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Abstract

In pavement engineering, the use of warm mix asphalt (WMA) technologies can ensure important environmental and technical benefits. However, several uncertainties about WMA still exist, such as long-term field performance and full compatibility with reclaimed asphalt pavement (RAP) or polymer modified bitumen (PMB). In this regard, a full-scale trial section (including three test fields with warm recycled mixtures prepared with different WMA chemical additives and a reference test field with hot recycled mixtures, all containing PMB) was constructed along an Italian motorway and monitored for several years of service life. The evolution of the structural properties was assessed with in-situ Falling Weight Deflectometer (FWD) tests and laboratory tests on extracted cores, both immediately after the construction of the trial section and after more than three years under actual traffic loading. It was found that the reduced working temperatures adopted for the WMA mixes (40°C lower than hot mix asphalt (HMA)) did not penalise the workability and the stiffness immediately after the trial section construction, whereas the HMA mixture experienced higher structural damage (likely due to more severe aging) during the in-service life. The WMA mixes exhibited better stiffness homogeneity and, overall, superior performance and potentially longer service life with respect to the reference HMA mixture.

Keywords: warm mix asphalt (WMA); chemical additives; reclaimed asphalt pavement (RAP); polymer modified bitumen (PMB); Falling Weight Deflectometer (FWD); full-scale trial section.

1. Introduction

In the last decade, warm mix asphalt (WMA) technologies have gained growing interest in pavement engineering thanks to the significant environmental benefits deriving from the reduced production and compaction temperatures as compared to traditional hot mix asphalt (HMA). In fact, the reduction of the working temperatures with respect to HMA implies lower emissions of greenhouse gases and pollutants as well as lower fumes

(Stimilli et al., 2017a), improving also the working conditions. Moreover, from a technical point of view, the reduced working temperatures allow longer hauling distances, cold weather paving (e.g. by night, in winter) and shorter opening time to traffic and may also ensure reduced binder aging. In addition, some economic advantages may derive from the reduced energy consumption and the lower wear of the asphalt plant, even though some modifications to the asphalt plant and/or royalties may be needed (Capitão et al., 2012; Cheraghian et al., 2020; Rubio et al., 2012; Thives and Ghisi, 2017).

Nevertheless, despite these benefits, several uncertainties about the use of WMA technologies still exist. One of the main critical points is the lack of data concerning WMA long-term field performance, which is due to the relatively recent development of such technologies and to the fact that so far most research on this topic has been carried out at a laboratory scale rather than within full-scale trial projects (Capitão et al., 2012; Cheraghian et al., 2020; Rubio et al., 2012; Sol-Sánchez et al., 2016; Stimilli et al., 2017a, 2017b, 2017c). Some test sections have been studied in previous projects (Timm et al., 2016; Vargas-Nordcbeck and Timm, 2012), but the knowledge of the performance of WMA pavements is still limited.

Moreover, the combination of WMA technologies and recycling techniques can be beneficial from both the sustainability and the performance perspectives. Indeed, on the one hand the presence of reclaimed asphalt pavement (RAP) in the mix can offset the uncertain rutting resistance of WMA mixtures, on the other hand WMA technologies can allow to use higher RAP amount, while promoting the basic principles of circular economy (i.e. the re-use of end-of-life materials) (Guo et al., 2020; Hettiarachchi et al., 2019). However, incorporating RAP in WMA mixtures adds further uncertainties in terms of material mechanical behaviour, which are due to the intrinsic heterogeneity of RAP (Antunes et al., 2019).

Furthermore, little is still known about the interaction between WMA additives and polymers in the case of polymer modified bitumen (PMB), even though the latter is widely used in asphalt pavements because of its superior performance and durability as compared to unmodified bitumen. In fact, the properties of the ternary system formed by bitumen, polymer and additive are not predictable by simply superposing the effects of each component. In addition, it is unsure whether the WMA technologies could be fully effective for the production of asphalt mixtures containing PMB, as they usually require higher working temperatures to gain proper workability (Ferrotti et al., 2017; Porto et al., 2019; Stimilli et al., 2017b, 2017c). It is evident that all the above-mentioned unsolved matters represent significant gaps in the current literature, which limit the application of WMA.

Within this framework, the objective of this study was to monitor the evolution of the structural properties of warm recycled pavements over time with in-situ Falling Weight Deflectometer (FWD) tests and laboratory tests carried out on extracted cores. For this purpose, a full-scale trial section was constructed in April 2016 along the A1 Italian motorway and monitored for a period of three and a half years, during which it was subjected to actual motorway traffic. The trial section included three test fields constructed with warm recycled mixtures prepared with different WMA chemical additives and a reference test field constructed with hot recycled mixtures, all containing PMB.

2. Experimental program

2.1. Trial section

A 800 m full-scale trial section was constructed in April 2016 along the A1 Italian motorway (south carriageway) from km 513+000 to km 513+800. Specifically, the project involved the full-depth re-construction of the slow traffic lane after the milling of all existing asphalt layers. The trial section consisted in four test fields, including three test fields constructed with warm recycled mixtures prepared with different WMA chemical additives and a reference test field constructed with hot recycled mixtures, as follows:

- Test field with WMA chemical additive 2 (C2) from km 513+000 to km 513+200, coded as WC2;
- Test field with WMA chemical additive 1 (C1) from km 513+200 to km 513+400, coded as WC1;
- Test field with WMA chemical additive 3 (C3) from km 513+400 to km 513+600, coded as WC3;
- Reference HMA test field from km 513+600 to km 513+800, coded as HMA.

It is worth noting that all the test fields were characterized by the same boundary conditions in terms of subgrade, pavement structure (layers' characteristics and thicknesses), traffic, road geometry (straight and flat stretch) and weather conditions. The pavement consisted of an open-graded friction course (OGFC) with nominal thickness of 4 cm, a binder layer with nominal thickness of 10 cm, a base layer with nominal thickness of 15 cm and a cold recycled subbase layer (in-situ recycling with foamed bitumen) with nominal thickness of 25 cm. More details about the trial section and its construction can be found elsewhere (Stimilli et al., 2017b).

In terms of traffic loads, during the monitoring period (April 2016 – November 2019), the trial section was subjected to about $30 \cdot 10^6$ *equivalent single axle loads* (ESALs). The ESALs were computed by monitoring the actual traffic and considering a 12-ton single axle with twin wheels as reference axle and fatigue cracking as predominant distress, based on the approach developed at Norwegian Road Research Laboratory (Evensen and Senstad, 1992).

2.2. Materials

In each test field, the asphalt mixtures had the same gradation, the same type and dosage of virgin bitumen as well as identical RAP content, as described below (all the percentages are referred to the aggregate weight):

- OGFC: open-graded mixture with 15% of selected RAP (fraction 8/16 mm, deriving only from the milling of old motorway OGFCs), 83% of virgin basalt aggregate fractions and 2% of limestone filler. Fibers (composed of 70% cellulose and 30% glass) were added to the mix at a dosage of 0.3% to prevent draindown issues. The total bitumen content (virgin bitumen plus bitumen from RAP) was 5.25%;
- Binder layer: dense-graded mixture with 25% of RAP (0/14 mm, unfractioned, deriving from the milling of old binder and base motorway layers) and 75% of virgin limestone aggregate fractions, including 2% of filler. The total bitumen content was 4.8%;
- Base layer: dense-graded mixture with 30% of RAP (0/14 mm, unfractioned, deriving from the milling of old binder and base motorway layers) and 70% of

virgin limestone aggregate fractions, including 1% of filler. The total bitumen content was 4.5%.

For all the layers, the virgin bitumen used was a styrene-butadiene-styrene (SBS) polymer modified bitumen with 3.8% SBS by bitumen weight, which is widely employed for construction and maintenance activities in the Italian motorway network. The bitumen contained in the RAP (equal to 4% and 5% by aggregate weight for fraction 8/16 and unfractioned 0/14, respectively) was of the same origin as the virgin one, i.e. SBS polymer modified. Moreover, it is worth pointing out that the gradation and the RAP content considered were equal to the ones usually adopted in the case of HMA production.

Therefore, the mixtures differed only for the type of WMA additive and in terms of working temperatures. All the selected WMA products were chemical additives because in previous laboratory investigations (Frigio et al., 2016; Frigio and Canestrari, 2018) it was found that this type of WMA technology was more promising from the performance and durability point of view as compared to organic additives (i.e. waxes) and zeolites in the case of mixes containing RAP and PMB (for both open- and dense-graded mixtures), especially in terms of moisture resistance (comparable to HMA). The WMA products studied were characterized by different chemical composition and operational mechanisms, as it is known that WMA additives can act on the viscosity of the binder and/or on the friction properties at the binder/aggregate interface (Ingrassia et al., 2018).

Specifically, the additive C1 is mainly composed of ammine substances that act as surfactants and adhesion enhancers. C1 is a viscous liquid at 25°C with a density of about 1.0 g/cm^3, a minimum operational temperature point of about −8°C and a flash point higher that 140°C. The additive C2 is composed of alkylates and fatty acids that act as viscous regulators. At ambient temperature, C2 is an amber-coloured inodorous liquid. It is characterized by a density of 0.86-0.90 g/cm^3 at 20°C, a flash point higher than 220°C and a boiling point between 300 and 408 °C and is insoluble in water. Finally, the additive C3 is composed of surfactants. C3 is liquid at ambient temperature and presents a density of about 1.00 g/cm^3 at 20°C and a flash point greater than 200°C.

Within the single test field, all the asphalt mixtures (OGFC, binder and base layers) were prepared with the same WMA additive (without any additive for test field HMA) and at equal working temperatures, namely 130°C (production) and 120°C (compaction) in the case of warm mixtures, 170°C (production) and 160°C (compaction) in the case of hot mixtures.

The dosage of the additives was chosen in accordance with the range recommended by the producers. Specifically, the additive amount differed for each layer depending on the “working” bitumen content (given by virgin bitumen plus re-activated bitumen from RAP), in order to take into account the differences in terms of RAP source and content, as summarized in Table 1. As for the degree of RAP bitumen re-activation, a precautionary value of 60% was assumed for all WMA mixes, whereas the re-activation degree can be considered about 70% in the case of HMA, as shown by Stimilli et al. (2015).

Table 1. Main composition characteristics of the asphalt mixtures.

Layer	RAP [% by agg. weight]	Total bitumen [% by agg. weight]	WMA additive [% by virgin bitumen weight]
OGFC	15 (RAP 8/16 mm)	5.25	C1: 0.42
			C2: 0.70
			C3: 0.45
Binder	25 (RAP 0/14 mm)	4.80	C1: 0.50
			C2: 0.80
			C3: 0.50
Base	30 (RAP 0/14 mm)	4.50	C1: 0.55
			C2: 0.90
			C3: 0.55

The proper amount of WMA additive was supplied following a strict and reliable protocol at the asphalt plant right before adding each material component (i.e. virgin bitumen, virgin and RAP aggregates) to the mixing chamber. In this way, minor modifications to the asphalt plant were necessary for WMA production, leaving unaltered the possibility to easily switch to standard HMA production. Moreover, operating in this way allowed also to fully exploit the potential of WMA additives, as it is recommended by the producers to avoid storage and/or rest periods after the addition of the WMA product to the bitumen.

Except for the addition of the WMA additives, the operational steps followed at the asphalt plant for the production of warm mixes were the same usually adopted for hot mixes.

2.3. Testing program and procedures

The testing program involved the execution of two FWD campaigns at an interval of three and a half years. Specifically, the first series of FWD tests was performed in April 2016 after a week from the construction of the trial section, whereas the second series of FWD tests took place in November 2019. On both occasions, the FWD tests were performed with a step of about 20 m for each test field, for a total of 11 tests for WC2, WC1 and WC3 (10 tests for WC2 in 2019) and 8 tests for HMA. The FWD was configured with a 30 cm diameter loading plate and nine geophones positioned at 0, 200, 300, 400, 500, 600, 700, 800 and 1500 mm from the center of the loading plate. Moreover, the Ground Penetrating Radar (GPR) was used concurrently with the FWD tests in order to define the exact thickness of the layers at the testing points.

The testing program was completed with *indirect tensile stiffness modulus* (ITSM) tests carried out on a total of 80 specimens (100 mm diameter cores) taken from the test fields (40 specimens in 2016 and 40 specimens in 2019). The tests were performed at 20°C on 10 specimens cored from the binder layer at different positions of each test field. Repeated load pulses with a rise time of 124 ms and a pulse repetition period of 3 s were applied to the specimen using a servo-pneumatic machine, in compliance with Annex C of EN 12697-26 (2012). The peak load was adjusted to achieve a target peak horizontal deformation of 5 μm. For each specimen, the test was repeated along two perpendicular diameters and the average ITSM value was considered. The ITSM value was calculated as in Equation (1):

$$ITSM = \frac{F \cdot (\nu + 0.27)}{z \cdot h} \qquad (1)$$

Where F is the peak load of the applied repeated pulse, z is the amplitude of the horizontal deformation, h is the average thickness of the specimen and ν is the Poisson ratio (assumed equal to 0.35 in all cases).

The FWD tests allowed to assess the evolution of the structural properties of the test fields after three and half years of traffic loading, whereas the ITSM tests were useful to evaluate the stiffness of the WMA and HMA mixtures and its change over time.

It is worth mentioning that the experimental investigation described in this paper is part of a larger project concerning the in-depth characterization of warm recycled mixtures. The project involved also an extensive investigation (carried out in 2016) on the properties of mixtures produced at the asphalt plant and compacted in the laboratory, as well as a series of interface shear tests and cyclic tests performed on cores taken from the trial section both in 2016 and 2019. The results obtained in these studies, however, are not the focus of this paper and will be presented elsewhere.

3. Results and analyses

3.1. Stiffness from ITSM tests on field cores

Figure 1 shows the average ITSM values (10 specimens) obtained at 20°C from the binder layer cores for both 2016 and 2019 test campaigns. In the figure, the error bars represent the standard deviations and, for each mixture, the average air void content (V_v) is also indicated to take into account the influence of the volumetric properties.

In order to compare the different mixtures, a statistical analysis (one-factor *analysis of variance*, ANOVA) was carried out separately on 2016 and 2019 cores. The analysis allowed to evaluate the significance of the differences observed between two mixtures. A confidence level of 95% was chosen, meaning that the difference was considered significant (i.e. not random) when the *p-value* obtained was lower than 0.05. The results of the ANOVA analysis are summarized in Table 2.

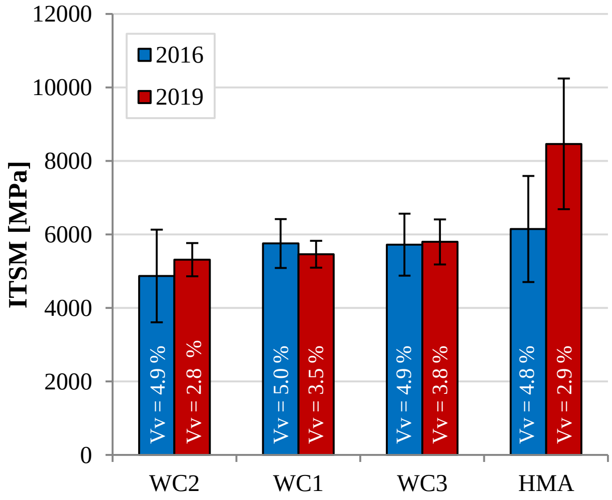

Figure 1. ITSM values obtained at 20°C for binder layer cores.

Table 2. Results of ANOVA analysis.

Mixtures	2016		2019	
	p-value	Significance	*p-value*	Significance
HMA vs. WC1	0.444	NO	< 0.0001	YES
HMA vs. WC2	0.050	YES	< 0.0001	YES
HMA vs. WC3	0.429	NO	0.0004	YES
WC1 vs. WC2	0.066	NO	0.4229	NO
WC1 vs. WC3	0.923	NO	0.1933	NO
WC2 vs. WC3	0.093	NO	0.0593	NO

As for the 2016 cores, from Figure 1 and Table 2, it can be observed that there were no significant differences in terms of stiffness between the mixtures (except for the pair HMA-WC2, for which the *p-value* was exactly equal to 0.05). This finding demonstrates that the reduced production and compaction temperatures adopted for WMA did not significantly penalise the stiffness properties of the mixtures as compared to the reference HMA mixture. Moreover, the similar air void content found in 2016 for the four mixtures (all compacted following the same in-situ compaction procedure) denotes the same workability for the reference HMA mix and the three WMA mixes produced with different additives.

Conversely, as for the 2019 cores, it can be noted that the HMA mixture exhibited significantly higher stiffness with respect to all WMA mixtures, which were characterized by similar ITSM values (Figure 1 and Table 2). Furthermore, the stiffness was basically unchanged for the three WMA mixes in three and a half years, whereas only the HMA mix underwent a remarkable stiffness increase (Figure 1). This difference can likely be attributed to a faster in-service aging rate of the reference HMA mixture.

In addition, it is very interesting to observe that the WMA mixes were characterized by a better homogeneity in terms of stiffness as compared to the HMA mix, as demonstrated by the considerably lower standard deviation values. In fact, based on the results obtained on 10 different specimens for each test field, the coefficient of variation (i.e. the ratio between the standard deviation and the average) was always lower than 10-15% for the WMA mixtures (except for WC2, 2016), whereas it was equal to 20-25% for the reference HMA mixture. As a possible explanation, it should be considered that the RAP (25% by aggregate weight for the binder layer) is not homogeneously distributed within the asphalt mixture at the specimen scale and that high production temperatures can have a marked effect on the RAP aging, resulting in a greater stiffness variability for the HMA mix with respect to the WMA mixes. This outcome suggests that the use of WMA technologies can ensure longer pavement service life thanks to the reduction of pavement areas with lower performance (which represent preferential points for the initiation of distresses that would easily extend to the rest of the pavement).

3.2. Stiffness from FWD tests

The limited extension of each test field, together with the fact that the trial section was a straight and flat stretch, allowed to consider each test field as a distinct homogeneous section for the analysis of FWD measurements. Therefore, for each test field, a representative basin was identified among those measured, with the final aim of estimating the stiffness moduli of the pavement layers through the back-calculation method.

In this regard, the measured deflection basins were first visually examined in order to detect inconsistent basins to be excluded from the analysis due to possible local inhomogeneities emerged during the FWD tests. After this preliminary step, for each test field, the remaining basins were considered to calculate the average basin. Finally, to identify the representative basin, the sum of the deviations from the average basin was calculated for every measured basin, as follows:

$$\Delta_i = \sum_{j=1}^{n} \left(\frac{d_{ij}}{d_{av,j}} - 1 \right)^2 \quad (2)$$

Where Δ_i is the sum of the deviations from the average basin for basin i, n is the number of geophones (i.e. 9), d_{ij} is the deflection belonging to basin i and measured by geophone j, $d_{av,j}$ is the average deflection corresponding to geophone j. The representative basin was the measured basin closest to the average one (i.e. the basin characterized by the lowest value of Δ_i).

Figures 2 and 3 show the deflection basins measured for each test field in 2016 and 2019, respectively. In the figures, the average basin is plotted in light blue (rhombi), the representative basin is plotted in green (triangles) and the discarded basins are plotted in red (asterisks). It can be immediately noted that, both in 2016 and in 2019, the HMA test field was characterized by a worse homogeneity with respect to the WMA test fields, as demonstrated by the higher dispersion of the FWD measurements. This result is consistent with what found from core testing (see Figure 1).

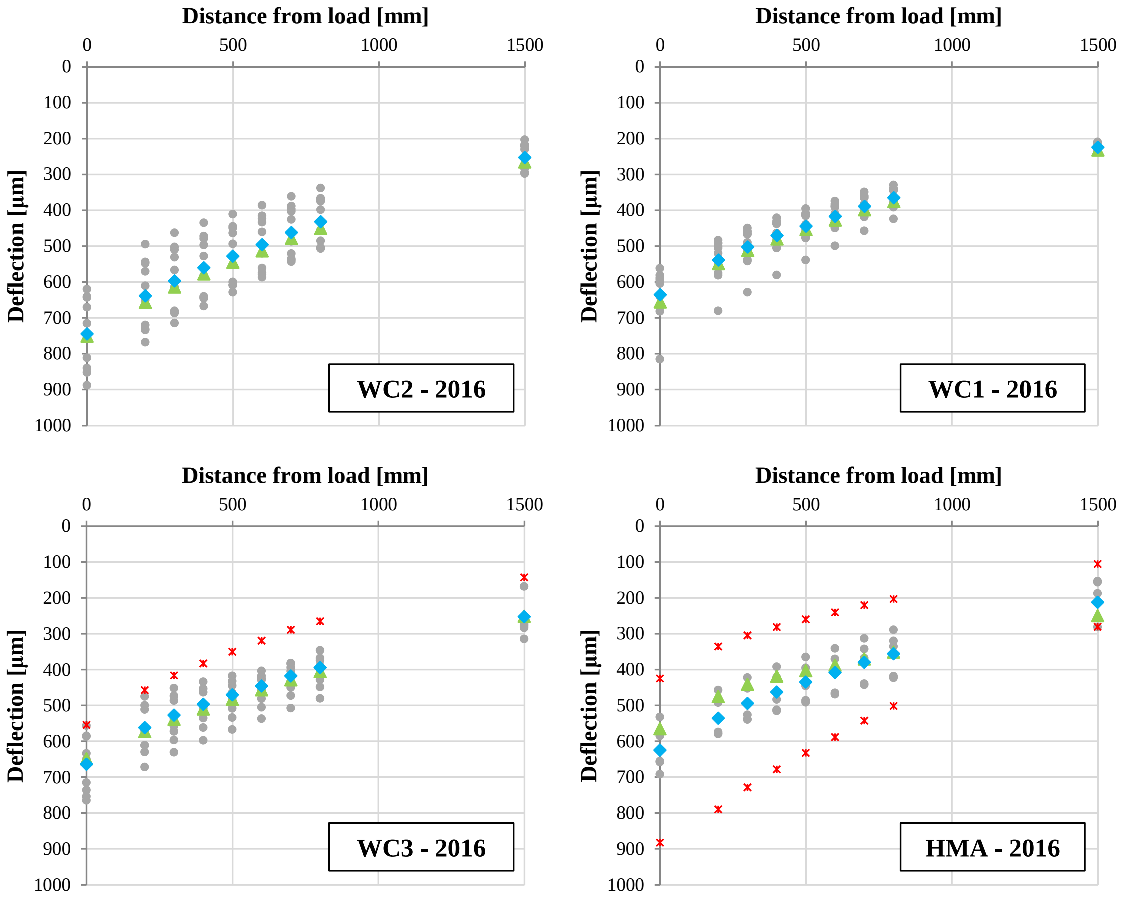

Figure 2. FWD basins measured in 2016 (the light blue rhombi represent the average basin, the green triangles represent the representative basin and the red asterisks represent the discarded basins).

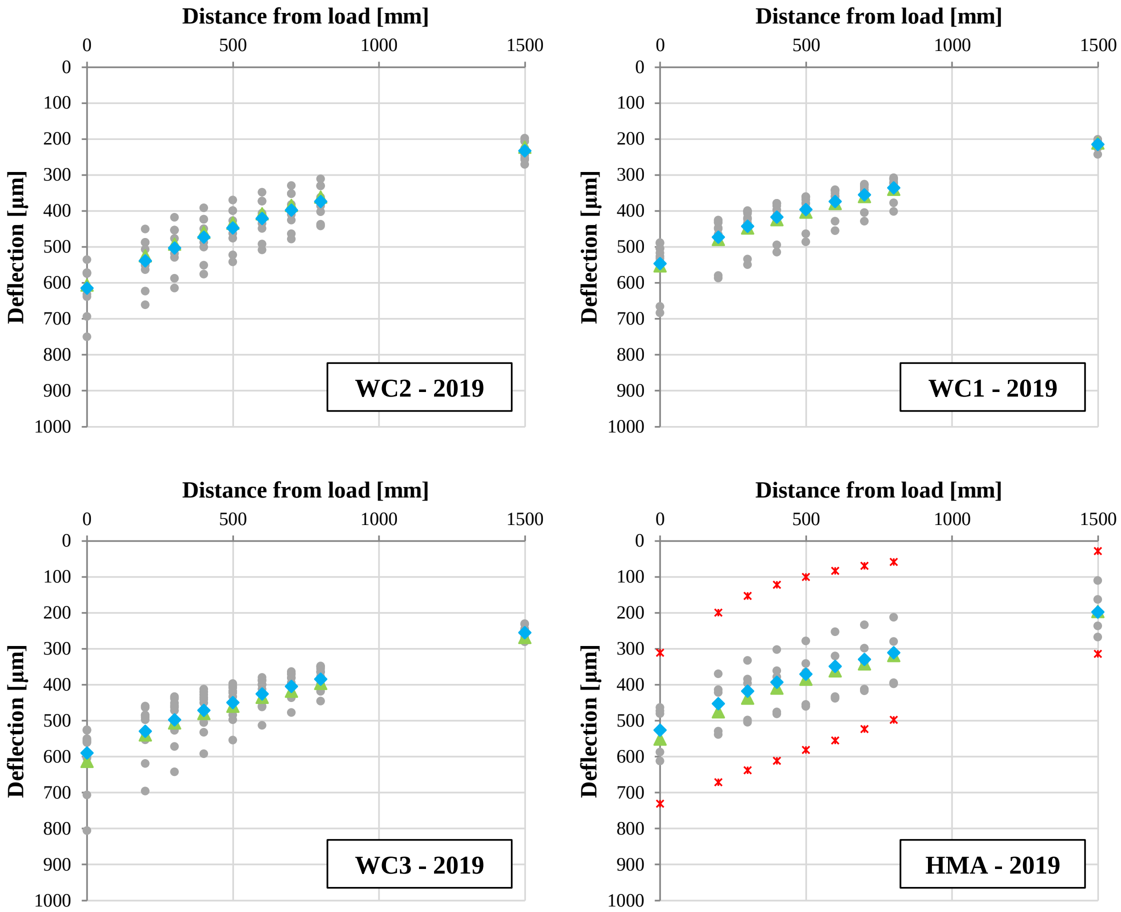

Figure 3. FWD basins measured in 2019 (the light blue rhombi represent the average basin, the green triangles represent the representative basin and the red asterisks represent the discarded basins).

The representative basins of the four test fields for both 2016 and 2019 campaigns are compared in Figure 4. It can be observed that in 2016 (i.e. immediately after the trial section construction) the HMA test field exhibited lower deflection values, probably due to the greater oxidation degree of the mixture caused by the higher production and compaction temperatures as compared to the WMA mixes. This outcome is in line with the ITSM values obtained from the binder layer cores in 2016 (Figure 1). Indeed, even though the statistical analysis did not show any statistically significant difference between the mixtures in terms of stiffness, the average ITSM value for the HMA mix was slightly higher than that of the WMA mixtures. Among the warm mixes, in 2016, WC2 showed the largest deflections, whereas the deflections measured for WC1 and WC3 were very similar (Figure 4), consistently with the corresponding ITSM values (Figure 1).

Moreover, from Figure 4, it can also be noted that the deflection values measured in 2016 by the geophone at 1500 mm from the center of the loading plate (which are mainly determined by the properties of the unbound layers) were comparable for all the test fields. Therefore, it can be concluded that the different deflections measured close to the loading plate were ascribable only to the properties of the asphalt layers.

On the other hand, the comparison of the representative basins obtained in 2019 shows – with respect to 2016 – a general decrease of the deflections and a reduced range of variability between the test fields (Figure 4), which are mainly attributable to the changes in the properties of the asphalt layers occurred in the three and a half years of in-service life. In fact, the deflection values measured at 1500 mm from the center of the loading plate were basically the same as those obtained in 2016.

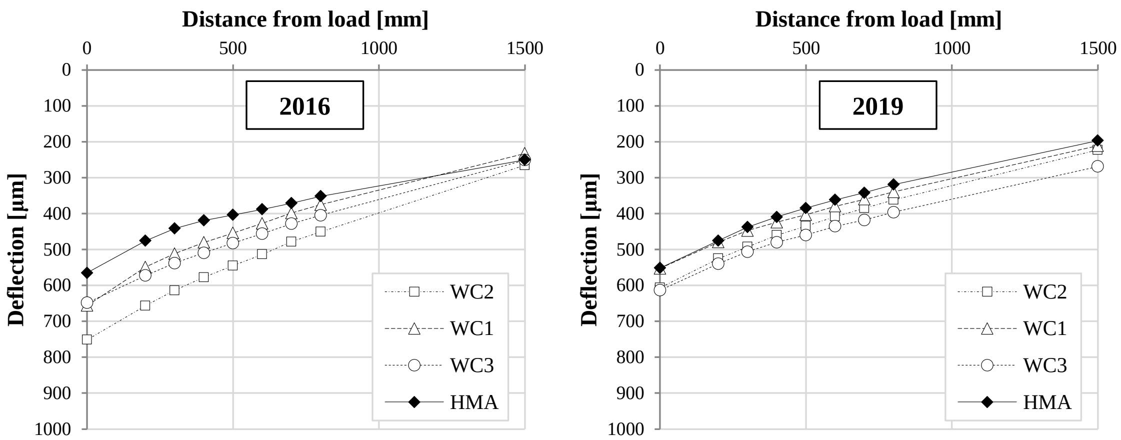

Figure 4. FWD representative basins.

After identifying the representative basin for each test field, the stiffness moduli of the layers were back-calculated with ELMOD 6 software. In the perspective of the back-calculation, the pavement was schematized as a three-layer elastic structure. The first layer was representative of all the asphalt layers (OGFC, binder and base) and was characterized by a thickness H1 (obtained from the GPR measurements) and a stiffness modulus E1. Obviously, such assumption affected the results in terms of modulus, as the OGFC had lower stiffness with respect to the dense-graded binder and base layers. Nevertheless, this assumption was necessary because for the back-calculation it is recommended (COST 336, 2005) that the minimum thickness of the analysed layers must be at least equal to half the radius of the loading plate used for the FWD tests (in the case of interest, the loading plate radius was 15 cm and thus the minimum thickness that could be analysed was 7.5 cm). The second layer corresponded to the subbase and was characterized by a thickness H2 (obtained from the GPR measurements) and a stiffness modulus E2. Finally, the subgrade was modelled as a half-space (infinite thickness) and was characterized by a stiffness modulus Es.

The stiffness moduli (E1, E2 and Es) obtained from the back-calculation for the representative basin of each test field are summarized in Table 3, along with the corresponding thicknesses (H1 and H2) measured with GPR and the average temperature of the asphalt layer (T1). From the table, it can be noted that there is a difference greater than 4°C between T1 obtained in 2016 (about 18.5-19 °C, April) and T1 obtained in 2019 (about 14-14.5 °C, November). For this reason, in order to make a reasonable comparison between the 2016 and 2019 data, all the moduli E1 determined with the back-calculation for the asphalt layer were corrected and referred to a temperature of 20°C.

The conversion from the pavement temperature to the reference temperature of 20°C was carried out considering some available results deriving from complex modulus tests conducted in 2016 on binder mixtures (WC2, WC1, WC3 and HMA) produced at the asphalt plant and compacted in the laboratory. Specifically, the temperature-dependency was assessed by developing the complex modulus isochronous curve at 20 Hz, which can be considered a typical FWD load frequency (CROW Report 17, 1998; Pożarycki et al., 2019). Since it was found that the four mixtures had similar temperature susceptibility (see Figure 5, showing the complex modulus shift factors, a_T), it was reasonable to consider a single isochronous curve, determined by averaging the results obtained for the four mixtures at 20 Hz, for each testing temperature (10, 20, 30, 40, 50 °C).

The experimental data were then fitted with the following function (CROW-report D05-06, 2005):

$$E^* = e^{(a \cdot T^3 + b \cdot T^2 + c \cdot T + d)} \quad (3)$$

Which can be re-written as follows:

$$\ln(E^*) = a \cdot T^3 + b \cdot T^2 + c \cdot T + d \quad (4)$$

Where E^* is the complex modulus, T is the temperature, a, b, c and d are the function parameters. The experimental data and the fitted function are shown in Figure 6 ($a = 5 \cdot 10^{-6}$, $b = -0.0007$, $c = -0.0243$, $d = 10.294$).

This law was used for all test fields to obtain the corrected asphalt layer moduli, denoted as $E1_{20°C}$ and reported in Table 3. Once again, it is worth pointing out that these moduli are representative of an overall response given by the OGFC, the binder layer and the base layer.

Table 3. Stiffness moduli from FWD tests.

Year	Test field	km	H1 [mm]	H2 [mm]	T1 [°C]	E1 [MPa]	E2 [MPa]	Es [MPa]	$E1_{20°C}$ [MPa]
2016	WC2	513+060	299	242	18.9	7851	441	159	7465
	WC1	513+220	276	230	18.3	9315	1134	117	8620
	WC3	513+420	297	237	18.4	9941	918	100	9241
	HMA	513+680	293	229	18.9	9832	2137	148	9348
2019	WC2	513+120	296	234	14.3	10946	394	161	8525
	WC1	513+400	291	231	14.3	11009	1333	112	8574
	WC3	513+600	287	235	14.3	10853	1186	123	8452
	HMA	513+720	292	230	14.1	10169	1185	111	7855

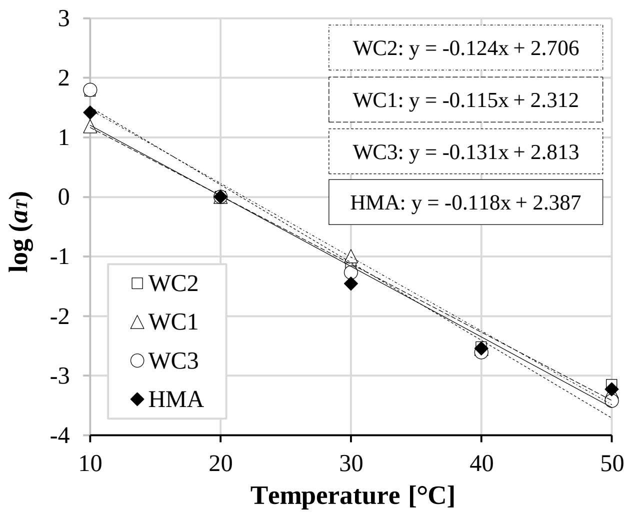

Figure 5. Complex modulus shift factors.

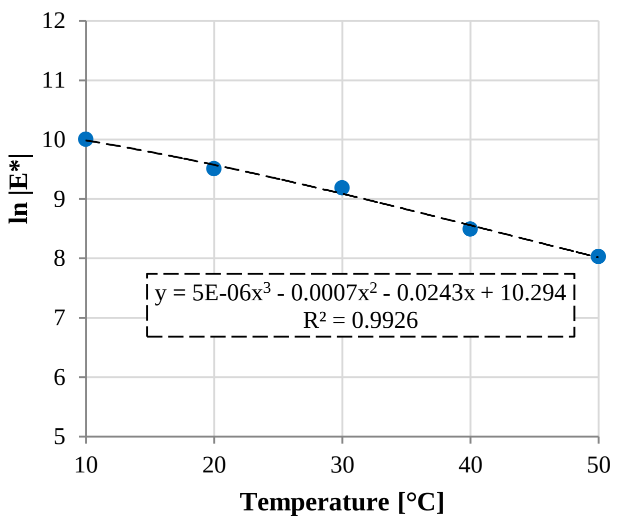

Figure 6. Complex modulus average isochronous curve at 20 Hz.

3.3. Interpretation of the results

The percentage variation of $E1_{20°C}$ in the monitored period (April 2016-November 2019) is provided in Table 4. It can be observed that $E1_{20°C}$ increased for WC2, decreased for HMA and remained almost unchanged for WC1 and WC3. Furthermore, it can be noted that a very similar value of $E1_{20°C}$ was obtained for all WMA test fields in 2019 (about 8500 MPa).

In addition, Table 4 shows also the percentage variation of the ITSM values measured in the laboratory (at 20°C) in 2016 and 2019 on binder layer cores (see Figure 1) concurrently with the FWD tests. In terms of ITSM, an increase for WC2 and almost no variation for WC1 and WC3 were observed, in accordance with the variation of $E1_{20°C}$. Moreover, even in terms of ITSM, all WMA mixes exhibited a similar stiffness modulus in 2019 (around 5500 MPa). The only discrepancy with respect to the back-calculation results is the marked increase in the ITSM value found for the HMA mixture in the monitoring period (+37.7%), which is in contrast with the decrease observed in terms of $E1_{20°C}$ (−16.0%).

Table 4. Stiffness variation (2019 vs. 2016).

Test field	$E1_{20°C}$ [MPa]			ITSM @20°C [MPa]		
	2016	2019	% variation	2016	2019	% variation
WC2	7465	8525	+14.2%	4868	5310	+9.1%
WC1	8620	8574	−0.5%	5752	5457	−5.1%
WC3	9241	8452	−8.5%	5718	5794	+1.3%
HMA	9348	7855	−16.0%	6146	8460	+37.7%

In order to verify the reliability of these results, the following considerations can be made. First of all, it should be recalled that the $E1_{20°C}$ values obtained from the back-calculation can be related to a typical FWD load frequency of 20 Hz (CROW Report 17, 1998; Pożarycki et al., 2019), whereas the ITSM values obtained in the laboratory refer to a frequency of about 2 Hz (Pasetto and Baldo, 2006). This observation explains why, for each single mixture, the ITSM value is always lower than $E1_{20°C}$, for both 2016 and 2019 data. The only exception is represented by the 2019 HMA mix.

Furthermore, it is necessary to assess whether the difference between the $E1_{20°C}$ values at 20 Hz and the ITSM values at 2 Hz is consistent or not with the viscoelastic properties of the materials investigated. In this regard, the available complex modulus data, obtained in 2016 for binder mixtures (WC2, WC1, WC3 and HMA) produced at the asphalt plant and compacted in the laboratory, can be considered. Specifically, the ratio between the complex modulus at 2 Hz and 20°C (denoted as E*(2 Hz)) and the complex modulus at 20 Hz and 20°C (denoted as E*(20 Hz)) can be compared with the ratio ITSM/$E1_{20°C}$, as shown in Table 5. It can be observed that the ratio E*(2 Hz)/E*(20 Hz) is almost identical for all WMA mixes (average value equal to 0.73), whereas it is slightly higher for the HMA mix (0.83). It is interesting to notice that, for all WMA mixes, the ratio ITSM/$E1_{20°C}$ in 2016 and 2019 (average value in both cases equal to 0.65) is slightly lower than the E*(2 Hz)/E*(20 Hz) value. The discrepancy between the values of ITSM/$E1_{20°C}$ and E*(2 Hz)/E*(20 Hz) can be explained by the fact that the ITSM value is determined in indirect tension configuration whereas E*(2 Hz) is determined in uniaxial compression configuration, which generally provides higher modulus values. Conversely, $E1_{20°C}$ represents a modulus calculated from the back-calculation assuming a tension-compression symmetric elastic model, which is to some extent comparable to E*(20 Hz). Consequently, the ITSM/$E1_{20°C}$ ratio is expected to be lower than the E*(2 Hz)/E*(20 Hz) ratio. In this sense, even in this case, the only inconsistency regards the 2019 HMA mixture, which exhibits an out of trend

ITSM/$E1_{20°C}$ ratio (equal to 1.08) that is higher than the corresponding E*(2 Hz)/E*(20 Hz) ratio, as can be seen from Table 5.

Table 5. Comparison with complex modulus results.

Test field	ITSM/$E1_{20°C}$		E*(2 Hz)/E*(20 Hz)
	2016	2019	2016
WC2	0.65	0.62	0.75
WC1	0.67	0.64	0.73
WC3	0.62	0.69	0.71
HMA	0.66	1.08	0.83

The inconsistency emerged for the HMA test field/mixture can be explained by considering that, in general, the back-calculation results express the overall performance of the analysed layer, which can be directly related to the material performance only if the layer is intact. In the event that the layer is affected by a progressive level of damage (e.g. presence of many micro-cracks and/or macro-cracks), the back-calculation would lead to decreasing stiffness modulus values over time (given a fixed layer thickness). On the other hand, it is well known that, due to aging, the stiffness modulus of asphalt mixtures tends to increase over time. Within this framework, it can be deduced that the HMA mixtures likely experienced the most severe aging (which made the layers prone to fracture and damage) during the three and a half years of in-service life, as demonstrated by the reduction of $E1_{20°C}$ and the concurrent increase of ITSM (see Table 4). This outcome is in line with the results of previous studies, which highlighted that WMA technologies may provide an anti-aging effect even in the long term (Diefenderfer, 2018; Wang et al., 2019).

In summary, based on the evolutive trend obtained from FWD tests as well as from laboratory tests, it can be concluded that the WMA mixtures studied could provide superior performance and longer service life with respect to the reference HMA mixture.

4. Conclusions

The objective of this study was to monitor the evolution of the structural properties of warm recycled pavements (composed of WMA mixtures containing RAP and PMB) over time with in-situ FWD tests and laboratory ITSM tests carried out on extracted cores. A full-scale trial section was constructed in April 2016 along the A1 Italian motorway and monitored for a period of three and a half years, during which it was subjected to actual motorway traffic. The trial section included three test fields constructed with warm recycled mixtures produced with different WMA chemical additives and a reference test field constructed with hot recycled mixtures. The asphalt mixtures differed only for the type of WMA additive and in terms of production and compaction temperatures (170°C and 160°C for HMA vs. 130°C and 120°C for WMA), whereas all the other conditions (gradation, type and dosage of virgin bitumen, RAP content, boundary conditions) were purposely kept the same.

The results and analyses presented led to the following conclusions:

- The differences observed in terms of ITSM between the mixtures immediately after the trial section construction (2016) were not statistically significant and all mixes were characterized by similar air void contents, meaning that the reduced working temperatures adopted for WMA did not penalise the stiffness and the workability;

- In 2016, the HMA test field exhibited lower deflections (and slightly higher ITSM) as compared to the WMA test fields, most likely due to the greater oxidation undergone during the production and compaction process;
- In 2019, the HMA mixture was characterized by a statistically significant higher stiffness with respect to all WMA mixtures that showed similar ITSM values, whereas the FWD measurements led to a general decrease of the deflections and a reduced range of variability between the test fields, attributable to the changes in the properties of the asphalt layers occurred in the three and a half years of in-service life;
- From the comparison between the 2016 and the 2019 results, the only inconsistency emerged for the HMA test field/mixture, which exhibited a reduction of the stiffness modulus back-calculated from FWD data and a concurrent increase of ITSM, likely ascribable to the more severe aging (and thus greater fracture and damage tendency) experienced during the in-service life with respect to all WMA mixes;
- Both the ITSM values and the measured FWD basins showed that the WMA mixes were characterized by a better homogeneity in terms of stiffness as compared to the HMA mix, probably due to the fact that the typical HMA working temperatures strongly increase the aging of RAP, which is not homogeneously distributed within the asphalt mixture. This outcome suggests that the use of WMA technologies can reduce pavement areas with lower performance, which represent preferential points for the initiation of distresses.

In summary, the evolutive trend obtained from FWD tests as well as from laboratory tests showed that the WMA mixtures investigated could provide superior performance and longer service life with respect to the reference HMA mixture.

The future monitoring of the trial section will allow to assess the long-term field performance of warm recycled pavements, thus filling some of the literature current gaps.

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