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Materials and Structures (2025) 58:271
https://doi.org/10.1617/s11527-025-02793-9

ORIGINAL ARTICLE

Experimental investigation on fatigue properties of asphalt mixtures with high content of RAP and recycled plastics

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Received: 21 May 2025 / Revised: 7 August 2025 / Accepted: 29 August 2025 / Published online: 29 September 2025

Abstract The experimental study described in the paper explored the synergistic effects of Reclaimed Asphalt Pavements (RAP) and a polymeric compound derived from recycled plastics on fatigue properties of dense-graded asphalt mixtures. The investigated mixtures were prepared in the laboratory by combining 50% of RAP with different dosages of the polymeric compound. A control mixture containing a highly SBS-modified binder was also prepared and tested for comparison purposes. The fatigue properties of mixtures were evaluated by means of flexural test in the four-point bending configuration under oscillatory loading. The tests were conducted in both controlled-stress and controlled-strain loading modes to understand the response of materials under different critical loading conditions in actual pavement structure. The experimental results indicated that the use of high percentages of RAP combined with the use of recycled plastics resulted in good fatigue performance of the mixture. The experimental data also indicated that recycled plastics at higher dosage used in this study yielded fatigue performance comparable or even slightly higher to that obtained with the use of control SBS-modified binder.

Keywords Reclaimed asphalt pavement · Rrecycled plastics · SBS-modified binder · Fatigue

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1 Introduction

Road paving industry is developing ambitious goals to significantly reduce the greenhouse gas emissions associated to asphalt mix production. In this context, the increased use of recycled materials in asphalt mixtures represents a valued industry-driven opportunity for the reduction of carbon footprint of road pavements [1]. Recycling contributes to minimize the impacts of raw material manufacturing; in addition, it allows to preserve natural resources and to decrease waste disposal in landfills. Among various recycled materials used in the production of asphalt mixtures, waste plastics and reclaimed asphalt pavement (RAP) hold a prominent role [2–6].

The disposal of plastic waste poses a global environmental challenge due to its non-biodegradable nature, causing it to persist in terrestrial and marine ecosystems for decades [7, 8]. Only a marginal proportion of plastic waste is successfully recycled, while the majority is managed through other means. Despite significant efforts in recent years [9], only 9% of plastic waste is currently recycled, while 22% is illegally dumped into the environment, 50% ends up in landfills, and 19% is incinerated [10]. Regarding the latter two components, road pavement engineers are exploring the potential of incorporating plastic waste in asphalt mixtures, both to give these materials a "second life" and to provide a sustainable alternative to traditional disposal methods [11–13].

Different investigations have reported an overall improvement in mechanical performances—such as stiffness, fatigue life, permanent deformation, and moisture damage resistance—when waste plastics are added into asphalt mixtures [14–16]. The extent of these improvements is highly dependent on the type and properties of the plastic used. Furthermore, the inclusion of plastics can partially replace bitumen, reducing its required content and contributing to more sustainable pavement solutions [8].

Plastics can be incorporated into asphalt pavement through different processes, which are generally distinguished depending on their melting point [17].

In the wet process, plastics are blended with asphalt binders at high temperatures to produce modified binders, that are subsequently mixed with hot aggregates. Plastics with low melting points are suitable for this process [18, 19]. In the dry process, recycled plastics are added directly into the mixture as either aggregate replacement or mixture modifiers. Aggregate replacement entails using plastics characterized by high melting points with consequent substitution of one or more natural stone fractions. When using plastics with melting point below the mixture production temperature, they partially melt upon mixing resulting in modified mixture with plastic-coated aggregates [20, 21]. The addition of low melting point plastics into the mixtures is also referred to hybrid process by some authors [17].

The wet approach is currently the most employed process in the road sector. However, several drawbacks exist. Phase separation between plastic and binder can occur when modified binder is stored and left to rest, with negative effects on the performance and durability of the overall asphalt mixture [21]. Furthermore, the wet method requires more machinery and equipment to shred the plastics into powders and mix them with hot asphalt binder, thus resulting in an expensive, energy- and time-consuming process, with consequent high production costs and increased environmental impact [17, 22, 23]. Conversely, the dry (and hybrid) process seems simple and energy-saving by adding the shredded plastics powders directly into mixing batch [24]. Moreover, several studies have highlighted the potential benefits in terms of stiffness modulus, rutting resistance, indirect tensile strength, and moisture damage of mixtures when plastics are added via dry or hybrid process compared to the wet one [8, 25–27].

In terms of fatigue, there is less consensus among researchers about the effect of recycled plastic compounds on mix performance. Several experimental works reported an improvement in fatigue properties of mixtures when plastics are incorporated via dry or hybrid approach [7, 8, 24, 28]; on the opposite, other studies highlighted better fatigue performances of asphalt mixtures modified via wet process [22, 29]. Some researchers finally found neither improvements nor deteriorations [30, 31] caused by addition of plastics using one or another method. These different outcomes can be associated to different origins and characteristics of plastics used in various studies, indicating that the type of plastic is a critical factor in affecting the performance characteristics of the final asphalt mixture.

RAP has been used for decades as recycled material in pavement construction. Typically, the incorporation of RAP in new hot mix asphalt is limited to 25–30% [19] even though the current trend is to increase RAP content even further [32, 33]. The main reason for limiting the amount of RAP in asphalt mixtures is related to the presence of aged binder. In fact, while RAP binder can increase the stiffness modulus and improve rutting resistance of mixture, it may negatively affect its fatigue performance [34–39]. Some studies suggest that fatigue life of mixtures can be improved when RAP content remains within a specific range [40, 41].

The combined use of recycled plastics and RAP may benefit from the synergistic effects provided by both components, representing a valuable even though challenging sustainable solution for the construction of asphalt pavements.

The fatigue properties of asphalt mixtures incorporating both RAP and recycled polymers has not been extensively studied, with few studies available in the literature and not completely agreeing with each other. Leng et al. [42] analysed the fatigue resistance of binder samples containing virgin bitumen, RAP bitumen at various dosages, and waste polyethylene terephthalate (PET) derived additives, obtained through an aminolysis process. The experimental investigation conducted through dynamic shear rheometer tests, highlighted that the combination of PET-modified binder and RAP at all investigated dosages consistently outperformed virgin binder fatigue resistance. However, increasing RAP bitumen content above 15% led to a decline in fatigue characteristics compared to lower dosages. Vijayan et al. [43] investigated asphalt mixtures, with and without a recycled plastic modifier, at a 50% recycling rate for two recycling cycles using a comprehensive volumetric and mechanical performance evaluation. The analysis of fatigue life indicated that the asphalt mixture modified with recycled plastic had superior fatigue resistance compared to the conventional mixture. Additionally, all the recycled mixtures with a 50% recycling rate also showed significant improvement in fatigue life. Saadeh et al. [44] employed an asphalt binder modified with low-density polyethylene, reactive elastomeric terpolymers and polyphosphoric acid, to prepare mixtures containing only raw aggregates and mixtures with 20% RAP. Based on the IDEAL cracking tests, they found that when using raw aggregate, the resistance to fatigue cracking was nearly the same regardless of using a polymer-modified binder or conventional binder. However, when RAP aggregate was used with a polymer-modified binder, there was a significant reduction in fatigue life.

The research work presented in this paper aimed to evaluate the fatigue properties of asphalt mixtures containing 50% RAP and different dosages of a polymeric compound derived from recycled plastic waste. A control asphalt mixture containing a standard SBS-modified binder was also evaluated for comparison purposes.

Fatigue properties of mixtures were investigated by means of flexural tests conducted in four-point bending (4 PB) configuration. Both controlled-stress and controlled-strain loading modes were adopted in laboratory testing, to simulate different critical loading conditions in pavement structure. The choice of using different loading modes was also dictated by the need of highlighting material-related effects associated to the type of modifier and the presence of RAP. In fact, strain-controlled fatigue tests can better reflect the effect of polymer compounds while stress-controlled tests are more reflecting the overall material stiffness [45, 46]. Moreover, different loading modes may differently emphasize the impact of RAP on fatigue resistance of mixtures, even though no clear consensus has been reached among researchers [37, 40, 41]. For both stress- and strain-controlled fatigue tests, different failure criteria were considered in the analysis and interpretation of test data.

2 Materials and testing

2.1 Materials

The asphalt mixtures investigated in this study were produced in the laboratory by combining different base materials including virgin aggregates, RAP, two asphalt binders, a polymeric compound derived from plastic waste, and a bio-based rejuvenating agent.

Virgin aggregates were of siliceous origin and were collected from a local quarry located in north-west Italy in three distinct size fractions (0/5, 8/16, and 16/20). RAP was provided by the same local quarry and sourced from two stockpiles named RAP 0/12 and RAP 0/20. RAP fractions were incorporated in asphalt mixtures using 30% by weight of total aggregate of RAP 0/12 and 20% by weight of total aggregates for RAP 0/20. Binder content of RAP fractions was determined using the ignition method [47], indicating a percentage of 4.15% and 5.45% by weight of aggregates for RAP 0/12 and RAP 0/20, respectively. The asphalt binders were an unmodified 50/70 paving-grade bitumen (Binder *A*) and a highly SBS-modified 45/80-70 bitumen (Binder *B*). Binder *A* was used in the production of mixtures containing polymeric compound, while binder *B* was used in the production of control SBS-modified mixture assumed as reference. Both binders were preliminarily characterized to determine their penetration grade, softening point, and Performance Grade (PG). The results are summarized in Table 1.

The polymeric compound was a commercially available product, entirely composed of recycled materials derived post-consumer and post-industrial waste plastics. It was produced according to a special

Table 1 Characteristics of asphalt binders

	Performance grade	Penetration at 25 °C (dmm)	Softening point (°C)
Binder *A*	PG 64-22	70	48.1
Binder *B*	PG 76-22	55	80.3

Fig. 1 Polymeric compound derived from plastic waste and its main characteristics

Aspect	Granules
Colour	Shades of grey
Apparent Density at 25 °C	0.4 - 0.6 g/cm^3
Softening point	160-180 °C

Table 2 Properties of the rejuvenator

Aspect	Liquid
Colour	Brown—purple
Density at 25 °C	0.93 ± 0.1 g/cm^3
Viscosity at 25 °C	100 ± 50 cP
Flash point	> 200 °C
Water content	< 2%

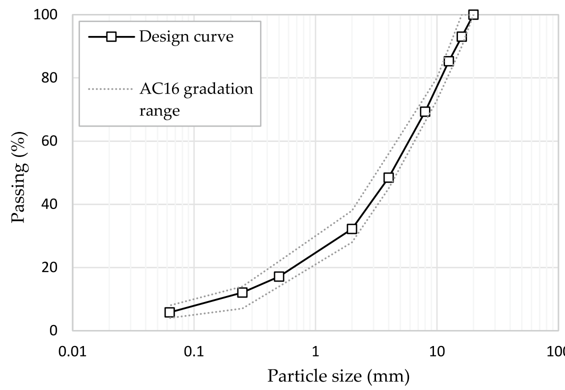


Fig. 2 Gradation limits and design curve of AC16 mixtures

processing scheme, including washing, separation, grinding, and pelletizing of waste plastics to obtain granules with a diameter of 4–6 mm. Although the compound is primarily composed of polyethylene (PE) and polypropylene (PP), its exact formulation remains unknown as it is a proprietary product. A visual representation of the compound along with its main characteristics are provided in Fig. 1. It is worth noting that the softening point of the compound is indicated between 160 °C and 180 °C. Since it does not exceed the mixing temperature by 30 °C, as recommended for fully dry modification [48], the incorporation of the compound of recycled plastics into the mixture is referred to hybrid approach in this paper.

The rejuvenator was a bio-based product, supplied by the same manufacturer of the polymeric compound. The main characteristics of the product are given in Table 2. The rejuvenator was applied at a single dosage of 0.3% by weight of the RAP, following manufacturer's recommendation. It was sprayed directly onto the RAP just before mixing, to ensure immediate interaction between the rejuvenator and the aged bitumen thus maximizing the effectiveness of the treatment [49].

The asphalt mixtures considered in this study were characterised by a nominal maximum aggregate size of 16 mm (AC16). The aggregates and RAP fractions were properly combined to achieve a dense-graded distribution curve that complies with gradation limits set by Italian technical specification assumed as reference [50] (Fig. 2).

Two test mixtures containing the polymeric compound at two different dosages (MIX 1 and MIX 2) and a control mixture containing SBS-modified binder (MIX REF) were produced in the laboratory.

Their job-mix formula and mixing procedure were determined from the mix design conducted in a previous research study described in [51]. The optimized compositions of the mixtures are summarized in Table 3.

It is worth noting that RAP contributed to 2.2% of the total binder in the final blend and all blends contained the same amount of virgin binder, equal to 2%. It is also worth noting that the rejuvenator and the polymeric compound were computed in the total binder content, in accordance with the hypothesis of considering them as binder extenders which actively contributed to the binding of aggregate structures.

Mixing procedure entailed virgin aggregate to be heated to a suitable temperature (280 °C) before being introduced into the mixer with RAP [52]. This last one was conditioned in an oven at 70 °C for a maximum of 2 h, in order to minimise additional ageing. The thermal shock caused by the contact between aggregates and RAP, allowed them to reach the target mixing temperature (170 °C) before adding the

Table 3 Composition of mixtures considered in the investigation

Mix code	RAP content* (%)	Binder type	Rejuvenator content by RAP (%)	Polymeric compound content** (%)	Virgin bitumen content** (%)	Total binder content** (%)
MIX 1	50	*A*	0.3	0.3	2.0	4.8
MIX 2		*A*		0.5	2.0	5.0
MIX REF		*B*		–	2.0	4.5

*by weight of RAP + aggregates, **by weight of total mixture

remaining components (virgin binder, filler, polymeric compound).

2.2 Testing

Mechanical characterization of asphalt mixtures involved performing flexural tests in 4 PB configuration for determination of their stiffness modulus and fatigue resistance.

Stiffness modulus tests were conducted as fingerprint tests under sinusoidal loading, following a procedure coherent with EN 12697-26 Annex B [53]. Measurements were carried out at temperature of 20 °C and at frequencies of 0.1, 0.2, 0.5, 1, 2, 5, 10, 20 Hz. All samples were conditioned at the target temperature for a minimum of four hours prior to testing to ensure consistency in results. To prevent specimen damage, the strain amplitude was limited to 50 micro-strains. Measurements were run in 26 replicates for each asphalt mixture.

Fatigue tests were carried out in both controlled-stress and controlled-strain loading modes, following a procedure coherent with EN 12697-24 [54].

All tests were conducted at 20 °C and 10 Hz, applying different stress and strain amplitudes to achieve a well-distributed dataset in a strain versus number of cycles plot. Specifically, 13 tests were performed for each asphalt mixture and loading mode, resulting in 78 tests overall. Stress amplitudes adopted in stress-controlled mode ranged in the intervals 1130–2000 MPa for MIX1, 1100–2000 MPa for MIX2, and 1200–2400 MPa for MIX REF. Strain amplitudes adopted in strain-controlled mode ranged in the intervals 145–320 μm/m for MIX 1, 164–250 μm/m for MIX 2, and 150–320 μm/m for MIX REF. These intervals allowed yielding number of cycles to failure (N_f) ranging from 10,000 to 3,000,000 cycles, with a few exceptions beyond this range.

Stiffness modulus and fatigue tests entailed using prismatic specimens measuring 50 × 50 × 410 mm. Specimens were obtained by means of sawing and cutting operations from larger asphalt slabs of 500 × 300 × 70 mm size, compacted using an electromechanical roller segment compactor with a head radius of 535 mm. Four beams were obtained from the core of each slab. The weight of material employed to produce slabs was carefully adjusted to achieve a target air void content of 4% ± 0.5%. Actual air voids of specimens were determined before testing by calculating the bulk density of each saturated surface dry beam according to EN 12697-8 [55]. Figure 3 presents the average air void contents along with the corresponding standard deviations obtained from measurements. The results indicate that the specimens of hybrid-modified asphalt mixtures exhibited slightly higher void content compared to those of control mixture containing SBS-modified binder. This can be attributed to the presence of the plastic additive: due to its plastomeric nature, plastic made the binder phase of the mixtures stiffer with consequent creation of more voids in the compacted specimens.

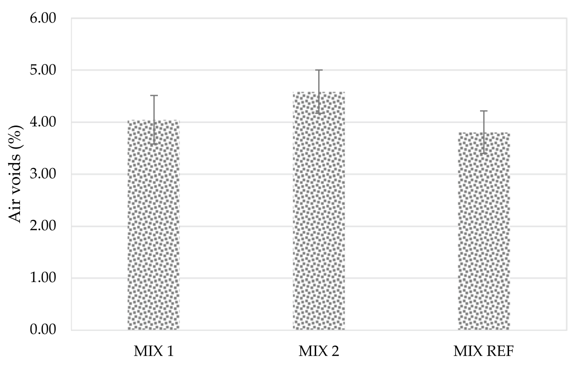


Fig. 3 Mean values of air void content of compacted asphalt mixtures

3 Fatigue life criteria

Fatigue analysis plays a key role in assessing the performance of asphalt mixtures, as fatigue represents one of the primary failure modes of road pavements [56]. Fatigue life can be defined as the capability of an asphalt mixture to withstand cyclic tensile loads (repeated traffic) without developing significant cracking [57]. Several factors influence the evaluation of fatigue properties of asphalt mixtures in the laboratory, including specimen geometry and compaction, testing type, loading mode, ageing, temperature, and air voids content [58].

Fatigue testing typically presents three distinct phases as depicted in Fig. 4a [59, 60]. Phase I, known as the adaptation phase, is characterised by a rapid decline in stiffness modulus. While fatigue damage begins to develop during this phase, its progression is largely governed by external factors such as specimen heating and binder thixotropy, which influence the initial response of the material. At this stage, microcracks appear in the specimen. Phase II, known as the quasi-stationary phase, is characterised by the predominance of fatigue damage, with the stiffness modulus decreasing in an almost linear manner. While external effects remain present, they become secondary in influence [61]. Moreover, their impact varies depending on the type of test: in beam tests, these effects are less pronounced due to the non-uniform distribution of stress and strain within the specimen [56]. During this phase, micro-cracks propagate within the material. Phase III, referred to as the failure phase, is marked by a sharp decline in the stiffness modulus. In controlled-tension tests, this reduction ultimately leads to specimen failure. At this stage, micro-cracks coalesce into macro-cracks, which propagate and result in the physical rupture of the specimen [62].

Different criteria can be identified in literature to estimate fatigue life, broadly categorized into empirical and energy-based criteria.

The most widely used empirical criterion is the number of cycles $N_{f,50}$, corresponding to a 50% reduction in the initial stiffness modulus S_0 [63]. According to EN 12697-24 D [54], i_0 is established as the average stiffness modulus between cycles 98 and 102. Several studies [61, 64] have shown that this criterion may underestimate fatigue life, especially in modified asphalt mixtures.

From an energetic perspective, asphalt materials display a hysteresis loop in a stress–strain diagram under cyclic loading. Due to the viscoelastic nature of asphalt, the unloading path deviates from the loading path, and the area enclosed by the loop represents the dissipated energy. This energy dissipation occurs in the form of heat and material degradation, contributing to the overall fatigue damage of the specimen [65].

The dissipated energy per cycle is computed using the following equation:

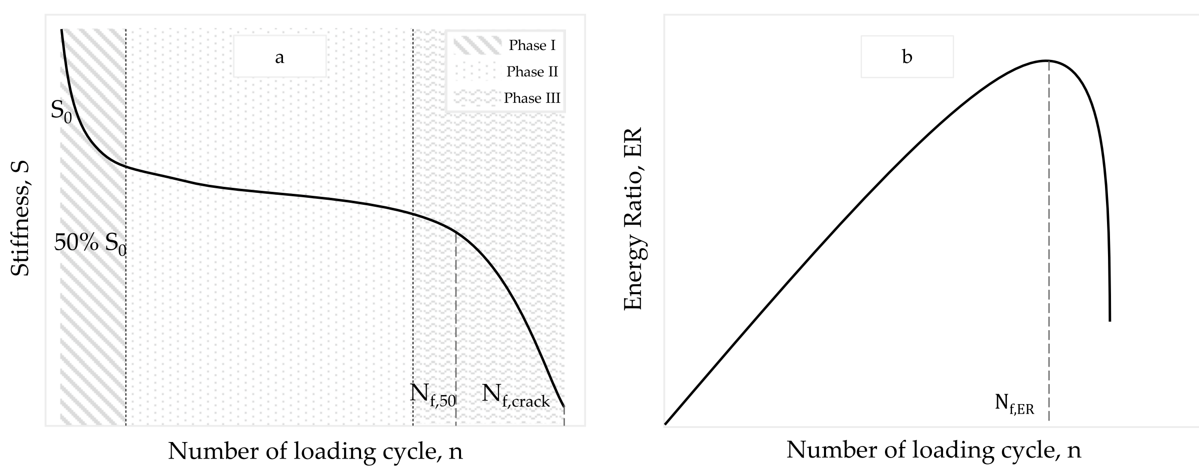


Fig. 4 **a** Evolution of stiffness as a function of number of loading cycle; **b** Evolution of energy ratio as a function of number of loading cycle

$$W_{\mathrm{i}} =_{\mathrm{i}} \varepsilon_{\mathrm{i}} \pi \sin\left(_{\mathrm{i}}\right) = \varepsilon_{\mathrm{i}}^{2} E_{\mathrm{i}} \pi \sin\left(_{\mathrm{i}}\right) = \frac{\overset{2}{\mathrm{i}}}{E_{\mathrm{i}}} \pi \sin\left(_{\mathrm{i}}\right) \quad (1)$$

where W_i represents the dissipated energy in the ith cycle, σ_i and ε_i denote the stress and the strain levels, respectively, E_i indicates the stiffness modulus and φ_i is the phase angle. In strain-controlled tests, W_i decreases progressively cycle by cycle [66]. Throughout the loading history, the hysteresis loop undergoes shape modifications. Several studies [67–69] attribute these changes to material damage, highlighting the correlation between dissipated energy and fatigue degradation.

Based on previous works of Van Dijk and Visser [70, 71], Hopman et al. [72] proposed a fatigue failure criterion based on the number of cycles ($N_{f,ER}$) at which crack initiation is considered to occur. They introduced the energy ratio (ER) as the ratio of the product of the initial dissipated energy W_0 and the current cycle number n to the dissipated energy at that cycle W_n:

$$\mathrm{ER} = \frac{n \cdot W_0}{W_n} \quad (2)$$

By substituting Eq. 1 in Eq. 2, removing the constant terms, and considering that the variation in sin φ is negligible compared to the change in the complex modulus [73], ER can be expressed just in terms of the number of cycles and the complex modulus, as defined in Eq. 3 and Eq. 4 for stress-controlled and strain-controlled tests, respectively:

$$\mathrm{ER} = n \cdot E_i \quad (3)$$

$$\mathrm{ER} = \frac{n}{E_i} \quad (4)$$

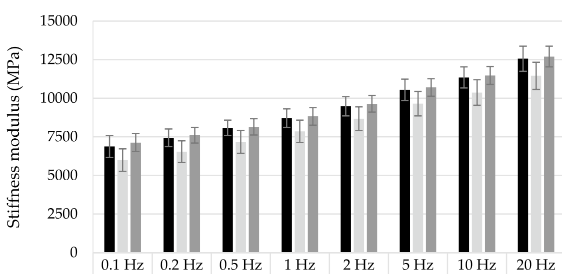
In stress-controlled tests, fatigue life can be defined as the peak in the ER vs number of cycles curve, as illustrated in Fig. 4b. Conversely in strain-controlled tests, the ER vs number of cycles diagram displays a monotonically increasing trend, with fatigue life traditionally identified at the cycle where the curve deviates from the initial linear progression [72]. However, such an approach remains arbitrary and relies on engineering judgment [74]. To address this limitation, Rowe and Bouldin [75] proposed extending the ER criterion used in stress-controlled tests to strain-controlled tests, providing a more consistent and objective methodology for fatigue life determination across different loading conditions.

In this study, both empirical $N_{f,50}$ and energy-based $N_{f,ER}$ were determined from fatigue test data. In the specific case of controlled-stress mode tests, the number of cycles $N_{f,crack}$ corresponding to the physical rupture of specimen was also considered.

4 Experimental results

Figure 5 displays the values of stiffness modulus at 20 °C obtained from fingerprint testing carried out on the asphalt mixtures investigated in the study. The table below the graph reports the mean values at each frequency, while the error bars represent the corresponding standard deviations.

MIX 1 showed stiffness values approximately 11% higher than MIX 2, whereas MIX REF exceeded MIX 2 by up to 20%. The stiffness of MIX 1 and MIX REF are very similar to each other at any frequency, with a slight predominance of MIX REF. Due to higher content of added plastics in MIX 2, this last one was expected to exhibit higher moduli compared to MIX 1. Such deviation from expected outcome can be attributed to the higher air void content of MIX 2, as reported in previous Fig. 3. The similarity in air voids of MIX 1 and MIX REF, which is reflected on the similarity of corresponding stiffness moduli, suggested the prominent effect of air void of compacted specimen on their stress–strain response under external loading. Eskandarsefat et al. [45] emphasized the critical role of stiffness modulus during fatigue testing, since stiffer mixtures require higher forces to



	0.1 Hz	0.2 Hz	0.5 Hz	1 Hz	2 Hz	5 Hz	10 Hz	20 Hz
■ MIX 1	6882	7442	8084	8714	9482	10551	11343	12558
■ MIX 2	5993	6538	7178	7858	8672	9650	10369	11452
■ MIX REF	7130	7612	8145	8824	9640	10698	11477	12698

Fig. 5 Stiffness moduli at 20 °C of investigated asphalt mixtures

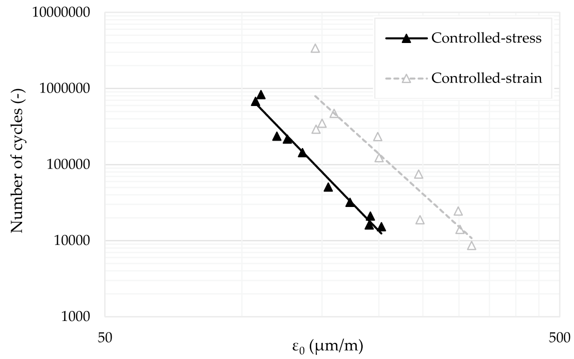


Fig. 6 Fatigue life versus initial strain for MIX1 under different control loading modes

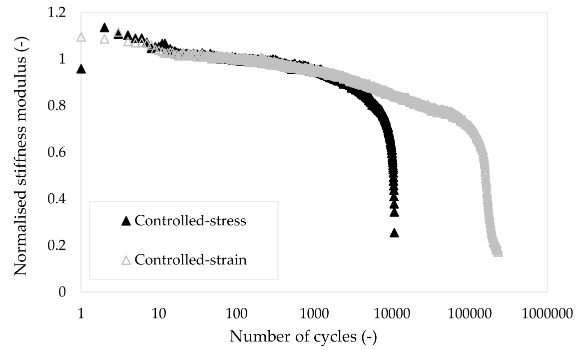


Fig. 7 Evolution of normalized stiffness modulus as a function of loading cycles under different control loading modes (MIX 2 in the example tested at initial strain of 200 μm/m)

obtain the same initial strain level, and this may especially impact Phase I of material response.

Figure 6 displays the results of fatigue tests gathered for MIX 1, plotted in terms of fatigue life $N_{f,50}$ versus initial strain ε_0 (measured at 100th cycle) in the log–log scale. The two trend curves corresponding to the two different controlled loading modes are clearly distinguished from each other. The curve obtained in controlled-strain mode is characterized by a higher variability in test results and appears to be shifted horizontally towards the right side of the diagram, indicating an increase in fatigue life compared to that determined by the controlled-stress mode for any level of initial strain applied. The relative difference in fatigue life between the two controlled loading modes is essentially the same regardless of the criterion used.

The same outcome was observed for the other two mixtures (MIX 2 and MIX REF). This outcome, coherent with expectations, can be explained by considering the evolution of stress–strain response exhibited under cycling loading in the two different modes. Referring to the example reported in Fig. 7, which shows the normalised stiffness modulus as a function of load repetitions obtained from two tests with the same initial deformation levels imposed, it can be observed that the change rule of modulus under controlled-stress and under controlled-strain was almost coincident up to a certain number of cycles. Beyond this point, the behaviours began to diverge. Under controlled-stress, the modulus decreased more quickly until the complete rupture of test specimen; in the case of controlled-strain mode, the rate of change was lower, and this slowed down the degradation of the material. This phenomenon can be detected also in binder fatigue testing and the threshold point corresponding to the divergence between two controlled loading modes generally depends on test conditions [76].

Figure 8 compares the results obtained for the different mixtures tested in the controlled-stress mode. The three graphs, represented in the form of Wohler's diagrams, refer to the three different fatigue life criteria considered.

By taking into account deviations of individual test results, the trend lines only provide an estimate of the behaviour of considered materials. Even though the differences are not huge, some interesting considerations can be made. The control mixture containing the SBS-modified binder exhibited slightly higher fatigue lives than the hybrid-modified mixtures. The fatigue behaviour of MIX 1 and MIX 2 differed as a function of the applied strain: at low levels of initial strain, the mixture with a higher dosage of polymer compound demonstrated slightly better fatigue resistance, whereas this trend was reversed at higher levels of strain. A crossing zone between the curves emerges between 140 and 150 μm/m, depending on the failure criterion considered. MIX 2 manifested the steepest slope in absolute value, indicating the greatest sensitivity to cyclic loading. Slopes showed by MIX 1 and MIX REF were very similar.

Figure 9 compares the results obtained for the different mixtures tested in the controlled-strain mode. In this case, only $N_{f,50}$ and $N_{f,ER}$ criteria have been considered in the analysis.

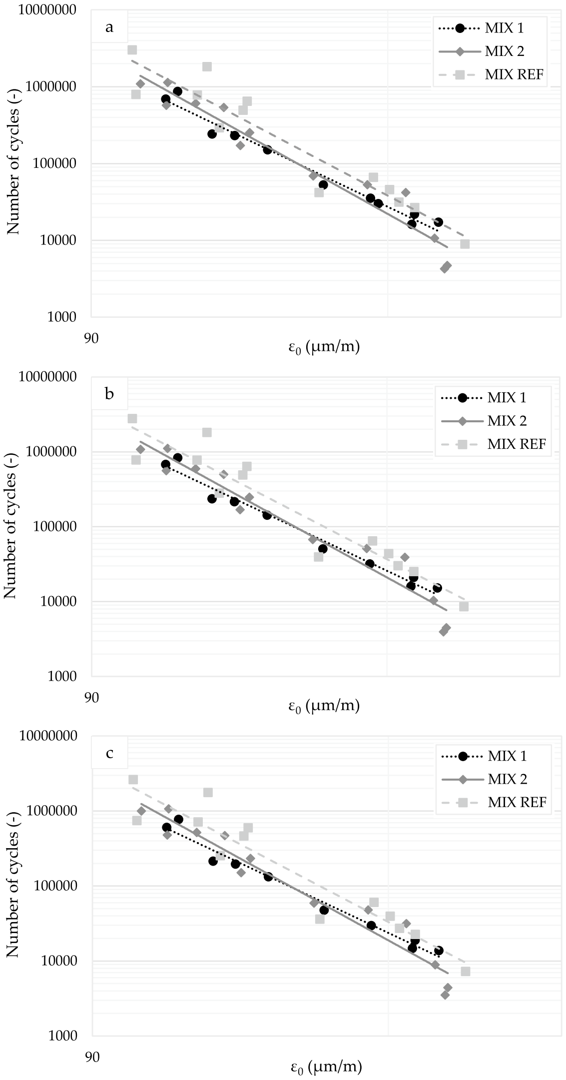


Fig. 8 Number of cycles to failure versus ε_0 in controlled-stress mode: **a** $N_{f,crack}$, **b** $N_{f,50}$, **c** $N_{f,ER}$

In general terms, data point trends allow to make a clear distinction between MIX 2 and other two mixtures, while MIX1 and MIX REF showed a more similar response when compared to each other.

More in detail, MIX 1 was characterised by the least slope, indicating the lowest susceptibility to fatigue degradation, while the highest slope value was exhibited by MIX 2. Also in this case, a crossover zone between MIX 1 and MIX 2 is observed at approximately 200–205 μm/m, depending on the failure criterion considered. Similarly, a crossover between MIX 2 and MIX REF is observed at around

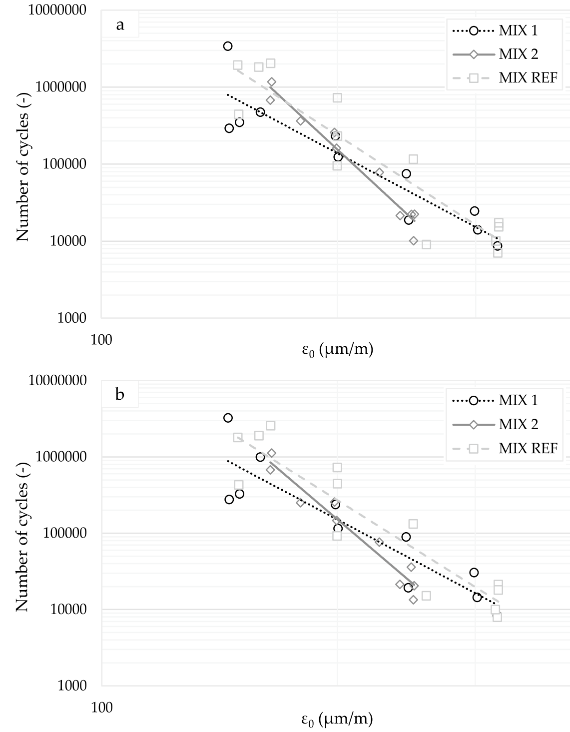


Fig. 9 Number of cycles to failure versus initial strain in controlled-strain mode: **a** $N_{f,50}$, **b** $N_{f,ER}$

170 μm/m for $N_{f,50}$ and 155 micro-strains for $N_{f,ER}$. As a consequence of the observed trends, the ranking order in of the material depends on the fatigue regions that can be identified in the Wohler's diagram. In the low cycle fatigue domain, that corresponds to high levels of stress imposed, MIX 1 behaved very similarly to MIX REF showing numbers of cycles at failure higher than MIX 2. Usually, low cycle fatigue is associated to localized plastic deformation around pre-existing defects within the material or at the tips of crack when it has already been initiated [77]. In the higher cycle fatigue domain corresponding to lower stresses, MIX 2 showed higher number of cycles to failure than MIX 1 and similar values to MIX REF, with a tendency to outperform this last one for even higher cycles. The cyclic loading in high cycle regime typically induces small elastic strain, with crack initiation often occurring at microstructural defects [77].

This outcome can be explained by considering that at high levels of initial strain, material stiffness-related effects are preponderant as they contribute to amplifying localized stress concentrations. In the present case, MIX 1 and MIX REF exhibited higher stiffness which was reflected in lower stress concentrations, thus resulting in better performance in the low cycle regime. Conversely, at low levels of imposed strain the response was more affected by the stiffness of the binder phase than by bulk volume effects. It can be argued that the higher dosage of plastomeric compound in the MIX 2 reasonably resulted in a stiffening of the binder mastic that was reflected in increased fatigue life of the material. It is worth noting that pavement design is generally set to achieve low levels of strains at the bottom of bound layers and to keep them within linear visco-elastic region of the materials.

It can be also hypothesized that plastic compound may form structural networks within the binder matrix, and this may influence fatigue resistance of mixture differently depending on loading mode. This aspect deserves to be better investigated in future by means of micro-structural analysis.

Figure 10 shows the fatigue life parameter $N_{\mathrm{f,ER}}$ plotted as a function of $N_{\mathrm{f,50}}$. All data points follow a common trend with a strong relationship between two criteria. This indicates that no significant difference exists between ranking orders of materials based on one criterion or another.

The relationship between number of cycles at failure $N_{\mathrm{f,50}}$ and initial level of tensile strain ε_0 in the log–log scale can be described by the following equation:

$$\log N_{\mathrm{f,50}} = A_0 + A_1 \cdot \log \varepsilon_0 \tag{5}$$

where the parameters A_0 and A_1 are the fitting coefficients determined from linear regression analysis. An additional analysis on residuals have been performed to identify potential outliers. Residuals can be defined as the difference between experimentally observed values and those predicted by the model. Outliers were detected and removed using the interquartile range method to ensure data consistency [78]. Such a statistical methodology identified four tests to be removed from the investigated dataset. Since the number of excluded tests is low, this supports the reliability of the obtained data, which exhibits low dispersion relative to the identified fatigue lines.

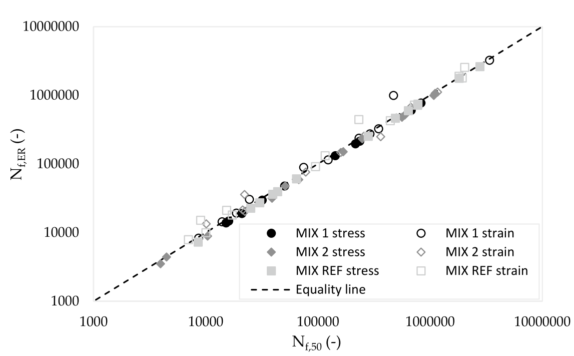


Fig. 10 $N_{\mathrm{f,50}}$ versus $N_{\mathrm{f,ER}}$

Fatigue regression coefficients were used to determine ε_6, which represents the strain value corresponding to 1,000,000 cycles at failure. Such a parameter is commonly used as performance indicator to rank fatigue resistance of asphalt mixtures.

Table 4 summarizes the ε_6 values along with the corresponding regression coefficients obtained from the analysis. The goodness of the fit, expressed in terms of R^2 parameter, is also reported.

The results first confirm that the various criteria are essentially equivalent, as they led to very similar values of ε_6 for each mixture.

In the case of the controlled-stress tests, MIX REF and MIX 2 showed very similar performance with a slight prevalence of the first one. The gap is actually very small, even compared to MIX 1: in fact, the relative difference between the hybrid mixture with the lowest compound content and the reference mixture is of order of 10%.

The mixtures MIX REF and MIX 2 showed a very similar response also in controlled-strain tests, but in this case MIX 2 slightly outperform the reference one with ε_6 value well above 160 µm/m. The gap with MIX 1 is more pronounced than in controlled-stress mode, with ε_6 value still around 140 µm/m.

Table 4 ε_6 values and regression parameters for considered asphalt mixtures

Loading mode	Mix code	$N_{\mathrm{f,50}}$			
		ε_6	A_0	A_1	R^2
Controlled—stress	MIX 1	98	18.36	−6.18	0.98
	MIX 2	107	20.58	−7.21	0.94
	MIX REF	110	19.86	−6.78	0.90
Controlled—strain	MIX 1	140	17.64	−5.43	0.85
	MIX 2	164	26.92	−9.44	0.95
	MIX REF	161	20.58	−6.61	0.86

It is worth noting that ε_6 levels above 120–130 μm/m at 20 °C are usually taken as indicative of good fatigue performance in the field for standard dense graded asphalt mixtures [79, 80]. Despite the presence of a high amount of RAP, the optimized composition of the mixtures allowed yielding adequate fatigue lives for all of them.

A final observation can be made on R^2 coefficient. R^2 values were found to be higher than 0.9 for tests conducted in stress-controlled mode, indicating a high goodness of fit. In the case of strain-controlled tests, R^2 values obtained for two mixtures (MIX1 and MIX REF) were slightly lower. This outcome reveals that strain-controlled mode was characterized by greater variability of test data with respect to stress-controlled mode.

5 Summary and conclusions

This experimental study compared the fatigue properties of different modified asphalt mixtures containing 50% RAP. Two mixtures (MIX 1 and MIX 2) modified with recycled plastics and a reference mixture (MIX REF) modified with SBS polymer were investigated. Compositions of the mixtures, in terms of aggregate and RAP fraction proportions, rejuvenator dosage, polymer compound and binder content, had been previously optimized based on a laboratory mix design. Mixtures were subjected to 4 PB fatigue tests at 20 °C and 10 Hz. Tests were carried out in controlled-stress and controlled-strain modes in order to consider the response of materials under different critical loading modes in pavement structures. Before conducting fatigue tests, stiffness modulus tests were also performed to characterise the stress–strain response of prismatic beams under cyclic loading.

The experimental results indicated that the use of high percentages of RAP combined with the use of recycled plastics resulted in adequate fatigue performance of mixture. The experimental data also indicated that recycled plastics used in this study yielded fatigue performance comparable or even slightly higher to that obtained with the use of SBS-modified binder.

Specific conclusions from the study can be drawn as follows:

- MIX 2 containing 0.5% polymer compound showed lower stiffness than MIX 1 containing 0.3% and control MIX REF, at all frequencies investigated,
- MIX 2 showed superior fatigue performance, expressed in terms of ε_6, with respect to MIX 1. The relative difference between ε_6 values were about 6% in controlled-stress mode and 18% in controlled-strain mode,
- MIX 2 and MIX REF prevailed over one another depending on the loading mode. Specifically, in controlled-stress mode MIX REF exhibited slightly higher performance, while in controlled-strain mode MIX 2 slightly outperformed the reference one,
- a very strong correlation between $N_{f,50}$ and $N_{f,ER}$ failure criteria was found. As a consequence, their use led to the same ranking order of the materials,
- fatigue tests conducted in strain-controlled mode were characterized by a greater variability of data with respect to tests conducted in stress-controlled mode, as revealed by corresponding R^2 values obtained from regression analysis.

Overall, the outcome of the experimental work highlights the great potential in terms of fatigue performance of hybrid-modified asphalt mixtures containing high amount of RAP and waste plastics. This appears very promising in the perspective of full-scale development of such technology, as the environmental benefits deriving from the use of recycled materials are not jeopardized by reduced service life of the pavement.

However, further experimental work is needed to support the conclusions of the study. This should entail expanding the array of materials to include plastics waste and RAP of different types and origins. The effects of ageing and their implications on long-term fatigue performance also deserve to be investigated in future work. Finally, the results obtained from laboratory testing should be validated with field testing data.

Acknowledgements Loretta Venturini and Carlo Carpani are warmly acknowledged for their technical support in providing some of the testing materials and valuable discussion of the results.

Funding Open access funding provided by Politecnico di Torino within the CRUI-CARE Agreement. This publication is part of the project PNRR-NGEU which has received funding from the MUR –DM352/2022.

Data availability Data sets generated during the current study are available from the corresponding author on reasonable request.

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