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Research article

Reuse of waste tennis strings as recycled polymer fibers in asphalt concrete

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ABSTRACT

The reuse of end-of-life polymer-based products as secondary raw materials is an effective strategy to reduce waste and promote more sustainable material systems. In this study, waste tennis strings were investigated as recycled polymer fibers for asphalt concrete, in order to assess their suitability as a reinforcing phase in a composite paving material. An experimental program was conducted to evaluate the effect of the recycled fibers on the mechanical and performance-related properties of asphalt concrete. Stiffness, fatigue resistance, and fracture behavior were examined to determine the influence of fiber addition on the response of the material. The results showed that waste tennis-string fibers can beneficially modify the mechanical behavior of asphalt concrete, with particularly significant improvements in fracture-related performance and a generally positive effect on selected properties. These findings indicate that waste tennis strings can be successfully valorized as reinforcing fibers in asphalt concrete, offering a technically viable recycling route for a post-consumer polymeric waste. More broadly, the study demonstrates the potential of unconventional recycled fibers to enhance the performance of engineering composite materials while contributing to more sustainable resource use.

1. Introduction

The issue of waste disposal is currently recognized as one of the most significant challenges faced worldwide by developed and emerging countries [1–4]. In particular, the industry has experienced a remarkable increase in plastic production, rising globally from approximately 2 million tons in 1950s to 400 million tons in 2022 [5,6], with an unavoidable consequent increase in waste to be disposed of. The use of recycled plastics in asphalt pavements has been extensively analyzed in several research projects over the years, starting from the late 1980s [7,8]. Such a recycling possibility has increasingly attracted attention from transportation agencies due to the combined environmental, economic and technical benefits which stem from preventing plastic pollution, reducing production costs, and enhancing pavement performance [9–18].

The approaches adopted to add plastic waste in the production of asphalt mixtures rely on the so-called wet and dry processes. The wet process involves pre-blending of plastic waste with bitumen, followed by mixing of the resulting binder with aggregates, whereas in the dry process, plastic waste is mixed with aggregates before adding the binder during asphalt mixture production.

The performance of asphalt mixtures modified with waste plastic has been reported to be quite promising, with several studies showing improvements in stiffness modulus [14,19], workability and compactability [8], moisture sensitivity [16,18], fracture and fatigue resistance [14,16,19], and anti-rutting potential [16,18]. However, the type of recycled plastics (which are characterized by a non-negligible variability in composition and melting point) and their dosage significantly affect mixture behavior. As an example, Baghaee Moghaddam et al. [19] observed that stiffness modulus of mixtures increased with lower plastic contents, while Pasetto et al. [14] found that higher dosages of waste plastic resulted in greater stiffness and permanent deformation resistance. Nevertheless, in most cases reported in the literature it was highlighted that this plastic recycling solution provides outstanding environmental, economic, and social benefits [9,10,18].

Numerous studies have demonstrated notable enhancements in asphalt mixture properties when waste plastics are added in the form of fibers [20–29]. Kim et al. [22] observed that the inclusion of nylon fibers enhanced mechanical characteristics (including Marshall stability, indirect tensile strength, and moisture susceptibility), while Yin et al. demonstrated [23] that recycling waste nylon fibers led to specific improvements in crack resistance. While investigating the use of recycled

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Table 1
Gradation of employed aggregates (by mass).

Sieve size (mm)	Passing (%)		
	Sand 0/2 mm	Sand 0/5 mm	Crushed gravel 5/15 mm
16	100.0	100.0	100.0
10	100.0	100.0	64.9
4	99.7	92.2	7.6
2	99.3	69.3	4.9
0.5	71.7	27.1	2.2
0.25	52.8	15.7	1.6
0.063	26.6	4.4	1.1

polyester fibers, Xia et al. [29] observed a noteworthy improvement in the fatigue life of pavements incorporating such materials. Consistent findings were reported by Zarie et al. [25], who evaluated the fracture energy of asphalt mixtures at low and intermediate temperatures, which improved significantly in the presence of polyester fibers. Similarly, Qiu et al. [28] achieved comparable results by investigating the fracture properties, assessed both in the pre-cracking and post-cracking phase, by using fracture mechanics approaches and by utilizing polyester fibers and calcium lignosulfonate powder.

One of the largest sources of waste plastic production is the sports industry, and in particular this is the case of tennis strings. Research in such a domain has shown that the loss of tension occurring in strings over time calls for frequent replacements [30], thereby leading to the generation of waste filaments the amount of which is massive as a result of the high number of players worldwide (87 million in 2021 according to the International Tennis Federation, expected to reach 120 million by 2030 [31]). Thus, sustainable solutions and recycling initiatives are urgently needed to mitigate the environmental impact associated to tennis strings disposal [30,32]. Typical composition of commercially available tennis strings includes polyester, nylon, natural gut, polyolefin, and Kevlar [30,33]. As indicated by Yusuf et al. [30], approximately 60% of tennis strings are in polyester, while 30% are composed of nylon.

Although previous studies have investigated the use of polyester and nylon fibers in asphalt mixtures, no research has specifically addressed the reuse of end-of-life tennis strings as a reinforcing component in asphalt concrete. This gap is relevant because waste tennis strings do not simply represent another source of polymeric fibers, but a specific post-consumer material characterized by its own morphology, geometry, and practical variability, which may influence its interaction with the asphalt mixture. Therefore, a preliminary research study was initiated to assess the impact of waste tennis string fibers (WTSFs) on the performance of asphalt mixtures. In particular, the experimental investigation described in this paper focused on the characterization of dense-graded asphalt mixtures in which WTSFs were hypothesized to form an internal reinforcing network. The testing program included the evaluation of workability, stiffness, fatigue resistance, and fracture properties. Based on the obtained results, the role of WTSFs as a recycled reinforcing component in asphalt mixtures was critically evaluated and discussed.

2. Materials

Asphalt mixtures considered in the experimental investigation were prepared by combining selected mineral aggregate fractions of different sizes with a commercially available neat bitumen and with a mixture of WTSFs obtained from damaged tennis strings ready to be disposed of.

2.1. Components materials

The employed bitumen was a 50–70 penetration grade neat binder as per EN 12591 [34]. Mineral aggregates included sand, supplied in two size fractions (0/2 mm and 0/5 mm), and crushed gravel (5/15 mm). Gradation (as per EN 933–1 [35]) and apparent density (as per EN 1097–6 [36]) of the aggregate fractions are shown in Table 1 and Table 2, respectively.

Table 2
Density of employed aggregates.

	Sand 0/2 mm	Sand 0/5 mm	Crushed gravel 5/15 mm
Apparent density (Mg/m^3)	2.886	2.773	2.820

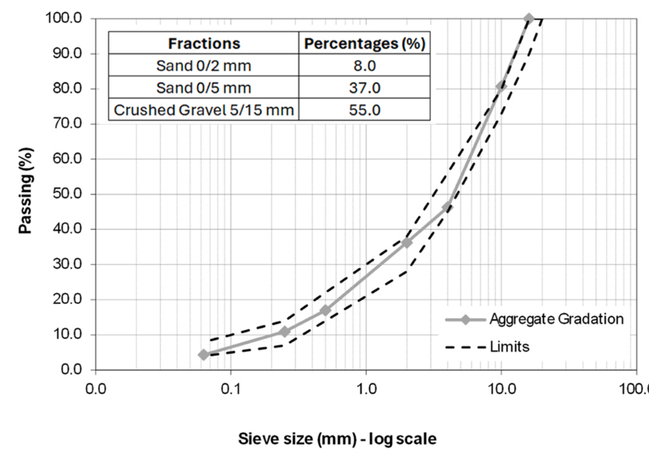

Fig. 1. Target aggregate gradation compared to upper and lower limits of technical specifications.

Damaged tennis strings, sourced from a local stringer, were of the monofilament type. Although detailed technical information on such strings is not available due to patent cover, the supplier reported that they are essentially made of polyester, a synthetic polymer derived from petroleum-based materials, and they present a heptagonal cross-section with a diameter ranging between 1.15 mm and 1.30 mm.

Length of the available waste strings was variable, depending upon the cutting operations to which they were subjected by the stringer (approximately 1.5 m per filament). Density was found to be equal to 1.380 Mg/m^3. Based on information which can be found in the literature [32,37–39] and on preliminary testing (carried out by keeping the material in the oven at different temperatures for 60 min), melting temperature was estimated to be above 275 °C. Such a finding suggests that fibres obtained from these strings will not melt whenever included in the production of asphalt mixtures at common mixing temperatures.

As indicated by Guo et al. [26], the length of reinforcing fibers is a factor that can have a significant influence on the performance of asphalt mixtures. Based on data gathered from the literature [22,26,37,38,40–48], fiber lengths commonly adopted for asphalt-mixture reinforcement typically range from approximately 6 mm to 25 mm. Therefore, the available waste strings were manually cut into fibers approximately 10 mm to 15 mm in length, which was considered a reasonable preliminary compromise between adequate dispersion within the dense-graded AC16 mixture and the ability of the fibers to act as reinforcing elements. In fact, excessively long fibers may hinder homogeneous distribution and interfere with the aggregate skeleton, whereas excessively short fibers may reduce the reinforcing effect. In the present study, fiber length was intentionally kept fixed within a preliminary range in order to focus on the feasibility and mixture-scale mechanical response of WTSFs.

Considering the length and the diameter range, the estimated aspect ratio of the WTSFs was approximately 8–13. Although tensile strength and detailed surface morphology would be useful to further clarify the reinforcing mechanisms, such information was not available in the present preliminary study because detailed technical data on the strings were not accessible due to patent restrictions.

Table 3
Average results of volumetric mix design.

Mixture	P_b (%)	TMD (Mg/m^3)	v (%)	VMA (%)	VFB (%)
RM	5.0	2.527	4.0	15.4	74.1
F0.3	5.3	2.524	4.0	16.0	74.9
F0.5	5.3	2.514	4.0	16.0	75.0

2.2. Asphalt mixtures

Composition of the asphalt mixtures was defined in accordance with the technical specifications commonly adopted in Italy for AC 16 dense-graded mixtures. The target particle size distribution was obtained by using the relative percentages of mineral fractions reported in Fig. 1, which also shows the resulting aggregate distribution, plotted against the acceptance gradation band suggested for AC 16 mixtures.

Waste tennis string fibers (WTSFs) were added to the other mixture components by means of the dry process, which is the most convenient option to avoid inhomogeneities in the mixtures when the melting point of fibres is significantly higher than mixing temperature [49,50]. Based on data retrieved from the literature [38,40,41,43–45,50–52], three different WTSF contents were selected, equal to 0.3%, 0.5%, and 1% by the weight of aggregates, and the resulting mixtures were labelled as F0.3, F0.5, and F1.0, respectively. The selected dosages were conceived as a preliminary literature-based range for feasibility assessment, including low, intermediate, and relatively high content, in order to explore both the reinforcing potential of the fibers and the possible onset of volumetric/compaction limitations. An AC 16 dense-graded mixture containing no WTSFs (indicated as RM) was also prepared for comparative purposes.

Mix design of the selected asphalt mixtures was carried out in accordance with the volumetric procedure described in AASHTO R 35 [53], whereby specimens of each mixture with different binder contents were compacted by means of the gyratory technique [54]. In agreement with the abovementioned technical specifications, target void content was set equal to 4.0%, and specimens were compacted at 100 gyrations (N_{design}). Compaction was carried out after the short-term conditioning procedure suggested by AASHTO R 30, which recommends a pre-compaction conditioning time of 2 h in an oven set at 135°C [55]. Average results obtained in the mix design stage are provided in Table 3, in which they are expressed in terms of design binder content P_b (by weight of aggregates and, when included, of fibres), theoretical maximum density (TMD), voids content (v) at N_{design}, voids in mineral aggregate (VMA) at N_{design}, and voids filled with bitumen (VFB) at N_{design} [56–58].

It is worth mentioning that the F1.0 mixture revealed clear practical issues during compaction. In fact, different binder contents ranging from 4.5% to 6.0% were considered, but in all cases the resulting specimens exhibited relatively high air-void contents, ranging from 5.8% to 8.6%. Therefore, target volumetric conditions could not be achieved, and the mixture remained outside the acceptable range defined by the Technical Specifications commonly adopted for this type of mixture, whose upper limit is 5% voids. This behavior is most probably due to the fact that the high fiber dosage led to the creation of an excessively stiff network that interfered with the densification process, as also reported in previous studies [39,41,59]. Consequently, the F1.0 mixture was not considered for further mechanical testing.

As expected, the TMD of asphalt mixtures decreased as fiber dosage increased because of the density of fibers, which is significantly lower than the density of aggregates. Furthermore, it can be observed that the inclusion of fibers led to higher design binder contents, due to the increase of the surface area caused by their inclusion in the mixtures.

Since the F0.3 and F0.5 mixtures exhibited similar mix design results, only one of them was selected for the evaluation of mechanical properties. In particular, since the final goal of the study was to assess the potential of using relevant quantities of recycled materials in asphalt mixtures, the selected mixture was F0.5, the one characterized by the higher WTSF content while still satisfying the volumetric requirements. As previously mentioned, the RM mixture was also included in the investigation for comparative purposes.

3. Methods

3.1. Preparation of specimens

Both asphalt mixtures considered in the investigation (RM and F0.5) were prepared in a high-capacity automatic laboratory mixer, by adopting the procedure described in the following, which ensures achievement of an adequate homogeneity:

(i) preheating aggregate fractions and bitumen at mixing temperature (165 °C);
(ii) mixing the preheated aggregate coarser fractions (sand 0/5 mm and crushed gravel 5/15 mm) at low speed for 30 s;
(iii) pouring one third of the design binder quantity, followed by mixing at low speed for 30 s;
(iv) adding half of the finest aggregate fraction (sand 0/2 mm) and, when included, half of the fibers (stored at the room temperature), followed by mixing at low speed for 30 s;
(v) pouring one third of the design binder quantity, followed by mixing at low speed for 30 s;
(vi) adding the second half of the finest aggregate fraction (sand 0/2 mm) and, when included, the second half of the fibers, followed by mixing at low speed for 30 s;
(vii) pouring one third of the design binder quantity, followed by mixing at low speed for 30 s;
(viii) final mixing at high speed for 3 min.

During the different mixing stages, it was clearly observed that WTSFs, as expected, did not melt.

For the assessment of mechanical properties, samples of both mixtures were compacted at 150 °C by means of the gyratory technique, applying a number of gyrations that allowed the target 4% voids to be achieved.

3.2. Compactability

The influence on compactability of the inclusion of WTSFs in the considered asphalt mixture was assessed by means of the compaction energy indices *CDI* (Construction Densification Index) and *TDI* (Traffic Densification Index), introduced by Bahia et al. [60]. The *CDI* is defined as the area under the *N*-%*C* compaction curve (where *N* is the number of gyrations and %*C* is percent compaction) from the point of initial densification (identified at 8 gyrations) to the number of gyrations corresponding to 92% compaction. The *TDI* corresponds to the area comprised between 92% and 96% compaction. In their original definition, *CDI* was hypothesized to be associated to the energy required for compaction during construction, while *TDI* was assumed to be related to the effort necessary for the densification process of asphalt mixtures occurring under traffic loadings until reaching the target voids content of 4%.

3.3. Stiffness

Stiffness characterization was performed in the indirect tensile configuration accordingly to EN 12697–26, Annex C [61]. This method provides a measurement of the so-called stiffness modulus (also indicated as resilient stiffness) by subjecting a cylindrical specimen of asphalt mixture to 5 compressive haversine pulses along its vertical diameter, and by measuring the corresponding horizontal deformation. In the present study, pulse loads were applied with a rise time of 124 ms and the target maximum horizontal deformation was set equal to 5 μm,

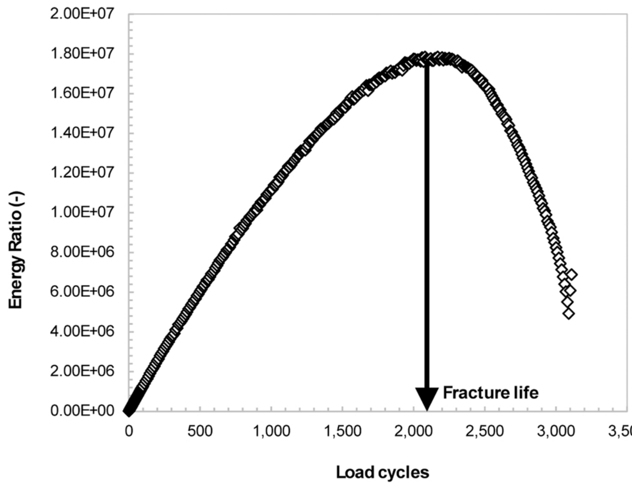

Fig. 2. Fatigue life criterion based on the dissipated energy approach [62].

7.5 µm, and 9 µm (for test temperatures of 10 °C, 20 °C, and 30 °C, respectively).

Six gyratory specimens of each mixture, with a diameter of 100 mm, were subjected to testing. The stiffness modulus E was determined by using the following formula (Eq. 1):

$$E = \frac{F \bullet (\nu + 0.27)}{(z \bullet h)} \tag{1}$$

where F is the peak loading force, ν is Poisson's ratio (assumed to be equal to 0.35 for all temperatures), z is the amplitude of the resilient horizontal deformation, and h is the mean thickness of the specimen.

3.4. Fatigue resistance

The same specimens employed for the evaluation of stiffness were subjected to fatigue tests carried out in the indirect tensile configuration as per EN 12697–24, Annex F [62]. According to the standard testing procedure, a harmonic sinusoidal cyclic compressive load, with a loading frequency of 10 Hz and without rest periods, is repetitively applied to a cylindrical specimen until reaching failure.

Tests were carried out at 10 °C by applying constant horizontal stresses that were varied among the different tests to obtain fatigue failure after the application of a number of loading cycles comprised between 10^3 and 10^6. The adopted failure criterion, based on the concept of dissipated energy, is expressed in terms of the energy ratio (ER) which is calculated as a function of loading cycles (n) as indicated in the following formula (Eq. 2):

$$ER(n) = n \bullet S_{mix,n} \tag{2}$$

where $S_{mix,n}$ is the stiffness modulus measured at the n-th loading cycle.

The fatigue life of an asphalt mixture is given by the number of loading cycles corresponding to the maximum energy ratio as indicated in Fig. 2.

Experimental data obtained at the various stress levels were fitted to the following power-law equation, which identifies the characteristic fatigue line of an asphalt mixture (Eq. 3):

$$N_{f,W} = k_\varepsilon \bullet \left(\frac{1}{\varepsilon_a}\right)^{n_\varepsilon} \tag{3}$$

where $N_{f,W}$ is the number of loading cycles until failure (according to energy ratio criterion), ε_a is the maximum initial horizontal strain amplitude at the center of the specimen (expressed in µm/m), k_ε and n_ε are the material dependent regression parameters of the fatigue function.

3.5. Fracture properties

Fracture properties were investigated by means of the semicircular bending (SCB) test, performed at 0 °C in accordance with EN 12697–44 [63].

Cylindrical specimens (150 mm in diameter and 115 mm in height), compacted by using the gyratory technique (with a target voids content of 4%), were cut into halves to obtain semicylindrical specimens (50 mm in thickness and 74 mm in height). Furthermore, the semicircular specimens were notched in their centre (0.35 mm width and 10 mm depth).

The suggested strain rate indicated in EN 12697–44 (50 mm/min) was not deemed appropriate to adequately highlight the effect of the inclusion of the WTSFs, which can act as physical barriers to the propagation of microcracks. Thus, as suggested by Ma and Hesp [64], a reduced strain rate of 0.5 mm/min was adopted for testing in order to better highlight the contribution of WTSFs to fracture resistance, as excessively high loading rates may promote a brittle response and mask the effect of fiber reinforcement. Since the SCB tests were performed at 0 °C, viscoelastic effects were expected to be significantly reduced compared to higher temperatures, making a linear elastic fracture mechanics (LEFM)-based interpretation reasonable for comparative purposes.

Data modelling and analysis was based on the concepts of fracture toughness and fracture energy, which are presented in EN 12697–44 [63] and AASHTO T394 [65], respectively. Fracture toughness (K_{Ic}) was used as a mode-I fracture parameter to describe the resistance of the investigated mixtures to crack propagation; a higher K_{Ic} indicates a greater ability of the material to oppose crack growth under the adopted SCB configuration.

Fracture toughness (K_{Ic}) was determined by means of the following equation (Eq. 4):

$$K_{Ic} = \sigma_{\max} \bullet Y_1 \bullet \sqrt{\pi a} \tag{4}$$

where $\sigma_{\max}$ is the maximum stress at failure, Y_1 is the normalized stress intensity factor, and a is the notch depth (equal to 10 mm).

Fracture energy (G_f) was obtained by making use of the following formula (Eq. 5):

$$G_f = \frac{W_f}{A_{lig}} \tag{5}$$

where W_f is the work of fracture (corresponding to the area under the loading portion of the load versus deflection curve), and A_{lig} is the so-

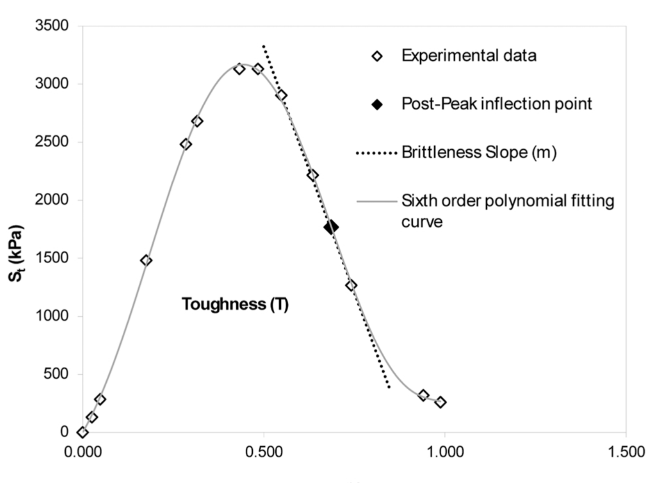

Fig. 3. Example of stress versus estimated strain data and definition of parameters for calculation of N_{flex} [68].

Table 4
Average values of compactability parameters of the investigated asphalt mixtures.

Mixture	v_8 (%)	v_{design} (%)	v_{180} (%)	N_{92}	N_{96}	CDI	TDI
RM	12.0	4.0	2.5	26	96	43	170
F0.5	11.9	4.0	2.6	25	96	38	174

called ligament area, which represents the area forming in front of the initial crack, equal the product of the ligament length (given by the difference between the specimen radius and the notch length) and the thickness of the specimen.

A minimum of two replicates was considered for both mixtures, thereby referring to average results in the following analysis.

To complete the analysis of the investigated mixtures, an additional performance index was considered, the N_{flex} factor, recently introduced to determine and distinguish the resistance of different bituminous mixtures to cracking-related damage [66,67]. In accordance with AASHTO TP141 [68], N_{flex} is determined by using the Indirect Tensile Strength test, and its expression is given by the following formula (Eq. 6):

$$N_{flex} = \frac{T}{|m|} \tag{6}$$

where T is the so-called toughness, and m is the brittleness slope, obtained by analyzing stress (S_t)-estimated strain (ε') curves as shown in Fig. 3.

As per AASHTO TP141, toughness T is determined by mathematically integrating the best-fit polynomial equation from the start of the test to the post-peak inflection point (defined as the first point past the peak stress with a curvature of zero), while the brittleness slope m represents the slope of the tangent at the post-peak inflection point.

Indirect Tensile Strength tests (ITS) were carried out at 10 °C by adopting a constant vertical deformation rate of 50 mm/min. By making use of cylindrical specimens prepared by means of the gyratory technique (with a target voids content of 4%), at least two replicates were considered for both mixtures, thereby referring to average results in the subsequent analyses.

4. Results and discussion

4.1. Compactability

Specimens of the investigated mixtures (RM and F0.5) were compacted at 150 °C by applying 180 gyrations. Mean values of the obtained results are reported in Table 4, where they are expressed in terms of voids at 8, 100, and 180 gyrations (indicated as v_8, v_{design}, v_{180}, respectively), gyrations corresponding to 92% and 96% compaction (indicated as N_{92} and N_{96}, respectively), *CDI*, and *TDI*.

Void contents at the three selected numbers of gyrations (v_8, v_{design}, v_{180}) and results relative to reference compaction levels (N_{92} and N_{96}) highlighted no differences between the two mixtures in terms of their overall densification behaviour. Moreover, slight differences were observed when considering compaction energy indices (*CDI* and *TDI*), with mixture F0.5 displaying, in comparison to the reference mixture, a lower *CDI* and a higher *TDI*. As reported in literature, lower values of *CDI* indicate more workable asphalt mixtures which require reduced compaction energy during construction, while higher values of *TDI* reflect a lower tendency to densify under the effects of traffic loadings, thereby leading to an improved resistance to rutting [60,69,70]. Obtained results suggest that the slightly better workability recorded for the mixture containing WTSFs may be due to the higher binder dosage (5.3% instead of 5.0%) and to the presence of fibres, which as a result of their specific characteristics (length, diameter and texture) most probably facilitate the spatial reorganization of aggregate particles under gyratory shear loading. Moreover, the slightly higher *TDI* value recorded in the presence of WTSFs, which may suggest a possible improvement in anti-rutting behaviour, could be related to the formation of a stiffening fibre network that counteracts the densification process due to the traffic loading.

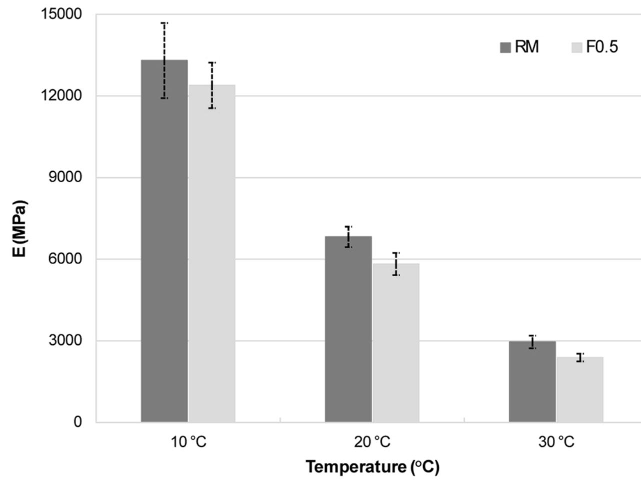

Fig. 4. Stiffness modulus of the investigated asphalt mixtures as a function of test temperature.

Table 5
Results of fatigue tests.

Specimen ID	σ_a (kPa)	ε_a (μm/m)	$N_{f,w}$
Mixture RM			
RM_1	650	92	2184
RM_2	580	75	5504
RM_3	400	49	31720
RM_4	300	38	123738
RM_5	250	31	352485
Mixture F0.5			
Specimen ID	σ_a (kPa)	ε_a (μm/m)	$N_{f,w}$
F0.5_1	570	91	2094
F0.5_2	500	83	5736
F0.5_3	450	65	18674
F0.5_4	300	46	60064
F0.5_5	270	42	139826
F0.5_6	200	37	393333

4.2. Stiffness

Average values of the stiffness moduli E obtained at 10 °C, 20 °C, and 30 °C are presented in Fig. 4, which also displays the corresponding data dispersion bars. It should be noted that tested specimens had average voids contents of 4.1% (varying from 3.7% to 4.6%) and 4.0% (ranging from 3.5% up to 4.2%) for the RM reference mixture and F0.5 mixture, respectively. It should be noted that the air-void contents of the tested specimens were very close to the target value of 4% and remained within a relatively narrow range for both mixtures. Therefore, although the influence of air-void content on stiffness properties cannot be completely excluded, the limited volumetric variability suggests that the observed differences are mainly related to the presence of WTSFs rather than to substantial differences in compaction level.

The results indicate that the addition of WTSFs did not lead to an increase in stiffness under the investigated conditions; on the contrary, a slight reduction was observed for the fiber-reinforced mixture. This trend may be related to the combined effect of the slightly higher binder content adopted for the F0.5 mixture and the presence of WTSFs within the asphalt matrix. However, on the basis of the present experimental program, the respective contribution of these two factors cannot be

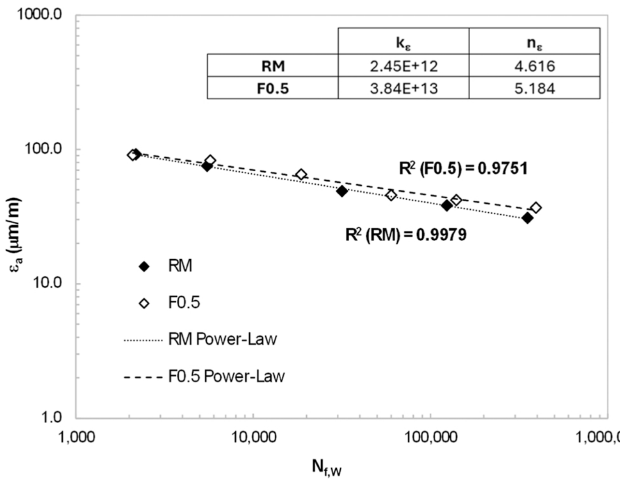

Fig. 5. Fatigue lines and regression parameters of the investigated asphalt mixtures.

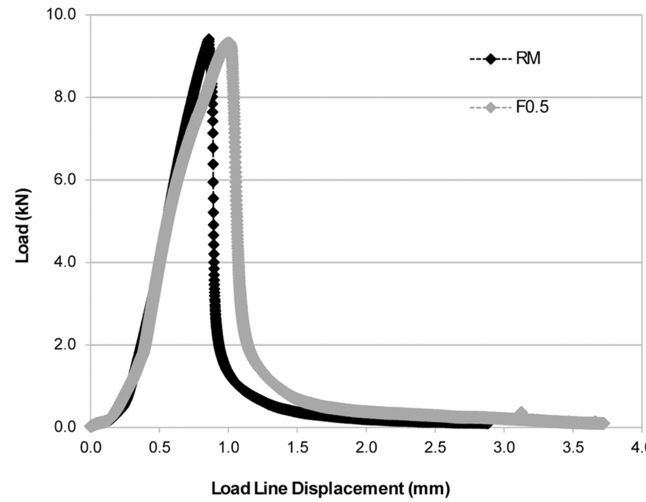

Fig. 6. Example of loading curves of the SCB tests performed at 0 °C on the investigated asphalt mixtures.

Table 6
Average results of the SCB tests performed at 0 °C on the investigated asphalt mixtures.

Mixture	σ_{max} (MPa)	ε_{max} (%)	K_{Ic} (N/mm$^{1.5}$)	G_f (J/m^2)
RM	1.1	1.1	29.2	1232
F0.5	1.1	1.3	29.2	1542

isolated unambiguously. Therefore, the observed stiffness response should be interpreted as the overall effect of fiber addition under the adopted mixture design and production conditions, rather than as the direct consequence of a single mechanism. From the data displayed in Fig. 4 it can also be noted that the WTSFs, as expected, did not change the temperature susceptibility of the considered mixture.

4.3. Fatigue resistance

Data obtained from fatigue tests performed on individual specimens are listed in Table 5, in which they are expressed in terms of applied horizontal stress (σ_a), corresponding initial tensile strain amplitude (ε_a), and loading cycles to failure ($N_{f,w}$).

Data shown in Table 5 are synthesized in Fig. 5, in which the characteristic fatigue lines of the two asphalt mixtures are shown together with regression parameters k_ε and n_ε.

In line with the findings of other researchers, it can be observed that the inclusion of WTSFs slightly enhanced fatigue resistance of the asphalt mixtures as a consequence of the presence of the reinforcing network created by the fibers, which also distribute and dissipate stresses more evenly throughout the material under loading [24,26,29,40,43,44,51,71–73]. Consistently with the results obtained by Wu et al. [40], the improvement in the resistance to fatigue was found to be greater at low stress levels since the presence of fibers reduces more effectively the formation and coalescence of microcracks at low initial tensile strains. Such behavior can be better highlighted by considering the strain amplitude corresponding to a fatigue life of 10^6 cycles (ε^6), which represents a useful indicator for the comparison of the fatigue resistance of different asphalt mixtures. By employing the regression parameters of the fatigue lines (indicated in Fig. 5), values of ε^6 equal to 24 and 29 were obtained for the RM and F0.5 mixture, respectively, which translate into an improvement in the fatigue resistance caused by the WTSFs of the order of 20%. It is worth noting that, since the two regression lines are relatively close and the fatigue campaign was conducted on a limited number of replicates (six), this result should be interpreted as a preliminary indication of a possible improvement in fatigue resistance, rather than as definitive proof of enhanced durability under traffic loading.

4.4. Fracture properties

Average data gathered from SCB tests are summarized in Table 6, in which they are expressed in terms of maximum stress at failure (σ_{max}), strain at failure (ε_{max}), fracture toughness (K_{Ic}), and fracture energy (G_f). It is worth mentioning that for the calculation of fracture energy, the work of fracture was calculated as the area under the load versus displacement curve until the load dropped below 0.1 kN. Tested specimens had an average voids content of 3.3% and 3.4% for the RM reference mixture and F0.5 mixture, respectively.

As known, higher K_{Ic} means greater resistance to crack propagation under mode-I loading. Contrary to expectations, the same average value of the fracture toughness K_{Ic} was recorded for the two investigated bituminous mixtures, suggesting that the inclusion of WTSFs did not significantly affect crack-initiation resistance as captured by this parameter. However, obtained values of the fracture energy G_f, which are representative of the external energy required for crack growth [74], led to a different conclusion. In fact, the asphalt mixture containing the WTSFs showed an average G_f value which is 25% higher than that of the reference mixture, indicating an improved resistance to crack propagation. Such a difference can be captured by observing the typical load line displacement - load curves obtained from SCB tests (Fig. 6).

As reported in literature [75,76], during SCB tests, while the applied load is increasing, stress concentration occurs, with the consequent formation of a non-linear fracture process zone (FPZ) near the notch. When cracks start to coalesce forming macrocracks, the applied load decreases and propagation of cracks take place. It can be observed that, despite achieving similar peak loads, the strain at failure ε_{max} and shape of the curves obtained for the two asphalt mixtures (with and without WTSFs) were different, highlighting a behavior which required further interpretation. Although fracture toughness and fracture energy were derived from the same SCB test, they do not describe the same stage of the fracture process. In the adopted framework, fracture toughness is governed by the peak load and therefore mainly reflects crack-initiation resistance at the notch tip, whereas fracture energy is related to the entire load–displacement response and is therefore more sensitive to post-peak crack propagation and energy dissipation. The present results suggest that WTSFs did not significantly modify the critical condition for crack initiation but rather improved the post-peak crack propagation stage by reducing brittleness and increasing the work of fracture.

In particular, the initial ascending part of both curves, the slope of

Table 7
Average values of the pre-peak and after-peak fracture energies derived from SCB tests performed at 0 °C on the investigated asphalt mixtures [28].

Mixture	G_{Pf} (J/m^2)	G_{Af} (J/m^2)
RM	884	348
F0.5	1039	503

which can be used for the determination of stiffness, was almost the same for the two mixtures with only a small deviation close to the peak load. Such an occurrence is consistent with the stiffness data obtained in the previous stage of the study, where a negligible reduction of stiffness was observed when including the WTSFs in the asphalt mixture. Moreover, the area under the loading portion of the load versus deflection curves after the peak load (i.e. the post-peak work of fracture) was found to be different for the two mixtures, indicating a change in terms of microcracks coalescence. For this reason, the approach proposed by Qiu et al. [28] was considered as the most appropriate for data analysis. According to this methodology, the fracture energy G_f can be divided in two contributions, indicated as pre-peak (G_{Pf}) and after-peak (G_{Af}) fracture energy, evaluated by substituting the work of fracture W_f of Eq. 5 with the pre-peak (W_{Pf}) and after-peak (W_{Af}) work of fracture, calculated by means of the following equations (Eq. 7 and Eq. 8):

$$G_{Pf} = \frac{W_{Pf}}{A_{lig}} \tag{7}$$

$$G_{Af} = \frac{W_{Af}}{A_{lig}} \tag{8}$$

Obtained values of G_{Pf} and G_{Af} are listed in Table 7.

It can be observed that both partial fracture energies showed higher values in the case of the asphalt mixture containing WTSFs, with the greatest increase (close to 45%) recorded after the peak load. This finding, which is consistent with other results published in the literature, suggest that WTSFs caused an increase of the resistance to crack propagation after reaching failure, thereby reducing the speed of crack growth [28,40,42,77–79]. As highlighted by Morea et al. [42], the greater enhancement of the fracture properties in the post-peak region is more evident at low temperatures, where asphalt mixtures become progressively more brittle and the WTSFs may have a sewing effect across the formed cracks, absorbing part of the applied stresses and hindering further crack propagation. This behavior was confirmed by the presence of the WTSFs filaments which appeared on the fractured surfaces after execution of SCB tests (Fig. 7).

Results obtained with the Indirect Tensile Strength test are presented in Table 8, in which they are expressed in terms of horizontal Indirect Tensile Strength (*ITS*), tensile strain at failure (ε'_{max}), toughness (T), and indirect tensile factor (N_{flex}). Tested specimens had average voids contents of 3.9% and 4.2% for the RM reference mixture and F0.5 mixture, respectively.

Data displayed in Table 8 indicate that the two mixtures, although characterized by a similar strength, exhibited different behaviour in terms of crack-related damage, with the F0.5 mixture being associated to higher values of toughness and N_{flex}, which are a consequence of the presence of the WTSF network. This evidence, together with the higher tensile strain at failure, also proves that the inclusion of WTSFs led to a more ductile response under loading, which matches with the findings reported by Jia et al. [38].

5. Conclusions and future perspectives

The research study presented in this paper focused on the preliminary laboratory characterization of asphalt mixtures containing recycled fibers derived from end-of-life tennis strings (indicated as WTSFs), which were hypothesized to form a reinforcement network.

Obtained results (which refer to the material investigated and the specific WTSF source considered in this study), indicate some promising trends and can be summarized as follows:

- compactability characteristics were slightly positively affected by the inclusion of WTSFs, as proven by the slightly lower *CDI* and higher *TDI* values;
- the presence of WTSFs did not cause major changes in stiffness modulus, which was found to be more sensitive to binder content;
- the inclusion of WTSF appears to confirm a slight trend towards improved fatigue resistance;
- fracture properties showed the greatest improvements when making use of the WTSFs, suggesting that their presence may hinder crack propagation after failure and may act more as crack-propagation tougheners than as crack-initiation strengtheners under the adopted laboratory conditions (SCB and ITS tests).

These findings need further confirmation in light of the boundaries of the present experimental study (single WTSFs source, limited number of replicates for fracture tests, no long-term durability assessment). Future investigations will include a more detailed characterization of the waste tennis string fibers, including tensile properties and surface morphology, in order to better understand their interaction with the asphalt matrix and the mechanisms governing reinforcement, highlighting the influence of the type of waste strings (i.e. multifilament or synthetic gut) and of their geometrical characteristics (i.e. length and diameter). Moreover, additional experimental tests are required to assess the effects of WTSFs on the thermal properties, permanent deformation performance, and long-term durability of asphalt mixtures, including ageing- and moisture-related aspects. Finally, although the present results support the potential sustainability of this recycling approach, no quantitative environmental or economic assessment was included in this study. In particular, life-cycle assessment (LCA) and life-cycle cost analysis (LCCA) are currently in an embryonic stage, mainly due to the ongoing collection of primary data required for a sufficiently robust evaluation.

Table 8
Average results of Indirect Tensile tests performed at 10 °C on the investigated asphalt mixtures.

Mixture	ITS (MPa)	ε'_{max} (%)	T (J/m^3)	N_{flex}
RM	3.5	0.25	867.4	0.08
F0.5	3.3	0.38	1438.7	0.18

Fig. 7. Example of crack growth (left) and WTSF filaments observed on fractured surfaces (right) after SCB tests.

CRediT authorship contribution statement

Mozhgan Hajiali: Writing – review & editing, Writing – original draft, Investigation, Data curation. **Sadegh Yeganeh:** Writing – original draft, Methodology, Investigation. **Pier Paolo Riviera:** Writing – review & editing, Writing – original draft, Visualization, Methodology, Investigation, Data curation, Conceptualization. **Ezio Santagata:** Writing – review & editing, Supervision, Methodology, Conceptualization.

Declaration of Competing Interest

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

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