC raveling susceptibility due to this factor. It is believed that if the vehicle braking condition would also include a progressive speed reduction, the R.E. would have been much lower.

• The negative influence of extreme high load in PFCs raveling susceptibility was verified in Cases E and F, where the results showed an average of the total CZE with an R.E.<30% of 18.58% and 20.61%, respectively. These results almost doubled those from the control case.

• Case G, which combines the most unfavorable conditions, presented an average CZE with R.E. < 30% of 89.7%. This result suggests that the damage caused by the different parameters here evaluated is cumulative. This also shows that one single moving wheel pass is enough to degrade the PFC mixture and promote raveling when these factors are simultaneous presented during the operation of the pavement structure.

• After one wheel pass with the loading conditions specified in Case C and G, 10.17 and 32.21% of the total CZE presented R.E. <10%. This translates in the fact that the PFC mixtures might loss between 28 and 91 aggregates contacts and between 8 and 26 aggregate particles, respectively (i.e. 1.6 and 5.1% aggregate loss of the whole PFC mixtures).

In general, the proposed methodology proved to be an effective tool to evaluate raveling susceptibility of PFC mixtures. This tool can be further used to assess the mechanical response of different design approaches of PFC mixtures, to quantify the structural reliability of these structures using multiple 2D random microstructures and to provide information about the progressive degradation of these mixtures that could be used as part of life cycle cost analysis of these mixtures.
8.1. Summary

This dissertation presents the principles and results of several computational micromechanics models and an accompanying experimental plan to evaluate raveling in PFC mixtures. Overall, this research effort is considered a novel contribution to the current state of knowledge related to the internal strength of PFC mixtures and the degradation mechanisms causing raveling.

Initially, PFC microstructure geometries were obtained from X-ray CT images. These 2D images were used as input components in FE computational mechanical models in Abaqus®, which were used to obtain a first quantification of the internal strength and overall structural contribution of these layers (Chapter III), as well as the degradation processes within the internal stone-on-stone contact network of PFCs that might cause raveling (Chapter IV).

The results obtained in these initial computational approaches provided relevant information about the behavior of these mixtures but they also evidenced the need of having 2D PFC microstructure geometries that could better represent the mechanical behavior of actual 3D PFCs, as well as the need to count with more reliable data on the viscoelastic and fracture properties of the asphalt mortar located at the stone-on-stone contacts. Therefore, a novel methodology to randomly generate 2D PFC microstructures that represent the internal mechanical behavior of actual PFC mixture was proposed (Chapter V). In addition, an experimental plan was conducted in order to obtain the viscoelastic and fracture properties of the asphalt mortar material located at the stone-on-stone contacts of PFCs affected by different environmental conditions (Chapter VI).

The microstructure geometries obtained using the proposed methodology and the data obtained from the experimental tests were used in Chapter VII as input parameters in new computational models. The objective of these models was to assess the mechanical response of PFC mixtures under realistic in-service conditions. The results from these new simulations permitted to identify the field road operational conditions and PFC properties that promote or prevent raveling. This information could be efficiently used to improve the long-term durability of PFC mixtures. Next, some of the main conclusions obtained from this study are presented.

8.2. General conclusions

The results obtained in the initial computational FE models demonstrated that the PFC layers contribute to the structural capacity of the pavements evaluated but that this structural contribution is strongly related with different geometrical and microstructural characteristics of the PFC microstructures. In fact, the PFC thickness, the number of aggregates and the number of contacts – without considering floating aggregates– were observed to be critical in impacting the structural contribution of these layers.
Moreover, the numerical results conducted showed that the use of the 2, 4 and 6 cm PFC layers with 20 and 25% AV provided a structural contribution equivalent to 0.5–3.0 cm of the HMA layer or to 1.0–9.0 cm of the UGB layer. It is important to stress that if the thickness of any layer of a flexible pavement is reduced due to the placement of a PFC layer, it is necessary to guarantee that the service life of the PFC is similar to that of regular HMA layers and/or that the project has efficient management strategies to maintain or replace the PFC layer when necessary.

The initial computational models of raveling susceptibility in flexible pavements allowed to evaluate of the relative role of different parameters in promoting this degradation process in PFC mixtures. It was found that raveling is mainly a mechanical surface-contact problem associated with Mode I of failure and that the mixture design plays a main role in the promotion of this damage in PFC materials. When increasing the AV content, the quality of the skeleton of the mixture negatively impacted the mechanical response of the mastic-mastic contact elements and, consequently, there was an increase in the susceptibility of the mixtures to raveling. This suggest that it may be convenient to limit the maximum AV content of the mixture in the field, probably by making it dependent on PFC layer thickness or on certain characteristic property related to its gradation.

Besides, it was proven that PFC mixtures are more prone to develop raveling under low speed traffic conditions and in zones where the vehicles are forced to brake frequently, which justifies current recommendations provided by several highway agencies regarding the use of these mixtures only in high-speed roads.

The initial computation models also evidenced the need for having PFC microstructures that better represent the mechanical behavior of 3D PFC mixtures. Considering this, a novel methodology to randomly generate 2D PFC microstructures was proposed. This method combines the MG software to generate random coarse-aggregate particles with controlled morphological properties, and the LMGC90 DE software, to generate the coarse aggregate fraction of 2D PFC microstructures using gravimetric techniques. The proposed technique was calibrated by conducting computational $|E^*|$ tests and comparing the results with experimental data. The results demonstrate that this technique is able to generate random 2D PFC microstructures that represent the stiffness of actual 3D PFC mixtures.

From the initial computational models and the literature review, it was also evident the need of counting with material properties that characterize the viscoelastic and fracture behavior of the material between the stone-on-stone contacts under different environmental and loading conditions. For this reason, an experimental plan to assess the coupled effects of aging and moisture on the linear viscoelastic and fracture behavior of a PFC asphalt mortar was developed. The PFC mortar specimens were aged at two different states (short and long term) and subjected to three different dry-wet-dry moisture vapor conditioning cycles. The PFC mortar specimens were then subjected to DMA and SCB testing. It was found that the dry-wet-dry cycles did not significantly impact the dynamic shear modulus of the PFC mortar specimens in both aging states. In contrast, under any moisture condition, the long-term aged PFC mortar specimens were between 35 and 53.0% stiffer than the short-term aged specimens.

SCB test results showed that the effect of moisture vapor conditioning cycles had a minor effect on fracture behavior when the mortar was short-term aged. In contrast, long-term aged PFC mortar specimens were highly susceptible to moisture. LA specimens experienced higher peak and faster/earlier
fracture than SA specimens. Besides, after three dry-wet-dry cycles, $P_{\text{max}}$, $G_f$ and $CRI$ of the long-term aged specimens were 30, 43 and 21% smaller in comparison to the dry condition.

In addition, the low fracture resistance of the PFC mortar in the LA condition after the multiple dry-wet-dry cycles in comparison with the SA condition in dry state was observed to be a primary reason of the high raveling potential and overall low durability of PFC mixtures. For instance, CRI and FI in the LA mortar specimens subjected to three dry-we-dry cycles were 3.5 and 9.9 times less resistant to fracture than the SA mortar specimens in dry condition.

Considering the previous findings, a new set of FE pavement models—with PFC surfaces that were randomly generated using the proposed gravimetric technique and reliable material properties—were evaluated. These models assess the susceptibility to raveling of different PFC mixtures after several years of in-service conditions and used zero thickness CZE at the mortar-mortar contacts to have a more realistic quantification of the potential to raveling. The proposed FE pavement models were subjected to the pass of one moving wheel at different loading and material properties conditions. After this, the remaining fracture energy (R.E.) of the CZE was subtracted and analyzed.

These simulations showed that the control case (A), which includes average loading and material degradation conditions, presented an average of 10.0% of CZE in the aggregate contacts with R.E.<30%. This percentage reveals the susceptibility to raveling of the PFCs due to the environmental conditions (i.e. aging and moisture) at which these mixtures are subjected during their service life.

Besides, the fatigue damage case (B) resulted in an average of 13.97% of the total CZE with R.E.<30%. This value is 3.97% higher than the control case and shows the relevance of including fatigue damage as a factor that increase the susceptibility to raveling. Once again, low speed proved to be critical in inducing raveling susceptibility (Case C). Case D, which considered wheel braking, presented an average of 13.0% of the total CZE with R.E.<30%. This result is 3.0% higher than the control case and exemplifies a slight increase in PFC raveling susceptibility due to braking conditions. In addition, the negative influence of extreme high load in PFCs raveling susceptibility was verified in Cases E and F, where the results almost doubled those from the control case.

Case G, which combines the most unfavorable conditions, presented an average CZE with R.E. < 30% of 89.7%. This result suggests that the damage caused by the different parameters here evaluate is cumulative and that one single wheel pass could be enough to degrade the PFC mixture and promote raveling when these unfavorable factors are present. In fact, under these conditions, one wheel pass can generate a loss of 8 to 26 aggregate particles from the PFC section analyzed (i.e. 1.6 and 5.1% aggregate loss of the whole PFC mixtures).

Finally, the new methodologies proposed in this dissertation permitted to evaluate raveling susceptibility of PFC mixtures and their structural contribution. These tools can be further used to assess the mechanical response of different design approaches of PFC mixtures among other applications, as explain in next section.
8.3. Recommendations

Considering the findings made in this work, a list of practical recommendations to reduce the susceptibility of PFC mixture to develop raveling is presented:

- The mixture design of the PFC plays a fundamental role in the development of raveling, especial attention should be put to the gradation of the PFC mixture.
- The long-term effect of aging and moisture damage on PFC mixtures should be part of the mixture design methodology.
- The author recommends the use of limestone aggregates and high polymer modified asphalts with anti-stripping agents in the PFC mixture design.
- Particular attention should be put on the construction methods since any change in the BC and AV content will lead to higher chances to developing raveling.
- High vehicle loads and zones where vehicles usually brake or transit at low speed (i.e. < 60 km/h) should be avoided in order to prevent raveling in PFC mixtures.

8.4. Future work

Based on the findings obtained in this dissertation and on the new methodology proposed to randomly generate 2D PFC microstructures, the following research topics have been identified as relevant to be considered in future works:

- The methodology proposed to randomly generate 2D PFC microstructures can be used to assess the mechanical response of different design approaches for PFC mixtures, after considering the uncertainty induced by the heterogeneity of the PFC microstructures (i.e. computational analysis of multiple randomly generated PFC microstructures as part of reliability-based design approaches).
- The proposed methodology can also be used to assess the structural contribution of different PFC mixtures designs to different pavement structures, making a further analysis of the influence of the geometrical parameters over their mechanical behavior, and developing a guideline that includes the Voids in Coarse Aggregates (VCA) volumetric parameter and other desired properties that a PFC mixture should account for.
- The methodology can further be used to generate multiple random microstructures of any PFC mixture to conduct computational probabilistic and statistical studies of the functionality, durability and mechanical response of PFC mixtures under different field conditions. This, in turn, would provide information about the performance reliability of these PFC mixtures.
- The proposed methodology can be used to study the force distribution within the contact network of the coarse aggregates of the PFC mixtures and determine the optimal gradation (grain size) that brings better mechanical responses on PFC mixtures. The evaluation of isotropic and anisotropic system can be developed.
- An experimental plan can be designed to obtain the fracture properties between the aggregate-mortar interface (i.e. adhesive damage) under different environmental conditions. The fracture
properties obtained from this experimental plan can be included in the FE models to evaluate raveling susceptibility due to both cohesive and adhesive damage.

- An experimental plan can be designed to quantify the material properties degradation induced by fatigue degradation. The results of this experimental plan can be combined and included in the computational models to evaluate raveling. These results could be used as part of life cycle cost analysis of PFC mixtures.
- Further analysis can be done to analyze the influence of the filler content within the mechanical behavior and degradation mechanisms in PFC mixtures.
- The loading conditions of the models proposed to evaluate raveling susceptibility can be improved by including more complex tire-pavement interactions, multiple loading cycles and Multiphysics simulations.
- An experimental plan should be developed in order to validate the results of the FE models proposed in Chapter VII.
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