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Comparison of asphalt mixtures containing polymeric compounds and polymer-modified bitumen based on the VECD theory

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Abstract

The ‘dry’ method that can be used to produce modified asphalt mixtures is a less expensive, less energy-consuming, and faster process than the well-established ‘wet’ method. Moreover, the dry method allows the incorporation of hard plastics, even those plastics obtained from waste products. Although researchers agree that the dry method can improve the stiffness and rutting resistance (i.e., high-temperature performance) of asphalt mixtures, they have conflicting opinions regarding mixture fatigue and cracking resistance. In this regard, this paper aims to evaluate, through the application of viscoelastic continuum damage theory, the fatigue behavior of two compound asphalt mixtures that have been modified using the dry method. One of the studied compounds is composed of plastomeric polymer and the other is composed of waste plastic with the addition of graphene. A reference mixture containing polymer-modified bitumen (representing the wet modification method) was used for comparison. The experimental program involved dynamic modulus tests and uniaxial cyclic fatigue tests of laboratory-compacted specimens and cores extracted from full-scale field test sections. The test results from the laboratory-compacted specimens and field cores were input to FlexPAVETM for pavement performance simulations. Under the same volumetric conditions, the three dense-graded mixtures broadly had comparable stiffness and fatigue resistance values at the material level. However, in the pavement-level simulations, the reference mixture exhibited much less damage after 30 years of service than the compound mixtures. Concerning the field test track, the air void contents of the mixtures varied due to workability issues related to the presence of the compounds. Optimum performance was obtained for asphalt layers that could be characterized by an intermediate stiffness level that ensured an adequate load distribution without negative consequences for the mixture’s fatigue resistance and thermal resistance.

Keywords: Polymer modification, waste plastic, viscoelastic continuum damage (VECD), field test track, FlexPAVETM

1. Introduction

Within the framework of pavement engineering, researchers are currently studying new processes and materials that could improve environmental sustainability and lower production costs and thus improve affordability for industries and asphalt plants. Polymer materials, such as styrene-butadiene-styrene (SBS) and ethylene vinyl acetate (EVA), are widely used as bitumen modifiers because they enhance the mixture's rutting and fatigue resistance and mitigate its susceptibility to temperature variations [1][2][3][4]. Such polymers usually are *ad hoc* engineered and added to the asphalt binder using the 'wet' modification method, which consists of blending the polymer with the binder first at a high temperature and then mixing the modified bitumen with the aggregate [5]. Because the wet method aims to ensure complete dispersion and chemical modification, the polymers used for this purpose should have a melting point that is lower than the production temperature, and special equipment is needed during transport and storage to avoid phase separation [5][6][7].

An alternative way to modify asphalt mixtures is the 'dry' method. In this case, the polymer is added and mixed with hot aggregate directly in the asphalt plant prior to the addition of the binder [5][6]. Therefore, this modification method is suitable also for polymers, such as hard plastics, that have a melting point that is comparable to the production temperature. This dry method reduces costs and energy consumption because there is no risk of binder phase separation [6][8][9]. However, the dry method implies less control of the binder/mixture properties.

In addition, the dry method enhances pavement sustainability because the modification process can employ polymeric compounds obtained from waste plastics. In this way, the huge amount of plastic currently sent to incineration facilities or landfills can be reduced, thereby mitigating serious negative impacts on the quality of air, water, and the environment [6][10]. Waste plastics are complex materials that typically are composed of many different grades of plastic, which can complicate their homogenization within the asphalt mixture and compatibility with bitumen [6][11]. The most common plastics found in municipal solid waste are polyethylene and polypropylene [5][11], both of which are known to increase the stiffness and rutting resistance of asphalt mixtures and reduce their susceptibility to thermal loading [12][13][14][15][16]. However, to date, researchers have not reached consensus regarding the impact of polymeric compounds on the fatigue and cracking resistance of asphalt mixtures [5], mainly because the resistance to fatigue and cracking could be diminished by the increase in stiffness caused by the addition of polymer/plastic [8]. As fatigue cracking is one of the major distress types for asphalt pavements over the course of their service life, it needs to be investigated carefully in the context of asphalt mixture modification.

An advanced and reliable method to predict the fatigue performance of asphalt mixtures is based on viscoelastic continuum damage (VECD) theory. VECD theory is built on three fundamental concepts: continuum damage mechanics, the elastic-viscoelastic correspondence principle, and the time-temperature superposition principle with growing damage. Continuum damage mechanics is based on the work potential theory [17] for modeling the effects of microcracks via internal state variables, which allows the material to be considered as a continuous and homogeneous body [18]. The elastic-viscoelastic correspondence principle is based on the pseudo-strain concept, which allows to reduce the viscoelastic behavior of the material into the corresponding elastic behavior [17][19]. The time-temperature superposition principle with growing damage (and viscoplastic

strain) takes into account the combined effects of time/rate and temperature, even outside the viscoelastic domain of the material's behavior [20]. The Simplified VECD (S-VECD) model is an approach that allows the fatigue performance of a mixture to be characterized based on the results of uniaxial cyclic fatigue tests and dynamic modulus tests [21]. The mechanistic-based predictive capability of the S-VECD model distinguishes it from conventional fatigue modeling approaches where individual index parameters are empirically defined [22]. Moreover, the S-VECD model properties obtained from laboratory tests can be used directly as input data for pavement performance simulations carried out using FlexPAVETM software, which allows the prediction of the long-term fatigue performance of the pavement [23][24].

Given this background, the aim of this study is to evaluate the fatigue performance of three mixtures based on the S-VECD modeling approach. A reference mixture (coded as H) that contains SBS polymer-modified bitumen represents the wet modification method and two mixtures that contain compounds (coded as GC and PC, respectively) represent the dry modification method. The GC compound was made of recycled plastic with graphene and the PC compound was made of plastomeric polymers. An open-graded mixture with SBS polymer-modified bitumen (coded as OG), typically used for wearing layers, also was studied. First, the laboratory-compacted mixture specimens were tested. Then, specimens from a full-scale field test track were investigated to evaluate the effects of compaction in the field. Finally, pavement simulations were carried out using FlexPAVETM under three scenarios in which the material properties of the asphalt layers, which were set as inputs, were varied. In this regard, the OG mixture was studied mainly to reproduce the pavement structure of the field test track in the FlexPAVETM simulations. Note that the S-VECD modeling approach has never been used before to study the fatigue behavior of asphalt mixtures modified with plastics via the dry method.

This study is part of a larger project in which, first, D'Angelo et al. assessed the rheological behavior and adhesion properties of the same modified binders in this study [25]. Then, Cardone et al. investigated the effects of modification using the same two compound mixtures (GC and PC) at the mixture level [26]. Specifically, Cardone et al. evaluated the stiffness, rutting resistance, and fatigue resistance of the same mixtures tested in this study using a traditional performance-related approach based on conventional tests (i.e., indirect tensile tests and triaxial cyclic tests) and performed Falling Weight Deflectometer (FWD) tests at the field test track [26]. The findings of these previous studies are recalled in the following sections to enrich the discussion of the results obtained in this investigation.

2. Experimental program

2.1. Materials

The dense-graded asphalt mixtures investigated in this study, H, GC, and PC, are the same as the mixtures investigated by Cardone et al. [26]. The aggregate gradation, reported in **Figure 1**, is the same for the three mixtures and is typical for base layers placed in Italian motorways. The mixtures contain 30% reclaimed asphalt pavement (RAP) by aggregate weight and their nominal maximum aggregate size (NMAS) is 20 mm. The total binder content (virgin bitumen plus bitumen from RAP) is 4.3% by aggregate weight for all the mixtures. The H mixture, which currently serves as the reference mixture for Italian motorway pavements, was produced with SBS polymer-modified bitumen with 3.8% polymer by binder weight. The GC and PC mixtures were produced using a neat 50/70 penetration grade bitumen and then adding the two compounds via the dry method, with a

dosage of 5.2% by total binder weight. The compounds, supplied in the form of hard pellets, consisted of hard recycled plastic and graphene for the GC mixture and a blend of plastomeric polymers for the PC mixture. The chemical characterization of the compounds carried out by D'Angelo et al. [25] showed that both compounds were made mostly of polyethylene and polypropylene, with the small addition of graphene for the GC mixture. **Figure 1** also reports the aggregate gradation of the OG mixture with the NMAS of 14 mm. The OG mixture binder is a SBS polymer-modified bitumen (3.8% polymer by binder weight) with 5.1% binder content by aggregate weight. A dosage equal to 0.3% by aggregate weight of cellulose and glass fiber was added to prevent drain-down issues.

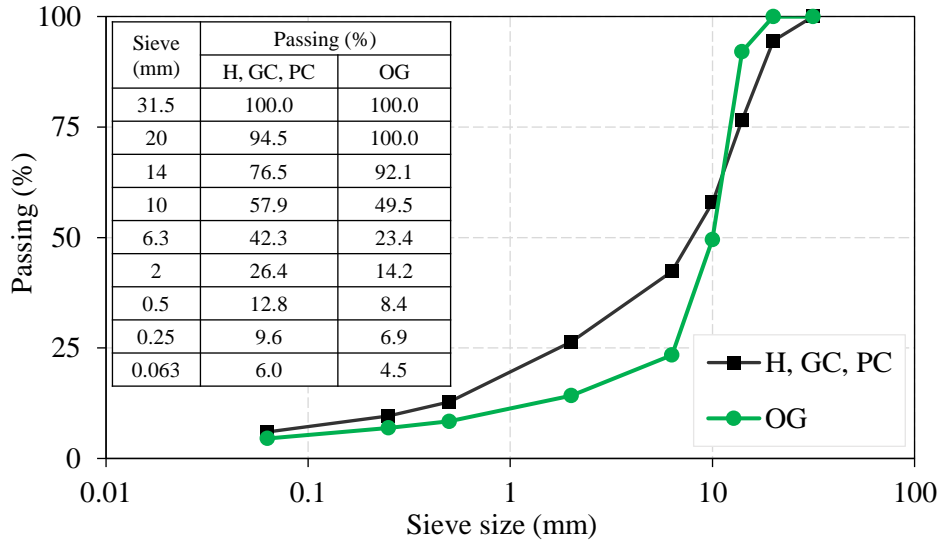


Figure 1. Aggregate gradations of investigated mixtures.

2.2. Field test track

A 600-m full-scale field test track was constructed in September 2020 as part of the A12 Italian motorway in the city of Fiumicino and consists of milled and reconstructed asphalt layers [26]. The field test track is composed of three 200-m sections, each characterized by the same pavement structure, as follows:

- An open-graded wearing layer with nominal thickness of 4 cm, constructed with the OG mixture for all three sections.
- A dense-graded layer with a total nominal thickness of 25 cm, constructed with a different material for each section, i.e., the reference mixture H (representing the wet modification method), and the GC and PC mixtures modified via the dry method. The construction was carried out via the consecutive compaction of two layers: a 15-cm lower layer on the foundation, hereafter called *Field_inf*, and a 10-cm upper layer, hereafter called *Field_sup*.
- An unbound foundation layer with nominal thickness of 35 cm, placed on the subgrade.

2.3. Testing program and procedures

2.3.1. Testing program

Figure 2 presents the testing program, which was divided into three stages. In the first stage, laboratory-compacted specimens of all the study mixtures were investigated. The specimens were produced using the same mixtures that were used for the field test track. The loose mixtures were produced at 170°C at the asphalt plant and then sampled and immediately compacted at 160°C by a gyratory compactor in accordance with EN 12697-31 [27]. As prescribed by AASHTO R 83 [28] and AASHTO PP 99 [29], the gyratory-compacted samples had a diameter of 150 mm and height of 180 mm. Then, one large (100-mm diameter, 150-mm height) test specimen or four small (38-mm diameter, 110-mm height) test specimens were vertically extracted from the inner portion of the gyratory-compacted sample. Due to their high air void contents, only the large specimens were considered for the OG mixture. First, given the peculiarities of the investigated materials, proficiency checks were carried out by the North Carolina State University (NCSU, USA) research team and the Università Politecnica delle Marche (UNIVPM, Italy) research team with the aim of comparing the results for the same mixtures, H and GC, obtained from the two institutions' individual laboratories. Then, all the remaining tests were carried out at the UNIVPM laboratory. During the second stage of the testing program, the dense-graded cores extracted from the field test track (*Field_sup* and *Field_inf*) were studied to assess the effects of *in situ* compaction. A set of 200-mm diameter cores was extracted from each field test section in February 2021. After cutting the layers at the interface, small specimens were horizontally extracted from the *Field_sup* and *Field_inf* layers [29]. The open-graded wearing layer was not included in the investigation due to its limited thickness. During the first and second stages of the testing program, the fatigue behavior at the material level was evaluated using the S-VECD model approach by carrying out dynamic modulus tests and cyclic fatigue tests. Finally, in the third stage, the fatigue performance of the mixtures was investigated at the structural level by performing FlexPAVETM pavement simulations based on the laboratory test results.

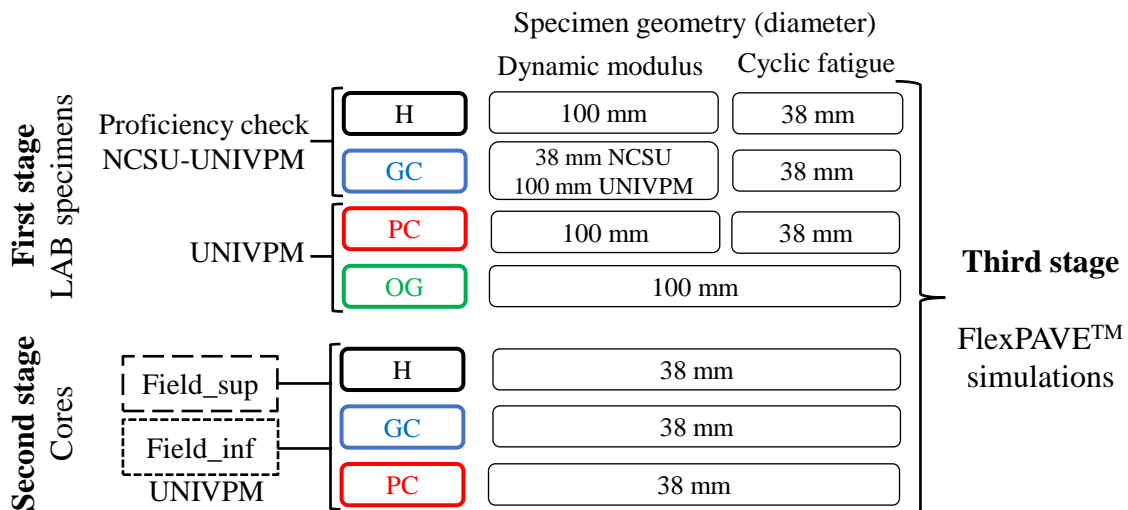


Figure 2. Testing program.

2.3.2. Dynamic modulus tests

Dynamic modulus tests were conducted using large specimens in accordance with AASHTO T 378 [30] and small specimens in accordance with AASHTO TP 132 [31] by means of an Asphalt Mixture Performance Tester (AMPT). The test temperatures that were needed to obtain the viscoelastic properties (dynamic modulus $|E^*|$ and phase angle δ) of the materials, i.e., 4°C, 20°C, and 40°C, were selected in accordance with the reference standards [31][32]. The frequencies investigated for each temperature were 0.1 Hz, 0.5 Hz, 1 Hz, 5 Hz, and 10 Hz, plus 0.01 Hz at 40°C (to improve the prediction of the behavior at low reduced frequencies). The specimens were subjected to sinusoidal axial compression loading; the amplitude was varied to maintain average strain levels of 100 $\mu\epsilon$ and 63 $\mu\epsilon$ for the large and small specimens, respectively. At least two and three replicate specimens were tested for each mixture for the large and small specimens, respectively.

The mastercurves and shift factors were determined by minimizing the error between the experimental data and results from the 2S2P1D model that was applied for the storage modulus ($E_1 = |E^*| \cdot \cos \delta$) [30][31]. Equation (1) presents the 2S2P1D model, which is represented by a combination of two springs, two parabolic elements, and one dashpot.

$$E_1(i\omega\tau) = E_{1,0} + \frac{E_{1,\infty} - E_{1,0}}{1 + \delta(i\omega\tau)^{-k} + (i\omega\tau)^{-h} + (i\omega\beta\tau)^{-1}} \quad (1)$$

where $E_{1,0}$ and $E_{1,\infty}$ are the storage modulus values for frequency $\omega \rightarrow 0$ and $\omega \rightarrow \infty$, respectively, and describe the behavior of the springs; k and h describe the behavior of the parabolic elements; δ is a proportional constant between the parabolic elements; and β is linked to the Newtonian viscosity of the dashpot [33].

2.3.3. Cyclic fatigue tests

Cyclic fatigue tests were conducted using small specimens for the three dense-graded mixtures in accordance with AASHTO TP 133 [34] and large specimens for OG in accordance with AASHTO T 107 [35] by means of the AMPT. Prior to fatigue testing, a dynamic modulus fingerprint test was performed to analyze the specimen-to-specimen variability and calibrate the strain level needed for the fatigue tests. The fingerprint test consisted of measuring the dynamic modulus in the tension compression mode of loading in a strain range of 50 $\mu\epsilon$ to 75 $\mu\epsilon$ at the target test temperature and at the frequency of 10 Hz. The ratio of the fingerprint modulus value to the reference modulus value that is derived from the mastercurve is referred to as the ‘dynamic modulus ratio’, and its value should be within the range of 0.85 to 1.15 for each specimen to limit specimen-to-specimen variability.

Direct tension cyclic fatigue tests were conducted at the frequency of 10 Hz in actuator displacement control mode of loading. The peak-to-peak on-specimen amplitude of the sinusoidal strain was selected to be between 190 $\mu\epsilon$ and 290 $\mu\epsilon$ in order to obtain a test duration between 2000 and 80000 cycles. As prescribed by the reference standards, the test temperature is the average temperature of the bitumen PG minus 3°C, which must not exceed 21°C. Because the expected PG for the binders in this study was PG 76-16, the testing temperature was 21°C.

215 The core experimental outcome of VECD theory is represented by the pseudo stiffness (C) versus
 216 damage (S) curve, referred to as the ‘damage characteristic curve’. The damage characteristic curve
 217 represents the relationship between the pseudo stiffness and amount of damage of a mixture and
 218 describes the fatigue damage evolution within the material [17]. Equation (2) presents the power
 219 function law that expresses this relationship, which represents a fundamental property of the
 220 material that is independent of mode of loading, temperature, frequency, and strain level [36].
 221 Therefore, a mixture can be characterized by three valid test results that show overlapping C versus
 222 S curves.

$$C = 1 - C_{11} \cdot S^{C_{12}} \quad (2)$$

223 where C is pseudo stiffness; S is damage; and C_{11} and C_{12} are fitting coefficients.

224 The D^R failure criterion was applied in this study to define material failure [37]. Equation (3)
 225 defines D^R as the slope of the linear relationship that passes through zero between the average
 226 reduction in pseudo stiffness and the number of cycles to failure (N_f) and is a measure of the
 227 material’s toughness. Following AASHTO TP 133 [34], the number of cycles to failure corresponds
 228 to the cycle in which the product of the peak-to-peak stress and cycle number reaches a maximum
 229 value after a stable increase during cyclic loading.

$$D^R = \frac{\int_0^{N_f} (1 - C) dN}{N_f} = \frac{\text{sum}(1 - C)}{N_f} \quad (3)$$

230 Equation (4) defines the apparent damage capacity index, S_{app} [34][35]. This index parameter was
 231 developed by Wang et al. [38] and takes into account all the properties that affect the cracking
 232 potential of a mixture within the pavement, i.e., stiffness, damage tolerance, and toughness. S_{app} is
 233 determined at a specific temperature that is based on the climatic PG of the location where the
 234 pavement is constructed. Given that the field test track is located in a PG 58-10 climatic zone [39],
 235 the S_{app} reference temperature used in this study was 21°C.

$$S_{app} = 1000^{\left(\frac{\alpha}{2}-1\right)} \cdot \frac{a_{T(S_{app})}^{\frac{1}{\alpha+1}} \cdot \left(\frac{D^R}{C_{11}}\right)^{\frac{1}{C_{12}}}}{|E^*|^{\frac{\alpha}{4}}_{LVE,S_{app}}} \quad (4)$$

236 where α is the damage growth rate; $\alpha_{T(S_{app})}$ is the time-temperature shift factor between the S_{app}
 237 temperature and the reference temperature considered for the dynamic modulus mastercurve; and
 238 $|E^*|^{\frac{\alpha}{4}}_{LVE,S_{app}}$ is the reference modulus calculated at the S_{app} reference temperature and at the
 239 reduced frequency of $62.8 \cdot \alpha_{T(S_{app})}$. S_{app} values usually vary within a range of 0 to 50, and higher
 240 values indicate better fatigue resistance.

241 3. Results and analysis

242 3.1. Proficiency check

243 The proficiency checks, which aimed to check for the reproducibility of the tests that were
 244 conducted at the NCSU and UNIVPM laboratories, involved laboratory-compacted specimens of
 245 the H and GC mixtures. First, the large specimen geometry (100-mm diameter) was selected,

because the NMAS of the mixtures was slightly higher than 19 mm, which is the maximum NMAS prescribed by the reference standards for small specimens [29][31][34]. However, to assess the feasibility of using the small specimen geometry (38-mm diameter), small specimens were considered for the dynamic modulus tests of the GC specimens at NCSU (see **Figure 2**). In order to exclude the influence of the volumetric properties, the tested specimens had air void contents within the range of 2.2% and 3.3%. The density of the specimens was measured using the automatic sealing method [40] at NCSU and the saturated surface dry method [41] at UNIVPM. According to AASHTO PP 99 [29], these two methods can be used indifferently for calculating the density of dense-graded asphalt specimens.

Figure 3 (a) and (b) respectively show the dynamic modulus mastercurves of the H and GC specimens tested at NCSU and UNIVPM. The results at the reference temperature of 21.1°C (70°F) overlap for both mixtures at both laboratories, demonstrating the good reproducibility and repeatability of the dynamic modulus tests.

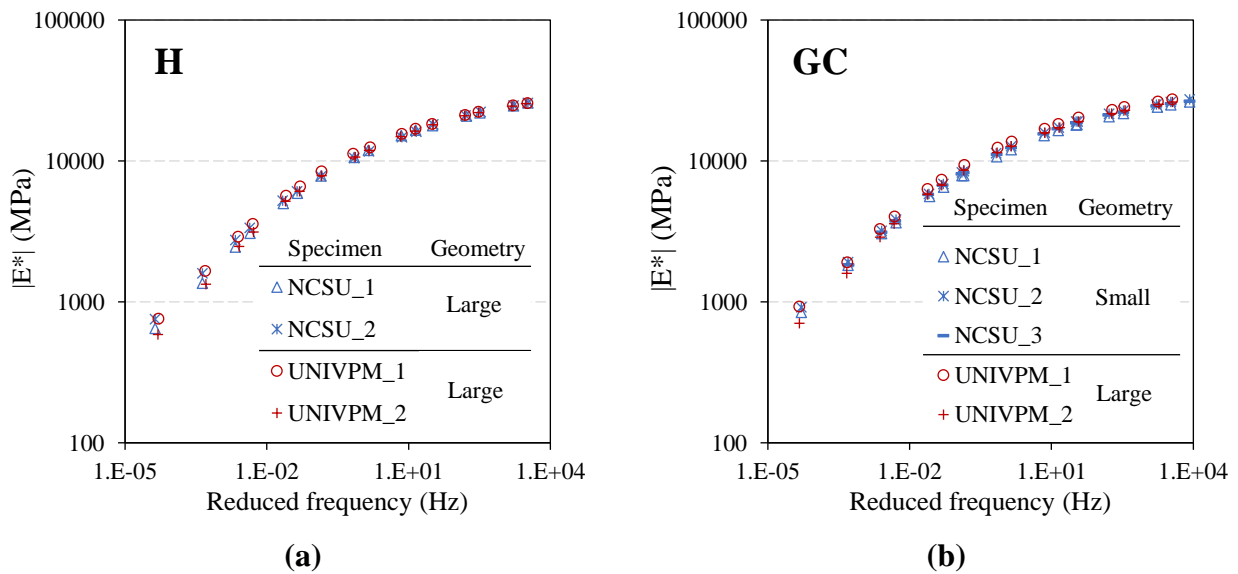
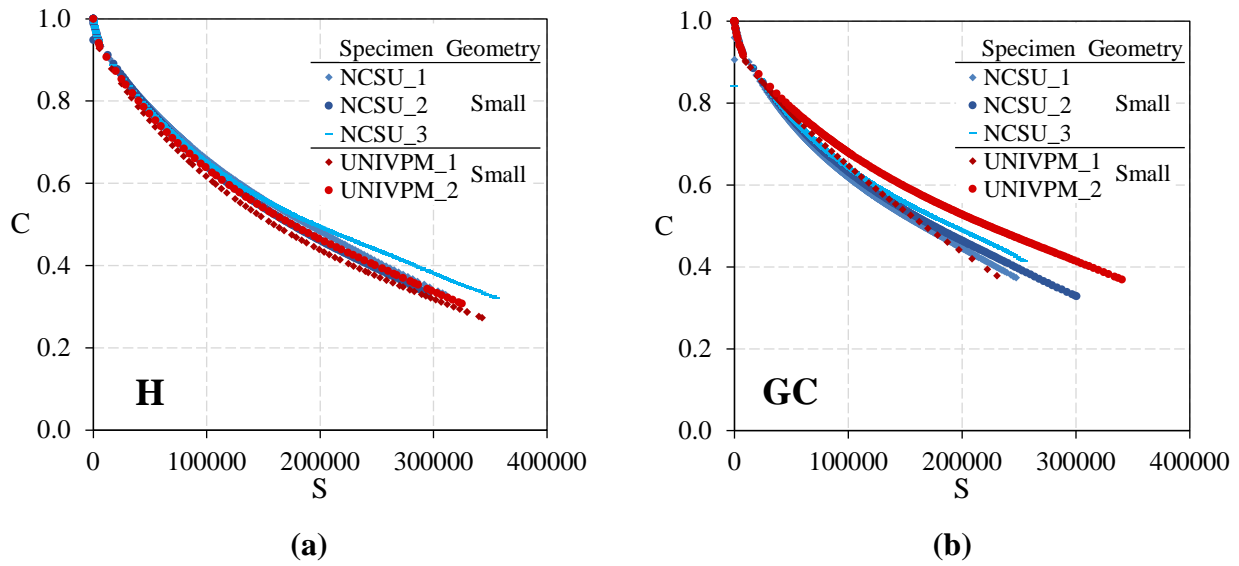


Figure 3. Proficiency check results: dynamic modulus mastercurves for individual specimens of (a) mixture H and (b) mixture GC at 21.1°C.

In addition, the overlapping results of the GC mixture obtained from specimens with different geometries, i.e., the large specimens tested at UNIVPM and the small specimens tested at NCSU (see **Figure 2**), demonstrate that the small specimen geometry complies with the representative volume element despite the NMAS of 20 mm. As a consequence, the small specimen geometry was adopted for the cyclic fatigue tests.

Figure 4 (a) and (b) respectively show the damage characteristic curves of H and GC obtained at NCSU and UNIVPM. Considering that the damage characteristic curve is a fundamental property of the material, and that it is not influenced by the testing and boundary conditions [36], the overlap of the C versus S curves indicates good reproducibility and repeatability of the fatigue tests.



271 **Figure 4.** Proficiency check: damage characteristic curves for individual specimens of (a) mixture
 272 H and (b) mixture GC.

273

274 3.2. Laboratory specimens

275 As a follow-up to the successful proficiency checks, laboratory specimens PC and OG also were
 276 investigated by means of dynamic modulus and cyclic fatigue tests. The following Sections 3.2.1
 277 and 3.2.2 present comparisons of the results obtained from the laboratory-compacted dense-graded
 278 mixtures, H, GC, and PC, and the results obtained for the open-graded mixture, OG, respectively.

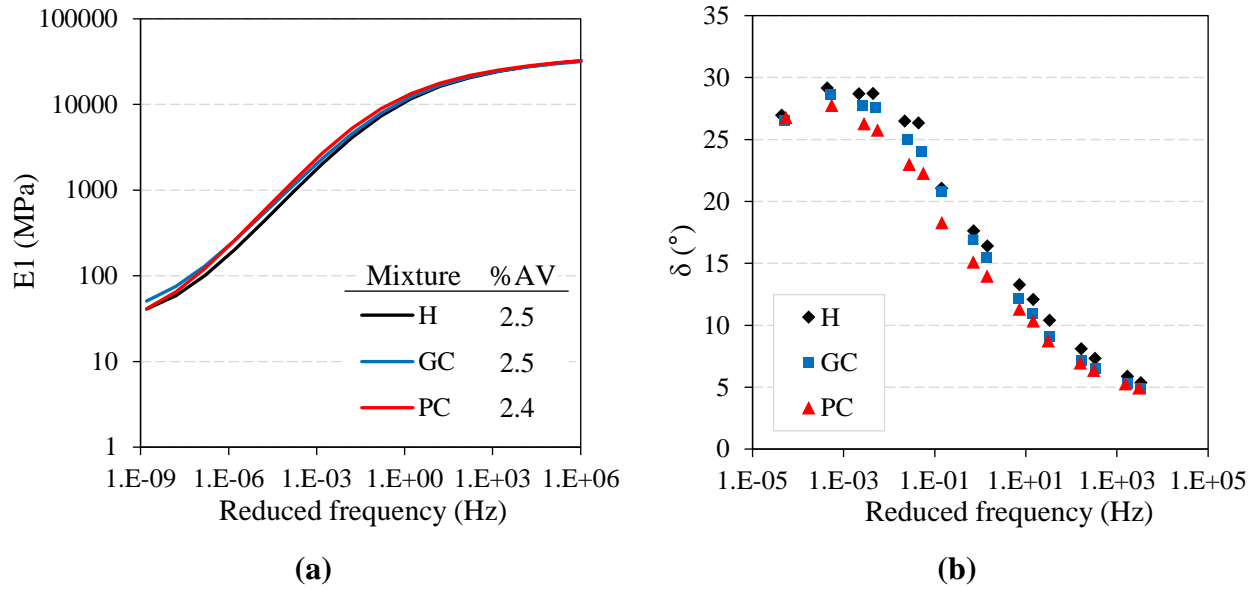
279 3.2.1. Dense-graded mixtures

280 **Figure 5** (a) and (b) respectively show the storage modulus mastercurves based on the 2S2P1D
 281 model and the phase angle mastercurves at the reference temperature of 21.1°C for all the dense-
 282 graded mixtures. Obviously, the 2S2P1D model predictions would be less accurate for very low and
 283 very high reduced frequencies, i.e., for conditions that are significantly different from those
 284 investigated in the laboratory (approximately between 10^{-5} Hz and 10^4 Hz). Note that the H, GC,
 285 and PC mixtures were characterized by the same volumetric properties.

286 **Figure 5** (a) shows that the dynamic modulus mastercurves basically overlap, meaning that H, GC,
 287 and PC have comparable stiffness values overall. This result confirms the findings by Cardone et al.
 288 [26], who observed that, under the same volumetric properties, the mixtures present similar indirect
 289 tensile stiffness modulus values (EN 12697-26) [42] when measured at 20°C and 2 Hz. Specifically,
 290 H exhibits a slightly lower stiffness value within a wide range of reduced frequencies compared to
 291 the mixtures modified by the dry method, whereas the PC and GC curves have slightly different
 292 dynamic modulus mastercurve shapes. **Figure 5** (a) also reports the average air void contents
 293 ('%AV') of each mixture, calculated using the saturated surface dry method.

294 The phase angle mastercurves presented in **Figure 5** (b) confirm that the mixtures modified by the
 295 dry method (especially PC) are more elastic and less viscous than the reference mixture H, as
 296 denoted by their lower phase angle values. Even at the binder level, D'Angelo et al. [25] observed a

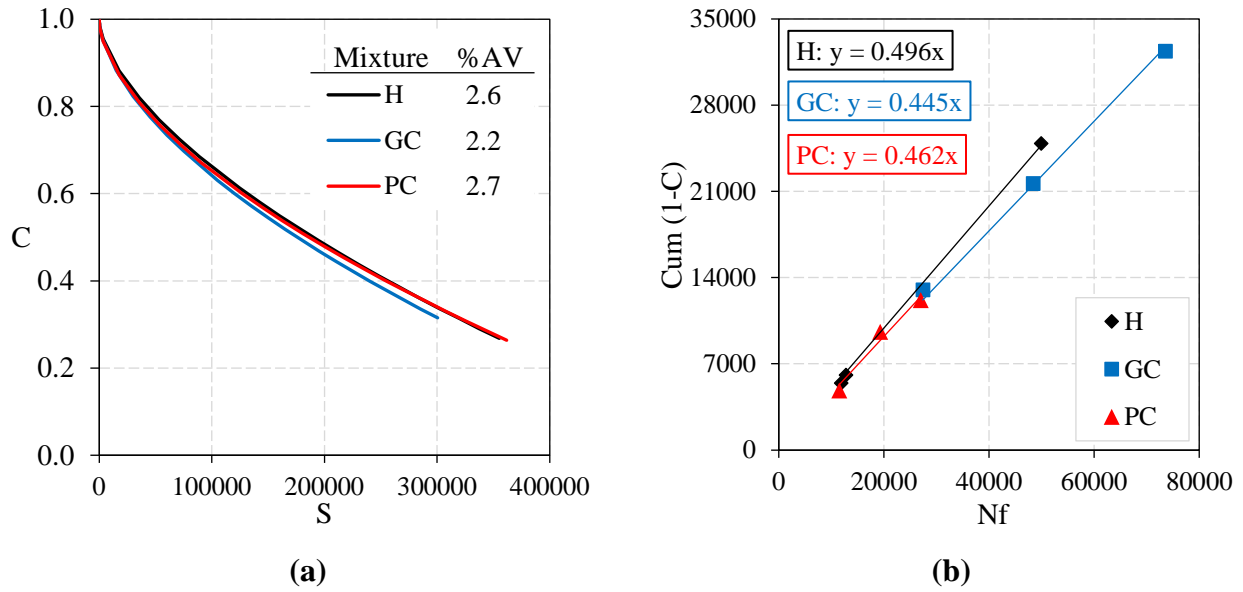
297 predominance of the elastic component of the dynamic modulus for the GC and PC binders
 298 compared to the H binder, especially at high temperatures. Moreover, lower phase angle values can
 299 be observed for the binders modified with the compounds compared to the polymer-modified
 300 bitumen [25].



301 **Figure 5.** Laboratory specimens, dense-graded mixtures: (a) storage modulus mastercurves
 302 (2S2P1D model) and (b) phase angle mastercurves at 21.1°C.

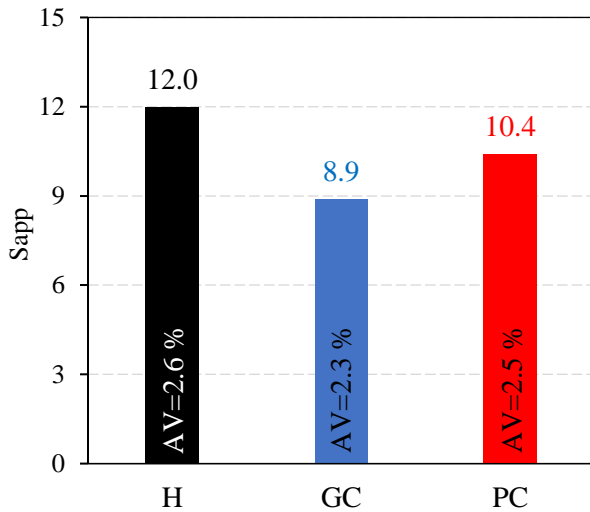
303
 304 **Figure 6 (a)** shows the fit of the C versus S curves for each mixture obtained from three replicate
 305 overlapping curves. The figure also reports the average air void contents of the specimens
 306 investigated in the fatigue tests. Note that these average air void contents are close to those of the
 307 specimens used in the dynamic modulus tests; see **Figure 5 (a)**. The curves are similar for all the
 308 dense-graded mixtures, which is probably due to their comparable stiffness values, also shown in
 309 **Figure 5**. Nevertheless, H and PC reached lower pseudo stiffness values at failure compared to GC,
 310 suggesting their greater capacity to tolerate damage. It is underlined that the fatigue resistance of the
 311 mixture cannot be evaluated solely based on the position of the damage characteristic curve (which
 312 mainly depends on the mixture's stiffness). The damage characteristics provided by the C versus S
 313 curve should be always combined with the mixture's toughness, quantified by the D^R value. This is
 314 possible thanks to the S_{app} parameter (Equation (4)), which allows to predict the fatigue
 315 performance of the mixture within the pavement.

316 **Figure 6 (b)** shows the results based on the failure criterion D^R , which is the slope of the linear
 317 relationship presented in Equation (3). The dense-graded mixtures basically are characterized by
 318 similar values. However, the slightly higher value observed for H (i.e., 0.496) indicates its greater
 319 ability to absorb energy before failure compared to GC and PC. This outcome could be ascribed to
 320 the different binder phases of the mixtures, i.e., the polymer-modified bitumen for mixture H versus
 321 the neat bitumen for mixtures GC and PC.



322 **Figure 6.** Laboratory specimens, dense-graded mixtures: (a) fit of damage characteristic curves and
 323 (b) D^R failure criterion.

324 **Figure 7** presents the S_{app} values and average air void contents obtained from the dynamic modulus
 325 and fatigue tests of the three dense-graded mixtures, which confirmed the rankings given by the D^R
 326 values. H is characterized by the highest value followed by PC and GC, suggesting that the fatigue
 327 resistance of the reference mixture is slightly better than that of the mixtures modified by the dry
 328 method.



329 **Figure 7.** Laboratory specimens, dense-graded mixtures: S_{app} values.

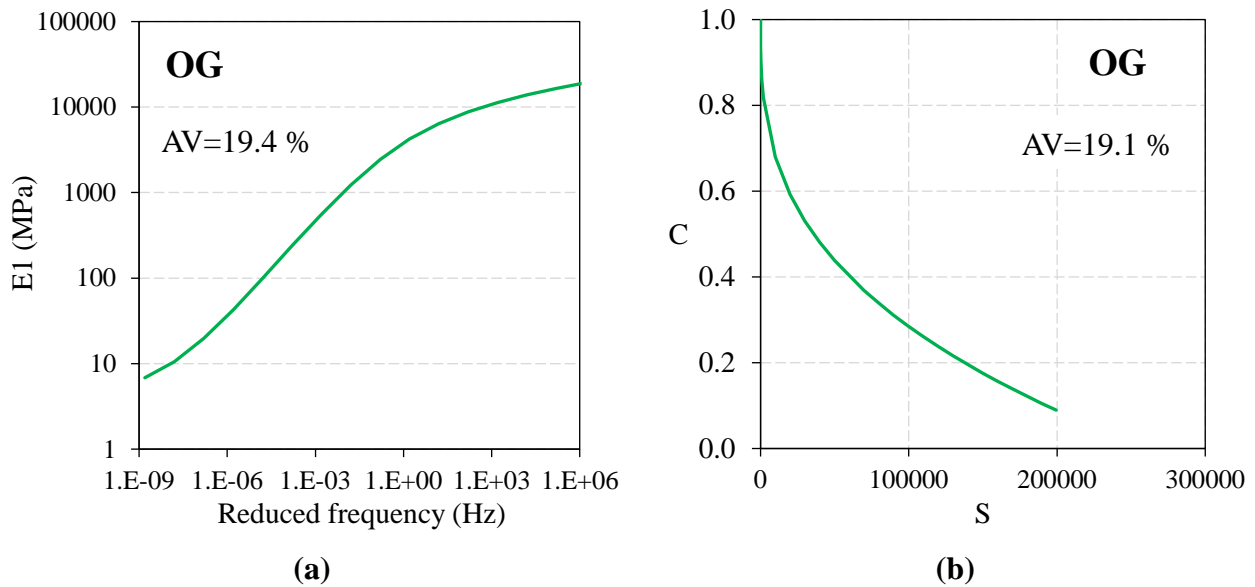
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332 3.2.2. Open-graded mixture

333 As mentioned, the open-graded mixture OG also was investigated through dynamic modulus and
 334 cyclic fatigue tests (see **Figure 2**). The S-VECD model and the related transfer functions have been
 335 mainly calibrated for dense-graded mixtures. Moreover, the service life of open-graded wearing
 336 layers mainly depends on functional aspects, as they are usually replaced due to ravelling and/or

337 clogging. However, the determination of the viscoelastic and damage properties of the OG mixture
 338 was necessary in order to run FlexPAVE™ simulations for the study pavement that includes the OG
 339 mixture as the wearing layer.

340 **Figure 8 (a)** shows the OG storage modulus mastercurve based on the 2S2P1D model at the
 341 reference temperature of 21.1°C. **Figure 8 (b)** shows the fit of the OG damage characteristic curve
 342 obtained from four replicate overlapping *C* versus *S* curves. These two figures also report the
 343 average air void contents of the tested OG specimens, calculated using the geometric method [41].



344 **Figure 8.** Mixture OG: (a) storage modulus mastercurve (2S2P1D model) at 21.1°C and (b) fit of
 345 damage characteristic curve.

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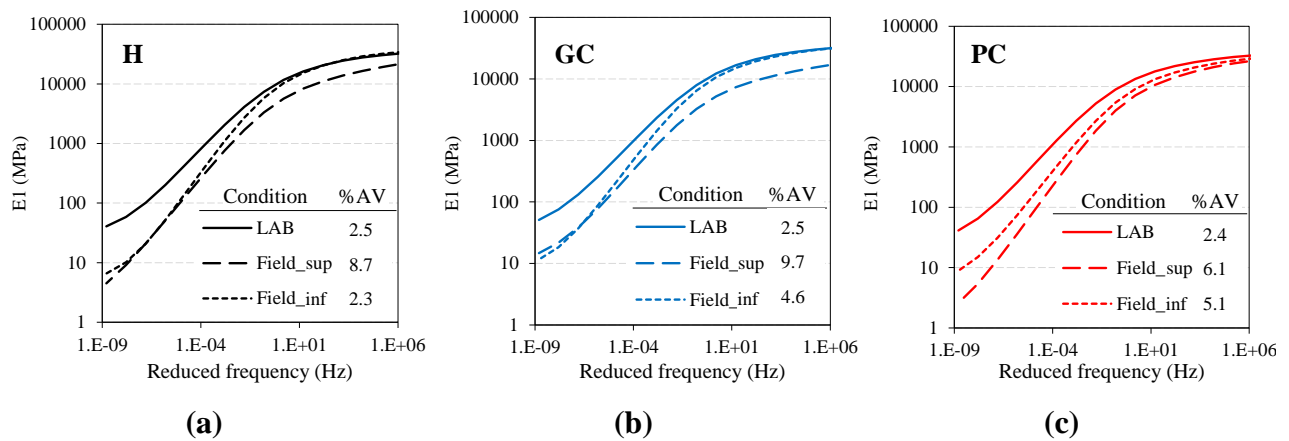
347 3.3. Field specimens

348 **Figure 9** (a), (b), and (c) present comparisons of the storage modulus mastercurves for the field and
 349 laboratory specimens for each of the three mixtures, respectively, obtained based on the 2S2P1D
 350 model at the reference temperature of 21.1°C. The figures also report the average air void contents
 351 for each mixture.

352 Note that, in the case of the *Field_inf* layer, the air void content is comparable to that of the
 353 laboratory specimens only for H, whereas both GC and PC show higher air void contents for the
 354 field specimens. Moreover, for all the mixtures, the air void content of the *Field_sup* layer is higher
 355 than that of the *Field_inf* layer and significantly higher than that of the laboratory specimens. As
 356 discussed in depth by Cardone et al. [26], the *Field_inf* layer of mixture H underwent additional
 357 compaction when *Field_sup* was constructed over it, whereas the workability of the GC and PC
 358 mixtures was diminished by the presence of the compounds, whose melting points are comparable
 359 to the compaction temperature (i.e., 160°C). Therefore, unlike the laboratory specimens
 360 (characterized by similar volumetric properties for all mixtures), the effect of the different air void
 361 contents must be considered when analyzing the behavior of field specimens.

362 **Figure 9** (a), (b), and (c) also show that all the mixtures exhibit a similar trend whereby the stiffness
 363 values of the laboratory specimens are higher than those of the *Field_sup* specimens, whereas the
 364 *Field_inf* specimens exhibit an intermediate stiffness value. The lowest stiffness value of the
 365 *Field_sup* specimens is due to the high air void content observed for all the mixtures. Regarding the
 366 *Field_inf* results, the dynamic modulus values are comparable to those of the laboratory specimens
 367 at high reduced frequencies and comparable to those of the *Field_sup* specimens at low reduced
 368 frequencies (even though the 2S2P1D model predictions could be less accurate at particularly low
 369 and high reduced frequencies). These findings indicate a faster transition from predominantly
 370 elastic behavior to predominantly viscous behavior. Moreover, recall that the dynamic modulus
 371 tests were conducted using large specimens under laboratory conditions and small specimens under
 372 field conditions. Therefore, the test geometry might emphasize the difference at low frequencies
 373 [43].

374 Different from the compound-modified mixtures, GC and PC, the intermediate stiffness values of
 375 the *Field_inf* specimens for the H mixture with SBS-modified bitumen could not be justified by the
 376 intermediate air void contents, because the laboratory and *Field_inf* specimens had similar air void
 377 contents of around 2.5%; see **Figure 9 (a)**. As a possible explanation, the mixtures were compacted
 378 by a gyratory compactor at a fixed height (180 mm) in the laboratory, whereas the lower 15-cm
 379 layer reached this compaction level in the field gradually thanks to the progressive passage of
 380 rollers on the pavement. The two compaction methods can imply different compaction energy and
 381 lead to different aggregate packing. In addition, the laboratory specimens were cored vertically
 382 from the gyratory-compacted samples, whereas the field specimens were cored horizontally within
 383 the layer (i.e., different orientation of the principal stresses).



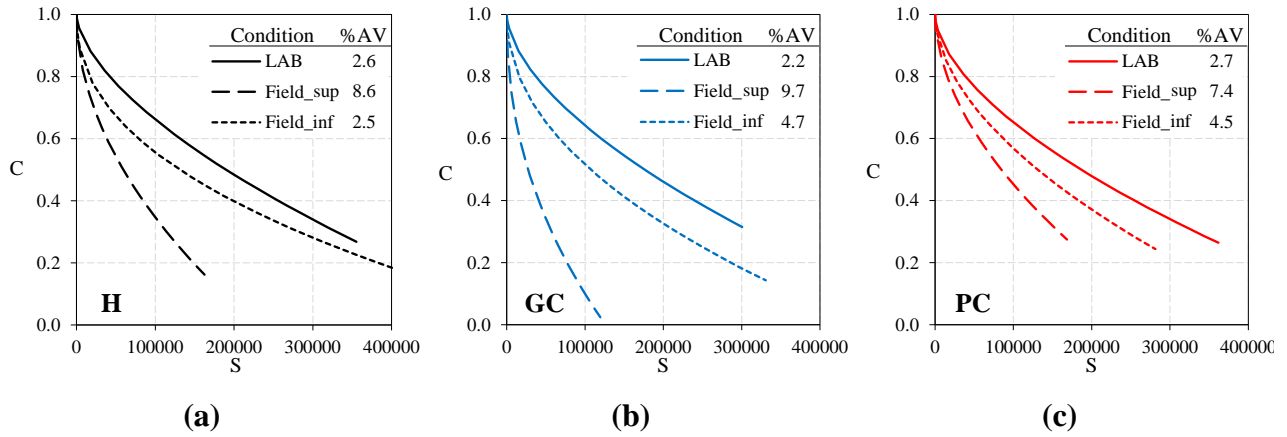
384 **Figure 9.** Field vs. laboratory specimens: storage modulus mastercurves (2S2P1D model) of (a) H,
 385 (b) GC, and (c) PC at 21.1°C.

386

387 **Figure 10** (a), (b), and (c) present comparisons of the fit of the damage characteristic curves
 388 obtained from the *Field_sup* and *Field_inf* specimens and from the laboratory specimens for each of
 389 the three mixtures, respectively. The figures also report the average air void contents of the
 390 specimens subjected to fatigue tests, which are comparable to the air void contents of the dynamic
 391 modulus specimens shown in **Figure 9**. **Figure 10** shows that, for all three dense-graded mixtures,
 392 the curve of the laboratory specimens is in the highest position, followed by that of *Field_inf* and

393 then *Field_sup*. These results are consistent with the stiffness properties and air void contents (in
 394 general, the curve of softer mixtures tends to stay below that of stiffer mixtures). Regarding mixture
 395 H, note that the LAB curve is above the *Field_inf* curve despite their similar air void contents,
 396 confirming that *C* versus *S* curves are dependent on the stiffness property, which in turn is affected
 397 by the compaction and coring conditions, as mentioned earlier.

398 The H and GC field specimens reached lower *C* values at failure than the laboratory specimens,
 399 suggesting the field specimens' greater tolerance to damage, which could be ascribed to the different
 400 volumetric properties and/or compaction method (whose effects are discussed above). Moreover, the
 401 high value of *S* at failure observed for H in the *Field_inf* condition suggests that this mixture is able
 402 to tolerate a greater amount of damage compared to all the other investigated mixtures. However,
 403 recall that the *C* versus *S* curve alone does not provide complete information on the fatigue resistance
 404 and thus should be always combined with D^R into the S_{app} value. The curve of *Field_sup* for the GC
 405 mixture, which is characterized by the *C* value at failure that is close to 0, is considered unreliable
 406 due to the high air void content (i.e., 10%), which could lead to problems related to the
 407 representativeness of the small specimen geometry. Conversely, for PC, all the curves can be
 408 characterized by similar pseudo stiffness values at failure.



409 **Figure 10.** Field vs. laboratory specimens: fit of damage characteristic curves for (a) H, (b) GC, and
 410 (c) PC.

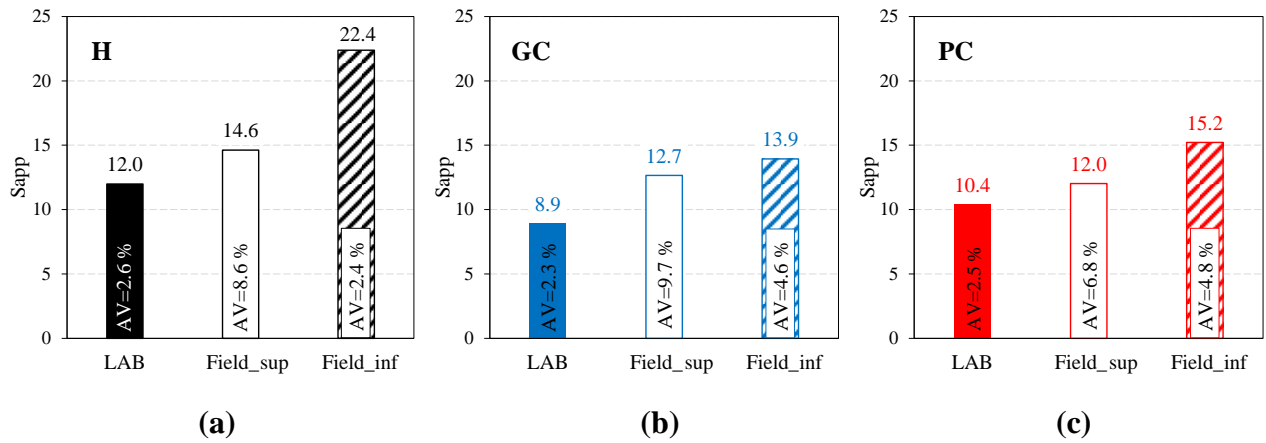
411
 412 **Table 1** presents the D^R values for all the mixtures. The toughness, i.e., the ability to absorb energy
 413 before fracture, of the field specimens is always significantly greater than that of the laboratory
 414 specimens, suggesting that the laboratory compaction process leads to a reduction in toughness (i.e.,
 415 possible over-compaction causing some broken aggregate, especially within the RAP fraction). The
 416 extremely high D^R value (0.876) observed for *Field_sup* of GC confirms the unrealistic results for
 417 this mixture. No clear trend related to air void content was observed in terms of D^R .

418 **Table 1.** Field vs. Laboratory Specimens: D^R Values

Condition	H	GC	PC
Laboratory	0.496	0.445	0.462
<i>Field_sup</i>	0.619	0.876	0.481
<i>Field_inf</i>	0.610	0.581	0.514

419

420 The S_{app} values in **Figure 11** (a), (b), and (c) confirm the findings from the damage characteristic
 421 curves and D^R failure criterion for the three mixtures. The figures also report the average air void
 422 contents of all the specimens used for the dynamic modulus and fatigue tests. Better fatigue
 423 resistance can be expected from the field specimens, especially from *Field_inf* specimens, which
 424 are characterized by an intermediate stiffness value that ensures good distribution of the stress
 425 within the pavement without compromising the mixture's toughness. The highest S_{app} value of 22.4,
 426 observed for H in the *Field_inf* condition, can be associated with the high S value at failure
 427 observed for this mixture; see **Figure 10** (a).



428 **Figure 11.** Field vs. laboratory specimens: S_{app} values for (a) H, (b) GC, and (c) PC.

429 Overall, based on the analysis of the mixtures at the material level, the compound-modified
 430 mixtures, GC and PC, show similar performance, which is slightly worse than the performance of
 431 the reference mixture, H. These results are in agreement with those obtained by Cardone et al. [26]
 432 through indirect tensile fatigue tests (EN 12697-24) [44]. Moreover, the analysis results indicate
 433 that an intermediate material stiffness value can lead to better fatigue resistance of the pavement, as
 434 expressed by the S_{app} parameter.

435 4. Pavement performance simulations

436 Pavement performance simulations were carried out using FlexPAVETM, which takes into account
 437 both the moving nature of traffic loads and climatic conditions that affect the pavement [23][24].
 438 Using FlexPAVETM, the pavement is modeled as a three-dimensional layered viscoelastic structure,
 439 and the finite element method with Fourier transform is embedded to predict long-term mechanical
 440 responses [45].

441 The spatial distribution response of the damage factor was analyzed in this study. The damage
 442 factor is defined as the ratio of the current number of cycles (N) to the number of cycles at failure
 443 (N_f). The damage factor ranges from 0 to 1, with 1 indicating an asphalt element that is completely
 444 cracked. Moreover, the time history (or evolution) of the percentage of damage, defined as the ratio
 445 of the sum of the damage factors within the cross-section area to the total area, also was evaluated
 446 in this study. Equation (5) defines the percentage of damage (%Damage). Specifically,
 447 FlexPAVETM calculates the %Damage based on a reference cross-section area affected by stresses
 448 and strains due to traffic loading [23].

$$\%Damage = \frac{\sum_{i=1}^M (N/N_f)_i \cdot A_i}{\sum_{i=1}^M A_i} \quad (5)$$

where i is the nodal point number in the finite element mesh; M is the total number of nodal points in the finite element mesh; and A_i is the area that corresponds to the nodal point i in the finite element mesh.

Equation (6) presents the sigmoidal transfer function proposed by Wang et al. [24] that was used to create a relationship between the $\%Damage$ in the cross-section provided by FlexPAVETM and the percentage of cracking ($\%Cracking$) on the surface of the pavement.

$$\%Cracking = \frac{50}{1 + C_{f1} \cdot \exp[C_{f2} \cdot (\log C_{f3} - \log \%Damage)]} \quad (6)$$

where 50 is the maximum $\%Cracking$; and C_{f1} , C_{f2} , and C_{f3} are calibration factors whose values are 0.342, 13.97, and 16.38, respectively.

4.1. Input data

In this study, FlexPAVETM analysis was performed for a 30-year pavement service life under both thermal effects and traffic loading. The structure of the simulated pavement is the same as the Fiumicino field test track and consists of a 4-cm open-graded wearing layer, a 25-cm dense-graded asphalt layer, and a 35-cm foundation on the subgrade. **Table 2** presents a summary of the three scenarios evaluated in the simulations. For all three scenarios, the upper 4-cm layer is modeled based on the OG results, whereas the material properties of the 25-cm layer used as input differ. Note that, in the context of FlexPAVETM simulations, a 25-cm layer is equivalent to two separate layers of 10 cm (*Field_sup*) and 15 cm (*Field_inf*) with the same properties, presenting perfect bonding at the interface, as preliminarily verified in this study. The decision to consider a single layer with 25-cm thickness was based on the fact that it requires less computational time. In Scenario 1, the dense-graded layer was modeled using the results of the laboratory specimens. In Scenario 2 and Scenario 3, the dense-graded layer was modeled using the results of *Field_sup* (excluding GC for the reasons mentioned in Section 3.3) and *Field_inf* specimens, respectively. The foundation and the subgrade were modeled as linear elastic materials with stiffness modulus values of 400 MPa and 150 MPa, respectively, determined from FWD tests [26].

Table 2. Pavement Performance Simulations: Material Properties Considered for Each Scenario

Scenario	Wearing layer (4 cm)	Dense-graded layer (25 cm)	Foundation (35 cm)	Subgrade
1	OG	LAB (H, GC, PC)	$E = 400$ MPa	$E = 150$ MPa
2	OG	<i>Field_sup</i> (H, PC)	$E = 400$ MPa	$E = 150$ MPa
3	OG	<i>Field_inf</i> (H, GC, PC)	$E = 400$ MPa	$E = 150$ MPa

As for the climatic conditions, based on a comparison of the annual temperatures and precipitation, the Italian city of Fiumicino (where the field test track is located) best matches San Luis Obispo in

California (whose climatic data are present in the FlexPAVETM database). Regarding the loading conditions, as revealed from traffic data studies conducted in 2019, the field test track is subjected to one million equivalent standard axle loads (ESALs) per year, where the reference ESAL is a 120-kN single axle with dual wheels. In order to assess the damage caused by motorway heavy traffic, a speed of 90 km/h and a tire inflation pressure of 800 kPa were considered. The tire-pavement contact area was set as circular, and linear traffic growth of 0.4% per year was estimated.

4.2. FlexPAVETM results

Figure 12 (a), (b), and (c) present the damage contours of H, GC, and PC, respectively, for Scenario 1 (i.e., with the material properties of the laboratory specimens for the dense-graded asphalt layer; see **Table 2**) after 30 years of service. Recall that the H, GC, and PC laboratory specimens present analogous volumetric properties, i.e., air void contents around 2.5%, as reported also in **Figure 12**. **Figure 12** clearly indicates that the damage is concentrated mainly in the upper part of the asphalt layers, in particular under the 4-cm wearing layer. This damage distribution is likely attributable to the thermal effects that are due to the difference in stiffness values between the open-graded and dense-graded layers. Note, too, that thermal loading is similar to the loading applied in a displacement-controlled test, and the resistance of the mixture to thermal loading depends on the mixture's capacity to relieve the stress and resist the damage that is due to the induced stress [22]. Mixture H is less prone to thermal damage than GC and PC, because the latter mixtures are slightly stiffer and more brittle (i.e., greater induced stress and lower tolerance to damage) and have lower phase angle values (i.e., less capacity to relieve stress) than H (**Figure 5**). Bottom-up cracking, which relates strictly to fatigue stress, is limited in all the cases because of the significant stiffness of the laboratory specimens (**Figure 5**), which determined the low tensile strain at the bottom of the asphalt layers.

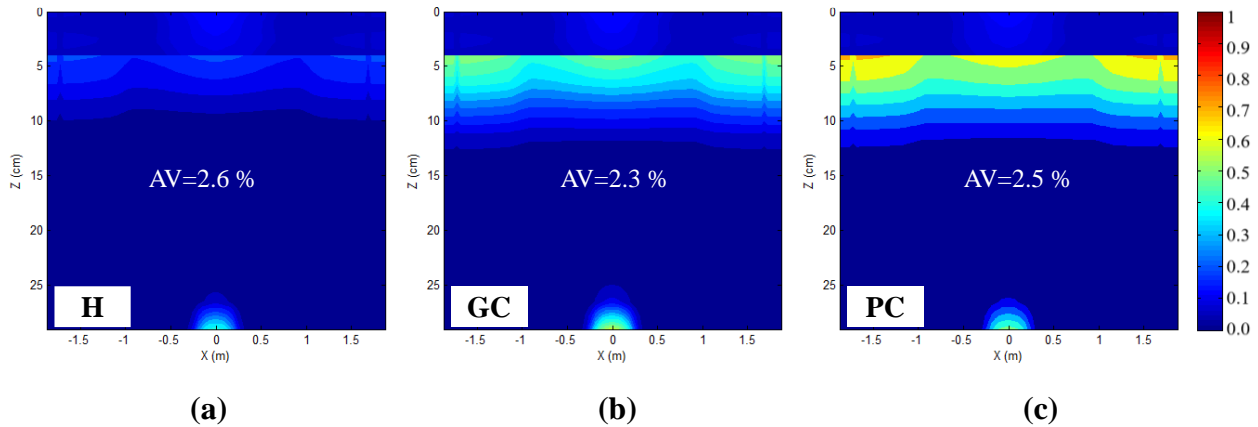
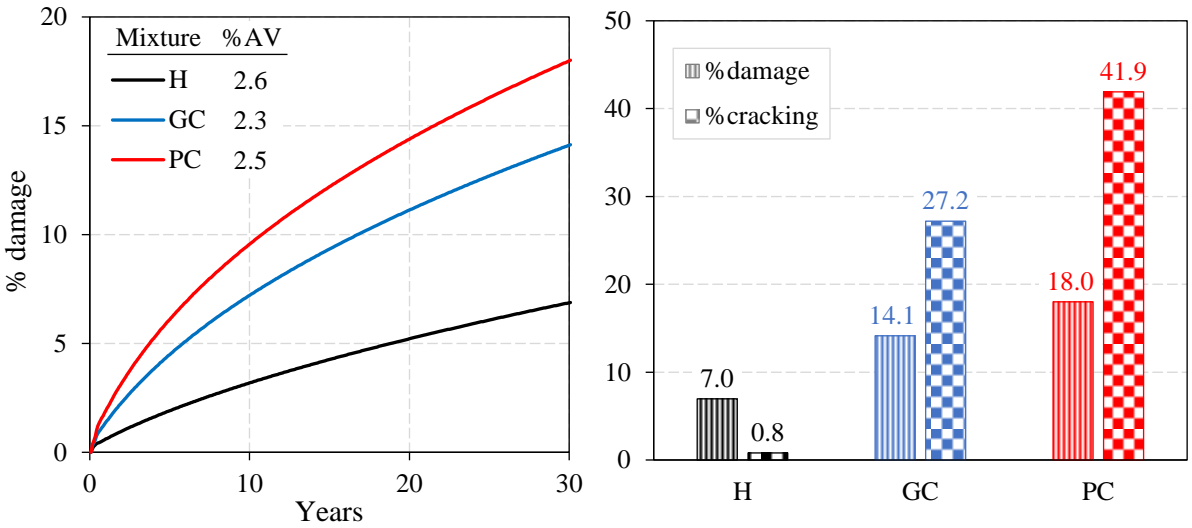


Figure 12. Scenario 1: Damage contours of (a) H, (b) GC, and (c) PC after 30 years.

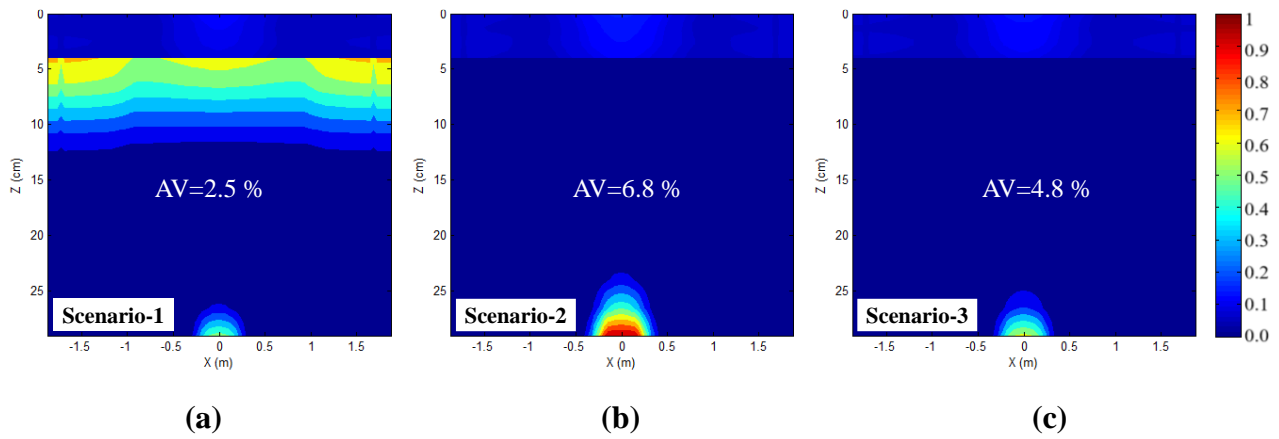
Figure 13 (a) illustrates the Scenario 1 damage evolution within the pavement during the 30 years of service, whereas **Figure 13** (b) reports the percentage of damage and the percentage of cracking after the 30 years of service. Percent of damage at the end of 30 years for H, GC, and PC is 7.0, 14.1 and 18.0%, respectively. After applying the transfer function in Equation (6), the percentage of cracking is 0.8%, 27.2%, and 41.9% for H, GC, and PC, respectively. The ratios of %Damage and %Cracking between GC and PC are about the same. However, %Cracking of H is much smaller

508 than the %Damage. In fact, according to the considered transfer function, when %Damage is lower
 509 than 10%, the corresponding %Cracking increases slowly and usually remains lower than 5%.
 510 Instead, when %Damage is higher than 10%, %Cracking increases dramatically [24]. The reason is
 511 that %Damage is calculated from the pavement cross-section whereas %Cracking is measured from
 512 pavement surface. That is, fatigue cracking cannot be seen on the pavement surface while fatigue
 513 damage grows within the asphalt layer. Overall, the better performance of H can be attributed to its
 514 polymer-modified bitumen.



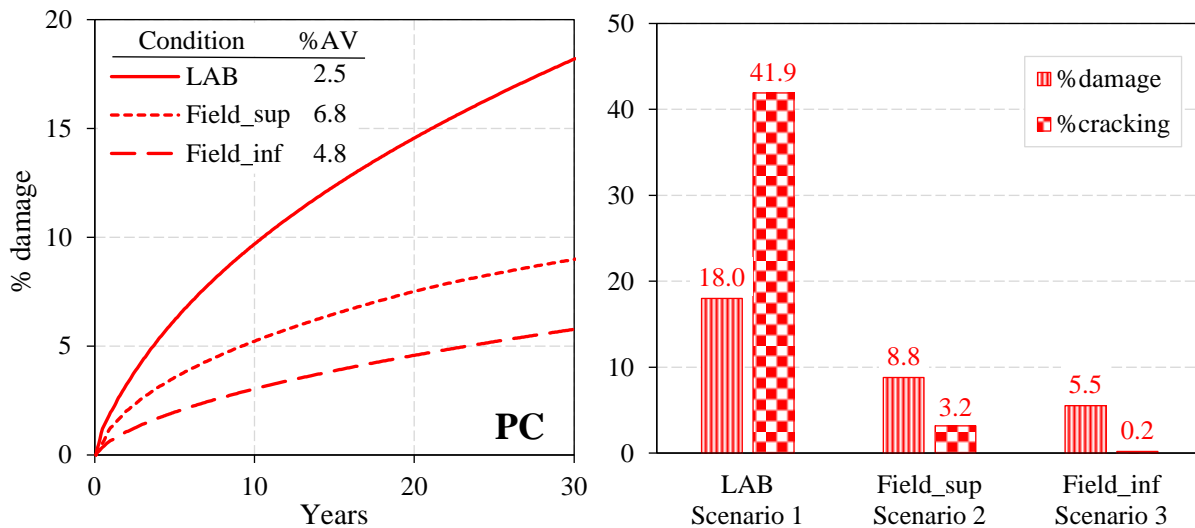
515
 516 **Figure 13.** Scenario 1: (a) damage evolution, and (b) percentages of damage and cracking after 30
 517 years.

518
 519 **Figure 14** (a), (b), and (c) respectively present the damage contours for the PC mixture in Scenario
 520 1 (laboratory specimens), Scenario 2 (*Field_sup* specimens), and Scenario 3 (*Field_inf* specimens)
 521 after 30 years of service, together with the average air void contents of the specimens. As stated
 522 earlier, in Scenario 1, the damage involves a large area under the wearing layer due to the difference
 523 in stiffness values between the layers, which led to significant thermal damage in this study. In
 524 Scenario 2, the damage is shown to be concentrated mostly at the bottom of the asphalt layer (i.e.,
 525 bottom-up cracking), even though the involved area is only 6 cm to 7 cm thick. This behavior is due
 526 to the lower stiffness values of the *Field_sup* specimens compared to the laboratory specimens
 527 (**Figure 9**) and relates strictly to the higher air void content. Finally, in Scenario 3, an intermediate
 528 behavior (that falls between the other two scenarios) can be observed whereby the pavement is
 529 subjected to limited damage at the bottom of the asphalt layer (bottom-up cracking) and the upper
 530 part of the asphalt layer (top-down cracking). However, the damage level observed for the open-
 531 graded wearing layer might not be fully reliable due to the limited data available in the literature
 532 regarding the S-VECD model characterization of open-graded mixtures (as anticipated in Section
 533 3.2.2). These findings suggest that the best performance at the structural level can be expected from
 534 dense-graded mixtures characterized by an intermediate stiffness level, which ensures a limited
 535 stress-strain level within the pavement without negatively affecting the mixture's toughness and
 536 thermal resistance. Similar observations could be made for H and GC, and thus, the results for H
 537 and GC are not shown here for brevity.



538 **Figure 14.** Damage contours for PC mixture: (a) Scenario 1, (b) Scenario 2, and (c) Scenario 3.

539 **Figure 15** (a) shows the damage evolution within the pavement during 30 years of service for the
 540 PC mixture, whereas **Figure 15** (b) reports the percentage of damage and the percentage of cracking
 541 after the 30 years of service for each Scenario. For the specimens obtained from the field test track,
 542 the percentage of damage after 30 years reached between 5% and 10%, corresponding to a very low
 543 percentage of surface cracking (lower than 3.2%), as explained earlier. A comparison of **Figure 14**
 544 and **Figure 15** shows that the wide difference in the percentage of cracking between the LAB and
 545 field conditions can be ascribed to the damage that is due to thermal stress. Given that only mixture
 546 H did not suffer thermal damage (see **Figure 12**), the viscoelastic properties derived from the
 547 modification of the binder can reasonably be assumed to provide beneficial effects, which are not
 548 provided through the dry modification of neat bitumen.



549 **Figure 15.** PC mixture: (a) damage evolution, and (b) percentages of damage and cracking after 30
 550 years under the three scenarios considered.
 551

552 5. Conclusions

553 This study aimed to compare the fatigue performance of a reference mixture (H) that contains
 554 bitumen modified via the wet method and two compound mixtures (GC and PC) modified via the
 555 dry method. The two compounds basically consist of polyethylene and polypropylene, plus
 556 graphene for GC. The analysis was conducted by applying the S-VECD modeling approach, which

includes dynamic modulus tests and direct tension cyclic fatigue tests whose results were used as input for FlexPAVETM pavement performance simulations. The mixtures were produced at an asphalt plant and were used to construct a 25-cm base layer (placed and compacted in two separate steps, 15 cm + 10 cm) in a field test track as part of an Italian motorway. Part of the mixtures also were compacted in the laboratory.

This study demonstrates for the first time the applicability of the S-VECD modeling approach also to non-conventional materials such as asphalt mixtures modified with plastics via the dry method. The main strength of this approach is that it allows to determine fundamental fatigue properties of the material, which are independent of the testing conditions, and to predict the long-term fatigue performance of the pavement (unlike the conventional empirical fatigue tests, whose results are valid only under the specific boundary and loading conditions considered).

Under the same volumetric properties, the investigated mixtures show comparable stiffness values and fatigue resistance overall, even though the mixtures with the compounds are slightly stiffer and less viscous than the reference mixture. However, based on the FlexPAVETM simulations, the mixtures with the compounds are much more prone to thermal damage compared to the reference mixture. This outcome is due mainly to the greater difference in stiffness values with the open-graded wearing layer. Compared to the laboratory specimens, the field specimens can be characterized by less stiffness as a consequence of their higher air void contents and different compaction conditions. In particular, the intermediate stiffness level, which characterizes the *Field_inf* specimens of all the mixtures, correlates with several positive effects: greater damage tolerance and toughness at the material level and less thermal damage and limited bottom-up cracking at the structural level.

Taking into account also the workability issues of GC and PC that are related to the melting point of plastics, the findings suggest that the performance of the reference mixture with the polymer-modified bitumen (H) is slightly better than the performance of the mixtures modified using the dry method (GC and PC). However, the addition of the polymeric compounds certainly improves the performance of mixtures with neat bitumen, even though graphene does not seem to add any positive contribution. Moreover, the environmental benefits generated from the dry modification method, i.e., reduced operational effort and the possibility of recycling waste plastics, also should be taken into consideration. Monitoring the existing motorway field test track over time will provide further data regarding the behavior of the mixtures in the field.

588

589 **Author contributions**

590 Conceptualization: Francesco Canestrari

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594 Formal analysis: Francesco Canestrari, Y. Richard Kim, Lorenzo Paolo Ingrassia, Sara Spadoni,
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596 Writing – original draft: Sara Spadoni, Lorenzo Paolo Ingrassia
597 Writing – review and editing: Francesco Canestrari, Y. Richard Kim, Douglas Mocelin.
598

599 **Declaration of Competing Interest**

600 The authors declare that they have no known competing financial interests or personal relationships
601 that could influence the work reported in this paper.
602

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607

608 **Data Availability Statement**

609 The datasets generated and/or analyzed during this study are available from the corresponding
610 author upon reasonable request.
611

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