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Laboratory and in-situ tests for estimating improvements in asphalt concrete with the addition of an LDPE and EVA polymeric compound

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HIGHLIGHTS

- Analysis of asphalt mixtures performances with a polymeric compound is carried out.
- Additive effects on stiffness, deformation and fatigue resistance are provided.
- Modification is proved to increase rutting resistance and stiffness modulus.
- Modified low-bitumen mixes show comparable fatigue resistance than high-bitumen ones.

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ABSTRACT

Pavement deformation is a critical issue in the design of pavement structures and the related mixture. Asphalt concretes may be very sensitive to this problem, in compliance with the viscoelastic behaviour of the adopted bitumen. To improve the material performance, many attempts have been made to introduce in the mixture other materials as "modifiers" or "additives" for increasing the permanent deformation resistance and the elastic modulus of the material. Among the different possible materials, polymers determined significant improvements in the road pavement performance.

In this paper, the authors tested the adoption of a specifically engineered polymeric compound, in order to evaluate its effects on a generic asphalt mixture. Several tests were used to prove the effectiveness of the modification. Tests were performed not only in the laboratory, but also in an actual pavement section of the International Airport of Palermo in Sicily. Tests results prove that a proper mix-design can assure a decrease in the permanent deformations, remarkable growth of the material modulus with a reduced bitumen percentage, with economical savings. Finally, regarding fatigue resistance, the modified mixture with low bitumen content assures performance comparable to the control mix containing higher bitumen percentage. Then, the proposed application can be very useful to improve pavement performance even when using softer binder, easier to find in the Italian context.

1. Introduction

Pavement engineers always aim to improve material performance and optimize the pavement design and construction, for assuring to users the highest levels of comfort and safety, reducing maintenance operations and costs [1]. In order to achieve these goals different researchers have focused on improving asphalt production technologies [2] or adding materials that can significantly improve asphalt binder performance [3], aiming also to increase sustainability of the production and construction processes [4]. Consequently, interesting studies and applications aimed to evaluate the performance of asphalt mixtures containing, for example, reclaimed asphalt materials [5,6], tire rubber [7–10], glass waste [11], or plastic materials [12–17] and other additives [18]. Among these solutions, the addition of polymeric materials may be very efficient. Indeed, plastic can actually improve the mixture and the pavement performance [19–25], because it can affect in a remarkable way the physical and mechanical behaviour of the binder. Previous studies proved that, in particular, polymers can improve rutting resistance, high-temperature stiffness, susceptibility to temperature variations and, sometimes, also fatigue cracking resistance [26–29].

However, there is still a need for further investigating performance improvements of asphalt mixtures modified by the addition of different polymers. In particular, in many critical scenarios (such

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as roads characterized by significant traffic or airport pavements) the required specifications to the asphalt concrete are very restrictive, since it is essential to build pavements with optimal performance both in terms of stability and durability. Then, the use of special asphalt mixtures, known in literature as EME (Enrobes à Module Élevé) [30] – high modulus asphalt mixtures – is common and effective. Unfortunately, these mixtures require the utilisation of specifically selected aggregates and very hard bitumen that are not always available in different geographical contexts. For this reason, it may be interesting to assess the possibility to obtain high performance mixtures, with similar performance to EME, with locally available and less valuable base materials by polymeric modification of the mixture.

In this paper, the authors investigated the effects of a specifically engineered polymeric compound, on average quality asphalt mixes, aiming to increase the physical and mechanical performance of the mixtures. In order to evaluate material performance and assess the eventual improvements, different types of experimental tests were performed, both in laboratory and in situ. In particular, considering a dry modification process, the research was developed in two different phases:

– Phase 1 (laboratory tests), for preliminary mix design and performance evaluation;
– Phase 2 (lab and field tests): for verifying, in situ, material performance on a real test section.

In detail, in phase 1, after an accurate mix design for optimizing the mixture – providing information on the optimal polymer and bitumen contents for binder/base mixes –, several laboratory tests were performed in order to evaluate the benefits of introducing the compound in terms of rutting resistance, fatigue resistance, and stiffness modulus. Then, the analysis was extended in phase 2, with the adoption of the modified mixtures to build a real pavement section in the International Palermo Airport, considering the importance of real scale testing [31]. The polymeric additive was actually adopted in surface, binder, and base courses of the pavement and, thus, it was possible to verify if in situ dynamic stiffness moduli of the materials (estimated through Heavy Weight Deflectometer equipment) were in line with the values estimated in the laboratory. As shown in the following section, the experimental results confirmed that the proposed modification can improve performance of soft bitumen asphalt mixtures especially in terms of permanent deformation resistance and stiffness modulus. Further, fatigue resistance of modified mixtures produced with softer bitumen proves to be comparable to that of the reference mixtures at higher bitumen content.

In the following sections, first, materials details are provided, then the testing methodologies are specifically presented. Finally, results of the tests performed in the two different analysis phases are presented and discussed for evidencing the effects of the additive on the reference mixtures.

2. Materials

The adopted aggregates, the bitumen, and the additives are described in the following. In detail, different mixtures were designed for the two test scenarios: one for phase 1 (binder/base course), and three for phase 2 (surface, binder, and base courses).

2.1. Aggregates

The aggregates used in the mixtures were crushed limestone from a local quarry. Tables 1 and 2 provide the aggregate composition and the physical and mechanical properties of the available aggregates for the material used in phase 1, while the same information for the material used in phase 2 (aggregates supplied by the same quarry, for the preliminary laboratory characterization, but about 6 months later) is provided in Tables 3 and 4. As expected, the results of the physical and mechanical characteristics are very consistent, being the slight differences only due to the testing repeatability. Furthermore, the adopted mix grading curves, are given in Fig. 1 (a for phase 1 mix, b, c, d respectively for surface, binder, and base mixes for phase 2), together with the gradation

Table 1
Composition of the aggregate available fractions for phase 1 mix.

Sieve (mm)	Passing (%) Fractions				
	20/25	10/15	6/10	0/6	filler
32	100	100	100	100	100
24	100	100	100	100	100
20	94.73	100	100	100	100
12	18.33	99.96	100	100	100
8	0.77	85.67	99.91	99.70	100
4	0.51	38.3	72.62	97.78	100
2	0.49	15.66	41.54	70.60	100
0.4	0.44	6.62	16.84	26.09	99.31
0.18	0.41	5.28	11.21	17.19	93.78
0.075	0.33	4.01	6.42	10.57	74.25

Table 3
Composition of the aggregate available fractions for phase 2 mixes.

Sieve (mm)	Passing (%) Fractions					
	25/30	20/25	10/15	5/10	0/6	filler
31.5	100	100	100	100	100	100
25	91.25	100	100	100	100	100
20	6.34	94.85	100	100	100	100
12.5	0.33	17.20	98.85	100	100	100
8	0.22	0.74	85.15	99.70	100	100
4	0.16	0.66	35.91	26.30	90.30	100
2	0.13	0.54	14.89	5.80	58.30	100
0.425	0.11	0.50	7.32	2.50	23.70	99.00
0.18	0.09	0.50	4.83	2.20	15.90	92.47
0.075	0.07	0.41	3.44	1.70	10.60	74.12

Table 2
Physical and mechanical characteristics of the available aggregates for phase 1.

Characteristics	Standard	Unit	Fractions				
			20/25	20/25	20/25	20/25	20/25
Bulk specific weight	EN 1097-7	g/cm^3					2.85
Apparent specific weight	EN 1097-6	g/cm^3	2.82	2.83	2.84	2.85	
Los Angeles abrasion	EN 1097-2	%	22.10	20.19	20.64	20.12	
Sand equivalent	EN 933-8	%			91.38	90.41	
Void ratio			0.80	0.79	0.71	0.73	
Absorption coefficient	EN 1097-6		0.64	0.51			

Table 4
Physical and mechanical characteristics of the available aggregates for phase 2.

Characteristics	Standard	Unit	Fractions					
			25/30	20/25	10/15	5/10	0/6	filler
Bulk specific weight	EN 1097-7	g/cm^3						2.85
Apparent specific weight	EN 1097-6	g/cm^3	2.81	2.81	2.83	2.83	2.84	
Los Angeles abrasion	EN 1097-2	%	21.76	21.94	20.07	20.38	20.02	
Sand equivalent	EN 933-8	%				90.60	89.79	
Void ratio			0.86	0.78	0.80	0.73	0.70	
Absorption coefficient	EN 1097-6		0.51	0.48	0.48			

limits as defined in the Technical specification for the construction works of the in situ application.

2.2. Bitumen

Although low penetration grade bitumen are more advantageous for high-modulus asphalt mixtures, the bitumen used in this research was a neat bitumen, with 50/70 penetration grade (Table 5). As previously said, the analysis focused on a softer binder easily available in contexts such as in Italy, for assessing if the improvements produced by the modification can determine its practical adoption, also in critical scenarios, where high modulus and good mechanical performance are required.

A dynamic mechanical analysis of this binder was conducted with a dynamic shear rheometer (DSR), for evaluating its rheological properties in terms of complex modulus $|G^*|$ and phase angle δ for a reference temperature of 30 °C (Fig. 2). These were obtained by frequency sweep tests, carried out in strain-controlled mode over a wide range of temperatures, according to the EN 14770 standard. The tests were carried out using parallel plate geometry, by applying strain amplitudes carefully checked to be within the linear visco-elastic response of the material. The testing temperature ranged from −10 °C to 80 °C, while the testing frequency ranged from 0.1 to 10 Hz. As expected, by observing the values of both the complex modulus $|G^*|$ and the phase angle in the whole range of service temperature (i.e. for the whole range of reduced frequencies), these tests confirmed that the 50/70 bitumen offers a lower elastic behaviour compared to that offered by harder binders typically used for EME production [32].

2.3. Additive

The mixture studied in the research was modified using a polymeric compound (PC) of selected polymers, designed for commercial purposes and represented in Fig. 3. This compound is a mix of low-density polyethylene (LDPE) and ethylene-vinyl acetate (EVA)

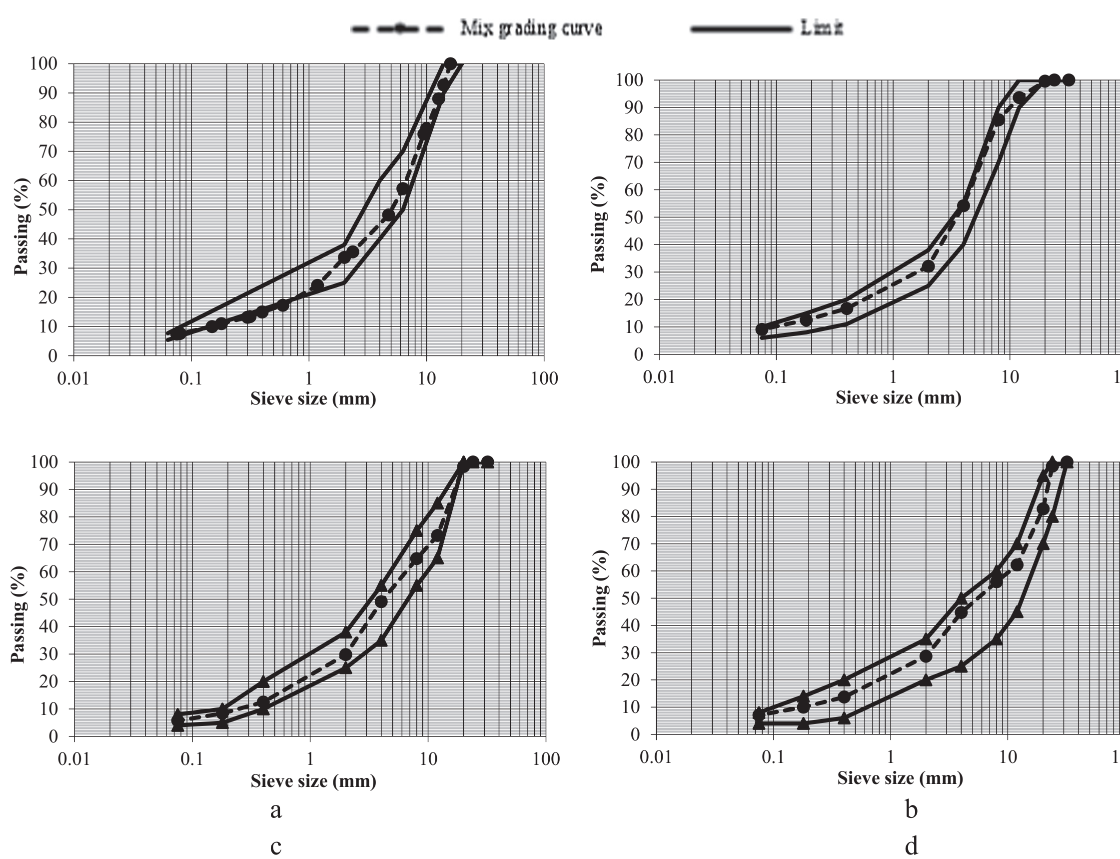

Fig. 1. Mix grading curves: (a) mix for phase 1, (b) surface mix phase 2, (c) binder mix phase 2, (d) base mix phase 2.

Table 5
Characteristics of 50/70 pen grade bitumen.

Characteristic	Standard	Unit	Value
Specific weight at 25 °C	EN 3838	g/cm^3	1.033
Penetration at 25 °C	EN 1426	dmm	68
Ring and Ball Softening Point	EN 1427	°C	50.5
Penetration Index	EN 12591		−0.21
Fraass Temperature	EN 12593	°C	−12
Ductility at 25 °C	ASTM D113	cm	>100
Viscosity at 60 °C	EN 13302	Pa·s	255.5
Viscosity at 100 °C	EN 13302	Pa·s	3.917
Viscosity at 135 °C	EN 13302	Pa·s	0.435
Viscosity at 150 °C	EN 13302	Pa·s	0.222
Mixing temperature (@ η = 0.17 Pa·s)	EN 13302	°C	155
Compaction temperature (@ η = 0.28 Pa·s)	EN 13302	°C	145
After RTFOT:			
Change in mass	EN 12607-1	%	0.19
Penetration at 25 °C	EN 1426	dmm	44
Ring and Ball Softening Point	EN 1427	°C	64.5
Viscosity at 60 °C	EN 13302	Pa·s	668

as well as others polymers with low molecular weight and medium melting point, that presents in semi-soft and flexible granules. It was not designed for modification of bitumen, but rather to improve the mechanical performance and durability of asphalt mixtures. Tentatively, a dosage of 4–8% on weight of bitumen is reasonable for this kind of polymers [33]. The physical properties listed in the technical sheet provided by the manufacturer are:

- aspect: granules;
- colour: shades of grey;
- dimensions: 2 ÷ 4 mm;
- softening point: 160 °C;
- melting point: 180 °C;
- melt index: 1 ÷ 5;
- specific weight: 0.934 g/ cm^3.

A differential scanning calorimetry (DSC) test was carried out on the compound in accordance with the ISO 11357-3 standard, in order to characterize its thermal behaviour. This test provides the thermal transitions of a polymer, that is temperature and enthalpy values corresponding to glass transition (T_g), melting point (T_m) and crystallization (T_c) through heating from 30 °C to 200 °C, cooling from 200 °C to 30 °C and again heating. For interpreting the DSC results, it is useful to recall that melting is an endothermic transition, because it is necessary to add energy to the polymer to make it melt, while crystallization is an exothermic transition because the polymer gives off heat when it crystallizes.

The test results are shown in Fig. 4 and confirm that the polymeric compound is actually a low-density polyethylene with a small quantity of high-density polyethylene and polypropylene. In fact, Fig. 4 shows 3 peaks: the first is at more or less at 106 °C and it is usually for low-density polyethylene, the second one at 120 °C is for the high-density polyethylene, while the last peak at 160 °C is for polypropylene. Since the area under the curve is proportional to the mass of the component polymers and the two areas at 120 °C and 160 °C are quite low, the quantity of polypropylene and high-density polyethylene (HDPE) is minimal. The other peaks are not important since they refer to crystallization of polymers at low temperatures [34]. It is interesting to notice that the DSC peaks in heating appear at around 100–120 °C, while typical production temperatures (mixing, mainly) are around 130–160 °C. This means that, during the production phases, the compound is in molten state, which is convenient for a good distribution into the mixture On the other hand, crystallization in cooling phase is visible at about 90 °C: this is a temperature conveniently lower than the laying temperature of the mixture, on site.

3. Methodology

3.1. Mix design

First it is fundamental to underline that, generally, the selected (apolar) additive has low affinity with the bitumen [35,36], making not practical or convenient to perform a wet modification process. Despite this issue, dry modification is very advantageous and may produce remarkable improvements in the mixture behaviour. For practical needs, it is much more advantageous to add the polymer in the asphalt mixture: the additive is added to the hot aggregates before mixing with the bitumen. In view of this, the optimal process to make a mixture is the succession of the following components: aggregates, additives, bitumen and filler [33,37].

The mix design was achieved by carrying out two type of tests:

– Marshall tests with different percentages of bitumen and compaction energy equal to 75 blows per face, according to the EN 12697-34 standard;
– compactibility test with a gyratory compactor (D = 150 mm), in accordance with the EN 12697-31 standard.

The volumetric properties (v, air voids, and VFB, voids filled with bitumen) were determined according to the requirements of the EN 12697-8 standard. The calculation of the maximum specific weight (γt) of the mixture was performed according to the “C” (mathematical) process specified by the EN 12697-5 standard, while the calculation of the apparent specific weight (γ_{app}) was performed according to the EN 12697-6 standard.

The mix design was different for the 2 phases of the analysis. Not only different materials and bitumen percentages were analysed in the two phases, but obviously the material performance was investigated from different perspectives and with different approaches. In particular, Phase 1 represented the first experimental approach to the new material and, thus, laboratory tests were numerous and aimed to preliminarily characterize the mixture behaviour and forecast performance. Consequently, mix design in phase 1 was deep and wide, consisting in two separates steps: first Marshall tests were focused on traditional mixtures without additives, to assess the physical and mechanical characteristics typically considered in the Italian Specifications and in compliance with the typical technical requirements; then, different percentages of the selected polymeric compound were added to the acceptable mixtures for performing compactibility tests by means of the gyratory compactor. As a consequence, phase 2 tests could rely on the results of the phase 1 analysis and aimed to confirm the efficiency of the material in practical applications, then the mix design resulted in a more expeditious and targeted procedure (compatible with execution time, on field), but in compliance with the specific technical requirements of the contract. In particular, phase 2 mix-design was performed using the Marshall methodology only on different specimens including the selected additive, by varying bitumen and additive percentages for different courses.

Mixture acceptability was determined in accordance with the technical specifications provided by the Italian National Authority on Public Works [38], whose limit values for Marshall Stability (S), Marshall Ratio (R) and voids (v) are listed in Table 6 for different courses.

The compactibility tests were performed using a gyratory compactor, designed to compact prepared HMA specimens at a constant consolidation pressure, a constant angle of gyration and a fixed speed of gyration. Moreover, it is equipped with a shear measurement system, which records the shear stress in terms of a unitless Gyratory Shear Ratio σ once per gyration. This is a measure of the internal stability of the mixture during the compaction, given

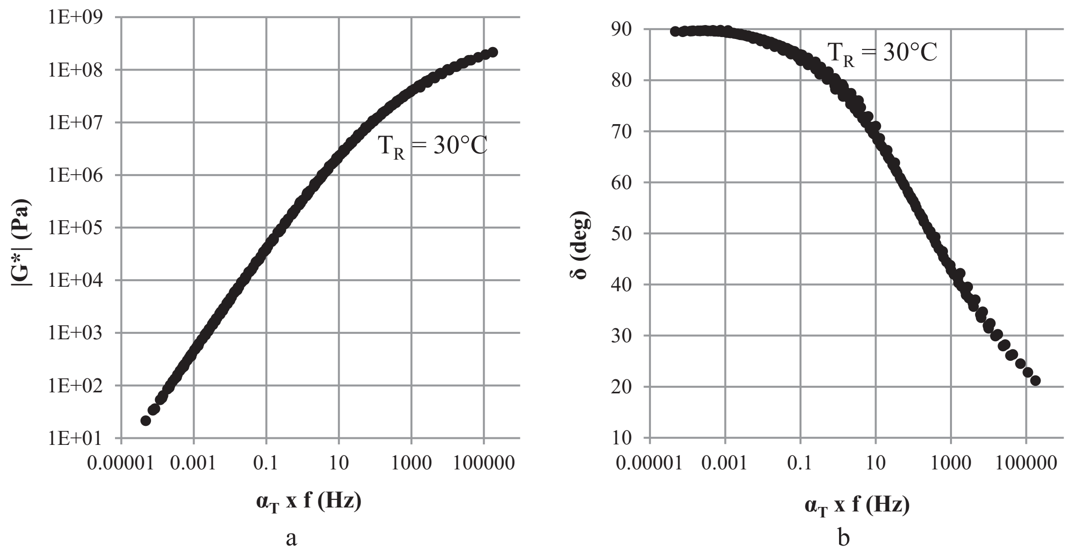

Fig. 2. Complex modulus (a) and phase angle (b) master curves of the adopted binder (50/70).

Fig. 3. Additive used in this study (PC).

by Eq. (1) as the ratio between the shear stress S and ram pressure P.

$$\sigma = \frac{(F \cdot d)/V}{R/A} \tag{1}$$

where F is a vertical force applied in order to achieve the gyration angle α during compaction, d is the lever arm distance, V is the specimen volume, R is the ram force applied to the bottom plate (opposed by an equal but opposite force at the fixed top plate) and A is the cross-sectional area of the mold [39].

Acquisition of this feature was carried out on specimens compacted up to a number of gyrations N equal to 200 rpm, in order to evaluate the behaviour of the mixtures in the different conditions of densification that affect it from the time of laying throughout the design life years. The densification curves recorded during the gyratory compaction made it possible to obtain parameters of the regression lines, K and C_1, that respectively define the workability and the self-densification of these mixtures.

3.2. Rutting resistance

In order to evaluate the material performance and the improvements produced by the polymer addition, comparisons in terms of rutting resistance can be very productive. In detail, rutting resistance was evaluated according to the EN 12697-22 standard, method B. The tests were performed at 60 °C on at least two specimens (slabs with dimension 305 × 305 × 50 mm) for each selected mixture (after mix design optimization). Air content was set constant and equal to v = 4.5%. Results consist in the average

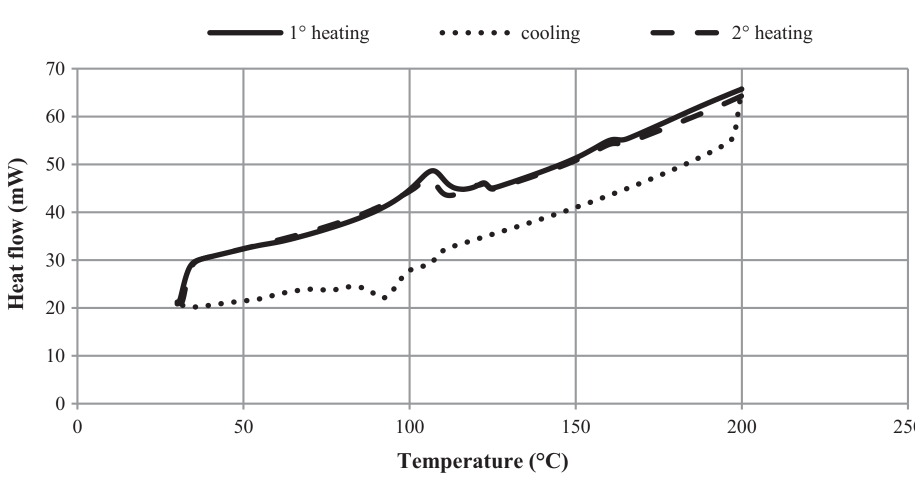

Fig. 4. DSC test results on the polymeric compound used.

Table 6
Marshall limit values in accordance with the Italian National specifications [38].

Required results	Unit	Course		
		Surface	Binder	Base
Marshall Stability, S	kN	>11	>10	>8
Marshall Ratio, R	kN/mm	3 ÷ 4.5	3 ÷ 4.5	>2.5
Marshall voids, v	%	3 ÷ 6	4 ÷ 6	4 ÷ 7

rut depth for the selected mixtures and in a parameter called wheel-tracking slope in air (WTS_{air}), i.e. the average rate at which rut depth increases with the number of passages (generally calculated between the 5000th and 10000th loading cycles). The lower these values for a mixture, the higher the related rutting deformation resistance.

3.3. Fatigue resistance

Comparisons were also made on the fatigue resistance of the different mixtures. In this case, tests were performed according to the EN 12697-24 standard, annex D. The fatigue behaviour of the optimized mixtures was studied through a four-point bending apparatus and the GCTS CATS software. The fatigue criterion used was the classical one, referenced as N_{f50}, corresponding to the number of cycles for which the modulus decreases to 50% of its initial value. The initial value was calculated at the 100th load cycle. The value of the strain amplitude leading to failure at one million cycles is hereafter called "ε_{10}^{6}". In detail, tests were conducted at 20 °C and 10 Hz on beams with dimensions 400 × 45 × 50 mm. The deformation was 350 με for short-duration tests and 150 με for long-term tests.

Fatigue test results made it possible to obtain the regression lines (Wöhler curves), represented by Eq. (2).

$$\varepsilon = a \times N^{-b} \tag{2}$$

where a is a constant and it depends on the physical and mechanical characteristics of the material, test temperature and frequency; b is the slope of the fatigue lines.

For the different tests, parameters a and b were calculated and compared. Moreover, the coefficient of determination (R^2) and the admissible strain level at $N = 10^6$ loading applications (ε_{10}^{6}) were evaluated in order to characterize the fatigue resistance.

3.4. Stiffness modulus

The stiffness modulus was the last parameter considered for evaluating the influence of the selected additive in asphalt mixtures and comparing the different mixtures. Stiffness modulus was calculated both in laboratory and in situ. Laboratory tests were performed according to the EN 12697-26 standard, annexes B and D, while in situ estimations relied on deflection measurements obtained through Heavy Weight Deflectometer (HWD) and processed using both the BAKFAA software program made available by the Federal Aviation Administration and Elmod6.

Concerning phase 1 both lab test types were performed. First, according to annex B, the loading configuration was that adopted for the fatigue tests. Beams had dimensions 400 × 50 × 45 mm (or 400 × 50 × 50 mm) and each test was carried up to 150 cycles. The deformation was 25 με, the temperature was 20 °C and the frequencies were 1, 10, 30 Hz and again 1 Hz in order to check that the specimen has not been damaged during the loading. Then, the dynamic modulus was also evaluated by means of a triaxial cell, according to annex D. The resulting recoverable axial strain response of the specimen is measured and used to calculate dynamic modulus. The loading configuration was direct compression on cylindrical specimens in controlled stress. The tests were conducted at 10, 20, 30 and 40 °C, at a confining pressure of 0 KPa and six frequencies per temperature (20, 10, 5, 1, 0.5 and 0.1 Hz) on cylinders with dimensions 100 × 150 mm obtained after compaction with a gyratory compactor. The stress levels applied were chosen in such a way that the strain response was kept within 50–150 με. Four replicates were tested for each mixture and the results were averaged.

The isotherms obtained were used for determination of the Master Curves. The values of the shift factor were calculated and optimized according to the formula of Arrhenius [40]. In this way, by horizontal translation of the shift of isotherms relating to the test temperatures it was possible to construct the master curve at a reference temperature of 20 °C for each bituminous mixture and analyse the material behaviour in the entire domain.

Regarding phase 2, stiffness moduli were evaluated in lab and in situ. Lab tests analysed separately surface, binder, and base course. In detail, four-points bending test was performed on surface and binder courses only, due to inconsistency between maximum aggregate diameter size and the testing equipment for the base course. On the contrary, triaxial cell tests on cylindrical specimens were performed on the three courses. Then, moduli were also evaluated in situ by means of deflection basin measurements performed through the HWD equipment. In a HWD test, an impulse load is applied to the pavement surface by dropping a weight onto a circular metal plate and the resulting pavement surface deflections are measured directly beneath the plate and at several radial offsets [41]. The deflection of an asphalt concrete pavement is almost vertical, forms a basin and represents an overall "system response" of the pavement layers to an applied load.

The deflected shape of the basin, known layer thicknesses and magnitude of the load, can be related to the moduli of the different layers, the values of which can be calculated using specific back-calculation procedures through iterative processes relying on different theoretical models [41,42]. In this study, the authors considered two different software for performing the back-calculation procedure: BAKFAA [43] and Elmod6 [44]. BAKFAA is a software that performs back-calculation of pavement layer modulus values using the FAA layered elastic analysis program called LEAF (Layered Elastic Analysis Program in Forward mode) and a downhill multidimensional simplex minimization method. The function minimized is the sum of the squares of the differences between vertical pavement surface deflections measured with a HWD and vertical pavement surface deflections computed with the layered elastic program. Elmod, instead, was used to perform back-calculation according to two different approaches, respectively called LET (Linear Elastic Theory) and MET (Method of Equivalent Thicknesses Theory) for further evaluations of the layer moduli.

4. Results

The comparison results confirmed the effectiveness of modifying bituminous mixtures with the addition of the polymeric compound. In the following paragraphs, numerical details concerning the different performed tests on the various considered mixtures are provided.

4.1. Phase 1: Laboratory tests

The first step in phase 1 testing process was the application of the Marshall method only to the traditional mixture (without additive) produced with 50/70 pen grade bitumen for preliminary mix design. Four percentages of bitumen were selected ($b'_1 = 4.8\%$, $b'_2 = 5.2\%$, $b'_3 = 5.5\%$, $b'_4 = 5.9\%$, by weight of the aggregates) and

Table 7
Marshall test results.

b (%)	v (%)	S (kN)	F (mm)	R (KN/mm)	VFB (%)	γ_{app} (g/cm^3)
4.8	6.03	12.24	4.27	2.87	69.22	2.46
5.2	2.88	13.39	4.07	3.29	82.90	2.52
5.5	2.52	14.67	4.40	3.33	86.05	2.52
5.9	1.82	13.30	5.33	2.49	89.75	2.53

four specimens for each percentage were produced, for a valid repetition.

The Marshall test results are reported in Table 7 and Fig. 5. In Fig. 5 for more clarity, the acceptability limits for binder and base courses, based on the Italian technical standard [38] and detailed in Table 6 are depicted too, in terms of S (S_{min} BASE, S_{min} BINDER), R (R_{min} BASE, R_{min} BINDER) and v (v_{min} – same for both binder and base –, v_{max} BASE and v_{max} BINDER). It is clear that mixtures with bitumen content between 4.8 and 5.1% are in compliance with the provided limitations.

The allowable mixtures were then subjected to gyratory compaction. The gyratory compactor test was initially carried out on two specimens with different bitumen contents (4.9, 5.1 and 5.4%) and three different PC contents (PC.0 = 0% of polymer, PC.3 = 0.3% of polymer, PC.6 = 0.6% of polymer, by weight of mineral aggregates). Later, the specimens with PC being subject to bleeding due to the excessive binder content, it was also decided to carry out the test on two specimens with two lower bitumen contents (4.3%, 4.6%), with a PC content equal to (0.3%). Fig. 6 represents the densification curves recorded during the gyratory compaction for the different mixtures. Table 8 provides the representative values related to the densification curves (C_1, K, R^2) and the percentage of air voids; regarding voids, Table 8 shows also the requirements at 10, 100 and 190 rpm defined by the technical specification of the National Agency for Roads [45].

Concerning permanent deformation resistance, the wheel-tracking test was carried out on the mixtures studied considering two percentages of bitumen for each percentage of polymer chosen and two slabs for each mixture, with dimensions 305 × 305 × 50 mm and air void content v = 4.5%. The average values are reported in Fig. 7. Results in terms of WTS_{air} are listed in Table 9.

Regarding stiffness modulus and fatigue cracking resistance, tests were performed only on the optimized mixtures defined through the previous tests. The results of these tests are provided in Table 10 and Fig. 8. In detail, Table 10 lists results of stiffness modulus tests carried out on prismatic specimens in the four-point bending test configuration ($|E^*|$ and ϕ) and the parameters of the fatigue lines (a, b, R^2, ε_{10}^6), while Fig. 8 represents the fatigue lines and the master curves obtained by means of a triaxial cell on cylindrical specimens.

4.2. Phase 2: Lab and field tests

After the first comparisons and investigations of phase 1, the tests were extended to verify in situ performance of the mixtures. This phase started from a new mix design of surface, binder and

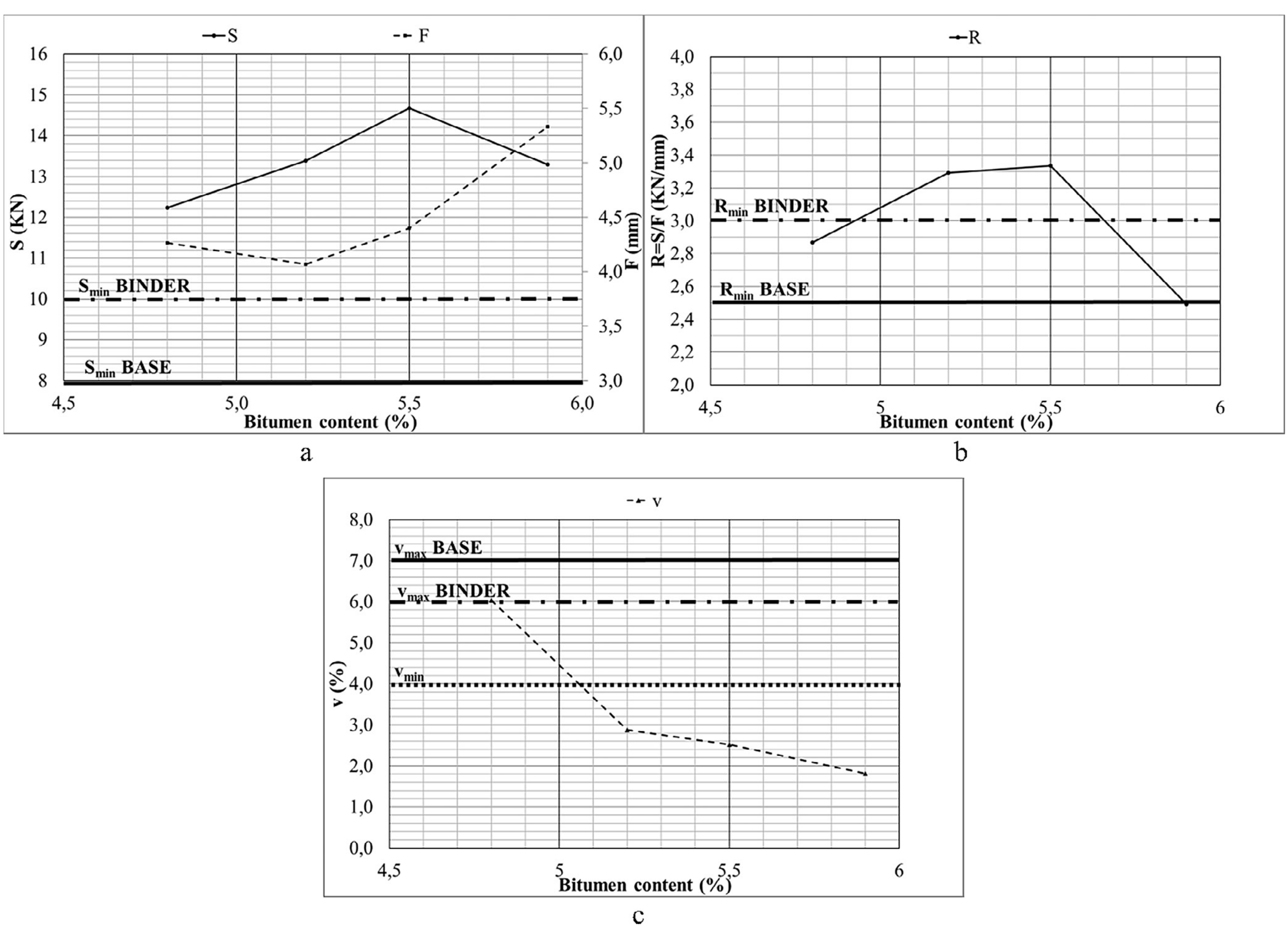

Fig. 5. Marshall values at different bitumen contents: (a) S and F, (b) R, (c) v.

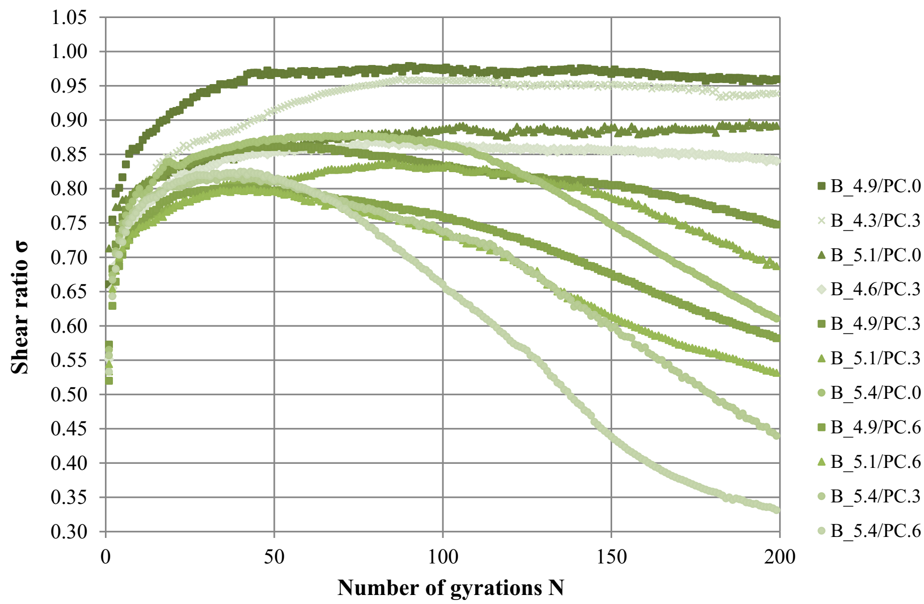

Fig. 6. Shear ratio lines of the different mixtures.

Table 8
Values of workability and self-densification and air voids (phase 1).

Mixture	b	p	% G_{mm} = C_1 + k * log (N)			Air voids (%)		
	%	%	C_1	K	R^2	N = 10	N = 100	N = 190
ANAS limits	–	–	–	–	–	11–15	3–6	≥ 2
B_4.3/PC.3	4.3	0.3	0.8009	0.0757	0.9980	12.6	4.7	2.8
B_4.6/PC.3	4.6	0.3	0.8072	0.0739	0.9984	12.1	4.4	2.6
B_4.9/PC.0	4.9	0.0	0.7972	0.0753	0.9989	13.0	5.1	3.2
B_4.9/PC.3	4.9	0.3	0.8141	0.0788	0.9952	11.1	2.7	1.0
B_4.9/PC.6	4.9	0.6	0.8278	0.0748	0.9900	10.2	2.0	0.7
B_5.1/PC.0	5.1	0.0	0.8190	0.0710	0.9988	11.2	3.8	2.0
B_5.1/PC.3	5.1	0.3	0.8168	0.0786	0.9940	10.8	2.4	0.8
B_5.1/PC.6	5.1	0.6	0.8274	0.0755	0.9880	10.2	1.9	0.6
B_5.4/PC.0	5.4	0.0	0.8141	0.0788	0.9952	9.9	1.7	0.4
B_5.4/PC.3	5.4	0.3	0.8345	0.0741	0.9837	9.6	1.4	0.3
B_5.4/PC.6	5.4	0.6	0.8366	0.0742	0.9702	9.5	1.0	0.3

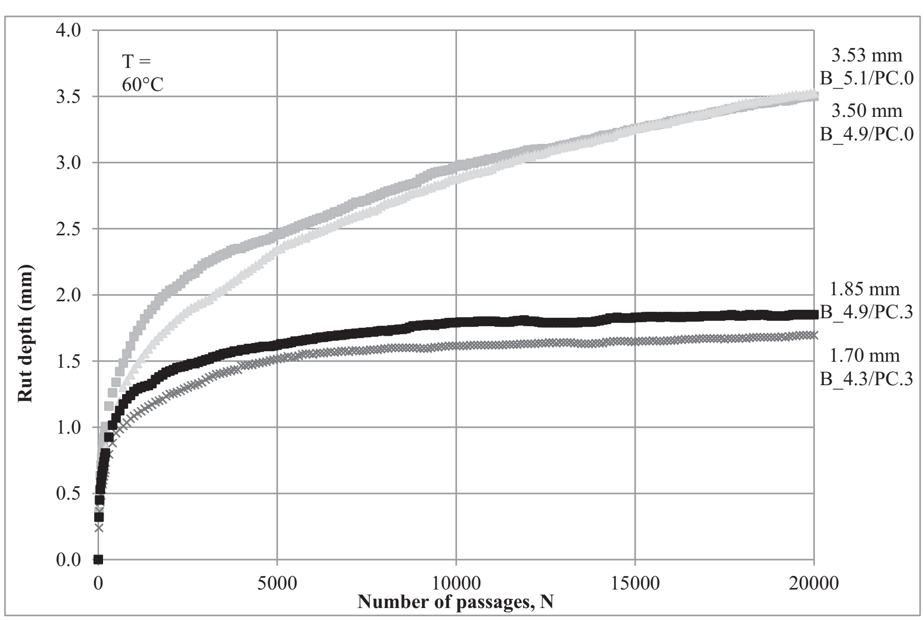

Fig. 7. Trend of rut depth in the wheel-tracking test.

Table 9
Values of WTS_{air} (phase 1).

Mixture	WTS_{air} (mm/10^3 cycles)
B_5.1/PC.0	0.130
B_4.9/PC.0	0.106
B_4.3/PC.3	0.016
B_4.9/PC.3	0.012

base mixtures, then, on the optimized ones, stiffness moduli were calculated both in laboratory and in situ. The mix design was performed through the Marshall method, using three bitumen contents (chosen respecting the specified technical requirements of the construction contract for the apron pavement in the airport of Palermo) and one PC content for different courses:

- b'_1 = 4.8%, b'_2 = 5.6%, b'_3 = 6.5%, by mass of aggregates, and PC = 8%, by mass of bitumen, for the surface course;
- b'_1 = 4.5%, b'_2 = 5.0%, b'_3 = 5.5%, by mass of aggregates, and PC = 5.5%, by mass of bitumen, for the binder course;
- b'_1 = 4.0%, b'_2 = 4.5%, b'_3 = 5.0%, by mass of aggregates, and PC = 5.5%, by mass of bitumen, for the base course.

The mixing temperature was 175 °C, while the minimum compaction temperature was 135 °C (except for the surface course, which was 150 °C). The results are reported in Table 11.

In compliance with specifications listed in Table 6 [38], for the surface course, the acceptable specimens are those containing 8% (by mass of bitumen) of PC and 5.6% (by mass of aggregates) of bitumen (i.e. around 0.4% of PC by mass of aggregates); for the binder course, the ones containing 5.5% (by mass of bitumen) of PC and 4.9% (by mass of aggregates) of bitumen (i.e. 0.3% of PC by mass of aggregates), and finally, for the base course, the ones containing 5.5% (by mass of bitumen) of PC and 4.1% (by mass of aggregates) of bitumen (i.e. 0.2% of PC by mass of aggregates).

Then, stiffness modulus tests were performed on these selected mixtures. Concerning laboratory tests, as in phase 1, the adopted methodologies were both the four-point bending tests on prismatic beams and triaxial test on cylindrical specimens. Results of four-point bending tests are listed in Table 12 (base mixture was not tested, due to inconsistency of D_{max} with the testing equipment), while isotherms and Master curves (reference T = 30 °C) calculated on cylindrical specimens are shown in Fig. 9.

Then, optimized mixtures were used for building a real pavement section of an access to the new apron of Palermo International Airport, whose layers are shown in Fig. 10. It is a semi-rigid pavement (an intermediate between a flexible pavement and a rigid pavement), where there are the surface, binder and base courses in polymer-modified asphalt (PMA, asphalt mixture with PC), a cement treated subbase, a cement stabilized soil and the bedrock formed by calcarenite stones.

Table 11
Marshall test results (phase 2).

Course	b (%)	S (kN)	F (mm)	R (KN/mm)	v (%)	γ_{app} (g/cm^3)
Surface	4.8	11.5	4.0	2.9	4.20	2.499
	5.6	12.0	2.9	4.1	3.60	2.514
	6.5	11.1	3.0	3.7	3.50	2.518
Binder	4.5	10.5	3.1	3.4	6.14	2.452
	5.0	10.8	3.3	3.3	5.45	2.472
	5.5	10.0	3.3	3.0	5.06	2.479
Base	4.0	11.0	3.5	3.1	4.95	2.489
	4.5	10.7	3.4	3.2	4.76	2.496
	5.0	9.4	3.6	2.6	3.30	2.499

Table 12
Values of complex modulus and phase angle for surface and binder courses.

Mixture	$\|E^*\|$ (MPa) Frequency (Hz)			ϕ (°) Frequency (Hz)		
	1	10	30	1	10	30
SURFACE	7037	11,250	11,903	21	13	13
BINDER	5193	8256	9461	23	16	16

Table 10
Values of complex modulus, phase angle and fatigue line parameters (phase 1). Test temp.: 20 °C.

Mixture	$\|E^*\|$ (MPa) Frequency (Hz)			ϕ (°) Frequency (Hz)			Fatigue line parameters			
	1	10	30	1	10	30	a	b	R^2	ε_{10}^{6}
B_4.9/PC.0	3692	7282	8756	36	24	23	4532	−0.240	0.9797	164.5
B_4.3/PC.3	4168	7528	9188	30	20	18	6328	−0.262	0.9828	169.5

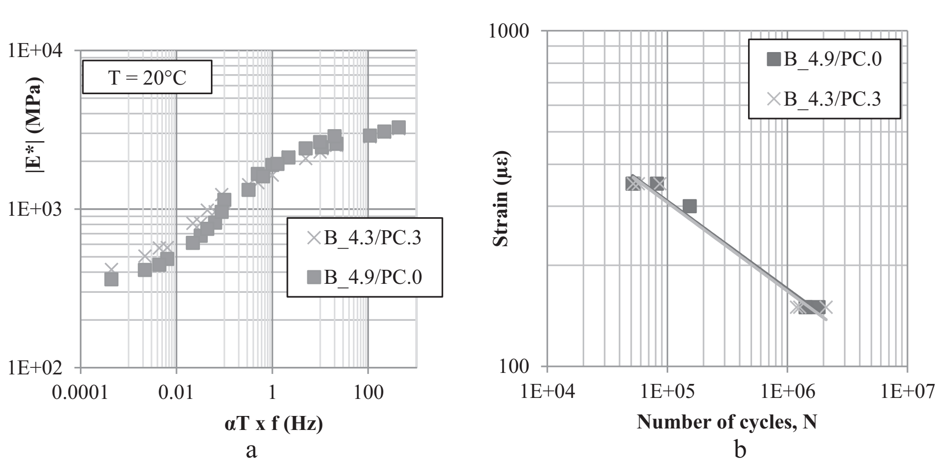

Fig. 8. Master curves (a) and fatigue lines (b) – phase 1.

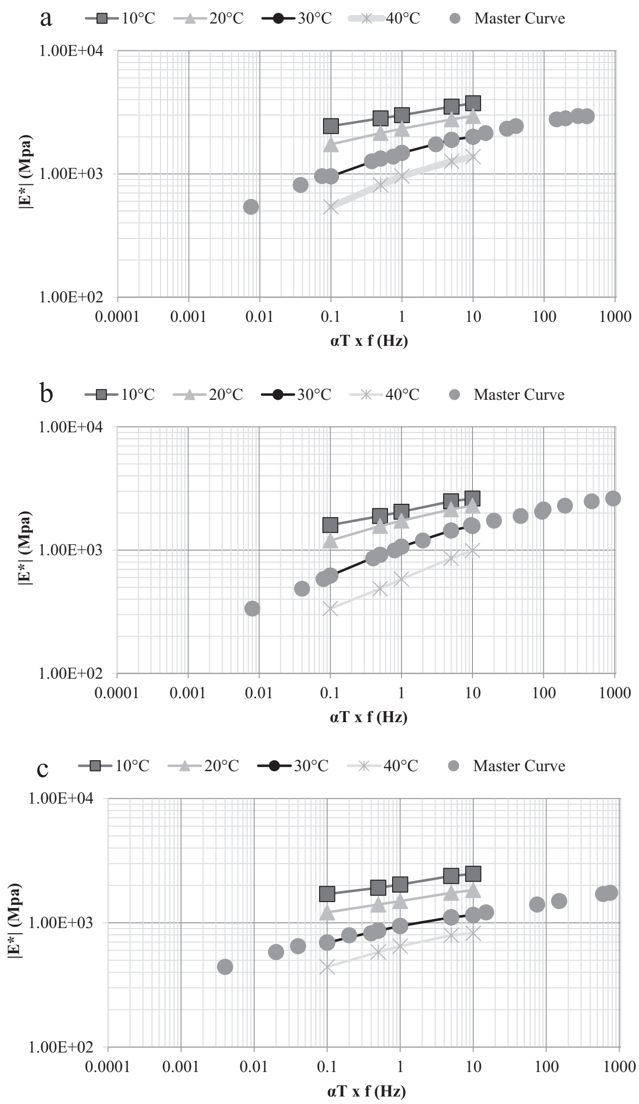

Fig. 9. Complex modulus master curves for the surface (a), binder (b), and base (c) courses – phase 2.

The in situ tests consisted in HWD measurements, considering a maximum load around 240 kN. Three drops for every station (2 different alignments, 4 station for each alignment) were carried out. The adopted HWD equipment is a Dynatest testing equipment with 9 geophones (D). The pavement temperature – very similar for all measurements, around 30 °C – was recorded during deflection tests, by means of a thermocouple equipped on the HWD device and the recorded temperatures have been taken into account in all the back-calculation methodologies.

First, deflections were then processed using the BAKFAA software. Only 7 geophones are admitted for this back-calculation procedure and, thus, geophones D6 and D9 were not considered. The input values adopted for the back-calculation process are listed in Table 13: in detail, interface parameter represents the bond between two pavement layers (from 0 = no bond to 1 = 100% bonding), while layer changeable represents whether the associated layer will allow the modulus seed value to be computed during the back-calculation process or not). Since it is good practice to avoid considering too many different layers in back-calculation (especially if they show relatively similar performance), surface, binder, and base layers were considered in a single layer (Layer 1 is representative of the PMA layers analysed in this paper). Regarding Elmod, the adopted structural scheme was the same, but the analysis was performed using all 9 available geophones. Moreover, for further testing the obtained values, since the target layers were surface ones, LET and MET theory were applied for matching deflection basins including the first 3 geophones only. This is due to the fact that the closer the geophone to the loading plate, the higher the correlation of the deflection value with the first layers. In Table 14, the moduli obtained for the first layer with the different approaches are shown.

5. Discussions

As shown in the previous section, experimental results were very positive, proving the effectiveness of the analysed PC as efficient modifier in order to adopt easily-to-find soft bitumen instead of harder ones.

Concerning the mix design in phase 1 and according to Italian specifications on Marshall results (Table 6) [38], initially the range [4.8%, 5.1%] (in weight of aggregates) seemed to be acceptable for the bitumen content for mixtures without additive (Table 7, Fig. 5). However, the Gyratory compactor analysis showed the effect of PC on the mixture, evidencing the opportunity to reduce the bitumen content for avoiding bleeding (up to 4.3%). Then, the presence of PC allows an actual optimization of the mixtures with lower binder contents, producing economical savings without affecting performance negatively (as proved in the following). At this regard, the performed tests cannot be used for investigating the actual reasons for the binder characteristics shown by PC. However, it would be very interesting to extend the experimental research on PC, focusing also on chemical and visco-elastic mechanisms of PC.

The analysis of the shear ratio is also very remarkable (Table 8, Fig. 6). In general, an increasing trend of the shear ratio σ in the initial stage of compaction (approximately, in the first 50 rpm)

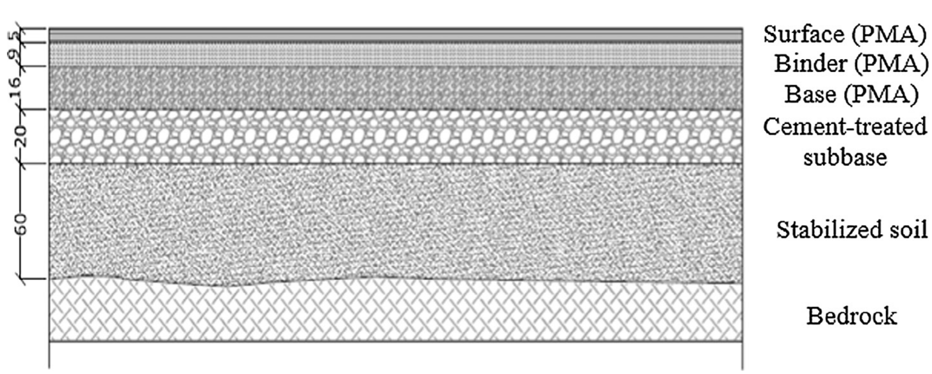

Fig. 10. Scheme of the testing pavement at the Palermo International Airport.

Table 13
Input values for BAKFAA.

Layer	Young's modulus (MPa)	Poisson's ratio	Thickness (mm)	Interface parameter	Layer changeable
Surface, Binder and base Course	500	0.35	300	1	Yes
Subbase	5000	0.20	200	1	Yes
Stabilized Soil	350	0.20	600	1	Yes
Bedrock	100	0.25	0	0	Yes

Table 14
Back-calculated moduli (signed values were excluded from calculation, as outliers).

	Stations [m]	E [MPa]					
		BAKFAA	ELMOD			Mean	Std Dev
		7 geoph.	LET		MET		
			9 geoph.	3 geoph.	3 geoph.		
Alignment 1	0.01	829.75	852.7	964	876.7	881	59
	0.04	841.15	1075.7	1128,2	1145.1	1116	36
	0.071	1424.34	1366.7	1389.4	1291.2	1368	56
	0.101	1344.05	1332.7	1389.4	1221,2	1322	71
Alignment 2	0	731.67	1775.6	833.6	751	772	54
	0.03	1152.15	1298.8	1157.5	1076.6	1171	93
	0.06	1129.8	1266.3	1157.5	1076.6	1158	80
	0.09	1380.28	1414.4	1391.5	1325.7	1378	38

and stabilization even beyond the maximum value of N, together with fulfilment of the volumetric requirements (VMA, VFA), ensure a correct formulation and good stability during operation [46]. This test gives a good idea of the job-site density values, according to course thickness, and allows an optimization of the mixture in terms of bitumen and PC contents. In detail, concerning the values reported in Table 8 (K, C_1, R^2), it can be observed that for the same aggregate skeleton the workability does not depend on the bitumen or polymer content. Instead, when the bitumen or polymer content increases, the values of the initial densification C_1 and, consequently, the compactness at any number of revolutions also increase. The shear ratio lines in Fig. 6 show that the specimens with 0.3% of PC and 4.3 or 4.6% of bitumen, and specimens with 0% of PC but 4.9 or 5.1 of bitumen, mobilize shear ratio values that are maintained constant during the design life years. By contrast, a slight excess of bitumen and a slight excess of additive cause a fall of the shear ratio and therefore the related content is not optimal. Similar trends and results can be evidenced by analysing voids in Table 8. Consequently, based on mix design results, for avoiding bleeding and assuring the best performance, 0.3% (in weight of mineral aggregates) was assumed as an optimum PC content and 5.1% (in weight of mineral aggregates) was confirmed as superior limit for bitumen content.

Permanent deformation resistance results on optimized mixtures are also very significant (Fig. 7, Table 9), confirming the great improvement assured by additive modification. It results that for the same percentage of polymer, there is no substantial difference in terms of rut depth; by contrast, at the same percentage of bitumen, the rut depth values of the mixture with PC are 50% lower than those of the mixture without additives. The WTS_{air} parameter confirms these conclusion (Table 9): in detail, one can observe that – as expected – this parameter increases when the percentage of bitumen increases, while it significantly decreases with the increase in the percentage of additive. In fact the addition of PC can produce a reduction in WTS_{air} values obtained of almost 90% with respect to those of the mixtures with no additive. Consequently, it is easy to understand that B_4.9/PC.3 may represent the optimized mixture with PC. The phase 1 investigation involved also stiffness modulus and fatigue resistance evaluation on the optimized mixture. For comparison, the optimized mix without PC was also further tested. Considering the four-point bending test results, the stiffness modulus test parameters are reported in Table 10 ($|E^*|$, ϕ) and show what it was reasonable to expect: for each mixture the stiffness modulus values are highest at high frequencies, and thus lowest at low frequencies, while it is the opposite in the case of the phase angle values. Furthermore, despite the lower bitumen content, the mixtures with PC are characterized by higher moduli and lower phase angles. On the other hand, the master curves obtained from results on cylindrical specimens (Fig. 8a) does not show any remarkable difference between the two mixtures for the entire range of frequency. This behaviour should be underlined, because obviously the two mixtures were characterized at different binder contents, but again PC addition overcame to this lack. Finally, the fatigue line (Fig. 8b) and the related parameters provided in Table 10 (a, b, R^2 and ε_{10}^{6}) synthetizes the fatigue behaviour of the mixtures. First, the fatigue lines have high values of the regression coefficients R^2 (around 0.98) and this means that the results are only slightly dispersed and very reproducible. Further, by comparing the fatigue lines, it can be noticed that the mixture with additive offers fatigue performance absolutely similar to that of the mixture without additives at higher bitumen content, which is known for being beneficial for this specific performance. In any case, the admissible strain values at 1.000.000 load application, ε_6 (>130 µstrain), can be considered very satisfactory for road paving applications, showing that the addition of polymers is a viable solution even when good fatigue resistance is required.

Then, phase 1 tests can be considered adequate to indicate the possibility of using PC to improve mechanical performances of bituminous mixtures produced with component of average quality. The optimized mixture is finally characterized by good stability and compaction values and PC assures a relevant positive influence regarding permanent deformation resistance and an increase in stiffness modulus.

Finally, phase 2 was performed to further investigate the PC benefits and verify whether similar high stiffness modulus values can be confirmed on actual pavement section, for justifying the adoption of such kind of modified mixture in critical scenarios, such as airport pavements. In this phase, mix design was specifically performed again through Marshall method for the different courses. According to the National Technical specifications [38] (Table 6) the optimized mixtures contain (by mass of aggregates):

– Surface: 0.4% of PC and 5.6% of bitumen;
– Binder: 0.3% of PC and 4.9% of bitumen;
– Base: 0.2% of PC and 4.1% of bitumen.

Obviously, the closer the layer to the pavement surface, the higher the required performance to the material, the higher the quantities of bitumen and PC needed. The selected mixtures were, thus, used for building the pavement section of the apron access at the Palermo International Airport. Moduli of the PMA layers were then estimated both in laboratory and in situ. Again, by analysing Table 12, it is noticed that for each mixture the stiffness modulus values are highest at high frequencies (and vice versa), while it is the opposite in the case of the phase angle values. In general, the high values obtained for these mixtures confirm what resulted in phase 1. However, comparing the four-point bending tests results for binder (Table 12) with those of phase 1 (Table 10) it is interesting to notice that for equal bitumen content, the addition of PC assures a relevant increase in modulus, especially at lower frequencies, where values are generally lower. In addition, the analysis of the master curves provided in Fig. 9 evidences stiffness modulus values in line with the results of phase 1.

Despite this, as said, the moduli were then also back-calculated considering deflections values measured with HWD equipment on the airport pavement section. Despite the possible uncertainties due to back calculation procedures and to the not identical testing processes affecting lab and in situ experiments, in situ estimations are widely in line with lab results. Since the asphalt temperature on the testing day was around 30 °C, it is possible to assess that the moduli of the PMA layer (Table 14) are comparable with those obtained through laboratory tests and represented in the master curves of Fig. 9. Actually, all the methodologies provided similar moduli (excluding some specific outliers that were not included in averages), with an overall mean value around 1100 MPa. Excluding station 1 values for both alignments that seems to be characterized by too lower modulus values, the mean value can rise up to 1237 MPa, widely comparable with lab outcomes. Obviously, the comparison is more qualitative than quantitative, since the back-calculated modulus is related to the 3 PMA layers together, making impractical a specific and direct evaluation of single layer moduli. However, phase 2 results further confirmed the high modulus values obtainable with the addition of PC, justifying the use of PC modified mixtures in strategical settings such airports, where pavements are subjected to high loads and, thus, to more rapid and significant degradation processes.

6. Conclusions

The laboratory and in situ tests presented in this paper widely proved the possible improvements that can be obtained by the modification of asphalt mixtures with an engineered polymeric compound (PC) designed for commercial purposes. The various experimental tests assessed the possibility to adopt medium quality asphalt mixtures (with lower binder content and softer bitumen) even for strategical applications, such as airport pavements. In fact, the addition of PC guarantees two times better performance in terms of permanent deformation resistance and stiffness modulus comparable to those of harder bitumen. These improvements have not drawbacks in terms of fatigue resistance, but on the contrary, PC addition guarantees performance of low-bitumen mixtures comparable to higher-bitumen ones. Numerical results evidenced possible optimal contents of bitumen and PC for practical applications, for properly determining the mixture composition. Moreover, PC modification was actually adopted for improving performance of the 50/70 soft bitumen mixtures designed for realizing the AC layers of a pavement section at the Palermo International Airport. In situ deflection measurements and particular back-calculation procedures were performed to further prove the high modulus assured by this modification, making practical the substitution of harder bitumen mixtures with more easily available softer ones.

It is possible to assess that:

– the use of PC guarantees improved performance in terms of permanent deformation resistance;
– in terms of fatigue resistance, the addition of PC compensates for the lack of bitumen;
– PC increases the stiffness modulus of the mixture;
– there is a good correlation between lab and in situ moduli;
– PC allows the adoption of less hard bitumens than those traditionally preferred for critical applications;
– the presence of PC guarantees a mix-design optimization with a lower binder content.

In conclusion, PC can represent a reliable and positive solution for numerous practical scenarios in which is required higher resistance to permanent deformation. Moreover, these advantages may produce remarkable economical savings for the involved subjects, assuring high performance levels with lower costs. From an economical point of view, the proposed solution assures significant reductions in costs, since in average the cost of the traditional binder adopted in the mixture is around half of EME binder and around 55–60% of soft or hard modified bitumen. Further, polymeric addition is very low in quantity and does not require expensive plant modifications. Finally, savings are also due to the use of locally available aggregates and this can also reduce construction impacts in terms of sustainability.

In future works, since the testing pavement section was not interested by airplane traffic yet, analysing its performance trend during its service life will give the opportunity to confirm forecasting on rutting and fatigue resistance improvements and to understand better the practical benefits of the SP modification.

Conflict of interest

There are no known conflicts of interest.

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